CECW-EG Engineer Manual 1110-2-2002	Department of the Army U.S. Army Corps of Engineers Washington, DC 20314-1000	EM 1110-2-2002 30 June 1995
	Engineering and Design EVALUATION AND REPAIR OF CONCRETE STRUCTURES	
	Distribution Restriction Statement Approved for public release; distribution is unlimited.	

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

EM 1110-2-2002

CECW-EG

Manual
No. 1110-2-2002

30 June 1995

Engineering and Design
EVALUATION AND REPAIR OF CONCRETE STRUCTURES

1. Purpose. This manual provides guidance on evaluating the condition of the concrete in a structure, relating the condition of the concrete to the underlying cause or causes of that condition, selecting an appropriate repair material and method for any deficiency found, and using the selected materials and methods to repair or rehabilitate the structure. Guidance is also included on maintenance of concrete and on preparation of concrete investigation reports for repair and rehabilitation projects. Considerations for certain specialized types of rehabilitation projects are also given.

2. Applicability. This manual is applicable to all HQUSACE elements, major subordinate commands, districts, laboratories, and field operating activities having civil works responsibilities.

FOR THE COMMANDER:

JAMES D. CRAIG
Colonel, Corps of Engineers
Chief of Staff

This manual supersedes EM 1110-2-2002, dated 25 July 1986.

DEPARTMENT OF THE ARMY EM 1110-2-2002
U.S. Army Corps of Engineers
CECW-EG **Washington, DC 20314-1000**

Manual
No. 1110-2-2002 30 June 1995

Engineering and Design
EVALUATION AND REPAIR OF CONCRETE STRUCTURES

Table of Contents

Chapter 1
Introduction

1-1. Purpose

This manual provides guidance on evaluating the condition of the concrete in a structure, relating the condition of the concrete to the underlying cause or causes of that condition, selecting an appropriate repair material and method for any deficiency found, and using the selected materials and methods to repair or rehabilitate the structure. Guidance is also included on maintenance of concrete and on preparation of concrete investigation reports for repair and rehabilitation projects. Considerations for certain specialized types of rehabilitation projects are also given.

1-2. Applicability

This manual is applicable to all HQUSACE elements and USACE commands having civil works responsibilities.

1-3. References

References are listed in Appendix A. Copies of all the references listed should be maintained in their most current versions by districts and divisions having civil works responsibilities. The copies should be kept in a location easily accessible to personnel responsible for concrete condition evaluations and concrete repair projects.

1-4. Definitions and Abbreviations

Terms frequently used in this manual are defined in the Glossary (Appendix B). Also, abbreviations used in this manual are explained in Appendix C.

1-5. Methodology for Repair and Rehabilitation

This manual deals primarily with evaluation and repair of concrete structures; however, a basic understanding of underlying causes of concrete deficiencies is essential to performing meaningful evaluations and successful repairs. If the cause of a deficiency is understood, it is much more likely that the correct repair method will be selected and that, consequently, the repair will be successful. Symptoms or observations of a deficiency must be differentiated from the actual cause of the deficiency, and it is imperative that causes and not symptoms be addressed in repairs. For example, cracking is a symptom of distress that may have a variety of causes. Selection of the correct repair technique for cracking depends upon knowing whether the cracking is caused by repeated freezing and thawing of the concrete, accidental loading, or some other cause. Only after the cause or causes are known can rational decisions be made concerning the selection of a proper method of repair and in determining how to avoid a repetition of the circumstances that led to the problem. The following general procedure should be followed for evaluating the condition and correcting the deficiencies of the concrete in a structure:

a. Evaluation. The first step is to evaluate the current condition of the concrete. This evaluation may include a review of design and construction documents, a review of structural instrumentation data, a visual examination, nondestructive testing (NDT), and laboratory analysis of concrete samples. Upon completion of this evaluation step, personnel making the evaluation should have a thorough understanding of the condition of the concrete and may have insights into the causes of any deterioration noted.

b. Relating observations to causes. Once the evaluation of a structure has been completed, the visual observations and other supporting data must be related to the mechanism or mechanisms that caused the damage. Since many deficiencies are caused by more than one mechanism, a basic understanding of causes of deterioration of concrete is needed to determine the actual damage-causing mechanism for a particular structure.

c. Selecting methods and materials. Once the underlying cause of the damage observed in a structure has been determined, selection of appropriate repair materials and methods should be based on the following considerations:

(1) Prerepair adjustments or modifications required to remedy the cause, such as changing the water drainage pattern, correcting differential foundation subsidence, eliminating causes of cavitation damage, etc.

(2) Constraints such as access to the structure, the operating schedule of the structure, and the weather.

(3) Advantages and disadvantages of making permanent versus temporary repairs.

(4) Available repair materials and methods and the technical feasibility of using them.

(5) Quality of those technically feasible methods and materials to determine the most economically viable to ensure a satisfactory job.

d. Preparation of plans and specifications. The next step in the repair or rehabilitation process is preparation of project plans and specifications. When required by a major rehabilitation project, a Concrete Materials Design Memorandum, in the form of a separate report or a part of the Rehabilitation Evaluation report, should be prepared as outlined in Chapter 9. Existing guide specifications should be used to the maximum extent possible. However, many of the materials and methods described in this manual are not covered in the existing guide specifications. If the materials and methods needed for a particular repair project are not covered in the guide specifications, a detailed specification based upon the guidance given in this manual and upon experience gained from similar projects should be prepared. Since the full extent of concrete damage may not be completely known until concrete removal begins, plans and specifications for repair projects should be prepared with as much flexibility with regard to material quantities as possible. A thorough condition survey, as outlined in Chapter 2, performed as close as possible to the time repair work is executed should help minimize errors in estimated quantities.

e. Execution of the work. The success of a repair or rehabilitation project will depend upon the degree to which the work is executed in conformance with plans and specifications. There is growing evidence, based upon experience gained on a number of projects, that concrete work on repair projects requires much greater attention to good practice than may be necessary for new construction. Because of the importance of the attention to detail and the highly specialized construction techniques required for most repairs, it is important that the design engineer responsible for the investigation of the distress and selection of repair materials and methods be intimately involved in the execution of the work. For example, many repair projects require placing relatively thin overlays, either vertically or horizontally. The potential for cracking in these placements is much greater than it is during placement of concrete in new construction because of the high degree of restraint.

Chapter 2
Evaluation of the Concrete in Concrete Structures

2-1. Introduction

This chapter presents information on how to conduct an evaluation of the concrete in a concrete structure. As was described in Chapter 1, a thorough and logical evaluation of the current condition of the concrete in a structure is the first step of any repair or rehabilitation project. When the condition of a structure indicates that major repair or rehabilitation is probably necessary, a comprehensive evaluation of the structure should be conducted to determine the scope of the work required. Such an evaluation could include the following: a review of the available design and construction documentation; a review of the operation and maintenance records; a review of the instrumentation data; a visual examination of the condition of the concrete in the structure; an evaluation of the structure by nondestructive testing means; a laboratory evaluation of the condition of concrete specimens recovered from the structure; a stress analysis; and a stability analysis of the entire structure. With the exception of performing stress and stability analyses, each of these general areas is described in detail in this chapter.

2-2. Review of Engineering Data

A thorough review of all of the pertinent data relating to a structure should be accomplished early in the evaluation process. To understand the current condition of the concrete in a structure, it is imperative to consider how design, construction, operation, and maintenance have interacted over the years since the structure was designed and constructed. Sources of engineering data which can yield useful information of this nature include project design memoranda, plans and specifications, construction history reports, as-built drawings, concrete report or concrete records (including materials used, batch plant and field inspection records, and laboratory test data), instrumentation data, operation and maintenance records, and periodic inspection reports. Instrumentation data and monument survey data to detect movement of the structure should be examined.

2-3. Condition Survey

A condition survey involves visual examination of exposed concrete for the purpose of identifying and defining areas of distress. A condition survey will usually include a mapping of the various types of concrete deficiencies that may be found, such as cracking, surface problems (disintegration and spalling), and joint deterioration. Cracks are usually mapped on fold-out sketches of the monolith surfaces. Mapping must include inspection and delineating of pipe and electrical galleries, filling and emptying culverts (if possible), and other similar openings. Additionally, a condition survey will frequently include core drilling to obtain specimens for laboratory testing and analysis. Stowe and Thornton (1984), American Concrete Institute (ACI) 207.3R, and ACI 364.1R[1] provide additional information on procedures for conducting condition surveys.

a. Visual inspection. A visual inspection of the exposed concrete is the first step in an on-site examination of a structure. The purpose of such an examination is to locate and define areas of distress or deterioration. It is important that the conditions observed be described in unambiguous terms that can later be understood by others who have not inspected the concrete. Terms typically used during a visual inspection are listed by category in Table 2-1. Each of the categories of terms in the table is discussed in detail in the following subparagraphs. Additional descriptions may be found in Appendix B, ACI 116R, and ACI 201.1R.

(1) Construction faults. Typical construction faults that may be found during a visual inspection include bug holes, evidence of cold joints, exposed reinforcing steel, honeycombing, irregular surfaces caused by improperly aligned forms, and a wide variety of surface blemishes and irregularities. These faults are typically the result of poor workmanship or the failure to follow accepted good practice. Various types of construction faults are shown in Figures 2-1 through 2-4.

(2) Cracking. Cracks that occur in concrete may be described in a variety of ways. Some of the more common ways are in terms of surface appearance, depth of cracking, width of cracking, current state of activity, physical state of concrete when cracking occurred, and structural nature of the crack. Various types of cracks based on these general terms are discussed below:

(a) Surface appearance of cracks. The surface appearance of cracks can give the first indication of the cause of cracking. Pattern cracks (Figures 2-5 through 2-7) are rather short cracks, usually uniformly distributed and interconnected, that run in all directions. Pattern cracking indicates restraint of contraction of the surface layer by the backing or inner concrete or possibly an

---

[1] All ACI references are listed with detailed information in Appendix A.

Table 2-1
Terms Associated with Visual Inspection of Concrete

Construction faults	Distortion or movement
Bug holes	Buckling
Cold joints	Curling or warping
Exposed reinforcing steel	Faulting
Honeycombing	Settling
Irregular surface	Tilting
Cracking	
Checking or crazing	Erosion
D-cracking	Abrasion
Diagonal	Cavitation
Hairline	Joint-sealant failure
Longitudinal	Seepage
Map or pattern	Corrosion
Random	Discoloration or staining
Transverse	Exudation
Vertical	Efflorescence
Horizontal	Incrustation
Disintegration	Spalling
Blistering	Popouts
Chalking	Spall
Delamination	
Dusting	
Peeling	
Scaling	
Weathering	

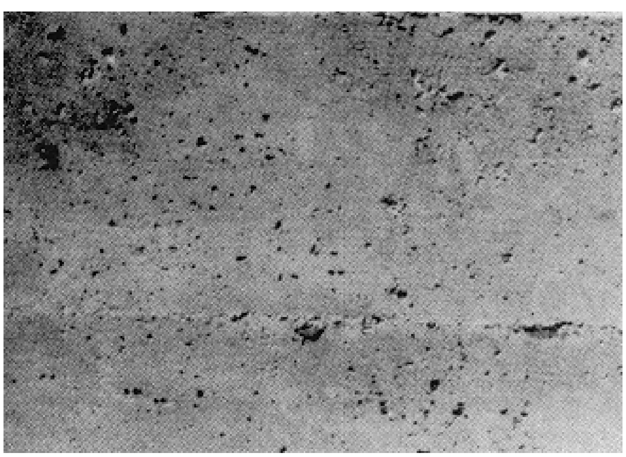


Figure 2-1. Bug holes in a vertical wall

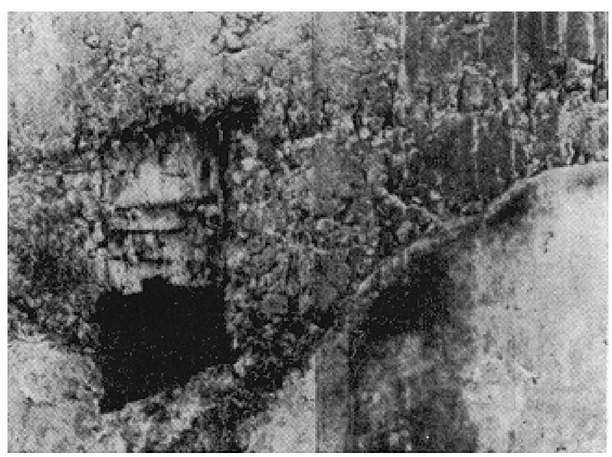


Figure 2-2. Honeycombing and cold joint

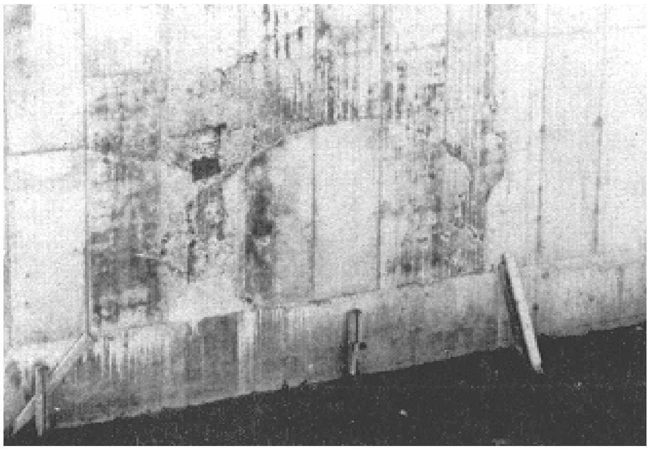


Figure 2-3. Cold joint

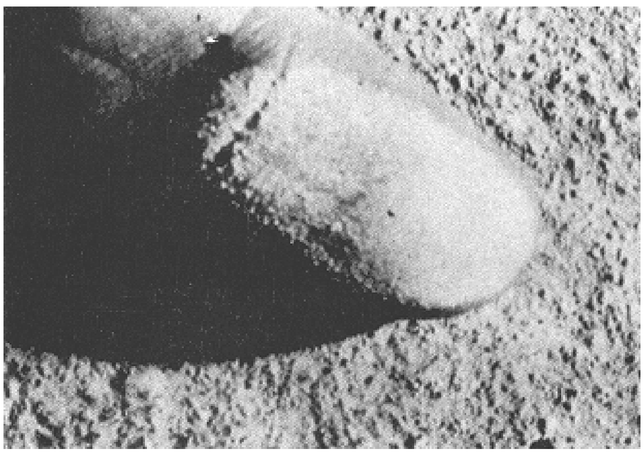


Figure 2-4. Dusting on horizontal finished surface

increase of volume in the interior of the concrete. Other terms used to describe pattern cracks are map cracks, crazing, and checking (see Glossary, Appendix B, for definitions). Another type of pattern crack is D-cracking. Figure 2-8 shows typical D-cracking in a concrete pavement. D-cracking usually starts in the lower part of a concrete slab adjacent to joints, where moisture accumulates, and progresses away from the corners of the slab. Individual cracks (Figures 2-9 through 2-11) run in definite directions and may be multiple cracks in parallel at definite intervals. Individual cracks indicate tension in the direction perpendicular to the cracking. Individual cracks are also frequently referred to as isolated cracks. Several terms may be used to describe the direction that an individual or isolated crack runs. These terms include diagonal, longitudinal, transverse, vertical, and horizontal.

(b) Depth of cracking. This category is self-explanatory. The four categories generally used to describe crack depth are surface, shallow, deep, and through.

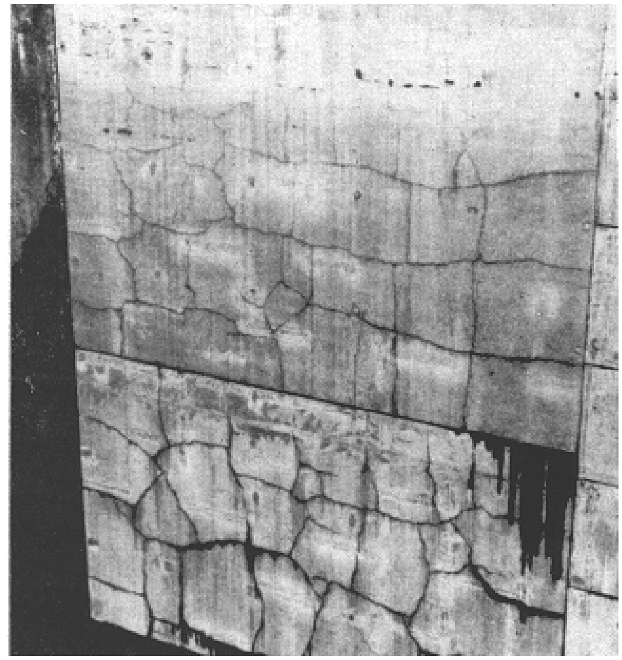


Figure 2-5. Pattern cracking caused by restrained volume changes

Figure 2-6. Pattern cracking resulting from alkali-slice reaction

(c) Width of cracking. Three width ranges are used: fine (generally less than 1 mm (0.04 in.)); medium (between 1 and 2 mm (0.04 and 0.08 in.)); and wide (over 2 mm (0.08 in.)) (ACI 201.1R).

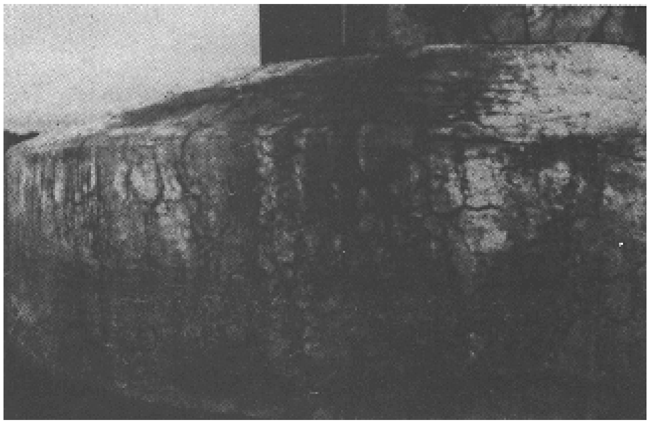


Figure 2-7. Pattern cracking caused by alkali-carbonate reaction

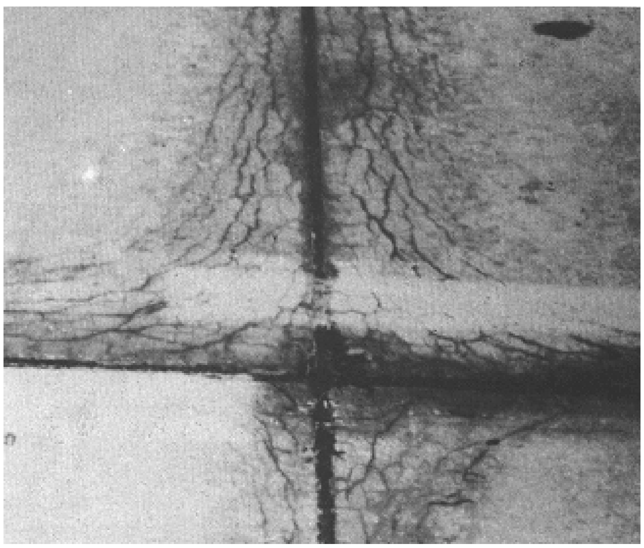


Figure 2-8. D-cracking in a concrete pavement

(d) Current state of activity. The activity of the crack refers to the presence of the factor causing the cracking. The activity must be taken into account when selecting a repair method. Two categories exist: Active cracks are those for which the mechanism causing the cracking is still at work. If the crack is currently moving, regardless of why the crack formed initially or whether the forces that caused it to form are or are not still at work, it must be considered active. Also, any crack for which an exact cause cannot be determined should be considered active. Dormant cracks are those that are not currently moving or for which the movement is of such magnitude that a repair material will not be affected by the movement.

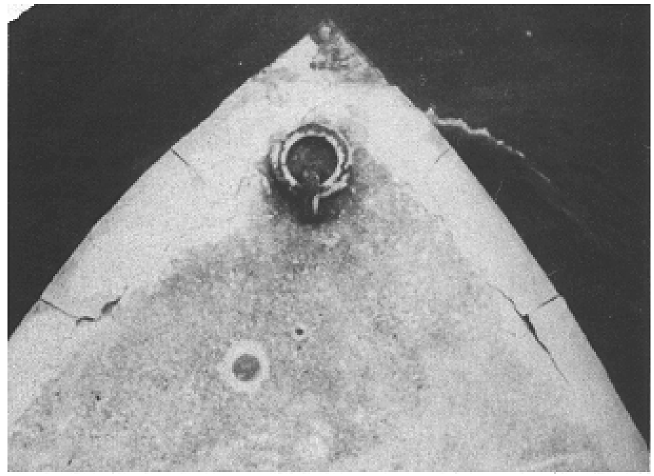


Figure 2-9. Isolated cracks as a result of restraint in the direction perpendicular to the crack

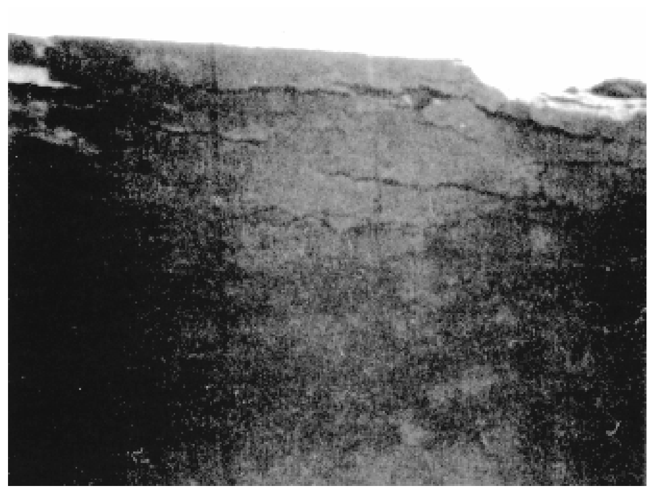


Figure 2-10. Parallel individual cracking caused by freezing and thawing

(e) Physical state of concrete when cracking occurred. Cracks may be categorized according to whether cracking occurred before or after the concrete hardened. This classification is useful to describe cracking that occurs when the concrete is fresh: for example, plastic shrinkage cracks.

(f) Structural nature of the crack. Cracks may also be categorized as structural (caused by excessive live or dead loads) and nonstructural (caused by other means). A structural crack will usually be substantial in width, and the opening may tend to increase as a result of continuous

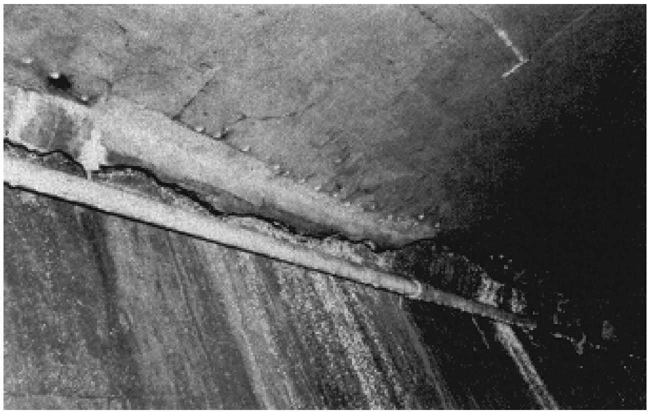


Figure 2-11. Isolated crack caused by structural overload

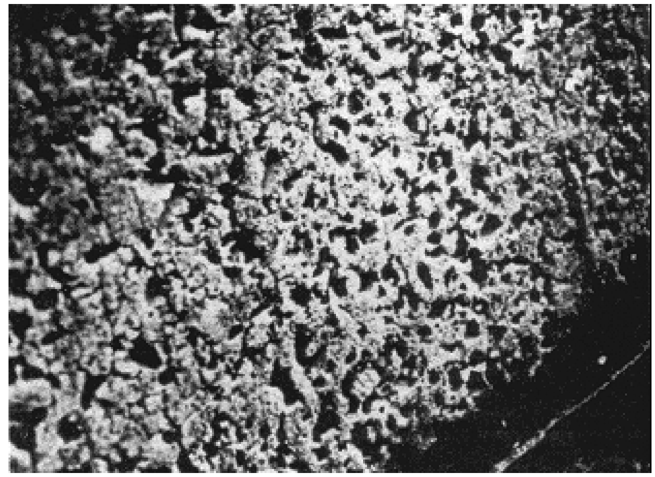


Figure 2-12. Disintegration of concrete caused by exposure to aggressive water

loading and creep of the concrete. In general, it can be difficult to determine readily during a visual examination whether a crack is structural or nonstructural. Such a determination will frequently require an analysis by a structural engineer. Any significant isolated crack that is discovered during a visual examination should be referred to a structural engineer and should be considered as possibly structural in nature.

(g) Combinations of descriptions. To describe cracking accurately, it will usually be necessary to use several terms from the various categories listed above. For example: (1) shallow, fine, dormant, pattern cracking that occurred in hardened concrete, (2) shallow, wide, dormant, isolated short cracks that occurred in fresh concrete, (3) through, active, transverse, isolated, diagonal cracks that occurred in hardened concrete.

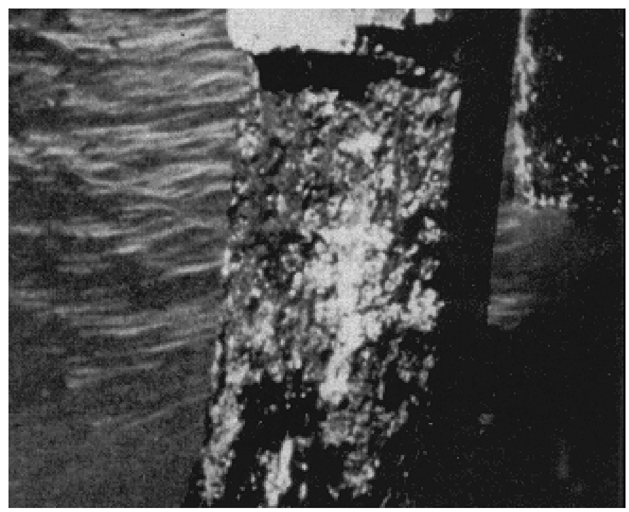


Figure 2-13. Disintegration of concrete caused by exposure to acidic water

(3) Disintegration. Disintegration of concrete may be defined as the deterioration of the concrete into small fragments or particles resulting from any cause. Disintegration may be differentiated from spalling by the mass of the particles being removed from the main body of concrete. Disintegration is usually the loss of small particles and individual aggregate particles, while spalling is typically the loss of larger pieces of intact concrete. Disintegration may be the result of a variety of causes including aggressive-water attack, freezing and thawing, chemical attack, and poor construction practices. Disintegration resulting from several different causes is shown in Figures 2-12 through 2-15. As is shown in Table 2-1, a wide variety of terms are used to describe disintegration. These terms are defined in the Glossary. Two of the most frequently used terms to describe particular types of disintegration are scaling and dusting.

(a) Scaling. Scaling is the localized flaking or peeling away of the near-surface portion of the hardened concrete or mortar. Scaling is frequently a symptom of freezing and thawing damage. Degrees of concrete scaling may be defined as follows (ACI 201.1R). Light spalling is loss of surface mortar without exposure of coarse aggregate (Figure 2-16). Medium spalling is loss of surface mortar up to 5 to 10 mm (0.2 to 0.4 in.) in depth and exposure of coarse aggregate (Figure 2-17). Severe spalling is loss of surface mortar 5 to 10 mm (0.2 to 0.4 in.) in depth with some loss of mortar surrounding aggregate particles 10 to 20 mm (0.4 to 0.8 in.) in depth, so that

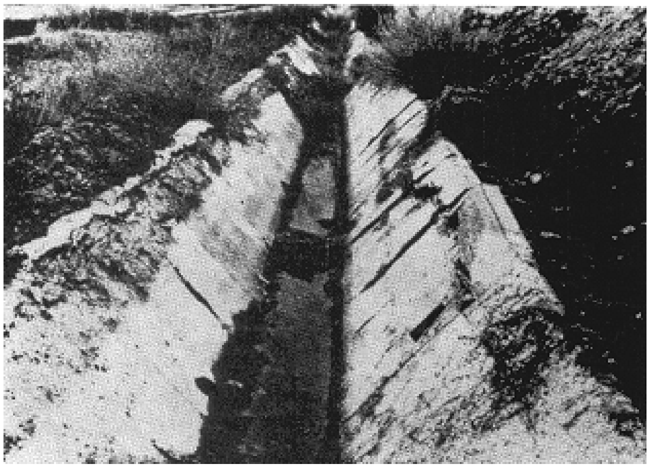


Figure 2-14. Disintegration of concrete caused by sulfate attack

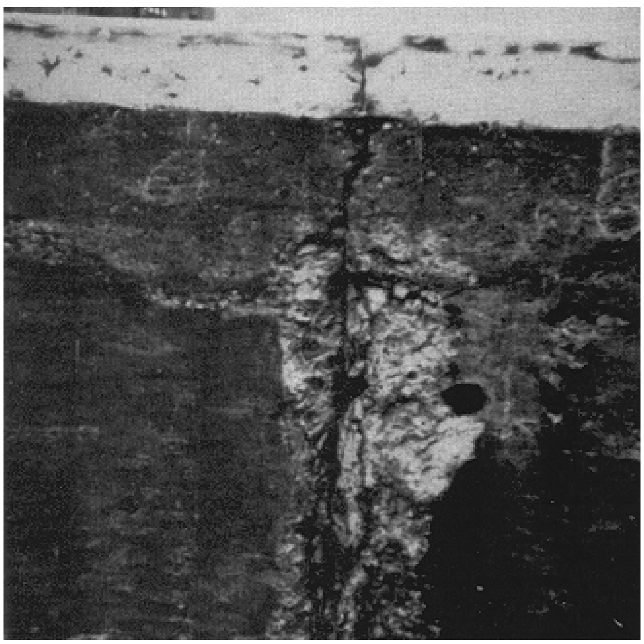


Figure 2-15. Disintegration at a monolith joint as a result of repeated cycles of freezing and thawing and barge impact

aggregate is clearly exposed and stands out from the concrete (Figure 2-18). Very severe spalling is loss of coarse aggregate particles as well as surface mortar and surrounding aggregate, generally to a depth greater than 20 mm (0.8 in.) (Figure 2-19).

(b) Dusting. Dusting is the development of a powdered material at the surface of hardened concrete. Dusting will usually be noted on horizontal concrete surfaces

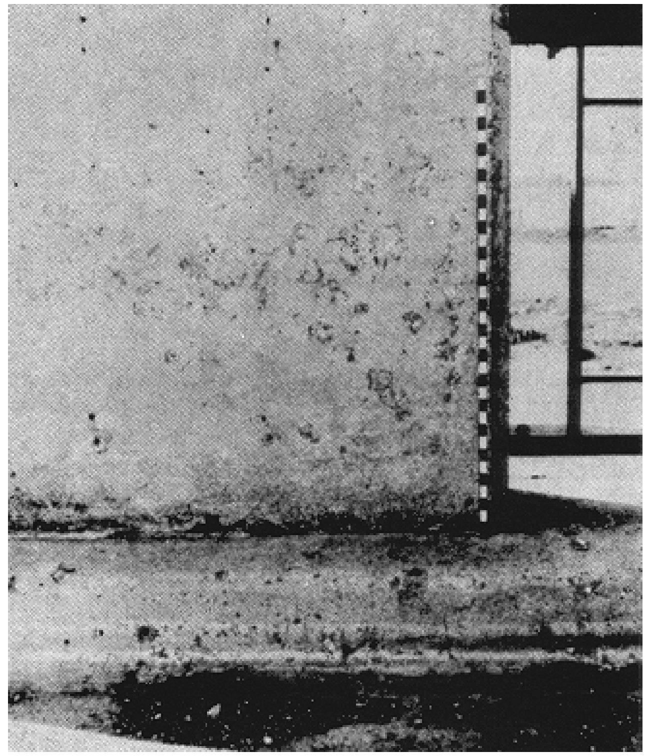


Figure 2-16. Light scaling

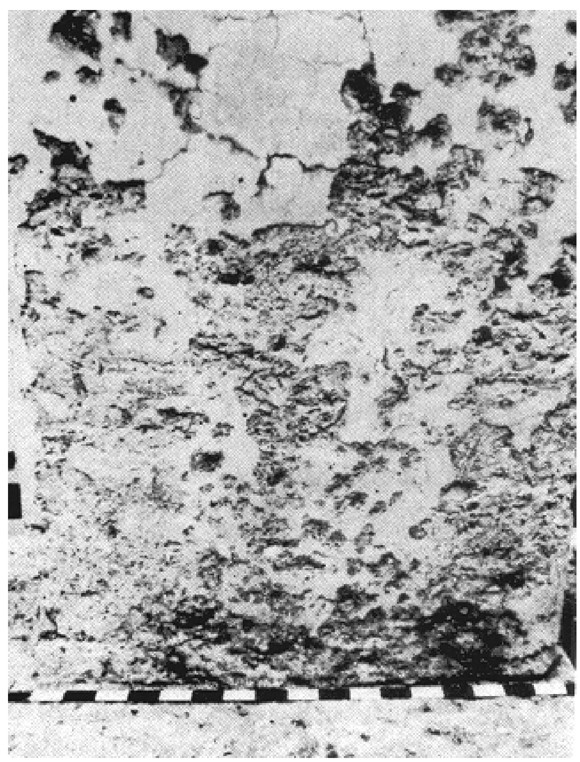


Figure 2-17. Medium scaling

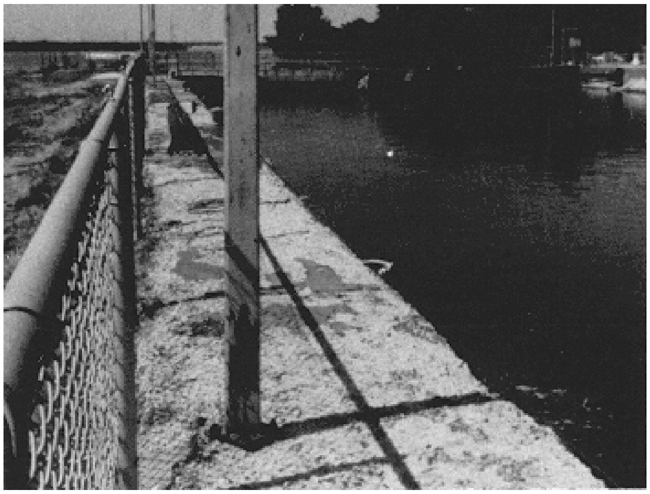


Figure 2-18. Severe scaling

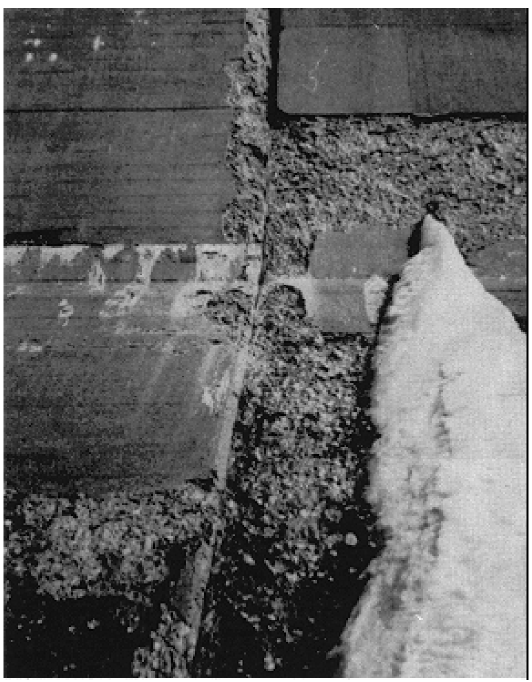


Figure 2-19. Very severe scaling

that receive a great deal of traffic. Typically, dusting is a result of poor construction practice. For example, sprinkling water on a concrete surface during finishing will frequently result in dusting.

(4) Distortion or movement. Distortion or movement, as the terms imply, is simply a change in alignment of the components of a structure. Typical examples would be differential movement between adjacent monoliths or the shifting of supported members on their supports. Review of historical data such as periodic inspection reports may be helpful in determining when movement first occurred and the apparent rate of movement.

(5) Erosion. Erosion of concrete may be categorized as one of two general types, each of which has a distinct appearance.

(a) Abrasion. Abrasion-erosion damage is caused by repeated rubbing and grinding of debris or equipment on a concrete surface. In hydraulic structures such as stilling basins, abrasion-erosion results from the effects of waterborne gravel, rock, or other debris being circulated over a concrete surface during construction or routine operation. Abrasion-erosion of this type is readily recognized by the smooth, well-worn appearance of the concrete (Figure 2-20).

(b) Cavitation. Cavitation-erosion damage is caused by repeated impact forces caused by collapse of vapor bubbles in rapidly flowing water. The appearance of concrete damaged by cavitation-erosion is generally different from that damaged by abrasion-erosion. Instead of a smooth, worn appearance, the concrete will appear very rough and pitted (Figure 2-21). In severe cases, cavitation-erosion may remove large quantities of concrete and may endanger the structure. Usually, cavitation-erosion occurs as a result of water velocities greater than 12.2 m/sec (40 ft/sec).

(6) Joint sealant failure. Joint sealant materials are used to keep water out of joints and to prevent debris from entering joints and making them ineffective as the concrete expands. Typical failures will be seen as

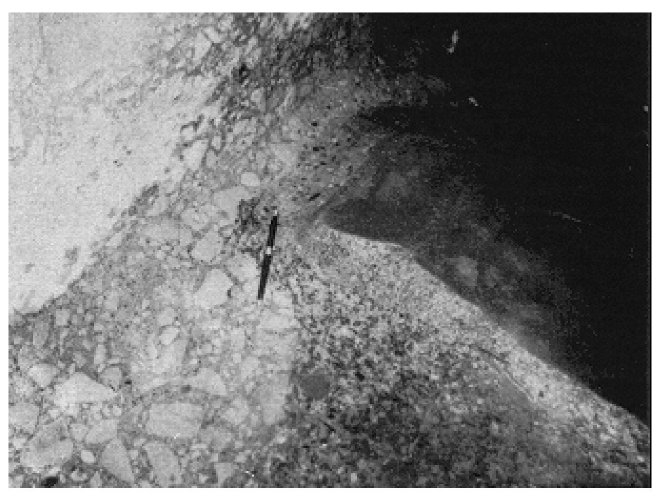


Figure 2-20. Smooth, worn, abraded concrete surface caused by abrasion of waterborne debris

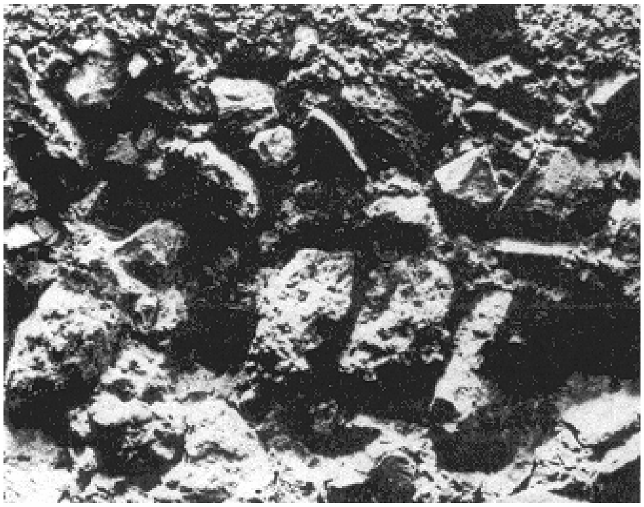


Figure 2-21. Rough, pitted concrete surface caused by cavitation

detachment of the sealant material from one or both sides of the joint or complete loss of the sealant material (Figures 2-22 and 2-23).

(7) Seepage. Seepage is defined in ACI 207.3R as "the movement of water or other fluids through pores or interstices." As shown in Table 2-1, the visual evidence of seepage could include, in addition to the presence of water or moisture, evidence of corrosion, discoloration, staining, exudations, efflorescence, and incrustations (Figures 2-24 through 2-28). (For definitions of these terms, see the Glossary, Appendix B). Although occurrences of this nature are quite common around hydraulic structures, they should be included in reports of visual inspections because the underlying cause may be significant. Seepage is another case in which review of historical data may be of benefit to determine whether rates are changing.

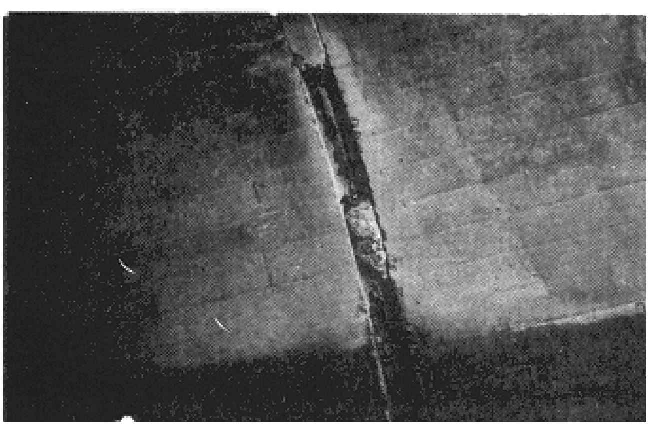


Figure 2-22. Deterioration of joint sealant

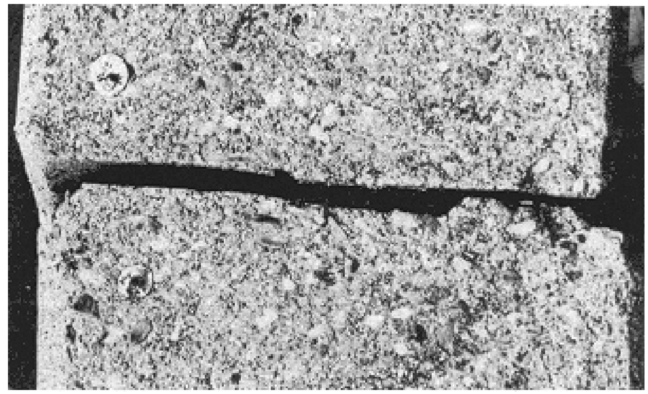


Figure 2-23. Loss of joint sealant

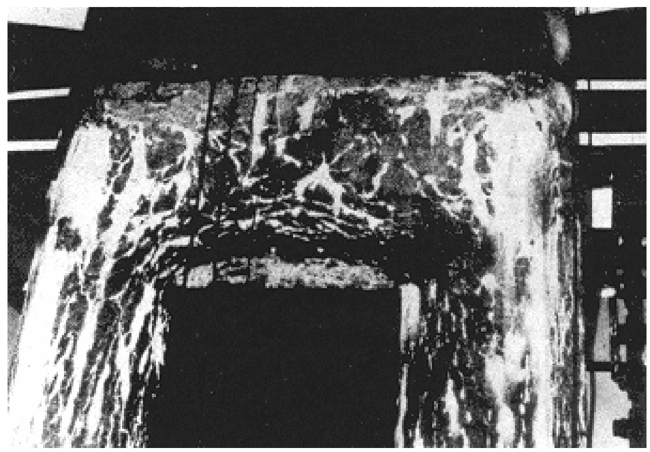


Figure 2-24. Efflorescence

(8) Spalling. Spalling is defined as the development of fragments, usually in the shape of flakes, detached from a larger mass. As noted in paragraph 2-3*a*(3), spalling differs from disintegration in that the material being lost from the mass is concrete and not individual aggregate particles that are lost as the binding matrix disintegrates. The distinction between these two symptoms is important in any attempt to relate symptoms to causes of concrete problems. Spalls can be categorized as follows:

(a) Small spall. Not greater than 20 mm (0.8 in.) in depth nor greater than 150 mm (6 in.) in any dimension (Figure 2-29).

Figure 2-25. Exudation

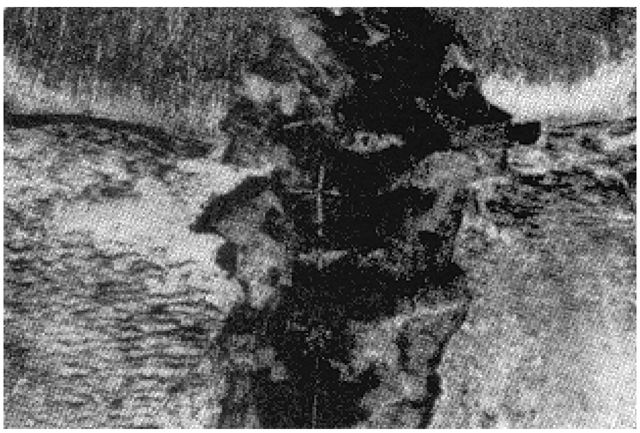


Figure 2-26. Incrustation

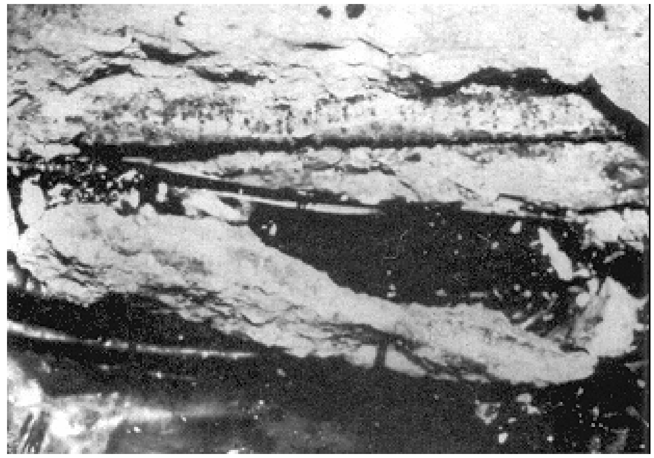


Figure 2-27. Corrosion

Figure 2-28. Water seepage through joint

(b) Large spall. Deeper than 20 mm (0.8 in.) and greater than 150 mm (6 in.) in any dimension (Figure 2-30).

(9) Special cases of spalling. Two special cases of spalling must be noted:

(a) Popouts. Popouts appear as shallow, typically conical depressions in a concrete surface (Figure 2-31). Popouts may be the result of freezing of concrete that contains some unsatisfactory aggregate particles. Instead of general disintegration, popouts are formed as the water in saturated coarse aggregate particles near the surface freezes, expands, and pushes off the top of the aggregate particle and the superjacent layer of mortar, leaving shallow pits. Chert particles of low specific gravity, limestone containing clay, and shaly materials are well known for this behavior. Popouts are easily recognizable by the shape of the pit remaining in the surface and by a portion

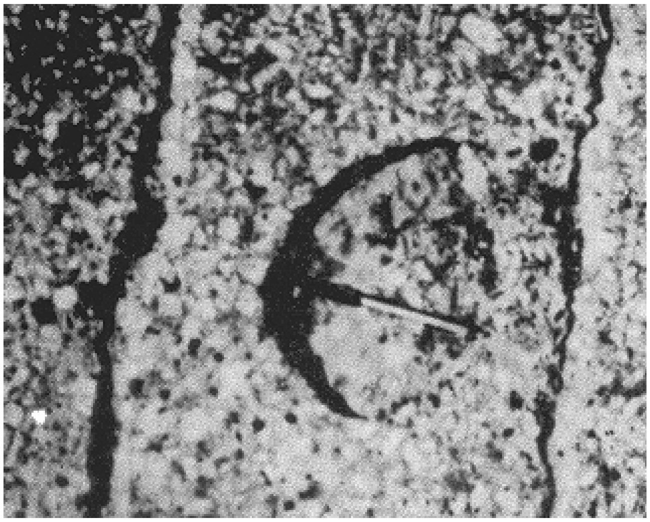


Figure 2-29. Small spall

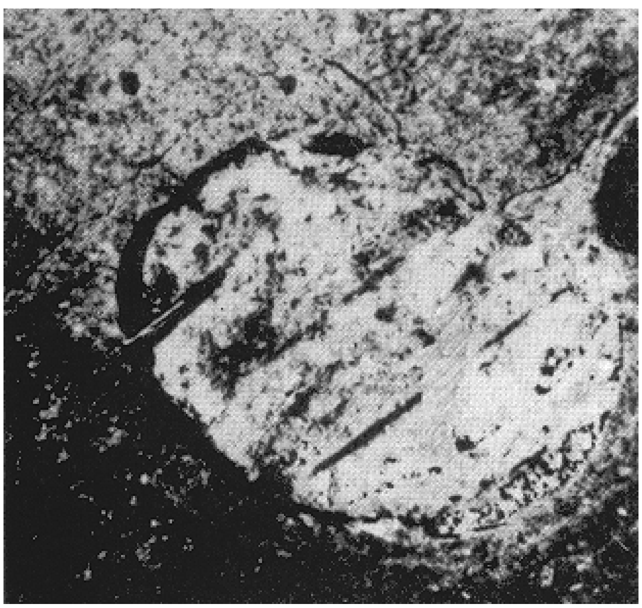


Figure 2-30. Large spall

of the offending aggregate particle usually being visible in the hole (Bach and Isen 1968).

(b) Spalling caused by the corrosion of reinforcement. One of the most frequent causes of spalling is the corrosion of reinforcing steel or other noncorrosion-resistant embedded metal in concrete. During a visual examination of a structure, spalling caused by corrosion of reinforcement is usually an easy symptom to recognize since the corroded metal will be visible along with rust staining,

Figure 2-31. Popout

and the diagnosis will be straightforward. Section 2-3*a*(10) discusses locating the delamination that occurs before the corrosion progresses to the point that the concrete spalls.

(10) Delamination. Reinforcing steel placed too near the surface or reinforcing steel exposed to chloride ions will corrode. The iron combines with the oxygen in water or air forming rust, or iron oxide, and a corresponding increase in volume up to eight times the original volume. The volume increase results in cracking over the reinforcing steel, followed by delamination along the mat of steel and eventually by spalling. This corrosion sometimes become evident early in the disruptive process when a rectangular pattern of cracking on the concrete surface can be traced to the presence of a reinforcing bar under each crack. Sounding of concrete with a hammer provides a low-cost, accurate method for identifying delaminated areas. Delaminated concrete sounds like a hollow “puck” rather than the “ping” of sound concrete. Boundaries of delaminations can easily be determined by sounding areas surrounding the first “puck” until “pings” are heard.

(a) Hammer-sounding of large areas generally proves to be extremely time consuming. More productive methods are available for sounding horizontal surfaces. Chain dragging accomplishes the same result as hammer-sounding. As the chain is dragged across a concrete surface, a distinctly different sound is heard when it crosses over a delaminated area.

(b) Infrared thermography is a useful method of detecting delaminations in bridge checks. This method is also used for other concrete components exposed to direct sunlight. The method works on the principle that as concrete heats and cools there is substantial thermal gradient within the concrete. Delaminations and other discontinuities interrupt the heat transfer through the concrete. These defects cause a higher surface temperature than that of the surrounding concrete during periods of heating, and a lower surface temperature than that of the surrounding concrete during periods of cooling. The equipment can record and identify areas of delaminations below the surface.

b. Cracking survey. A crack survey is an examination of a concrete structure for the purpose of locating, marking, and identifying cracks and determining the relationship of the cracks with other destructive phenomena (ACI 207.3R). 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. The first step in making a crack survey is to locate and mark the cracking and define it by type. The terms for and descriptions of cracks given in Section 2-3 should be used to describe any cracking that is found.

(1) Crack widths can be estimated using a clear comparator card having lines of specified width marked on the card. Crack widths can be measured to an accuracy of about 0.025 mm (0.001 in.) with a crack comparator, a hand-held microscope with a scale on the lens closest to the surface being viewed (Figure 2-32). Crack movement can be monitored with a crack measuring device. The crack monitor shown in Figure 2-33 gives a direct reading of crack displacement and rotation. It is important to make an initial reading when the monitor is attached because the monitor will not necessarily read zero after installation. If more accurate and detailed time histories are desired, a wide range of transducers and devices are available (EM 1110-2-4300).

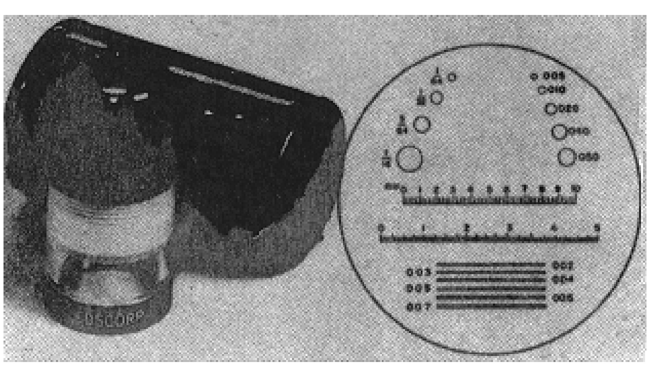


Figure 2-32. Comparator for measuring crack widths

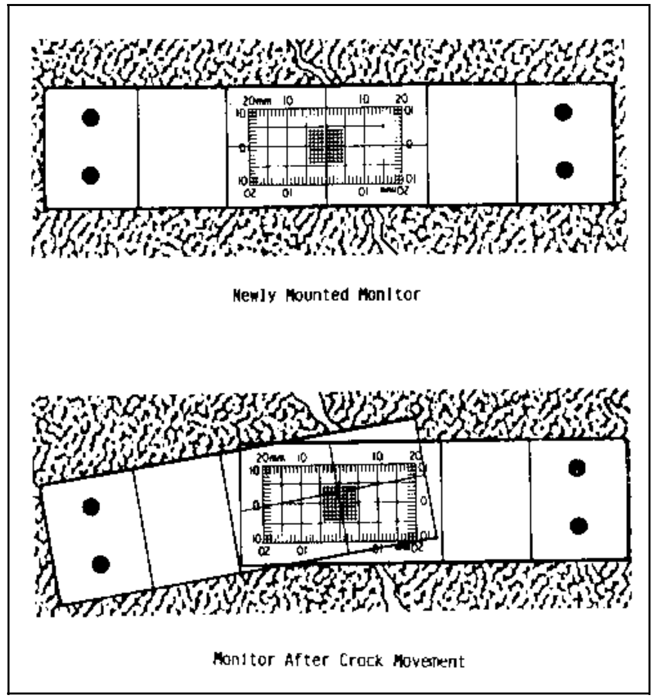


Figure 2-33. Crack monitor

(2) If possible, the crack depth should be determined by observation of edges or insertion of a fine wire or feeler gauge; however, in most situations, the actual depth may be indeterminable without drilling or using other detection techniques such as the pulse-velocity method described in Section 2-6*c*.

(3) Conditions which may be associated with cracking either over portions of the length or for the entire length should be noted. These conditions may include seepage through the cracks, deposits from leaching or other sources, spalling of edges, differential movement (offsets), etc. Chemical analyses of the seepage water and the deposits may be desirable.

(4) It may be worthwhile to repeat the survey under various loading conditions when change in crack width is suspected. Furthermore, tapping of surfaces with a hammer may detect shallow cracking beneath and parallel to the surface. A hollow sound generally indicates that such cracking is likely even though it cannot be seen. See Section 2-3*a*(10) for additional discussion on sounding to detect delamination.

c. Surface mapping.

(1) Surface mapping is a parallel procedure to a cracking survey in which deterioration of the surface concrete is located and described. Surface mapping may be accomplished by use of detailed drawings, photographs, movies, or video tapes. Items most often identified and mapped include: cracking, spalling, scaling, popouts, honeycombing, exudation, distortion, unusual discoloration, erosion, cavitation, seepage, conditions of joints and joint materials, corrosion of reinforcement (if exposed), and soundness of surface concrete. A list of items recommended for use in a surface mapping by hand is as follows (ACI 207.3R):

(a) Structure drawings, if available.

(b) Clipboard and paper or field book.

(c) Tape measure, 15 to 30 m (50 to 100 ft).

(d) Ruler graduated in 1/16 in. or 1 mm.

(e) Feeler gauge.

(f) Pocket comparator or hand microscope.

(g) Knife.

(h) Hammer, 1 kg (2 lb).

(i) Fine wire (not too flexible).

(j) String.

(k) Flashlight or lantern.

(l) Camera with flash and assortment of lenses.

(m) Assortment of film, color and high speed.

(2) Mapping should begin at one end of the structure and proceed in a systematic manner until all surfaces are mapped. Both external and internal surfaces should be mapped if access is possible. Use of three-dimensional (3-D) isometric drawings showing offsets or distortion of structural features is occasionally desirable. Areas of significant distress should be photographed for later reference. A familiar object or scale should be placed in the area to show the relative size of the feature being photographed. It is important to describe each condition mapped in clear, concise detail and to avoid generalizations unless reference is being made to conditions previously detailed in other areas. Profiles are advantageous for showing the depth of erosion.

d. Joint survey. A joint survey is a visual inspection of the joints in a structure to determine their condition. Expansion, contraction, and construction joints should be located and 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; for example, spalling or D-cracking, chemical attack, evidence of seepage, emission of solids, etc. Conditions of joint filler, if present, should be examined.

e. Core drilling. Core drilling to recover concrete for laboratory analysis or testing is the best method of obtaining information on the condition of concrete within a structure. However, since core drilling is expensive, it should only be considered when sampling and testing of interior concrete is deemed necessary.

(1) The presence of abnormal conditions of the concrete at exposed surfaces may suggest questionable quality or a change in the physical or chemical properties of the concrete. These conditions may include scaling, leaching, and pattern cracking. When such observations are made, core drilling to examine and sample the hardened concrete may be necessary.

(2) 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 Concrete Research Division (CRD)-C 26[1] and American Society for Testing and Materials (ASTM) C 823[2]. The core 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 not be less than three times the nominal maximum size of aggregate. For 150-mm (6-in.) maximum size aggregate concrete, 200- or 250-mm (8- or 10-in.)-diam cores are generally drilled because of cost, handling, and laboratory testing machine capabilities. Warning should be given against taking NX size 54-mm

---

[1] All CRD-C designations are from U.S. Army Engineer Waterways Experiment Station (USAEWES). 1949 (Aug). *Handbook for Concrete and Cement*, with quarterly supplements, Vicksburg, MS.

[2] All ASTM test methods cited are from the *Annual Book of ASTM Standards* (ASTM Annual).

(2-1/8-in.)-diam cores in concrete. When 50- to 150-mm (2- to 6-in.) maximum size aggregate concrete is cored, an NX size core will generally be recovered in short pieces or broken core. The reason for breakage is that there is simply little mortar bonding the concrete 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.

(3) Core samples must be properly identified and oriented with permanent markings on the material itself when feasible. Location of borings must be accurately described and marked 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 of fractured surfaces, unusual deposits, coloring or staining, distribution and size of voids, locations of observed construction joints, and contact with the foundation or other surface (ACI 207.3R). 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. Figure 2-34 illustrates a typical log for a concrete core recovered during a condition survey.

(4) When drill hole coring is not practical or core recovery is poor, a viewing system such as a borehole camera, bore hole television, or borehole televiewer may be used for evaluating the interior concrete conditions. A description and information on the availability of these borehole viewing systems can be found in EP 1110-1-10. Evaluation of distress in massive concrete structures may be desirable to determine in situ stress conditions. ACI 207.3R is an excellent guide to determining existing stress conditions in the structure.

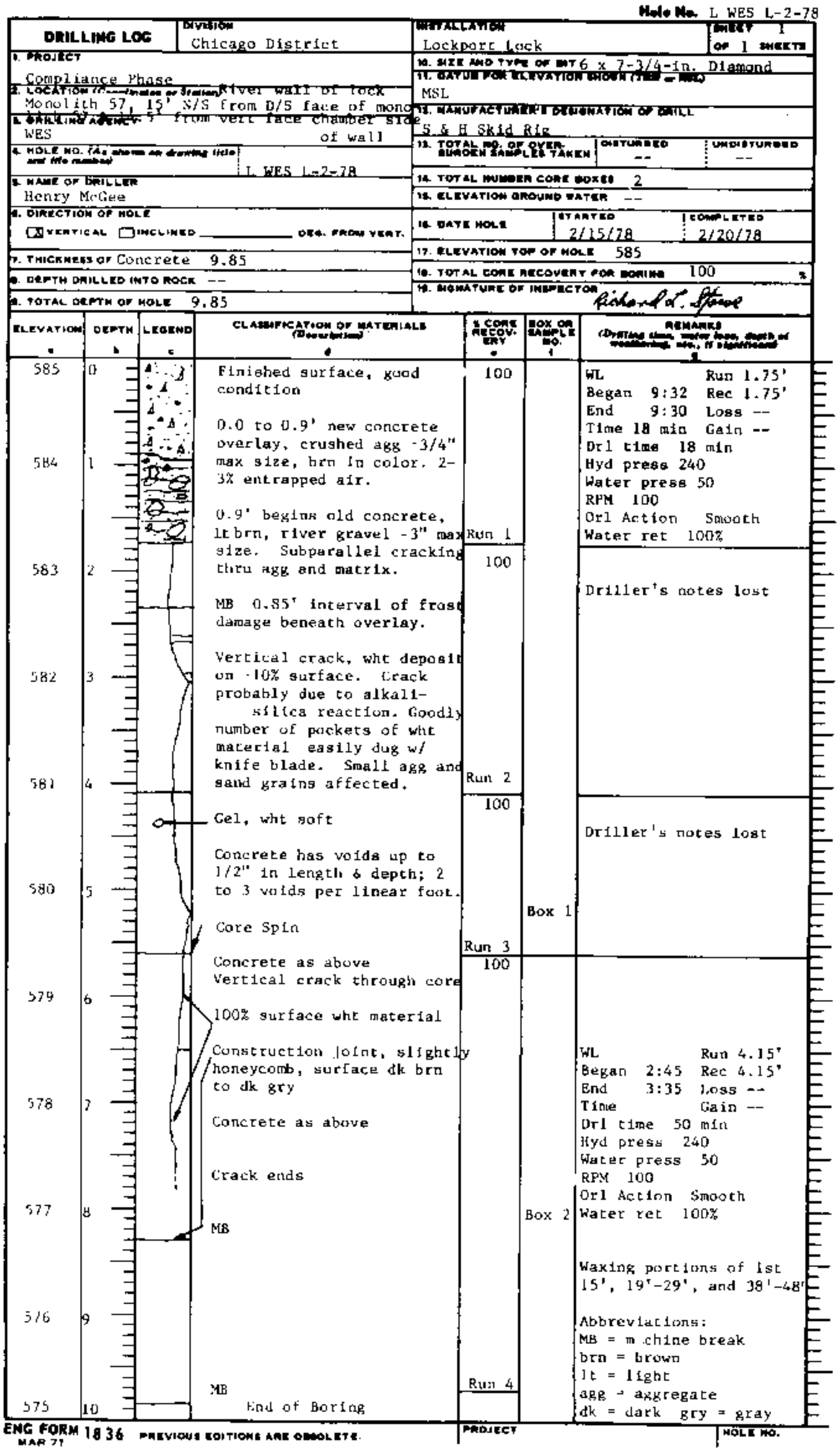
2-4. Underwater Inspection

A variety of procedures and equipment for conducting underwater surveys are available (Popovics and McDonald 1989). Included are several nondestructive techniques which can be used in dark or turbid conditions that preclude visual inspection. Some techniques originally developed for other purposes have been adapted for application in underwater inspections. Prior to an underwater survey, it is sometimes necessary for the surface of the structure to be cleaned. A number of procedures and devices for underwater cleaning of civil works structures are described by Keeney (1987).

a. Visual inspection by divers. Underwater surveys by divers are usually either scuba or surface-supplied diving operations. Basic scuba diving equipment is an oxygen tank, typically weighing about 34 kg (75 lb) which is carried by the diver. Surface-supplied diving, where the air supply is provided from the surface or shore, is a more elaborate operation in terms of equipment, safety concerns, diver skills, etc., especially when the diver approaches maximum allowable depths. Diver equipment for surface-supplied diving includes air compressors, helmets, weighted shoes, air supply lines, breastplates, etc., which can weigh as much as 90 kg (200 lb). The free-swimming scuba diver has more flexibility and maneuverability than the surface-supplied diver. However, he cannot dive as deep or stay underwater as long as a surface-supplied diver.

(1) Advantages. Underwater inspections performed by divers offer a number of advantages: they are (a) applicable to a wide variety of structures; (b) flexible inspection procedures; (c) simple (especially the scuba diver in shallow-water applications); and in most cases, (d) relatively inexpensive. Also, a variety of commercially available instruments for testing concrete above water have been modified for underwater use by divers. These instruments include a rebound hammer to provide data on concrete surface hardness, a magnetic reinforcing steel locator to locate and measure the amount of concrete cover over the reinforcement, and direct and indirect ultrasonic pulse-velocity systems which can be used to determine the general condition of concrete based on sound velocity measurements (Smith 1987).

(2) Limitations. Limitations on diver inspections include the regulations (Engineer Manual 385-1-1) that restrict the allowable depths and durations of dives and the number of repeat dives in a given period. Also, in turbid water a diver’s visibility may be reduced to only a few inches, or in extreme cases, a diver may be limited to a tactile inspection. Also, cold climates tend to reduce the diver’s ability to perform at normal levels. In any case, a diver’s visual, auditory, tactile, and spatial perceptions are different underwater from what they are in air. Therefore, he is susceptible to making errors in observations and recording data.



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' S/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. 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.	100		Driller's notes lost
582	3					
581	4			Run 2		
			Gel, wht soft Concrete has voids up to 1/2" in length & depth; 2 to 3 voids per linear foot. Core Spin	100		Driller's notes lost
580	5			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		
578	7		Concrete as above Crack ends			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%
577	8		MB		Box 2	Waxing portions of 1st 15', 19'-29', and 38'-48'
576	9					Abbreviations: MB = machine 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 2-34. Typical information included on a drill log for concrete core *b. Manned and unmanned underwater vehicles.*

(1) Underwater vehicles can be thought of as platformed, underwater camera systems with manipulator and propulsion systems. They consist of a video unit, a power source for propulsion, vehicle controllers (referred to as "joysticks"), and display monitor. Available accessories which allow the vehicles to be more functional include angle lens, lighting components, instrumentations for analyses, attachments for grasping, and a variety of other capabilities.

(2) There are five categories of manned underwater vehicles: (a) untethered, (b) tethered, (c) diver lockout, (d) observation/work bells, and (e) atmospheric diving suits. All are operated by a person inside, have viewports, are dry inside the pressure hull(s), and have some degree of mobility.

(3) There are six types of unmanned underwater vehicles: (a) tethered, free swimming, (b) towed, midwater, (c) towed, bottom-reliant, (d) bottom-crawling, (e) structurally reliant, and (f) untethered (Busby Associates, Inc. 1987). These remotely operated vehicles (ROV's) are primarily distinguished by their power source. All include a TV camera to provide real-time or slow-scan viewing, and all have some degree of mobility. They are controlled from the surface via operator-observed video systems. Joysticks are used to control propulsion and manipulation of the ROV and accessory equipment. Exceptions are the untethered types of ROV's which are self-propelled and operated without any connection to the surface. Most ROV's are capable of accommodating various attachments for grasping, cleaning, and performing other inspection chores. Specially designed ROV's can accommodate and operate nondestructive testing equipment.

(4) Underwater vehicles can compensate for the limitations inherent in diver systems because they can function at extreme depths, remain underwater for long durations, and repeatedly perform the same mission without sacrifice in quality. Also, they can be operated in environments where water temperatures, currents, and tidal conditions preclude the use of divers.

(5) Manned underwater vehicles are usually large and bulky systems which require significant operational support. Therefore, they are used less frequently than the smaller unmanned ROV's. Although the dependability of ROV's has steadily increased, some limitations remain. Most ROV systems provide two-dimensional (2-D) views only and, therefore, may not project the full extent of any defects. Murky water limits the effectiveness of ROV systems. In some situations, it may be difficult to determine the exact orientation or position of the ROV, thus impeding accurate identification of an area being observed (U.S. Dept. of Transportation 1989). Also, ROV's do not possess the maneuverability offered by divers. As a result, controlling the ROV in "tight" areas and in swift currents is difficult and can result in entanglement of the umbilical (REMR Technical Note CS-ES-2.6 (USAEWES 1985a)).

(6) Underwater vehicles are being increasingly accepted as a viable means to effectively perform underwater surveys in practically all instances where traditional diver systems are normally used. Manned underwater vehicles have been used in the inspection of stilling basins, in direct support of divers, and in support of personnel maintaining and repairing wellheads. Applications of ROV's include inspection of dams, breakwaters, jetties, concrete platforms, pipelines, sewers, mine shafts, ship hulls, etc. (Busby Associates, Inc. 1987). They have also been used in leak detection and structure cleaning.

c. Photography systems.

(1) Photography systems used in underwater inspection include still-photography equipment, video recording systems, video imaging systems, and any accessories.

(2) Still-photographic equipment includes cameras, film, and lighting. Most above-water cameras ranging from the "instamatic" type to sophisticated 35-mm cameras can be used underwater in waterproof cases (U.S. Dept. of Transportation 1989). There are also waterproof 35-mm cameras designed specifically for underwater photography (REMR Technical Note CS-ES-3.2 (USAEWES 1985b)). These cameras usually include specially equipped lens and electronic flashes to compensate for the underwater environment. Most film, color and black and white, can be used in underwater photography if ample lighting is provided. High-speed film that compensates for inherent difficulties in underwater photography is available.

(3) Underwater video equipment has improved dramatically in recent years (REMR Technical Note CS-ES-2.6 (USAEWES 1985a)). Video cameras can be used with an umbilical cable to the surface for real-time viewing on a monitor or for recording. Compact camera-recorder systems in waterproof housings can be used with or without the umbilical to the surface. These video systems can be configured to provide on-screen titles and

clock, as well as narration by a diver and surface observer.

(4) Video systems can provide pictorial representations of existing conditions, transmit visual data to topside personnel for analysis and interpretation, and provide a permanent record of the inspection process. Visual recordings can be used to monitor the performance of a structure with time. Additionally, video systems can penetrate turbid areas where the human eye cannot see. Video systems are typically used concurrently with divers and underwater vehicles.

d. High-resolution acoustic mapping system.

(1) Erosion and faulting of submerged surfaces have always been difficult to accurately map. To see into depressions and close to vertical surfaces requires a narrow beam. Also, there is a need to record exactly where a mapping system is located at any instant so that defects may be precisely located and continuity maintained in repeat surveys. These capabilities are provided by the high-resolution acoustic mapping system developed through a joint research and development effort between the U.S. Army Corps of Engineers and the U.S. Bureau of Reclamation (Thornton 1985 and Thornton and Alexander 1987).

(2) The system can be broken into three main components: the acoustic subsystem, a positioning subsystem, and a compute-and-record subsystem. The acoustic subsystem consists of a boat-mounted transducer array and the signal processing electronics. During a survey, each transducer generates acoustic signals which are reflected from the bottom surface and received at the transducer array. The time of flight for the acoustic signal from the transducer to the bottom surface and back is output to a computer. The computer calculates the elevation of the bottom surface from this information, and the basic data are recorded on magnetic disks.

(3) The lateral positioning subsystem consists of a sonic transmitter on the boat and two or more transponders in the water at known or surveyed locations. As each transponder receives the sonic pulse from the transmitter, it radios the time of detection of the survey boat. The position of the boat is calculated from this information and displayed by an onboard computer. The network can be easily reestablished, making it possible to return the survey boat and transducer array bar to a specific location.

(4) The compute-and-record subsystem provides for computer-controlled operation of the system and for processing, display, and storage of data. Survey results are in the form of real-time strip charts showing the absolute relief for each run, 3-D surface relief plots showing composite data from all the survey runs in a given area, contour maps selected areas, and printouts of the individual data points.

(5) The high-resolution acoustic mapping system is designed to operate in water depths of 1.5 m to 12 m (5 to 40 ft) and produce accuracies of ± 50 mm (2 in.) vertically and ± 0.3 m (1 ft) laterally. The major limitation of the system is that it can be used only in relatively calm water. Wave action causing a roll angle of more than 5 deg will automatically shut down the system.

(6) To date, the primary application of this system has been in rapid and accurate surveying of erosion damage in stilling basins. The system has been successfully used at a number of BuRec and Corps of Engineers (CE) dams including Folsom, Pine Flat, Ice Harbor, Locks and Dams 25 and 26 (Miss. River), Lookout Point, and Dexter.

e. Side-scan sonar.

(1) The side-scan sonar, which evolved from the echo sounding depth finders developed during World War II, basically consists of a pair of transducers mounted in a waterproof housing referred to as a "fish," a graphic chart-recorder set up for signal transmission and processing, and tow cable which connects the "fish" and recorder. The system directs sound waves at a target surface. The reflected signals are received by the transducers and transmitted to the chart-recorder as plotted images. The recorded image, called a sonograph, is characterized by various shades of darkened areas, or shadows, on the chart. Characteristics of the reflecting surface are indicated by the intensity of the reflected signals. Steel will reflect a more intense signal and produce a darker shaded area than wood, and gravel will reflect a more intense signal than sand. Acoustic shadows, shades of white, are projected directly behind the reflecting surface. The width of these shadows and the position of the object relative to the towfish are used to calculate the height of the object (Morang 1987).

(2) Electronic advances in the side-scan sonar have broadened its potential applications to include underwater surveying. In the normal position, the system looks at

vertical surfaces. However, it can be configured to look downward at horizontal surfaces in a manner similar to that of the high-resolution acoustic mapping system. The side-scan sonar is known for its photograph-like image. Current commercial side-scan sonar systems are available with microprocessors and advanced electronic features (built in or as accessory components) to print sonographs corrected for slant-range and true bottom distances (Clausner and Pope 1988).

(3) Side-scan sonar has proven useful in surveys of breakwaters, jetties, groins, port structures, and inland waterway facilities such as lock and dams. It has proven especially effective in examing the toe portion of rubble structures for scour and displacement of armor units (Kucharski and Clausner 1990). The ability of sonar to penetrate waters too turbid or dangerous for visual or optical inspection makes it the only effective means of inspecting many coastal structures.

f. Radar.

(1) Radar and acoustics work in a similar manner, except radar uses an electromagnetic signal which travels very fast compared to the relatively slow mechanical wave used in acoustics. In both cases, the time of arrival (TOA) is measured and a predetermined calibration velocity is used to calculate the depth of the reflecting interface. The two main factors that influence radar signals are electrical conductivity and dielectric constant of the material (Alongi, Cantor, Kneeter, and Alongi 1982 and Morey 1974). The conductivity controls the loss of energy and, therefore, the penetration depth. The dielectric constant determines the propagation velocity.

(2) The resistivity (reciprocal of conductivity) of concrete structures varies considerably in the dry, and the presence of water further complicates the measurement. Therefore, those who have a need for this type of underwater survey should contact one or more of the sources referenced for assistance in determining the proper measurement system for a given application.

g. Ultrasonic pulse velocity.

(1) Ultrasonic pulse velocity provides a nondestructive method for evaluating structures by measuring the time of travel of acoustic pulses of energy through a material of known thickness (Thornton and Alexander 1987). Piezoelectric transducers, housed in metal casings and excited by high-impulse voltages, transmit and receive the acoustic pulses. An oscilloscope configured in the system measures time and displays the acoustic waves. Dividing the length of the travel path by the travel time yields the pulse velocity, which is proportioned to the dynamic modulus of elasticity of the material. Velocity measurements through materials of good quality usually result in high velocities and signal strengths, while materials of poor quality usually exhibit decreased velocities and weak signals. For example, good quality, continuous concrete produces velocities in the range of 3,700 to 4,600 mps (12,000 to 15,000 fps); poor quality or deteriorated concrete, 2,400 to 3,000 mps (8,000 to 10,000 fps).

(2) The pulse-velocity method has provided reliable in situ delineations of the extent and severity of cracks, areas of deterioration, and general assessments of the condition of concrete structures for many years. The equipment can penetrate approximately 91 m (300 ft) of continuous concrete with the aid of amplifiers, is easily portable, and has a high data acquisition-to-cost ratio. Although most applications of the pulse-velocity method have been under dry conditions, the transducers can be waterproofed for underwater surveys.

h. Ultrasonic pulse-echo system.

(1) A new improved prototype ultrasonic pulse-echo (UPE) system for evaluating concrete has been developed by the U.S. Army Engineer Waterways Experiment Station (CEWES). The new system (Alexander and Thornton 1988 and Thornton and Alexander 1987) uses piezoelectric crystals to generate and detect signals and the accurate time base of an oscilloscope to measure the TOA of a longitudinal ultrasonic pulse in concrete.

(2) Tests have shown that the system is capable of delineating sound concrete, concrete of questionable quality, and deteriorated concrete, as well as delaminations, voids, reinforcing steel, and other objects within concrete. Also, the system can be used to determine the thickness of a concrete section in which only one surface is accessible. The system will work on vertical or horizontal surfaces. However, the present system is limited to a thickness of about 0.5 m (1.5 ft). For maximum use of this system, the operator should have had considerable experience using the system and interpreting the results.

(3) The system, which was originally developed to operate in a dry environment, was adapted for use in water to determine the condition of a reinforced concrete sea wall at a large marina (Thornton and Alexander 1988).

i. Sonic pulse-echo technique for piles.

(1) A sonic pulse-echo technique for determining the length of concrete and timber piles in dry soil or underwater has been developed at WES (Alexander 1980). Sonic energy is introduced into the accessible end of the pile with a hammer. If the pulse length generated by the hammer is less than round-trip echo time in the pile, then the TOA can be measured with the accurate time base of an oscilloscope. With a digital oscilloscope, the signal can be recorded on magnetic disc and the signal entered into the computer for added signal processing. If the length, mass, and hardness of the head of the hammer is such that the hammer generates energy in the frequency range that corresponds to the longitudinal resonant frequency of the pile, then the frequency can be measured with a spectrum analyzer.

(2) In addition to determining pile lengths to depths of tens of feet, this system can also detect breaks in a pile. Because the surrounding soil dissipates the energy from the hammer, the length-to-diameter ratio of the pile should be greater than 5 and less than 30. To date, work has been limited only to those applications where the impact end of the pile was above water.

2-5. Laboratory Investigations

Once samples of concrete have been obtained, whether by coring or other means, they should be examined in a qualified laboratory. In general, the examination should include petrographic, chemical, or physical tests. Each of these examinations is described in this paragraph.

a. Petrographic examination. Petrographic examination is the application of petrography, a branch of geology concerned with the description and classification of rocks, to the examination of hardened concrete, a synthetic sedimentary rock. Petrographic examination may include visual inspection of the samples, visual inspection at various levels of magnification using appropriate microscopes, X-ray diffraction analysis, differential thermal analysis, X-ray emission techniques, and thin section analysis. Petrographic techniques may be expected to provide information on the following (ACI 207.3R): (1) condition of the aggregate; (2) pronounced cement-aggregate reactions; (3) deterioration of aggregate particles in place; (4) denseness of cement paste; (5) homogeneity of the concrete; (6) occurrence of settlement and bleeding of fresh concrete; (7) depth and extent of carbonation; (8) occurrence and distribution of fractures; (9) characteristics and distribution of voids; and (10) presence of contaminating substances. Petrographic examination of hardened concrete should be performed in accordance with ASTM C 856 (CRD-C 57) by a person qualified by education and experience so that proper interpretation of test results can be made.

b. Chemical analysis. Chemical analysis of hardened concrete or of selected portions (paste, mortar, aggregate, reaction products, etc.) may be used to estimate the cement content, original water-cement ratio, and the presence and amount of chloride and other admixtures.

c. Physical analysis. The following physical and mechanical tests are generally performed on concrete cores:

(1) Density.

(2) Compressive strength.

(3) Modulus of elasticity.

(4) Poisson's ratio.

(5) Pulse velocity.

(6) Direct shear strength of concrete bonded to foundation rock.

(7) Friction sliding of concrete on foundation rock.

(8) Resistance of concrete to deterioration caused by freezing and thawing.

(9) Air content and parameters of the air-void system.

Testing core samples for compressive strength and tensile strength should follow the method specified in ASTM C 42 (CRD-C 27).

2-6. Nondestructive Testing

The purpose of NDT is to determine the various relative properties of concrete such as strength, modulus of elasticity, homogeneity, and integrity, as well as conditions of strain and stress, without damaging the structure. Selection of the most applicable method or methods of testing will require good judgment based on the information needed, size and nature of the project, site conditions and risk to the structure (ACI 207.3R). Proper utilization of NDT requires a "toolbox" of techniques and someone with the expertise to know the proper tool to use in the various circumstances. In this paragraph, the commonly used nondestructive testing techniques for evaluating in situ concrete will be discussed. Malhotra (1976), Thornton and Alexander (1987), and Alexander (1993) provide additional information on NDT techniques. Also, recent advances in nondestructive testing of concrete are summarized by Carino (1992). Test methods are classified into those used to assess in-place strength and those used to locate hidden defects. In the first category, recent developments are presented on the pullout test, the break-off test, the torque test, the pulloff test, and the maturity method. In the second category, a review is presented of infrared thermography, ground penetrating radar, and several methods based upon stress wave propagation. The principles of the methods, their advantages, and their inherent limitations are discussed. Where appropriate, requirements of relevant ASTM standards are discussed.

a. Rebound number (hammer).

(1) Description.

(a) The rebound number is obtained by the use of a hammer that consists of a steel mass and a tension spring in a tubular frame (Figure 2-35). When the plunger of the hammer is pushed against the surface of the concrete, the steel mass is retracted and the spring is compressed. When the mass is completely retracted, the spring is automatically released and the mass is driven against the plunger, which impacts the concrete and rebounds. The rebound distance is indicated by a pointer on a scale that is usually graduated from 0 to 100. The rebound readings are termed R-values. Determination of R-values is outlined in the manual supplied by the hammer manufacturer.

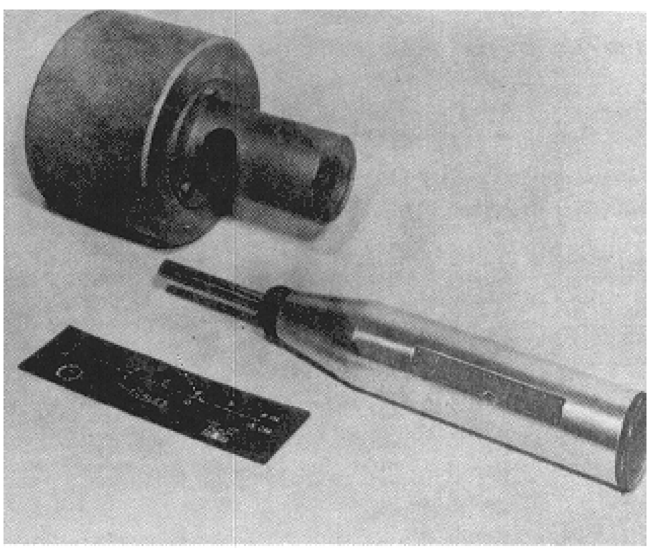


Figure 2-35. Rebound hammer

R-values indicate the coefficient of restitution of the concrete; the values increase with the "strength" of the concrete.

(b) Most hammers come with a calibration chart, showing a purported relationship between compressive strength of concrete and rebound readings. However, rather than placing confidence in such a chart, users should develop their own relations for each concrete mixture and each rebound hammer.

(2) Applications. Rebound numbers may be used to estimate the uniformity and quality of concrete. The test method is covered in ASTM C 805 (CRD-C 22).

(3) Advantages. The rebound hammer is a simple and quick method for NDT of concrete in place. The equipment is inexpensive and can be operated by field personnel with a limited amount of instruction. The rebound hammer is very useful in assessing the general quality of concrete and for locating areas of poor quality concrete. A large number of measurements can be rapidly taken so that large exposed areas of concrete can be mapped within a few hours.

(4) Limitations. The rebound method is a rather imprecise test and does not provide a reliable prediction of the strength of concrete. Rebound measurements on in situ concrete are affected by (a) smoothness of the concrete surface; (b) moisture content of the concrete; (c) type of coarse aggregate; (d) size, shape, and rigidity of specimen (e.g., a thick wall or beam); and (e) carbonation of the concrete surface.

b. Penetration resistance (probe).

(1) Description.

(a) The apparatus most often used for penetration resistance is the Windsor Probe, a special gun (Figure 2-36) that uses a 0.32 caliber blank with a precise quantity of powder to fire a high-strength steel probe into the concrete. A series of three measurements is made in each area with the spacer plate shown in Figure 2-37. The length of a probe extending from the surface of the concrete can be measured with a simple device, as shown in Figure 2-38.

(b) The manufacturer supplies a set of five calibration curves, each corresponding to a specific Moh's hardness for the coarse aggregate used in the concrete. With these curves, probe measurements are intended to be

Figure 2-36. Windsor probe apparatus showing the gun, probe, and blank cartridge

Figure 2-37. Windsor probe in use

converted to compressive strength values. However, use of the manufacturer's calibration curves often results in grossly incorrect estimates of the compressive strength of concrete. Therefore, the penetration probe should be calibrated by the individual user and should be recalibrated whenever the type of aggregate or mixture is changed.

(2) Applications. Penetration resistance can be used for assessing the quality and uniformity of concrete because physical differences in concrete will affect its

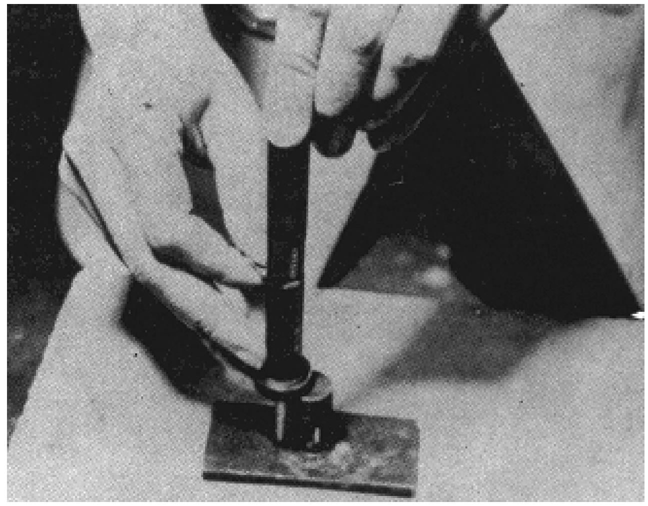


Figure 2-38. Device for measuring length of probe extending from surface of concrete

resistance to penetration. A probe will penetrate deeper as the density, subsurface hardness, and strength of the concrete decrease. Areas of poor concrete can be delineated by making a series of penetration tests at regularly spaced locations. The test method is covered in ASTM C 803 (CRD-C 59).

(3) Advantages. The probe equipment is simple, durable, requires little maintenance, and can be used by inspectors in the field with little training. The probe test is very useful in assessing the general quality and relative strength of concrete in different parts of a structure.

(4) Limitations. Care must be exercised whenever this device is used because a projectile is being fired; safety glasses should always be worn. The probe primarily measures surface and subsurface hardness; it does not yield precise measurements of the in situ strength of concrete. However, useful estimates of the compressive strength of concrete may be obtained if the probe is properly calibrated. The probe test does damage the concrete, leaving a hole of about 8 mm (0.32 in.) in diameter for the depth of the probe, and it may cause minor cracking and some surface spalling. Minor repairs of exposed surfaces may be necessary.

c. Ultrasonic pulse-velocity method.

(1) Description. The ultrasonic pulse-velocity method is probably the most widely used method for the nondestructive evaluation of in situ concrete. The method involves measurement of the time of travel of electronically pulsed compressional waves through a known

distance in concrete. From known TOA and distance traveled, the pulse velocity through the concrete can be calculated. Pulse-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.

(2) Applications. The ultrasonic pulse-velocity method has 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 or poor-quality concrete. The test method is described in ASTM C 597 (CRD-C 51).

(3) Advantages. The required equipment is portable (Figure 2-39) and has sufficient power to penetrate about 11 m (35 ft) of good continuous concrete, and the test can be performed quickly.

(4) Limitations. This method does not provide a precise estimate of concrete strength. Moisture variations and the presence of reinforcing steel can affect the results. Skilled personnel is required to analyze the results. The measurement requires access to opposite sides of the section being tested.

d. Acoustic mapping system.

(1) Description. This system makes possible, without dewatering of the structure, comprehensive evaluation of top surface wear on such horizontal surfaces as aprons, sills, lock chamber floors, and stilling basins, where turbulent flows carrying rock and debris can cause abrasion-erosion damage. The system uses the sonar principle, i.e., transmitting acoustic waves and receiving reflections from underwater structures.

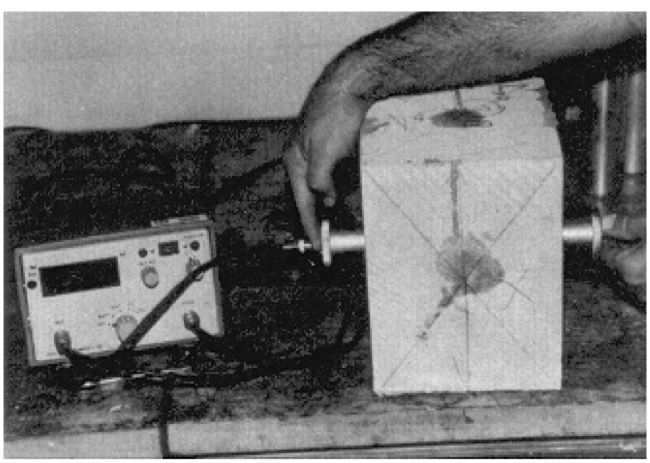


Figure 2-39. Ultrasonic pulse-velocity apparatus

(2) Application. The system can be used to perform rapid, accurate surveys of submerged horizontal surfaces in water depths of 71.5 to 12 m (5 to 40 ft) with accuracies of ± 50 mm (2 in.) vertically and ± .3 m (1 ft) laterally. Variations of the system may be used for other underwater applications such as repairing and investigating large scour holes or silt buildup. The system has been successfully used in surveying the stilling basin floor of Folsom Dam, a U.S. Bureau of Reclamation project (SONEX 1984), and the stilling basin of Ice Harbor Dam in Walla Walla District (SONEX 1983).

(3) Advantages. This method avoids the expense and user inconvenience associated with dewatering and the dangers and inaccuracies inherent in diver-performed surveys.

(4) Limitations. Vertical and lateral accuracy will decrease at depths greater than 9 m (30 ft). There are some operational restrictions associated with water velocity and turbulence.

e. Ultrasonic pulse-echo (UPE).

(1) Description. A variation of the pulse-velocity technique is the pulse-echo method wherein a compressional wave pulse is transmitted from a surface and its echo received back at the same surface location. Reflection times from interfaces, cracks, or voids, together with the known velocity within the concrete, permit calculation of distances from the discontinuity to the transmitting and receiving points. The system has been demonstrated to be feasible but is still under development (Alexander and Thornton 1988). An impact pulse-echo system for measurements on concrete piles is described by Alexander (1980).

(2) Applications. The system operates well for flatwork for dimensions less than 0.3 m (1 ft) in thickness. The system can detect foreign objects such as steel and plastic pipe. It can measure unknown thicknesses and presence of delaminations up to 0.3 m (1 ft) in thickness. Recently neural network algorithms were trained on some calibrated specimens to recognize the condition of concrete that has uniform microcracking.

(3) Advantages. The system has excellent resolution as it operates around a center frequently of 200 kHz. The wavelength is roughly 25 mm (1 in.) long in good-quality concrete, which provides better spatial resolution than radar. It can operate underwater or in the dry. The speed of sound in concrete does not vary by more than 5 percent from moist to dry concrete.

(4) Limitations. Presently the system exists as a laboratory prototype. The equipment presently is multicomponent and not very portable. Also, most measurement data need digital signal algorithms applied to the data to bring signals out of the noise, and this task requires the expertise of someone skilled in that discipline. The system presently does not have an onboard computer, and the data cannot be processed onsite in realtime. The system is not yet available commercially and is not a CRD or ASTM measurement standard. Plans are underway to commercialize the system and remedy the above-mentioned limitations.

f. Radar.

(1) Description. This is a reflection technique that is based on the principle of electromagnetic wave propagation. Similar to UPE in operation, the TOA of the wave is measured from the time the pulse is introduced into the concrete at the surface of the structure, travels to the discontinuity or interface, and is reflected back to original surface. Whereas the mechanical wave travels at the speed of sound for the UPE technique, the electromagnetic wave travels at the speed of light for radar.

(2) Applications. A radar unit operating at the frequency of 1 gHz has a wavelength about 150 mm (6 in.) in concrete. Presently systems can penetrate to a depth of about 0.5 m (1.5 ft) at this frequency. A void 150 mm (6 in.) deep in concrete must have a diameter of 50 to 75 mm (2 to 3 in.) to be detectable. At a depth of 0.3 m (1 ft), the void must be 75 to 100 mm (3 to 4 in.) in diameter to be detectable. Lower frequency systems can penetrate deeper than this, but the resolution is even poorer. Radar is especially sensitive for detecting steel reinforcement, but steel can also interfere with the measurements if one is looking for deterioration in the concrete. Radar is sensitive to moisture and may be useful for finding deteriorated areas, which tend to hold more water than sound concrete.

(3) Advantages. Radar is a noncontact method and data acquisition is very fast. Resolution and penetration are limited at the present time. Systems are available commercially.

(4) Limitations. Radar is steel in the process of development for use on concrete (Ahmad and Haskins 1993), and a measurement standard does not exist at this time. A radar unit may cost between $50K and $100K and requires someone highly trained to operate the equipment and interpret the data. Commercial systems being used for concrete are primarily designed to operate in the earth for geophysical applications. Better results can be obtained by applying signal processing techniques. The velocity of the pulse is dependent on the dielectric constant of the concrete and varies by almost 100 percent between dry concrete and moist concrete.

2-7. Stability Analysis

A stability analysis is often performed as part of an overall evaluation of the condition of a concrete structure. Guidelines for performing a stability analysis for existing structures are beyond the scope of this manual, but may be found in other CE publications. Information on requirements for stability analyses may be obtained from CECW-E.

2-8. Deformation Monitoring

A tool now available for a comprehensive evaluation of larger structures is the Continuous Deformation Monitoring System (CDMS) developed in Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. The CDMS uses the Navigation Satellite Timing and Ranging (NAVSTAR) Global Positioning Systems (GPS) to monitor the position of survey monuments installed on a structure. The system was demonstrated in a field test at Dworshak Dam (Lanigan 1992).

2-9. Concrete Service Life

a. Freeze-thaw deterioration. A procedure has been developed to predict the service life of nonair-entrained concrete subject to damage from freezing and thawing. The procedure addresses with a probabilistic method (Bryant and Mlakar 1991) both the known and uncertain qualities of the relevant material properties, environmental factors, and model of degradation resulting from freezing and thawing. Two important characteristics of this procedure are (1) it rationally addresses the uncertainties inherent in degradation of mass concrete caused by freezing and thawing, and (2) it is mathematically straightforward for implementation by CE offices.

(1) Current procedures for thermal modeling and analysis appear quite adequate for predicting temperatures in a concrete structure. Although 2-D analyses are better

for determining complex thermal response, in many cases a series of much simpler one-dimensional (1-D) analyses provide a very good estimation of temperatures. The external temperature inputs to a thermal analysis, i.e., water-air temperatures, were well represented by sinusoidal curves.

(2) The general understanding and analytical models for predicting moisture migration and degree of saturation are not as well developed as those for the thermal problem. A seepage model for predicting the degree of saturation appears to provide adequate answers for the prediction of service life; however, further study is appropriate to substantiate this indication.

(3) The procedure was demonstrated by hindcast application to the middle wall and landwall at Dashields Lock which exhibited an appreciable degree of measurable damage caused by freezing and thawing. Required data for application of the procedure, e.g., temperature and concrete properties, were available for these features, which were representative of typical CE projects.

(4) Damage predicted by the procedure was in agreement with observed damage resulting from freezing and thawing at each site. The general trends of location and spatial variation of damage were very similar to observations and measurements at the two sites. More encouragingly, the actual magnitudes of damage predicted by the procedure compared favorably with the previous measurements. This result provides the strongest indication that the procedure is rational and would enhance the ability of the CE to predict service life at its many other concrete structures.

b. Other deterioration mechanisms. A complete and comprehensive report by Clifton (1991) examines the basis for predicting the remaining service lives of concrete materials of nuclear power facilities. The study consisted of two major activities: the evaluation of models which can be used in predicting the remaining service life of concrete exposed to the major environmental stressors and aging factors; and, the evaluation of accelerated aging techniques and tests which can provide data for service life models or which themselves can be used to predict the remaining service life of concrete. Methods for service life prediction which are discussed in this report include: (1) estimates based on experience; (2) deductions from performance of similar materials; (3) accelerated testing; (4) applications of reliability and stochastic concepts; and (5) mathematical modeling based on the chemistry and physics of the degradation processes. Models for corrosion, sulfate attack, frost attack, and leaching were identified and analyzed. While no model was identified for distress caused by alkali-aggregate reactions, an approach for modeling the process was outlined.

2-10. Reliability Analysis

A reliability analysis may be required for major rehabilitation projects. Guidelines for performing a reliability analysis are beyond the scope of this manual. Information on requirements for reliability analyses may be obtained from CECW-E.

Chapter 3
Causes of Distress and Deterioration of Concrete

3-1. Introduction

a. General. Once the evaluation phase has been completed for a structure, the next step is to establish the cause or causes for the damage that has been detected. Since many of the symptoms may be caused by more than one mechanism acting upon the concrete, it is necessary to have an understanding of the basic underlying causes of damage and deterioration. This chapter presents information on the common causes of problems in concrete. These causes are shown in Table 3-1. Items shown in the table are discussed in the subsequent sections of this chapter with the following given for each: (1) brief discussion of the basic mechanism; (2) description of the most typical symptoms, both those that would be observed during a visual examination and those that would be seen during a laboratory evaluation; and (3) recommendations for preventing further damage to new or replacement concrete. The last section of the chapter presents a logical method for relating the symptoms or observations to the various causes.

b. Approach to evaluation. Deterioration of concrete is an extremely complex subject. It would be simplistic to suggest that it will be possible to identify a specific, single cause of deterioration for every symptom detected during an evaluation of a structure. In most cases, the damage detected will be the result of more than one mechanism. For example, corrosion of reinforcing steel may open cracks that allow moisture greater access to the interior of the concrete. This moisture could lead to additional damage by freezing and thawing. In spite of the complexity of several causes working simultaneously, given a basic understanding of the various damage-causing mechanisms, it should be possible, in most cases, to determine the primary cause or causes of the damage seen on a particular structure and to make intelligent choices concerning selection of repair materials and methods.

3-2. Causes of Distress and Deterioration

a. Accidental loadings.

(1) Mechanism. Accidental loadings may be characterized as short-duration, one-time events such as the impact of a barge against a lock wall or an earthquake.

Table 3-1
Causes of Distress and Deterioration of Concrete

Accidental Loadings
Chemical Reactions
Acid attack
Aggressive-water attack
Alkali-carbonate rock reaction
Alkali-silica reaction
Miscellaneous chemical attack
Sulfate attack
Construction Errors
Corrosion of Embedded Metals
Design Errors
Inadequate structural design
Poor design details
Erosion
Abrasion
Cavitation
Freezing and Thawing
Settlement and Movement
Shrinkage
Plastic
Drying
Temperature Changes
Internally generated
Externally generated
Fire
Weathering

These loadings can generate stresses higher than the strength of the concrete, resulting in localized or general failure. Determination of whether accidental loading caused damage to the concrete will require knowledge of the events preceding discovery of the damage. Usually, damage caused by accidental loading will be easy to diagnose.

(2) Symptoms. Visual examination will usually show spalling or cracking of concrete which has been subjected to accidental loadings. Laboratory analysis is generally not necessary.

(3) Prevention. Accidental loadings by their very nature cannot be prevented. Minimizing the effects of some occurrences by following proper design procedures (an example is the design for earthquakes) or by proper

attention to detailing (wall armor in areas of likely impact) will reduce the impacts of accidental loadings.

b. Chemical reactions. This category includes several specific causes of deterioration that exhibit a wide variety of symptoms. In general, deleterious chemical reactions may be classified as those that occur as the result of external chemicals attacking the concrete (acid attack, aggressive water attack, miscellaneous chemical attack, and sulfate attack) or those that occur as a result of internal chemical reactions between the constituents of the concrete (alkali-silica and alkali-carbonate rock reactions). Each of these chemical reactions is described below.

(1) Acid attack.

(a) Mechanism. Portland-cement concrete is a highly alkaline material and is not very resistant to attack by acids. The deterioration of concrete by acids is primarily the result of a reaction between the acid and the products of the hydration of cement. Calcium silicate hydrate may be attacked if highly concentrated acid exists in the environment of the concrete structures. In most cases, the chemical reaction results in the formation of water-soluble calcium compounds that are then leached away. In the case of sulfuric acid attack, additional or accelerated deterioration results because the calcium sulfate formed may affect the concrete by the sulfate attack mechanism (Section 3-2*b*(6)). If the acid is able to reach the reinforcing steel through cracks or pores in the concrete, corrosion of the reinforcing steel will result and will cause further deterioration of the concrete (ACI 201.2R).

(b) Symptoms. Visual examination will show disintegration of the concrete evidenced by loss of cement paste and aggregate from the matrix (Figure 2-13). If reinforcing steel has been reached by the acid, rust staining, cracking, and spalling may be present. If the nature of the solution in which the deteriorating concrete is located is unknown, laboratory analysis can be used to identify the specific acid involved.

(c) Prevention. A dense concrete with a low water-cement ratio (w/c) may provide an acceptable degree of protection against a mild acid attack. Portland-cement concrete, because of its composition, is unable to withstand attack by highly acidic solutions for long periods of time. Under such conditions, an appropriate surface coating or treatment may be necessary. ACI Committee 515 has extensive recommendations for such coatings (ACI 515.1R).

(2) Aggressive-water attack.

(a) Mechanism. Some waters have been reported to have extremely low concentrations of dissolved minerals. These soft or aggressive waters will leach calcium from cement paste or aggregates. This phenomenon has been infrequently reported in the United States. From the few cases that have been reported, there are indications that this attack takes place very slowly. For an aggressive-water attack to have a serious effect on hydraulic structures, the attack must occur in flowing water. This keeps a constant supply of aggressive water in contact with the concrete and washes away aggregate particles that become loosened as a result of leaching of the paste (Holland, Husbands, Buck, and Wong 1980).

(b) Symptoms. Visual examination will show concrete surfaces that are very rough in areas where the paste has been leached (Figure 2-12). Sand grains may be present on the surface of the concrete, making it resemble a coarse sandpaper. If the aggregate is susceptible to leaching, holes where the coarse aggregate has been dissolved will be evident. Water samples from structures where aggressive-water attack is suspected may be analyzed to calculate the Langlier Index, which is a measure of the aggressiveness of the water (Langlier 1936).

(c) Prevention. The aggressive nature of water at the site of a structure can be determined before construction or during a major rehabilitation. Additionally, the water-quality evaluation at many structures can be expanded to monitor the aggressiveness of water at the structure. If there are indications that the water is aggressive or is becoming aggressive, areas susceptible to high flows may be coated with a nonportland-cement-based coating.

(3) Alkali-carbonate rock reaction.

(a) Mechanism. Certain carbonate rock aggregates have been reactive in concrete. The results of these reactions have been characterized as ranging from beneficial to destructive. The destructive category is apparently limited to reactions with impure dolomitic aggregates and are a result of either dedolomitization or rim-silicification reactions. The mechanism of alkali-carbonate rock reaction is covered in detail in EM 1110-2-2000.

(b) Symptoms. Visual examination of those reactions that are serious enough to disrupt the concrete in a structure will generally show map or pattern cracking and a general appearance which indicates that the concrete is swelling (Figure 2-7). A distinguishing feature which differentiates alkali-carbonate rock reaction from alkali-silica reaction is the lack of silica gel exudations at cracks (ACI 201.2R). Petrographic examination in accordance with ASTM C 295 (CRD-C 127) may be used to confirm the presence of alkali-carbonate rock reaction.

(c) Prevention. In general, the best prevention is to avoid using aggregates that are or suspected of being reactive. Appendix E of EM 1110-2-2000 prescribes procedures for testing rocks for reactivity and for minimizing effects when reactive aggregates must be used.

(4) Alkali-silica reaction.

(a) Mechanism. Some aggregates containing silica that is soluble in highly alkaline solutions may react to form a solid nonexpansive calcium-alkali-silica complex or an alkali-silica complex which can imbibe considerable amounts of water and then expand, disrupting the concrete. Additional details may be found in EM 1110-2-2000.

(b) Symptoms. Visual examination of those concrete structures that are affected will generally show map or pattern cracking and a general appearance that indicates that the concrete is swelling (Figure 2-6). Petrographic examination may be used to confirm the presence of alkali-silica reaction.

(c) Prevention. In general, the best prevention is to avoid using aggregates that are known or suspected to be reactive or to use a cement containing less than 0.60 percent alkalies (percent Na_20 + (0.658) percent K_20). Appendix D of EM 1110-2-2000 prescribes procedures for testing aggregates for reactivity and for minimizing the effects when reactive aggregates must be used.

(5) Miscellaneous chemical attack.

(a) Mechanism. Concrete will resist chemical attack to varying degrees, depending upon the exact nature of the chemical. ACI 515.1R includes an extensive listing of the resistance of concrete to various chemicals. To produce significant attack on concrete, most chemicals must be in solution that is above some minimum concentration. Concrete is seldom attacked by solid dry chemicals. Also, for maximum effect, the chemical solution needs to be circulated in contact with the concrete. Concrete subjected to aggressive solutions under positive differential pressure is particularly vulnerable. The pressure gradients tend to force the aggressive solutions into the matrix. If the low-pressure face of the concrete is exposed to evaporation, a concentration of salts tends to accumulate at that face, resulting in increased attack. In addition to the specific nature of the chemical involved, the degree to which concrete resists attack depends upon the temperature of the aggressive solution, the w/c of the concrete, the type of cement used (in some circumstances), the degree of consolidation of the concrete, the permeability of the concrete, the degree of wetting and drying of the chemical on the concrete, and the extent of chemically induced corrosion of the reinforcing steel (ACI 201.1R).

(b) Symptoms. Visual examination of concrete which has been subjected to chemical attack will usually show surface disintegration and spalling and the opening of joints and cracks. There may also be swelling and general disruption of the concrete mass. Coarse aggregate particles are generally more inert than the cement paste matrix; therefore, aggregate particles may be seen as protruding from the matrix. Laboratory analysis may be required to identify the unknown chemicals which are causing the damage.

(c) Prevention. Typically, dense concretes with low w/c (maximum w/c = 0.40) provide the greatest resistance. The best known method of providing long-term resistance is to provide a suitable coating as outlined in ACI 515.1R.

(6) Sulfate attack.

(a) Mechanism. Naturally occurring sulfates of sodium, potassium, calcium, or magnesium are sometimes found in soil or in solution in ground water adjacent to concrete structures. The sulfate ions in solution will attack the concrete. There are apparently two chemical reactions involved in sulfate attack on concrete. First, the sulfate reacts with free calcium hydroxide which is liberated during the hydration of the cement to form calcium sulfate (gypsum). Next, the gypsum combines with hydrated calcium aluminate to form calcium sulfoaluminate (ettringite). Both of these reactions result in an increase in volume. The second reaction is mainly responsible for most of the disruption caused by volume increase of the concrete (ACI 201.2R). In addition to the two chemical reactions, there may also be a purely physical phenomenon in which the growth of crystals of sulfate salts disrupts the concrete.

(b) Symptoms. Visual examination will show map and pattern cracking as well as a general disintegration of the concrete (Figure 2-14). Laboratory analysis can verify the occurrence of the reactions described.

(c) Prevention. Protection against sulfate attack can generally be obtained by the following: Use of a dense, high-quality concrete with a low water-cement ratio; Use of either a Type V or a Type II cement, depending upon the anticipated severity of the exposure (EM 1110-2-2000); Use of a suitable pozzolan (some pozzolans, added as part of a blended cement or separately, have improved resistance, while others have hastened deterioration). If use of a pozzolan is anticipated, laboratory testing to verify the degree of improvement to be expected is recommended.

c. Construction errors. Failure to follow specified procedures and good practice or outright carelessness may lead to a number of conditions that may be grouped together as construction errors. Typically, most of these errors do not lead directly to failure or deterioration of concrete. Instead, they enhance the adverse impacts of other mechanisms identified in this chapter. Each error will be briefly described below along with preventative methods. In general, the best preventive measure is a thorough knowledge of what these construction errors are plus an aggressive inspection program. It should be noted that errors of the type described in this section are equally as likely to occur during repair or rehabilitation projects as they are likely to occur during new construction.

(1) Adding water to concrete. Water is usually added to concrete in one or both of the following circumstances: First, water is added to the concrete in a delivery truck to increase slump and decrease emplacement effort. This practice will generally lead to concrete with lowered strength and reduced durability. As the w/c of the concrete increases, the strength and durability will decrease. In the second case, water is commonly added during finishing of flatwork. This practice leads to scaling, crazing, and dusting of the concrete in service.

(2) Improper alignment of formwork. Improper alignment of the formwork will lead to discontinuities on the surface of the concrete. While these discontinuities are unsightly in all circumstances, their occurrence may be more critical in areas that are subjected to high-velocity flow of water, where cavitation-erosion may be induced, or in lock chambers where the "rubbing" surfaces must be straight.

(3) Improper consolidation. Improper consolidation of concrete may result in a variety of defects, the most common being bugholes, honeycombing, and cold joints. "Bugholes" are formed when small pockets of air or water are trapped against the forms. A change in the mixture to make it less "sticky" or the use of small vibrators worked near the form has been used to help eliminate bugholes. Honeycombing can be reduced by inserting the vibrator more frequently, inserting the vibrator as close as possible to the form face without touching the form, and slower withdrawal of the vibrator. Obviously, any or all of these defects make it much easier for any damage-causing mechanism to initiate deterioration of the concrete. Frequently, a fear of "overconsolidation" is used to justify a lack of effort in consolidating concrete. Overconsolidation is usually defined as a situation in which the consolidation effort causes all of the coarse aggregate to settle to the bottom while the paste rises to the surface. If this situation occurs, it is reasonable to conclude that there is a problem of a poorly proportioned concrete rather than too much consolidation.

(4) Improper curing. Curing is probably the most abused aspect of the concrete construction process. Unless concrete is given adequate time to cure at a proper humidity and temperature, it will not develop the characteristics that are expected and that are necessary to provide durability. Symptoms of improperly cured concrete can include various types of cracking and surface disintegration. In extreme cases where poor curing leads to failure to achieve anticipated concrete strengths, structural cracking may occur.

(5) Improper location of reinforcing steel. This section refers to reinforcing steel that is improperly located or is not adequately secured in the proper location. Either of these faults may lead to two general types of problems. First, the steel may not function structurally as intended, resulting in structural cracking or failure. A particularly prevalent example is the placement of welded wire mesh in floor slabs. In many cases, the mesh ends up on the bottom of the slab which will subsequently crack because the steel is not in the proper location. The second type of problem stemming from improperly located or tied reinforcing steel is one of durability. The tendency seems to be for the steel to end up near the surface of the concrete. As the concrete cover over the steel is reduced, it is much easier for corrosion to begin.

(6) Movement of formwork. Movement of formwork during the period while the concrete is going from a fluid to a rigid material may induce cracking and separation within the concrete. A crack open to the surface will allow access of water to the interior of the concrete. An internal void may give rise to freezing or corrosion problems if the void becomes saturated.

(7) Premature removal of shores or reshores. If shores or reshores are removed too soon, the concrete affected may become overstressed and cracked. In extreme cases there may be major failures.

(8) Settling of the concrete. During the period between placing and initial setting of the concrete, the heavier components of the concrete will settle under the influence of gravity. This situation may be aggravated by the use of highly fluid concretes. If any restraint tends to prevent this settling, cracking or separations may result. These cracks or separations may also develop problems of corrosion or freezing if saturated.

(9) Settling of the subgrade. If there is any settling of the subgrade during the period after the concrete begins to become rigid but before it gains enough strength to support its own weight, cracking may also occur.

(10) Vibration of freshly placed concrete. Most construction sites are subjected to vibration from various sources, such as blasting, pile driving, and from the operation of construction equipment. Freshly placed concrete is vulnerable to weakening of its properties if subjected to forces which disrupt the concrete matrix during setting. The vibration limits for concrete, expressed in terms of peak particle velocity and given in Table 3-2, were established as a result of laboratory and field test programs.

(11) Improper finishing of flat work. The most common improper finishing procedures which are detrimental to the durability of flat work are discussed below.

(a) Adding water to the surface. This procedure was discussed in paragraph 3-2*c*(1) above. Evidence that water is being added to the surface is the presence of a large paint brush, along with other finishing tools. The brush is dipped in water and water is "slung" onto the surface being finished.

(b) Timing of finishing. Final finishing operations must be done after the concrete has taken its initial set and bleeding has stopped. The waiting period depends on the amounts of water, cement, and admixtures in the mixture but primarily on the temperature of the concrete surface. On a partially shaded slab, the part in the sun will usually be ready to finish before the part in the shade.

(c) Adding cement to the surface. This practice is often done to dry up bleed water to allow finishing to proceed and will result in a thin cement-rich coating which will craze or flake off easily.

(d) Use of tamper. A tamper or "jitterbug" is unnecessarily used on many jobs. This tool forces the coarse aggregate away from the surface and can make finishing easier. This practice, however, creates a cement-rich mortar surface layer which can scale or craze. A jitterbug should not be allowed with a well designed mixture. If a harsh mixture must be finished, the judicious use of a jitterbug could be useful.

(e) Jointing. The most frequent cause of cracking in flatwork is the incorrect spacing and location of joints. Joint spacing is discussed in ACI 330R.

d. Corrosion of embedded metals.

(1) Mechanisms. Steel reinforcement is deliberately and almost invariably placed within a few inches of a concrete surface. Under most circumstances, portland-cement concrete provides good protection to the embedded reinforcing steel. This protection is generally attributed to the high alkalinity of the concrete adjacent to the steel and to the relatively high electrical resistance of the concrete. Still, corrosion of the reinforcing steel is among the most frequent causes of damage to concrete.

Table 3-2
Vibration Limits for Freshly Placed Concrete (Hulshizer and Desci 1984)

Age of Concrete at Time of Vibration (hr)	Peak Particle Velocity of Ground Vibrations
Up to 3	102 mm/sec (4.0 in./sec)
3 to 11	38 mm/sec (1.5 in./sec)
11 to 24	51 mm/sec (2.0 in./sec)
24 to 48	102 mm/sec (4.0 in./sec)
Over 48	178 mm/sec (7.0 in./sec)

(a) High alkalinity and electrical resistivity of the concrete. The high alkalinity of the concrete pore solution can be reduced over a long period of time by carbonation. The electrical resistivity can be decreased by the presence of chemicals in the concrete. The chemical most commonly applied to concrete is chloride salts in the form of deicers. As the chloride ions penetrate the concrete, the capability of the concrete to carry an electrical current is increased significantly. If there are differences within the concrete such as moisture content, chloride content, oxygen content, or if dissimilar metals are in contact, electrical potential differences will occur and a corrosion cell may be established. The anodes will experience corrosion while the cathodes will be undamaged. On an individual reinforcing bar there may be many anodes and cathodes, some adjacent, and some widely spaced.

(b) Corrosion-enhanced reduction in load-carrying capacity of concrete. As the corrosion progresses, two things occur: First, the cross-sectional area of the reinforcement is reduced, which in turn reduces the load-carrying capacity of the steel. Second, the products of the corrosion, iron oxide (rust), expand since they occupy about eight times the volume of the original material. This increase in volume leads to cracking and ultimately spalling of the concrete. For mild steel reinforcing, the damage to the concrete will become evident long before the capacity of the steel is reduced enough to affect its load-carrying capacity. However, for prestressing steel, slight reductions in section can lead to catastrophic failure.

(c) Other mechanisms for corrosion of embedded metals. In addition to the development of an electrolytic cell, corrosion may be developed under several other situations. The first of these is corrosion produced by the presence of a stray electrical current. In this case, the current necessary for the corrosion reaction is provided from an outside source. A second additional source of corrosion is that produced by chemicals that may be able to act directly on the reinforcing steel. Since this section has dealt only with the corrosion of steel embedded in concrete, for information on the behavior of other metals in concrete, see ACI 201.2R and ACI 222R.

(2) Symptoms. Visual examination will typically reveal rust staining of the concrete. This staining will be followed by cracking. Cracks produced by corrosion generally run in straight, parallel lines at uniform intervals corresponding to the spacing of the reinforcement. As deterioration continues, spalling of the concrete over the reinforcing steel will occur with the reinforcing bars becoming visible (Figure 2-27). One area where laboratory analysis may be beneficial is the determination of the chloride contents in the concrete. This procedure may be used to determine the amount of concrete to be removed during a rehabilitation project.

(3) Prevention. ACI 201.2R describes the considerations for protecting reinforcing steel in concrete: use of concrete with low permeability; use of properly proportioned concrete having a low w/c; use of as low a concrete slump as practical; use of good workmanship in placing the concrete; curing the concrete properly; providing adequate concrete cover over the reinforcing steel; providing good drainage to prevent water from standing on the concrete; limiting chlorides in the concrete mixture; and paying careful attention to protruding items such as bolts or other anchors.

e. Design errors. Design errors may be divided into two general types: those resulting from inadequate structural design and those resulting from lack of attention to relatively minor design details. Each of the two types of design errors is discussed below.

(1) Inadequate structural design.

(a) Mechanism. The failure mechanism is simple--the concrete is exposed to greater stress than it is capable of carrying or it sustains greater strain than its strain capacity.

(b) Symptoms. Visual examinations of failures resulting from inadequate structural design will usually show one of two symptoms. First, errors in design resulting in excessively high compressive stresses will result in spalling. Similarly, high torsion or shear stresses may also result in spalling or cracking. Second, high tensile stresses will result in cracking. To identify inadequate design as a cause of damage, the locations of the damage should be compared to the types of stresses that should be present in the concrete. For example, if spalls are present on the underside of a simple-supported beam, high compressive stresses are not present and inadequate design may be eliminated as a cause. However, if the type and location of the damage and the probable stress are in agreement, a detailed stress analysis will be required to determine whether inadequate design is the cause. Laboratory analysis is generally not applicable in the case of suspected inadequate design. However, for rehabilitation projects, thorough petrographic analysis and strength testing of concrete from elements to be reused will be necessary.

(c) Prevention. Inadequate design is best prevented by thorough and careful review of all design calculations. Any rehabilitation method that makes use of existing concrete structural members must be carefully reviewed.

(2) Poor design details. While a structure may be adequately designed to meet loadings and other overall requirements, poor detailing may result in localized concentrations of high stresses in otherwise satisfactory concrete. These high stresses may result in cracking that allows water or chemicals access to the concrete. In other cases, poor design detailing may simply allow water to pond on a structure, resulting in saturated concrete. In general, poor detailing does not lead directly to concrete failure; rather, it contributes to the action of one of the other causes of concrete deterioration described in this chapter. Several specific types of poor detailing and their possible effects on a structure are described in the following paragraphs. In general, all of these problems can be prevented by a thorough and careful review of plans and specifications for the project. In the case of existing structures, problems resulting from poor detailing should be handled by correcting the detailing and not by simply responding to the symptoms.

(a) Abrupt changes in section. Abrupt changes in section may cause stress concentrations that may result in cracking. Typical examples would include the use of relatively thin sections such as bridge decks rigidly tied into massive abutments or patches and replacement concrete that are not uniform in plan dimensions.

(b) Insufficient reinforcement at reentrant corners and openings. Reentrant corners and openings also tend to cause stress concentrations that may cause cracking. In this case, the best prevention is to provide additional reinforcement in areas where stress concentrations are expected to occur.

(c) Inadequate provision for deflection. Deflections in excess of those anticipated may result in loading of members or sections beyond the capacities for which they were designed. Typically, these loadings will be induced in walls or partitions, resulting in cracking.

(d) Inadequate provision for drainage. Poor attention to the details of draining a structure may result in the ponding of water. This ponding may result in leakage or saturation of concrete. Leakage may result in damage to the interior of the structure or in staining and encrustations on the structure. Saturation may result in severely damaged concrete if the structure is in an area that is subjected to freezing and thawing.

(e) Insufficient travel in expansion joints. Inadequately designed expansion joints may result in spalling of concrete adjacent to the joints. The full range of possible temperature differentials that a concrete may be expected to experience should be taken into account in the specification for expansion joints. There is no single expansion joint that will work for all cases of temperature differential.

(f) Incompatibility of materials. The use of materials with different properties (modulus of elasticity or coefficient of thermal expansion) adjacent to one another may result in cracking or spalling as the structure is loaded or as it is subjected to daily or annual temperature variations.

(g) Neglect of creep effect. Neglect of creep may have similar effects as noted earlier for inadequate provision for deflections (paragraph 3-2*e*(2)(c)). Additionally, neglect of creep in prestressed concrete members may lead to excessive prestress loss that in turn results in cracking as loads are applied.

(h) Rigid joints between precast units. Designs utilizing precast elements must provide for movement between adjacent precast elements or between the precast elements and the supporting frame. Failure to provide for this movement can result in cracking or spalling.

(i) Unanticipated shear stresses in piers, columns, or abutments. If, through lack of maintenance, expansion bearing assembles are allowed to become frozen, horizontal loading may be transferred to the concrete elements supporting the bearings. The result will be cracking in the concrete, usually compounded by other problems which will be caused by the entry of water into the concrete.

(j) Inadequate joint spacing in slabs. This is one of the most frequent causes of cracking of slabs-on-grade. Guidance on joint spacing and depth of contraction joints may be found in ACI 332R.

f. Abrasion. Abrasion damage caused by waterborne debris and the techniques used to repair the damage on several Corps' structures are described by McDonald (1980). Also, causes of abrasion-erosion damage and procedures for repair and prevention of damage are described in ACI 210R.

(1) Mechanism. Abrasion-erosion damage is caused by the action of debris rolling and grinding against a concrete surface. In hydraulic structures, the areas most

likely to be damaged are spillway aprons, stilling basin slabs, and lock culverts and laterals. The sources of the debris include construction trash left in a structure, riprap brought back into a basin by eddy currents because of poor hydraulic design or asymmetrical discharge, and riprap or other debris thrown into a basin by the public. Also barges and towboats impacting or scraping on lock wells and guide wells can cause abrasions erosion damage.

(2) Symptoms. Concrete surfaces abraded by water-borne debris are generally smooth (Figure 2-20) and may contain localized depressions. Most of the debris remaining in the structure will be spherical and smooth. Mechanical abrasion is usually characterized by long shallow grooves in the concrete surface and spalling along monolith joints. Armor plates is often torn away or bent.

(3) Prevention. The following measures should be followed to prevent or minimize abrasion-erosion damage to concrete hydraulic structures (Liu 1980 and McDonald 1980).

(a) Design. It appears that given appropriate flow conditions in the presence of debris, all of the construction materials currently being used in hydraulic structures are to some degree susceptible to erosion. While improvements in materials should reduce the rate of concrete damage caused by erosion, this improvement alone will not solve the problem. Until the adverse hydraulic conditions that can cause abrasion-erosion damage are minimized or eliminated, it will be extremely difficult for any of the construction materials currently being used to avoid damage by erosion. Prior to construction or repair of major structures, hydraulic model studies of the structure may be required to identify potential causes of erosion damage and to evaluate the effectiveness of various modifications in eliminating those undesirable hydraulic conditions. Many older structures have spillways designed with a vertical end-sill. This design is usually efficient in trapping the erosion-causing debris within the spillway. In some structures, a 45-deg fillet installed on the upstream side of the end sill has resulted in a self-cleaning stilling basin. Recessing monolith joints in lock walls and guide walls will minimize stilling basin spalling caused by barge impact and abrasion (See paragraph 8-1*e*(2)(e)).

(b) Operation. In existing structures, balanced flows should be maintained into basins by using all gates to avoid discharge conditions where eddy action is prevalent. Substantial discharges that can provide a good hydraulic jump without creating eddy action should be released periodically in an attempt to flush debris from the stilling basin. Guidance as to discharge and tailwater relations required for flushing should be developed through model and prototype tests. Periodic inspections should be required to determine the presence of debris in the stilling basin and the extent of erosion. If the debris cannot be removed by flushing operations, the basin should be cleaned by other means.

(c) Materials. It is imperative that materials be tested and evaluated, in accordance with ASTM C 1138 (CRD-C 63), prior to use in the repair of abrasion-erosion damaged hydraulic structures. Abrasion-resistant concrete should include the maximum amount of the hardest coarse aggregate that is available and the lowest practical w/c. In some cases where hard aggregate was not available, high-range water-reducing admixtures (HRWRA) and condensed silica fume have been used to develop high compressive strength concrete 97 MPa (14,000 psi) to overcome problems of unsatisfactory aggregate (Holland 1983). Apparently, at these high compressive strengths the hardened cement paste assumes a greater role in resisting abrasion-erosion damage, and as such, the aggregate quality becomes correspondingly less important. The abrasion-erosion resistance of vacuum-treated concrete, polymer concrete, polymer-impregnated concrete, and polymer portland-cement concrete is significantly superior to that of comparable conventional concrete that can also be attributed to a stronger cement matrix. The increased costs associated with materials, production, and placing of these and any other special concretes in comparison with conventional concrete should be considered during the evaluation process. While the addition of steel fibers would be expected to increase the impact resistance of concrete, fiber-reinforced concrete is consistently less resistant to abrasion-erosion than conventional concrete. Therefore, fiber-reinforced concrete should not be used for repair of stilling basins or other hydraulic structures where abrasion-erosion is of major concern. Several types of surface coatings have exhibited good abrasion-erosion resistance during laboratory tests. These include polyurethanes, epoxy-resin mortar, furan-resin mortar, acrylic mortar, and iron aggregate toppings. However, some difficulties have been reported in field applications of surface coatings, primarily the result of improper surface preparation and thermal incompatibility between coatings and concrete.

g. Cavitation. Cavitation-erosion is the result of relatively complex flow characteristics of water over concrete surfaces (ACI 210R).

(1) Mechanism. There is little evidence to show that water flowing over concrete surfaces at velocities less

than 12.2 m/sec (40 ft/sec) causes any cavitation damage to the concrete. However, when the flow is fast enough (greater than 12.2 m/sec) and where there is surface irregularity in the concrete, cavitation damage may occur. Whenever there is surface irregularity, the flowing water will separate from the concrete surface. In the area of separation from the concrete, vapor bubbles will develop because of the lowered vapor pressure in the region. As these bubbles are carried downstream, they will soon reach areas of normal pressure. These bubbles will collapse with an almost instantaneous reduction in volume. This collapse, or implosion, creates a shock wave which, upon reaching a concrete surface, induces very high stresses over a small area. The repeated collapse of vapor bubbles on or near the concrete surface will cause pitting. Concrete spillways and outlet works of many high dams have been severely damaged by cavitation.

(2) Symptoms. Concrete that has been damaged will be severely pitted and extremely rough (Figure 2-21). As the damage progresses, the roughness of the damaged area may induce additional cavitation.

(3) Prevention.

(a) Hydraulic design. Even the strongest materials cannot withstand the forces of cavitation indefinitely. Therefore, proper hydraulic design and the use of aeration to reduce or eliminate the parameters that trigger cavitation are extremely important (ACI 210R). Since these topics are beyond the scope of this manual, hydraulic engineers and appropriate hydraulic design manuals should be consulted.

(b) Conventional materials. While proper material selection can increase the cavitation resistance of concrete, the only totally effective solution is to reduce or eliminate the causes of cavitation. However, it is recognized that in the case of existing structures in need of repair, the reduction or elimination of cavitation may be difficult and costly. The next best solution is to replace the damaged concrete with more cavitation-resistant materials. Cavitation resistance of concrete can be increased by use of a properly designed low w/c, high-strength concrete. The use of no larger than 38-mm (1-1/2-in.) nominal maximum size aggregate is beneficial. Furthermore, methods which have reduced the unit water content of the mixture, such as use of a water-reducing admixture, are also beneficial. Vital to increased cavitation resistance are the use of hard, dense aggregate particles and a good aggregate-to-mortar bond. Typically, cement-based materials exhibit significantly lower resistance to cavitation compared to polymer-based materials.

(c) Other cavitation-resistant materials. Cavitation-damaged areas have been successfully repaired with steel-fiber concrete and polymer concrete (Houghton, Borge, and Paxton 1978). Some coatings, such as neoprene and polyurethane, have reduced cavitation damage to concrete, but since near-perfect adhesion to the concrete is critical, the use of the coatings is not common. Once a tear or a chip in the coating occurs, the entire coating is likely to be peeled off.

(d) Construction practices. Construction practices are of paramount importance when concrete surfaces are exposed to high-velocity flow, particularly if aeration devices are not incorporated in the design. Such surfaces must be as smooth as can be obtained under practical conditions. Accordingly, good construction practices as given in EM 1110-2-2000 should be followed whether the construction is new or is a repair. Formed and unformed surfaces should be carefully checked during each construction operation to confirm that they are within specified tolerances. More restrictive tolerances on surfaces should be avoided since they become highly expensive to construct and often impractical to achieve, despite the use of modern equipment and good construction practices. Where possible, transverse joints in concrete conduits or chutes should be minimized. These joints are generally in a location where the greatest problem exists in maintaining a continuously smooth hydraulic surface. One construction technique which has proven satisfactory in placement of reasonably smooth hydraulic surfaces is the traveling slipform screed. This technique can be applied to tunnel inverts and to spillway chute slabs. Hurd (1989) provides information on the slipform screed. Since surface hardness improves cavitation resistance, proper curing of these surfaces is essential.

h. Freezing and thawing.

(1) Mechanism. As the temperature of a critically saturated concrete is lowered during cold weather, the freezable water held in the capillary pores of the cement paste and aggregates expands upon freezing. If subsequent thawing is followed by refreezing, the concrete is further expanded, so that repeated cycles of freezing and thawing have a cumulative effect. By their very nature, concrete hydraulic structures are particularly vulnerable to freezing and thawing simply because there is ample opportunity for portions of these structures to become critically saturated. Concrete is especially vulnerable in areas of fluctuating water levels or under spraying conditions. Exposure in such areas as the tops of walls, piers, parapets, and slabs enhances the vulnerability of concrete to the harmful effects of repeated cycles of freezing and

thawing. The use of deicing chemicals on concrete surfaces may also accelerate damage caused by freezing and thawing and may lead to pitting and scaling. ACI 201.2R describes the action as physical. It involves the development of osmotic and hydraulic pressures during freezing, principally in the paste, similar to ordinary frost action.

(2) Symptoms. Visual examination of concrete damaged by freezing and thawing may reveal symptoms ranging from surface scaling to extensive disintegration (Figure 2-10). Laboratory examination of cores taken from structures that show surficial effects of freezing and thawing will often show a series of cracks parallel to the surface of the structure.

(3) Prevention. The following preventive measures are recommended by ACI 201.2R for concrete that will be exposed to freezing and thawing while saturated:

(a) Designing the structure to minimize the exposure to moisture. For example, providing positive drainage rather than flat surfaces whenever possible.

(b) Using a concrete with a low w/c.

(c) Using adequate entrained air to provide a satisfactory air-void system in the concrete, i.e., a bubble spacing factor of 0.20 mm (0.008 in.) or less, which will provide protection for the anticipated service conditions and aggregate size. EM 1110-2-2000 provides information on the recommended amount of entrained air.

(d) Selecting suitable materials, particularly aggregates that perform well in properly proportioned concrete.

(e) Providing adequate curing to ensure that the compressive strength of the concrete is at least 24 MPa (3,500 psi) before the concrete is allowed to freeze in a saturated state.

i. Settlement and movement.

(1) Mechanisms.

(a) Differential movement. Situations in which the various elements of a structure are moving with respect to one another are caused by differential movements. Since concrete structures are typically very rigid, they can tolerate very little differential movement. As the differential movement increases, concrete members can be expected to be subjected to an overstressed condition. Ultimately, the members will crack or spall.

(b) Subsidence. Situations in which an entire structure is moving or a single element of a structure, such as a monolith, is moving with respect to the remainder of the structure are caused by subsidence. In these cases, the concerns are not overcracking or spalling but rather stability against overturning or sliding. Whether portions of a single structural element are moving with respect to one another or whether entire elements are moving, the underlying cause is more than likely to be a failure of the foundation material. This failure may be attributed to long-term consolidations, new loading conditions, or to a wide variety of other mechanisms. In situations in which structural movement is diagnosed as a cause of concrete deterioration, a thorough geotechnical investigation should be conducted.

(2) Symptoms. Visual examination of structures undergoing settlement or movement will usually reveal cracking or spalling or faulty alignment of structural members. Very often, movement will be apparent in nonstructural members such as block or brick masonry walls. Another good indication of structural movement is an increase in the amount of water leaking into the structure. Since differential settlement of the foundation of a structure is usually a long-term phenomenon, review of instrumentation data will be helpful in determining whether apparent movement is real. Review by structural and geotechnical engineering specialists will be required.

(3) Prevention. Prevention of settlements and movements or corrective measures are beyond the scope of this manual. Appropriate structural and geotechnical engineering manuals should be consulted for guidance.

j. Shrinkage. Shrinkage is caused by the loss of moisture from concrete. It may be divided into two general categories: that which occurs before setting (plastic shrinkage) and that which occurs after setting (drying shrinkage). Each of these types of shrinkage is discussed in this section.

(1) Plastic shrinkage.

(a) Mechanism. During the period between placing and setting, most concrete will exhibit bleeding to some degree. Bleeding is the appearance of moisture on the surface of the concrete; it is caused by the settling of the heavier components of the mixture. Usually, the bleed water evaporates slowly from the concrete surface. If environmental conditions are such that evaporation is occurring faster than water is being supplied to the surface by bleeding, high tensile stresses can develop. These stresses can lead to the development of cracks on the concrete surface.

(b) Symptoms. Cracking caused by plastic shrinkage will be seen within a few hours of concrete placement. Typically, the cracks are isolated rather than patterned. These cracks are generally wide and shallow.

(c) Prevention. Determination of whether the weather conditions on the day of the placement are conducive to plastic shrinkage cracking is necessary. If the predicted evaporation rate is high according to ACI 305R, appropriate actions such as erecting windbreaks, erecting shade over the placement, cooling the concrete, and misting should be taken after placement. Additionally, it will be beneficial to minimize the loss of moisture from the concrete surface between placing and finishing. Finally, curing should be started as soon as is practical. If cracking caused by plastic shrinkage does occur and if it is detected early enough, revibration and refinishing of the cracked area will resolve the immediate problem of the cracks. Other measures as described above will be required to prevent additional occurrences.

(2) Drying shrinkage.

(a) Mechanism. Drying shrinkage is the long-term change in volume of concrete caused by the loss of moisture. If this shrinkage could take place without any restraint, there would be no damage to the concrete. However, the concrete in a structure is always subject to some degree of restraint by either the foundation, by another part of the structure, or by the difference in shrinkage between the concrete at the surface and that in the interior of a member. This restraint may also be attributed to purely physical conditions such as the placement of a footing on a rough foundation or to chemical bonding of new concrete to earlier placements or to both. The combination of shrinkage and restraints cause tensile stresses that can ultimately lead to cracking.

(b) Symptoms. Visual examination will typically show cracks that are characterized by their fineness and absence of any indication of movement. They are usually shallow, a few inches in depth. The crack pattern is typically orthogonal or blocky. This type of surface cracking should not be confused with thermally induced deep cracking which occurs when dimensional change is restrained in newly placed concrete by rigid foundations or by old lifts of concrete.

(c) Prevention. In general, the approach is either to reduce the tendency of the concrete, to shrink or to reduce the restraint, or both. The following will help to reduce the tendency to shrink: use of less water in the concrete; use of larger aggregate to minimize paste content; placing the concrete at as low a temperature as practical; dampening the subgrade and the forms; dampening aggregates if they are dry and absorptive; and providing an adequate amount of reinforcement to distribute and reduce the size of cracks that do occur. Restraint can be reduced by providing adequate contraction joints.

k. Temperature changes. Changes in temperature cause a corresponding change in the volume of concrete. As was true for moisture-induced volume change (drying shrinkage), temperature-induced volume changes must be combined with restraint before damage can occur. Basically, there are three temperature change phenomena that may cause damage to concrete. First, there are the temperature changes that are generated internally by the heat of hydration of cement in large placements. Second, there are the temperature changes generated by variations in climatic conditions. Finally, there is a special case of externally generated temperature change--fire damage. Internally and externally generated temperature changes are discussed in subsequent paragraphs. Because of the infrequent nature of its occurrence in civil works structures, fire damage is not included in this manual.

(1) Internally generated temperature differences.

(a) Mechanism. The hydration of portland cement is an exothermic chemical reaction. In large volume placements, significant amounts of heat may be generated and the temperature of the concrete may be raised by more than 38 °C (100 °F) over the concrete temperature at placement. Usually, this temperature rise is not uniform throughout the mass of the concrete, and steep temperature gradients may develop. These temperature gradients give rise to a situation known as internal restraint--the outer portions of the concrete may be losing heat while the inner portions are gaining (heat). If the differential is great, cracking may occur. Simultaneously with the development of this internal restraint condition, as the concrete mass begins to cool, a reduction in volume takes place. If the reduction in volume is prevented by external conditions (such as by chemical bonding, by mechanical interlock, or by piles or dowels extending into the concrete), the concrete is externally restrained. If the strains induced by the external restraint are great enough, cracking may occur. There is increasing evidence, particularly for rehabilitation work, that relatively minor temperature differences in thin, highly restrained overlays can lead to cracking. Such cracking has been seen repeatedly in lock wall resurfacing (Figure 2-5) and in stilling basin overlays. Measured temperature differentials have typically been much below those normally associated with thermally induced cracking.

(b) Symptoms. Cracking resulting from internal restraint will be relatively shallow and isolated. Cracking resulting from external restraint will usually extend through the full section. Thermally induced cracking may be expected to be regularly spaced and perpendicular to the larger dimensions of the concrete.

(c) Prevention. An in-depth discussion of temperature and cracking predictions for massive placements can be found in ACI 207.1R and ACI 207.2R. In general, the following may be beneficial: using as low a cement content as possible; using a low-heat cement or combination of cement and pozzolans; placing the concrete at the minimum practical temperature; selecting aggregates with low moduli of elasticity and low coefficients of thermal expansion; cooling internally or insulating the placement as appropriate to minimizing temperature differentials; and minimizing the effects of stress concentrators that may instigate cracking.

(2) Externally generated temperature differences.

(a) Mechanism. The basic failure mechanism in this case is the same as that for internally generated temperature differences--the tensile strength of the concrete is exceeded. In this case the temperature change leading to the concrete volume change is caused by external factors, usually changing climatic conditions. This cause of deterioration is best described by the following examples: First, a pavement slab cast in the summer. As the air and ground temperatures drop in the fall and winter, the slab may undergo a temperature drop of 27 °C (80 °F), or more. Typical parameters for such a temperature drop (coefficient of thermal expansion of 10.8×10^{-6}/°C (6×10^{-6}/°F) indicate a 30-m (98-ft) slab would experience a shortening of more than 13 mm (1/2 in.). If the slab were restrained, such movement would certainly lead to cracking. Second, a foundation or retaining wall that is cast in the summer. In this case, as the weather cools, the concrete may cool at different rates--exposed concrete will cool faster than that insulated by soil or other backfill. The restraint provided by this differential cooling may lead to cracking if adequate contraction joints have not been provided. Third, concrete that experiences significant expansion during the warmer portions of the year. Spalling may occur if there are no adequate expansion joints. In severe cases, pavement slabs may be lifted out of alignment, resulting in so-called blowups. Fourth, concretes that have been repaired or overlayed with materials that do not have the same coefficient of thermal expansion as the underlying material. Annual heating and cooling may lead to cracking or debonding of the two materials.

(b) Symptoms. Visual examination will show regularly spaced cracking in the case of restrained contraction. Similarly, spalling at expansion joints will be seen in the case of restrained expansion. Problems resulting from expansion-contraction caused by thermal differences will be seen as pattern cracking, individual cracking, or spalling.

(c) Prevention. The best prevention is obviously to make provision for the use of contraction and expansion joints. Providing reinforcing steel (temperature steel) will help to distribute cracks and minimize the size of those that do occur. Careful review of the properties of all repair materials will help to eliminate problems caused by temperature changes.

l. Weathering. Weathering is frequently referred to as a cause of concrete deterioration. ACI 116R defines weathering as “Changes in color, texture, strength, chemical composition, or other properties of a natural or artificial material due to the action of the weather.” However, since all of these effects may be more correctly attributed to other causes of concrete deterioration described in this chapter, weathering itself is not considered to be a specific cause of deterioration.

3-3. Relating Symptoms to Causes of Distress and Deterioration

Given a detailed report of the condition of the concrete in a structure and a basic understanding of the various mechanisms that can cause concrete deterioration, the problem becomes one of relating the observations or symptoms to the underlying causes. When many of the different causes of deterioration produce the same symptoms, the task of relating symptoms to causes is more difficult than it first appears. One procedure to consider is based upon that described by Johnson (1965). This procedure is obviously idealized and makes no attempt to deal with more than one cause that may be active at any one time. Although there will usually be a combination of causes responsible for the damage detected on a structure, this procedure should provide a starting point for an analysis.

a. Evaluate structure design to determine adequacy. First consider what types of stress could have caused the observed symptoms. For example, tension will cause cracking, while compression will cause spalling. Torsion or shear will usually result in both cracking and spalling. If the basic symptom is disintegration, then overstress may be eliminated as a cause. Second, attempt to relate the probable types of stress causing the damage noted to the locations of the damage. For example, if cracking resulting from excessive tensile stress is suspected, it would not be consistent to find that type of damage in an area that is under compression. Next, if the damage seems appropriate for the location, attempt to relate the specific orientation of the damage to the stress pattern. Tension cracks should be roughly perpendicular to the line of externally induced stress. Shear usually causes failure by diagonal tension, in which the cracks will run diagonally in the concrete section. Visualizing the basic stress patterns in the structure will aid in this phase of the evaluation. If no inconsistency is encountered during this evaluation, then overstress may be the cause of the observed damage. A thorough stress analysis is warranted to confirm this finding. If an inconsistency has been detected, such as cracking in a compression zone, the next step in the procedure should be followed.

b. Relate the symptoms to potential causes. For this step, Table 3-3 will be of benefit. Depending upon the symptom, it may be possible to eliminate several possible causes. For example, if the symptom is disintegration or erosion, several potential causes may be eliminated by this procedure.

c. Eliminate the readily identifiable causes. From the list of possible causes remaining after symptoms have been related to potential causes, it may be possible to eliminate two causes very quickly since they are relatively easy to identify. The first of these is corrosion of embedded metals. It will be easy to verify whether the cracking and spalling noted are a result of corrosion. The second cause that is readily identified is accidental loading, since personnel at the structure should be able to relate the observed symptoms to a specific incident.

d. Analyze the available clues. If no solution has been reached at this stage, all of the evidences generated by field and laboratory investigations should be carefully reviewed. Attention should be paid to the following points:

(1) If the basic symptom is that of disintegration of the concrete surface, then essentially three possible causes remain: chemical attack, erosion, and freezing and thawing. Attempts should be made to relate the nature and type of the damage to the location in the structure and to the environment of the concrete in determining which of the three possibilities is the most likely to be the cause of the damage.

(2) If there is evidence of swelling of the concrete, then there are two possibilities: chemical reactions and temperature changes. Destructive chemical reactions such as alkali-silica or alkali-carbonate attack that cause swelling will have been identified during the laboratory investigation. Temperature-induced swelling should be ruled out unless there is additional evidence such as spalling at joints.

(3) If the evidence is spalling and corrosion and accidental loadings have been eliminated earlier, the major causes of spalling remaining are construction errors, poor detailing, freezing and thawing, and externally generated temperature changes. Examination of the structure should have provided evidence as to the location and general nature of the spalling that will allow identification of the exact cause.

(4) If the evidence is cracking, then construction errors, shrinkage, temperature changes, settlement and movement, chemical reactions, and poor design details remain as possible causes of distress and deterioration of concrete. Each of these possibilities will have to be reviewed in light of the available laboratory and field observations to establish which is responsible.

(5) If the evidence is seepage and it has not been related to a detrimental internal chemical reaction by this time, then it is probably the result of design errors or construction errors, such as improper location or installation of a waterstop.

e. Determine why the deterioration has occurred. Once the basic cause or causes of the damage have been established, there remains one final requirement: to understand how the causal agent acted upon the concrete. For example, if the symptoms were cracking and spalling and the cause was corrosion of the reinforcing steel, what facilitated the corrosion? Was there chloride in the concrete? Was there inadequate cover over the reinforcing steel? Another example to consider is concrete damage caused by freezing and thawing. Did the damage occur because the concrete did not contain an adequate air-void system, or did the damage occur because the concrete used was not expected to be saturated but, for whatever reason, was saturated? Only when the cause and its mode of action are completely understood should the next step of selecting a repair material be attempted.

Appendix A
References

A-1. Required Publications

TM 5-822-6/AFM 88-7, Chapter 1
Rapid Pavements for Roads, Streets, Walks, and Open Storage Areas

TM 5-822-9/AFM 88-6, Chapter 10
Repair of Rigid Pavements Using Epoxy-Resin Grouts, Mortars, and Concrete

EP 1110-1-10
Borehole Viewing Systems

EM 385-1-1
Safety and Health Requirements Manual

EM 1110-1-3500
Chemical Grouting

EM 1110-2-2000
Standard Practice for Concrete

EM 1110-2-2005
Standard Practice for Shotcrete

EM 1110-2-2006
Roller-Compacted Concrete

EM 1110-2-2102
Waterstops and Other Joint Materials

EM 1110-2-3506
Grouting Technology

EM 1110-2-4300
Instrumentation for Concrete Structures CH1

U.S. Army Engineer Waterways Experiment Station 1949
U.S. Army Engineer Waterways Experiment Station. 1949 (Aug). *Handbook for Concrete and Cement*, with quarterly supplements (all CRD-C designations), Vicksburg, MS. Note: Use latest edition of all designations.

U.S. Army Engineer Waterways Experiment Station 1985
U.S. Army Engineer Waterways Experiment Station. 1985. *The REMR Notebook*, with periodic supplements, Vicksburg, MS, including:

a. "Video Systems for Underwater Inspection of Structures," REMR Technical Note CS-ES-2.6

b. "Underwater Cameras for Inspection of Structures in Turbid Water," REMR Technical Note CS-ES-3.2

c. "Removal and Prevention of Efflorescence on Concrete and Masonry Building Surfaces," REMR Technical Note CS-MR-4.3

d. "Cleaning Concrete Surfaces," REMR Technical Note CS-MR-4.4

e. "General Information of Polymer Materials," REMR Technical Note CS-MR-7.1

f. "Antiwashout Admixtures for Underwater Concrete," REMR Technical Note CS-MR-7.2

g. "Rapid-Hardening Cements and Patching Materials," REMR Technical Note CS-MR-7.3

h. "Handling and Disposal of Construction Residue," REMR Technical Note EI-M-1.2

Handbooks and reports published by the Waterways Experiment Station may be obtained from: U.S. Army Engineer Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

American Concrete Institute (Annual)
American Concrete Institute. Annual. *Manual of Concrete Practice*, Five Parts, Detroit, MI, including:

"Cement and Concrete Terminology," ACI 116R

"Guide for Making a Condition Survey of Concrete in Service," ACI 201.lR

"Guide to Durable Concrete," ACI 201.2R

"Mass Concrete," ACI 207.lR

"Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete," ACI 207.2R

"Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions," ACI 207.3R

"Roller Compacted Mass Concrete," ACI 207.5R

"Erosion of Concrete in Hydraulic Structures," ACI 210R

"Corrosion of Metals in Concrete," ACI 222R

"Standard Practice for the Use of Shrinkage-Compensating Concrete," ACI 223

"Causes, Evaluation, and Repair of Cracks in Concrete Structures," ACI 224.1R

"Guide for Concrete Floor and Slab Construction," ACI 302.1R

"Guide for Measuring, Mixing, Transporting, and Placing Concrete," ACI 304R

"Guide for the Use of Preplaced Aggregate Concrete for Structural and Mass Concrete Applications," ACI 304.1R

"Placing Concrete by Pumping Methods," ACI 304.2R

"Hot Weather Concreting," ACI 305R

"Building Code Requirements for Reinforced Concrete," ACI 318

"Guide for the Design and Construction of Concrete Parking Lots," ACI 330R

"Guide to Residential Cast-in-Place Concrete Construction," ACI 332R

"State-of-the-Art Report on Anchorage to Concrete," ACI 355.1R

"State-of-the-Art Report on High-Strength Concrete," ACI 363R

"Guide for Evaluation of Concrete Structures Prior to Rehabilitation," ACI 364.1R

"Use of Epoxy Compounds with Concrete," ACI 503R

"Standard Specification for Bonding Plastic Concrete to Hardened Concrete With a Multi-Component Epoxy Adhesive," ACI 503.2

"Guide for the Selection of Polymer Adhesives with Concrete," ACI 503.5R

"Guide to Sealing Joints in Concrete Structures," ACI 504R

"Guide to Shotcrete," ACI 506R

"A Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete," ACI 515.lR

"Guide for Specifying, Proportioning, Mixing, Placing, and Finishing Steel," ACI 544.3R

"Guide for the Use of Polymers in Concrete," ACI 548.1R

"State-of-the-Art Report on Polymer-Modified Concrete," ACI 548.3R

"Standard Specification for Latex-Modified Concrete (LMC) Overlays," ACI 548.4

ACI 226 1987
ACI Committee 226. 1987 (Mar-Apr). "Silica Fume in Concrete," *ACI Materials Journal*, Vol 84, No. 2, pp 158-166.

ACI publications may be obtained from: American Concrete Institute, Member/Customer Services Department, Box 19150, Detroit, MI 48219-0150.

American Society for Testing and Materials (Annual)
American Society for Testing and Materials. Annual. *Annual Book of ASTM Standards*, Philadelphia, PA. Note: Use the latest available issue of each ASTM standard.

ASTM Standards and Publications may be obtained from: American Society for Testing and Materials, 1916 Race Street, Philadelphia, PA 19103.

A-2. Related Publications

ABAM Engineers 1987a
ABAM Engineers, Inc. 1987a (Jul). "Design of a Precast Concrete Stay-in-Place Forming System for Lock Wall Rehabilitation," Technical Report REMR-CS-7, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

ABAM Engineers 1987b
ABAM Engineers, Inc. 1987b (Dec). "A Demonstration of the Constructibility of a Precast Concrete Stay-in-Place Forming System for Lock Wall Rehabilitation," Technical Report REMR-CS-14, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

ABAM Engineers 1989
ABAM Engineers, Inc. 1989 (Dec). "Concepts for Installation of the Precast Concrete Stay-in-Place Forming System for Lock Wall Rehabilitation in an Operational Lock," Technical Report REMR-CS-28, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Ahmad and Haskins 1993
Ahmad, Falih H., and Haskins, Richard. 1993 (Sep). "Use of Ground-Penetrating Radar in Nondestructive Testing for Voids and Cracks in Concrete," The REMR Bulletin, Vol 10, No. 3, pp 11-15.

Alexander 1980
Alexander, A. M. 1980 (Apr). "Development of Procedures for Nondestructive Testing of Concrete Structures; Report 2, Feasibility of Sonic Pulse-Echo Technique," Miscellaneous Paper C-77-11, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Alexander 1993
Alexander, A. Michel. 1993 (Apr). "Impacts on a Source of Acoustic Pulse-Echo Energy for Nondestructive Testing of Concrete Structures," Technical Report REMR-CS-40, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Alexander and Thornton 1988
Alexander, A. M., and Thornton, H. T., Jr. 1988. "Developments in Ultrasonic Pitch-Catch and Pulse-Echo for Measurements in Concrete," SP-112, American Concrete Institute, Detroit, MI.

Alongi, Cantor, Kneeter, Alongi 1982
Alongi, A. V., Cantor, T. R., Kneeter, C. P., and Alongi, A., Jr. 1982. "Concrete Evaluation by Radar Theoretical Analysis," *Concrete Analysis and Deterioration*, Transportation Research Board, Washington, DC.

Anderson 1984
Anderson, Fred A. 1984 (May). "RCC Does More," *Concrete International*, American Concrete Institute, Vol 6, No. 5, pp 35-37.

Bach and Isen 1968
Bache, H. H., and Isen, J. C. 1968 (Jun). "Model Determination of Concrete Resistance to Popout Formation," *Journal of the American Concrete Institute*, *Proceedings*, Vol 65, pp 445-450.

Bean 1988
Bean, Dennis L. 1988 (Apr). "Surface Treatments to Minimize Concrete Deterioration; Report 1, Survey of Field and Laboratory Application and Available Products," Technical Report REMR-CS-17, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Best and McDonald 1990a
Best, J. Floyd, and McDonald, James E. 1990 (Jan). "Spall Repair of Wet Concrete Surfaces," Technical Report REMR-CS-25, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Best and McDonald 1990b
Best, J. Floyd, and McDonald, James E. 1990 (Jan). "Evaluation of Polyester Resin, Epoxy, and Cement Grouts for Embedding Reinforcing Steel Bars in Hardened Concrete," Technical Report REMR-CS-23, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bischoff and Obermeyer 1993
Bischoff, John A., and Obermeyer, James R. 1993 (Apr). "Design Considerations for Raising Existing Dams for Increased Storage," *Geotechnical Practice in Dam Rehabilitation*, American Society of Civil Engineers, pp 174-187.

Bryant and Mlakar 1991
Bryant, Larry M., and Mlakar, Paul F. 1991 (Mar). "Predicting Concrete Service Life in Cases of Deterioration Due to Freezing and Thawing," Technical Report REMR-CS-35, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Busby Associates, Inc. 1987
Busby Associates, Inc. 1987. "Undersea Vehicles Directory," Arlington, TX.

Campbell 1982
Campbell, Roy L., Sr. 1982 (Apr). "A Review of Methods for Concrete Removal," Technical Report SL-82-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Campbell 1994
Campbell, Roy L., Sr. 1994 (Feb). "Overlays on Horizontal Concrete Surfaces: Case Histories," Technical Report REMR-CS-42, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Carino 1992
Carino, Nicholas J. 1992 (Jan). "Recent Developments in Nondestructive Testing of Concrete," *Advances in Concrete Technology*, Canada Center for Mineral and Energy Technology, Ottawa, Canada, pp 281-328.

Carter 1993
Carter, Paul. 1993 (Jan). "Developing a Performance-Based Specification for Concrete Sealers on Bridges," *Journal of Protective Coatings & Linings*, pp 36-44.

Cazzuffi 1987
Cazzuffi, D. 1987 (Mar). "The Use of Geomembranes in Italian Dams," *Water Power & Dam Construction.*

Clausner and Pope 1988
Clausner, J. E., and Pope, J. 1988 (Nov.) "Side-Scan Sonar Applications for Evaluating Coastal Structures," Technical Report CERC-88-16, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Clear and Chollar 1978
Clear, K. C., and Choller, B. H. 1978 (Apr). "Styrene-Butadiene Latex Modifiers for Bridge Peak Overlay Concrete," Report No. FHWA-RD-35, Federal Highway Administration, Washington, DC.

Clifton 1991
Clifton, James R. 1991 (Nov). "Predicting the Remaining Service Life of Concrete," Report NISTIR 4712, National Institute of Standards Technology, Gaithersburg, MD.

Concrete Repair Digest 1993
Concrete Repair Digest. 1993 (Feb/Mar). "Removing Some Common Stains from Concrete."

Dahlquist 1987
Dahlquist, M. S. 1987 (Oct). *REMR Bulletin*, Vol 4, No. 2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Davis 1960
Davis, R. E., "Prepakt Method of Concrete Repair," *ACI Journal,* Vol 32, pp 155-1752.

Davis, Jansen, and Neelands 1948
Davis, R. E., Jansen, E. C., and Neelands, W. T. 1948 (Apr). "Restoration of Barker Dam," *ACI Journal*, Vol 19, No. 8, pp 633-688.

Dobrowolski and Scanlon 1984
Dobrowolski, Joseph A., and Scanlon, John M. 1984 (Jun). "How to Avoid Deficiencies in Architectural Concrete Construction," Technical Report SL-84-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Emmons 1993
Emmons, Peter H. 1993. *Concrete Repair and Maintenance Illustrated*, R. S. Means Co., Inc., Kingston, MA, 295 pp.

Emmons and Vaysburd 1995
Emmons, Peter H., and Vaysburd, Alexander M. 1995 (Mar). "Performance Criteria for Concrete Repair Materials, Phase I," Technical Report REMR-CS-47, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Emmons, Vaysburd, and McDonald 1993
Emmons, Peter H., Vaysburd, Alexander M., and McDonald, James E. 1993 (Sep). "A Rational Approach to Durable Concrete Repairs," *Concrete International*, Vol 15, No. 9, pp 40-45.

Emmons, Vaysburd, and McDonald 1994
Emmons, P. H., Vaysburd, A. M., and McDonald, J. E. 1994 (Mar). "Concrete Repair in The Future Turn Of The Century - Any Problems?," *Concrete International*, Vol 16, No. 3, pp 42-49.

Fenwick 1989
Fenwick, W. B. 1989 (Aug). "Kinzua Dam, Allegheny River, Pennsylvania and New York; Hydraulic Model Investigation," Technical Report HL-89-17, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Gerwick 1988
Gerwick, B. C. 1988 (Sep). "Review of the State of the Art for Underwater Repair Using Abrasion-Resistant Concrete," Technical Report REMR-CS-19, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Gurjar and Carter 1987
Gurjar, Suresh, and Carter, Paul. 1987 (Mar). "Alberta Concrete Patch Evaluation Program," Report No. ABTR/RD/RR-87/05, Alberta Transportation & Utilities, Edmonton, Alberta, Canada.

Hacker 1986
Hacker, Kathy. 1986 (Dec). "Precast Panels Speed Rehabilitation of Placer Creek Channel," *The REMR Bulletin*, Vol 3, No. 3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hammonds, Garner, and Smith 1989
Hammons, M. I., Garner, S. B., and Smith, D. M. 1989 (Jun). "Thermal Stress Analysis of Lock Wall, Dashields

Locks, Ohio River," Technical Report SL-89-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hansen 1989
Hansen, Kenneth D. 1989 (Oct). "Performance of Roller-Compacted Concrete Dam Rehabilitations," *Proceedings from the 6th ASDSO Annual Conference*, Association of State Dam Safety Officials, pp 21-26.

Hansen and France 1986
Hansen, Kenneth D., and France, John W. 1986 (Sep). "RCC: A Dam Rehab Solution Unearthed,"*Civil Engineering*, American Society of Civil Engineers, Vol 56, No. 9, pp 60-63.

Hepler 1992
Hepler, Thomas E. 1992. "RCC Buttress Construction for Camp Dyer Diversion Dam," *Proceedings from the 9th ASDSO Annual Conference*, Association of State Dam Safety Officials, pp 21-26.

Holland 1983
Holland, T. C. 1983 (Sep). "Abrasion-Erosion Evaluation of Concrete Mixtures for Stilling Basin Repairs, Kinzua Dam, Pennsylvania," Miscellaneous Paper SL-83-16, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Holland 1986
Holland, T. C. 1986 (Sep). "Abrasion-Erosion Evaluation of Concrete Mixtures for Stilling Basin Repairs, Kinzua Dam, Pennsylvania," Miscellaneous Paper SL-86-14, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Holland and Gutschow 1987
Holland, Terence C., and Gutschow, Richard A. 1987 (Mar). "Erosion Resistance with Silica-Fume Concrete," *Concrete International*, Vol 9, No. 3, pp 32-40.

Holland, Husbands, Buck, and Wong 1980
Holland, T. C., Husbands, T. B., Buck, A. D., and Wong, G. S. 1980 (Dec). "Concrete Deterioration in Spillway Warm-Water Chute, Raystown Dam, Pennsylvania," Miscellaneous Paper SL-80-19, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Holland, Krysa, Luther, and Liu 1986
Holland, T. C., Krysa, A., Luther, M. D., and Liu, T. C. 1986. "Use of Silica-Fume Concrete to Repair Abrasion-Erosion Damage in the Kinzua Dam Stilling Basin," *Fly Ash, Silica Fume, Slag, and Natural Pozzolans in Concrete*, SP-91, Vol 2, American Concrete Institute, Detroit, MI.

Holland and Turner 1980
Holland, T. C., and Turner, J. R. 1980 (Sep). "Construction of Tremie Concrete Cutoff Wall, Wolf Creek Dam, Kentucky," Miscellaneous Paper SL-80-10, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Houghton, Borge, and Paxton 1978
Houghton, D. L., Borge, O. E., and Paxton, J. H. 1978 (Dec). "Cavitation Resistance of Some Special Concretes," *ACI Journal*, Vol 75, No. 12, pp 664-667.

Hulshizer and Desai 1984
Hulshizer, A. J., and Desai, A. J. 1984 (Jun). "Shock Vibration Effects on Freshly Placed Concrete," *ASCE Journal of Construction Engineering and Management*, Vol 110, No. 2, pp 266-285.

Hurd 1989
Hurd, M. K. 1989. "Formwork for Concrete," 5th Edition, SP-4, American Concrete Institute, Detroit, MI.

Husbands and Causey 1990
Husbands, Tony B., and Causey, Fred E. 1990 (Sep). "Surface Treatments to Minimize Concrete Deterioration; Report 2, Laboratory Evaluation of Surface Treatment Materials," Technical Report REMR-CS-17, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

ICOLD 1991
ICOLD. 1991. *Watertight Geomembranes for Dams - State of the Art*, Bulletin 78, International Commission on Large Dams, Paris.

Johnson 1965
Johnson, S. M. 1965. *Deterioration, Maintenance, and Repair of Structures*, McGraw-Hill, New York.

Kahl, Kauschinger, and Perry 1991
Kahl, Thomas W., Kauschinger, Joseph L., and Perry, Edward B. 1991 (Mar). "Plastic Concrete Cutoff Walls for Earth Dams," Technical Report REMR-GT-15, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Keeney 1987
Keeney, C. A. 1987 (Nov). "Procedures and Devices for Underwater Cleaning of Civil Works Structures," Technical Report REMR-CS-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Khayat 1991
Khayat, K. H. 1991. "Underwater Repair of Concrete Damaged by Abrasion-Erosion," Technical Report REMR-CS-37, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Kottke 1987
Kottke, Edgar. 1987 (Aug). "Evaluation of Sealers for Concrete Bridge Elements," Alberta Transportation and Utilities, Alberta, Canada.

Krauss 1994
Krauss, P. D. 1994 (Mar). "Repair Materials and Techniques for Concrete Structures in Nuclear Power Plants," ORNL/NRC/LTR-93/28, Oak Ridge National Laboratory, Oak Ridge, TN.

Kucharski and Clausner 1990
Kucharski, W. M., and Clausner, J. E. 1990 (Feb). "Underwater Inspection of Coastal Structures Using Commercially Available Sonars," Technical Report REMR-CO-11, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Langelier 1936
Langelier, W. F. 1936 (Oct). "The Analytical Control of Anti-Corrosion Water Treatment," *Journal of the American Water Works Association*, Vol 28, No. 10, pp 1500-1521.

Lanigan 1992
Lanigan, Carl, A. 1992. "Continuous Deformation Monitoring System (CDMS)," Technical Report REMR-CS-39, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Lauer 1956
Lauer, K. R., and Slate, F. O. 1956 (Jun). "Autogenous Healing of Cement Paste," *ACI Journal,* Proceedings, Vol 27, No. 10, pp 1083-1098.

Liu 1980
Liu, T. C. 1980 (Jul). "Maintenance and Preservation of Concrete Structures; Report 3, Abrasion-Erosion Resistance of Concrete," Technical Report C-78-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Liu and Holland 1981
Liu, T. C., and Holland, T. C. 1981 (Mar). "Design of Dowels for Anchoring Replacement Concrete to Vertical Lock Walls," Technical Report SL-81-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Liu and McDonald 1981
Liu, T. C., and McDonald, J. E. 1981. "Abrasion-Erosion Resistance of Fiber-Reinforced Concrete," *Cement, Concrete, and Aggregates,* Vol 3, No. 2.

Mailvaganam 1992
Mailvaganam, Noel P. 1992. "Repair and Protection of Concrete Structures," CRC Press, London.

Malhotra 1976
Malhotra, V. M. 1976. "Testing Hardened Concrete: Nondestructive Methods," ACI Monograph No. 9, Detroit, MI.

Marold 1992
Marold, W. J. 1992 (Feb). "Design of the Boney Falls RCC Emergency Spillway," *Roller Compacted Concrete III*, American Society of Civil Engineers, pp 476-490.

McDonald 1980
McDonald, J. E. 1980 (Apr). "Maintenance and Preservation of Concrete Structures; Report 2, Repair of Erosion-Damaged Structures," Technical Report C-78-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McDonald 1986
McDonald, James E. 1986 (Nov). "Repair of Waterstop Failures: Case Histories," Technical Report REMR-CS-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McDonald 1987a
McDonald, James E. 1987 (Jul). "Precast Concrete Stay-in-Place Forming System for Lock Wall Rehabilitation," *The REMR Bulletin*, Vol 4, No. 1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McDonald 1987b
McDonald, J. E. 1987b (Dec). "Rehabilitation of Navigation Lock Walls: Case Histories," Technical Report REMR-CS-13, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McDonald 1988
McDonald, J. E. 1988 (Jul). "A Precast Stay-in-Place Forming System for Lock Wall Rehabilitation," Video Report REMR-CS-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McDonald 1989
McDonald, J. E. 1989 (Feb). "Evaluation of Vinylester Resin for Anchor Embedment in Concrete," Technical Report REMR-CS-20, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McDonald 1990
McDonald, J. E. 1990 (Oct). "Anchor Embedment in Hardened Concrete Under Submerged Conditions," Technical Report REMR-CS-33, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McDonald 1991
McDonald, James E. 1991 (Mar). "Properties of Silica-Fume Concrete," Technical Report REMR-CS-32, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McDonald and Curtis 1995
McDonald, J. E, and Curtis, N. 1995 (Apr). "Applications of Precast Concrete in Repair of Civil Works Structures." Technical Report (REMR-CS-48), U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Mech 1989
Mech, George J. 1989 (Oct). "Rehabilitation of Peoria Lock Using Preplaced-Aggregate Concrete," *The REMR Bulletin*, Vol 6, No. 4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Meyers 1994
Meyers, John G. 1994 (Aug/Sep). "Slabjacking Sunken Concrete," *Concrete Repair Digest*, Vol 6, No. 4, The Aberdeen Group, Addison, Il.

Miles 1993
Miles, William R. 1993 (Oct). "Comparison of Cast-in-Place Concrete Versus Precast Concrete Stay-in-Place Forming Systems for Lock Wall Rehabilitation," Technical Report REMR-CS-41, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Minnotte 1952
Minnotte, J. S. 1952 (Oct). "Lock No. 5 Monongahela River Refaced by Grout Intrusion Method," *Civil Engineering*, Vol 22, pp 872-875.

Mitscher 1992
Mitscher, Kurt A. 1992 (Dec). "Sheet-Pile and Precast Concrete U-Flume Low-Flow Channels for the Blue River Paved Reach Project," 1991 Corps of Engineer Structural Engineering Conference, pp 471-480.

Monari 1984
Monari, F. 1984. "Waterproof Covering for the Upstream of the Lago Nero Dam," *Proceedings, International Conference on Geomembranes*, Denver, CO, pp 105-110.

Monari and Scuero 1991
Monari, F., and Scuero, A. M. 1991. "Aging of Concrete Dams: The Use of Geocomposites for Repair and Future Protection," International Commission on Large Dams, 17th Congress, June 17-21.

Montani 1993
Montani, Rick. 1993 (May/Jun). "High Molecular Weight Methacrylates," *Concrete Repair Bulletin*, Vol 6, No. 3, pp 6-9.

Morang 1987
Morang, A. 1987 (Dec). "Side-Scan Sonar Investigation of Breakwaters at Calumet and Burns Harbors on Southern Lake Michigan," Miscellaneous Paper CERC-87-20, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Morey 1974
Morey, R. M. 1974 (Mar). "Application of Downward Looking Impulse Radar," *Proceedings of 13th Annual Canadian Hydrographic Conference*, Canada Center for Inland Waters, Burlington, Ontario.

NACE International 1991
National Association of Corrosion Engineers. 1991 (Oct). "Coatings for Concrete Surfaces in Non-Immersion and Atmospheric Services," Standard Recommended Practice RP0591-91, Houston,TX.

Neeley 1988
Neeley, B. D. 1988 (Apr). "Evaluation of Concrete Mixtures for Use in Underwater Repairs," Technical Report REMR-CS-18, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Neeley and Wickersham 1989
Neeley, B. D., and Wickersham, J. 1989 (Oct). "Repair of Red Rock Dam," *Concrete International: Design and Construction*, Vol 11, No. 10, American Concrete Institute, Detroit, MI.

Neeley, Saucier, and Thornton 1990
Neeley, B. D., Saucier, K. L., and Thornton, H. T., Jr. 1990 (Nov). "Laboratory Evaluation of Concrete Mixtures and Techniques for Underwater Repairs," Technical

Report REMR-CS-34, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Norman, Campbell, and Garner 1988
Norman, C. D., Campbell, R. L., Jr., and Garner, S. 1988 (Aug). "Analysis of Concrete Cracking in Lock Wall Resurfacing," Technical Report REMR-CS-15, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Oswalt 1971
Oswalt, N. R. 1971. "Pomona Dam Outlet Stilling Basin Modifications," Memorandum Report, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

PCA 1990
"Upper Michigan Dam Rehabilitated with RCC." 1990 (Spring/Summer). *RCC Newsletter*, Vol 5, No. 1, Portland Cement Association, Skokie, IL.

Pfeifer and Scali 1981
Pfeifer, D. W., and Scali, M. J. 1981 (Dec). "Concrete Sealers for Protection of Bridge Structures," National Cooperative Highway Research Program Report No. 244, Transportation Research Board, Washington, DC.

Pinney 1991
Pinney, Stephen G. 1991 (Feb). "Preparation, Application and Inspection of Coatings for Concrete," *REMR Bulletin,* Vol 8, No. 1, Vicksburg, MS.

Popovics and McDonald 1989
Popovics, S., and McDonald, W. E. 1989 (Apr). "Inspection of the Engineering Condition of Underwater Concrete Structures," Technical Report REMR-CS-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Rail and Haynes 1991
Rail, R. D., and Haynes, H, H. 1991 (Dec). "Underwater Stilling Basin Repair Techniques Using Precast or Prefabricated Elements," Technical Report REMR-CS-38, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Ramakrishnan 1992
Ramakrishnan, V. 1992 (Aug). "Latex-Modified Concretes and Mortars," National Cooperative Highway Research Program, Synthesis of Highway Practice 179, Transportation Research Board, Washington, DC.

Schrader 1980
Schrader, E. K. 1980 (Oct). "Repair of Waterstop Failures," *ASCE Journal of the Energy Division,* Vol 106, No. EY2, pp 155-163.

Schrader 1981
Schrader, E. K. 1981 (Jun). "Deterioration and Repair of Concrete in the Lower Monumental Navigation Lock Wall," Miscellaneous Paper SL-81-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Schrader 1992
Schrader, Ernest K. 1992 (Dec). "Mistakes, Misconceptions, and Controversial Issues Concerning Concrete and Concrete Repairs," *Concrete International*, American Concrete Institute, Vol 14, No. 11, pp 54-59.

Schrader and Kaden 1976
Schrader, E. K., and Kaden, R. A. 1976 (Jul-Aug). "Stilling Basin Repairs at Dworshak Dam," *The Military Engineer*, Vol 68, No. 444, Alexandria, VA.

Simons 1992
Simons, B. P. 1992 (Mar). "Abrasion Testing for Suspended Sediment Loads," *Concrete International*, Vol 14, No. 3, pp.

Smith 1987
Smith, A. P. 1987 (Apr). "New Tools and Techniques for the Underwater Inspection of Waterfront Structures, OTC 5390, *19th Annual Offshore Technology Conference*, Houston, TX.

Solomon and Jaques 1994
Solomon, Joseph, and Jaques, Mike. 1994 (Jun-Jul). "Stopping Leaks With Polyurethane Grouts," *Concrete Repair Digest*, Vol 5, No. 3, pp 180-185.

SONEX 1983
SONEX, LTD. 1983 (Sep). "Sonic Inspection of Ice Harbor Dam Spillway Stilling Basin," prepared for U.S. Army Corps of Engineers, U.S. Army Engineer District, Walla Walla, Walla Walla, WA, under Contract DACW39-83-M-3397, revised Feb 1984.

SONEX 1984
SONEX, LTD. 1984 (Jan). "Final Report: High Resolution Acoustic Survey, Folsom Dam Stilling Basin Floor," prepared for U.S. Army Engineer Waterways Experiment

Station, Vicksburg, MS, under Purchase Order DACW39-83-M-4340.

Stowe and Thornton 1984
Stowe, R. L., and Thornton, H. T., Jr. 1984 (Sep). "Engineering Condition Survey of Concrete in Service," Technical Report REMR-CS-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Stowe and Campbell 1989
Stowe, R. L., and Campbell, R. L., Sr. 1989. "User's Guide: Maintenance and Repair Materials Database for Concrete and Steel Structures," Technical Report REMR-CS-27, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS.

Stratton, Alexander, and Nolting 1982
Stratton, F. W., Alexander, R., and Nolting, W. 1982 (May). "Development and Implementation of Concrete Girder Repair by Post-Reinforcement," Report No. FHWA-KS-82-1, Kansas Department of Transportation, Topeka, 31 pp.

Sumner 1993
Sumner, Andrew C. 1993. "Rehabilitation of Crescent and Vischer Ferry Dams, Construction Techniques & Problem Solutions," *Geotechnical Practice in Dam Rehabilitation*, American Society of Civil Engineers, New York, NY.

Thornton 1985
Thornton, H. T., Jr. 1985 (Mar). "Corps-BuRec Effort Results in High-Resolution Acoustic Mapping System," *The REMR Bulletin*, Vol 2, No. 1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Thornton and Alexander 1987
Thornton, H. T., and Alexander, A. M. 1987 (Dec). "Development of Nondestructive Testing Systems for In Situ Evaluation of Concrete Structures," Technical Report REMR-CS-10, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Thornton and Alexander 1988
Thornton, H. T., Jr., and Alexander, A. M. 1988 (Mar). "Ultrasonic Pulse-Echo Measurements of the Concrete Sea Wall at Marina Del Rey, Los Angeles County, California," *The REMR Bulletin*, Vol 5, No. 1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Trout 1994
Trout, John. 1994 (Mar-Apr). "Comparing High Pressure and Low Pressure Injection," *Concrete Repair Bulletin*, Vol 7, No. 2, pp 20-21.

U.S. Army Engineer District, Walla Walla 1979
U.S. Army Engineer District, Walla Walla. 1979 (Jan). "Dworshak Dam and Reservoir, Inspection Report No. 6," Walla Walla, WA.

U.S. Army Engineer Division, Missouri River 1974
U.S. Army Engineer Division, Missouri River. 1974 (Apr). "Development of Equipment and Techniques for Pneumatic Application of Portland Cement Mortar in Shallow Patches," Omaha, NE.

U.S. Department of Transportation 1989
U.S. Department of Transportation. 1989 (Nov). "Underwater Inspection of Bridges," Report No. FHWA-DP-80-1, Federal Highway Administration, Washington, DC.

Warner 1984
Warner, J. 1984 (Oct). "Selecting Repair Materials," *Concrete Construction*, Vol 29, No. 10, pp 865-871.

Webster and Kukacka 1988
Webster, R. P., and Kukacka, L. E. 1988 (Jan). "In Situ Repair of Deteriorated Concrete in Hydraulic Structures: Laboratory Study," Technical Report REMR-CS-11, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Webster, Kakacka, and Elling 1989
Webster, R. P., Kukacka, L. E., and Elling, D. 1989 (Apr). "In Situ Repair of Deteriorated Concrete in Hydraulic Structures: A Field Study," Technical Report REMR-CS-21, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Webster, Kakacka, and Elling 1990
Webster, R. P., Kukacka, L. E., and Elling, D. 1990 (Sep). "In Situ Repair of Deteriorated Concrete in Hydraulic Structures: Epoxy Injection Repair of a Bridge Pier," Technical Report REMR-CS-30, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Wickersham 1987
Wickersham, J. 1987 (Dec). "Concrete Rehabilitation at Lock and Dam No. 20, Mississippi River," *The REMR*

Bulletin, Vol 4, No. 4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Wong, Feldsher, Wright, and Johnson 1992
Wong, Noel C., Feldsher, Theodore B., Wright, Robert S., and Johnson, David H. 1992 (Feb). "Final Design and Construction of Gibraltar Dam Strengthening," *Roller Compacted Concrete III*, American Society of Civil Engineers, pp 440-458.

Wong, Forrest, and Lo 1993
Wong, Noel C., Forrest, Michael P., and Lo, Sze-Hang. 1993 (Sep). "Littlerock Dam Rehabilitation: Another RCC Innovation," *Proceedings from the 10th ASDSO Annual Conference*, Association of State Dam Safety Officials, pp 303-314.

Xanthakos 1979
Xanthakos, P. P. 1979. *Slurry Walls*, McGraw-Hill Book Co., New York.

Appendix B
Glossary

Terms related to evaluation and repair of concrete structures as used herein are defined as follows:

Abrasion resistance
Ability of a surface to resist being worn away by rubbing and friction.

Acrylic resin
One of a group of thermoplastic resins formed by polymerizing the esters or amides of acrylic acid; used in concrete construction as a bonding agent or surface sealer.

Adhesives
The group of materials used to join or bond similar or dissimilar materials; for example, in concrete work, the epoxy resins.

Air-water jet
A high-velocity jet of air and water mixed at the nozzle; used in cleanup of surfaces of rock or concrete such as horizontal construction joints.

Alkali-aggregate reaction
Chemical reaction in mortar or concrete between alkalies (sodium and potassium) from portland cement or other sources and certain constituents of some aggregates; under certain conditions, deleterious expansion of the concrete or mortar may result.

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 reactivity (of aggregate)
Susceptibility of aggregate to alkali-aggregate reaction.

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.

Autogenous healing
A natural process of closing and filling of cracks in concrete or mortar when kept damp.

Bacterial corrosion
The destruction of a material by chemical processes brought about by the activity of certain bacteria which may produce substances such as hydrogen sulfide, ammonia, and sulfuric acid.

Blistering
The irregular raising of a thin layer at the surface of placed mortar or concrete during or soon after completion of the finishing operation, or in the case of pipe after spinning; also bulging of the finish plaster coat as it separates and draws away from the base coat.

Bug holes
Small regular or irregular cavities, usually not exceeding 15 mm in diam, resulting from entrapment of air bubbles in the surface of formed concrete during placement and compaction.

Butyl stearate
A colorless oleaginous, practically odorless material ($C_{17}H_{35}COOC_4H_9$) used as an admixture for concrete to provide dampproofing.

Cavitation damage
Pitting of concrete caused by implosion; i.e., the collapse of vapor bubbles in flowing water which form in areas of low pressure and collapse as they enter areas of higher pressure.

Chalking
Formation of a loose powder resulting from the disintegration of the surface of concrete or an applied coating such as cement paint.

Checking
Development of shallow cracks at closely spaced, but irregular, intervals on the surface of plaster, cement paste, mortar, or concrete.

Cold-joint lines
Visible lines on the surfaces of formed concrete indicating the presence of joints where one layer of concrete had hardened before subsequent concrete was placed.

Concrete, preplaced-aggregate
Concrete produced by placing coarse aggregate in a form and later injecting a portland-cement-sand grout, usually with admixtures, to fill the voids.

Corrosion
Destruction of metal by chemical, electrochemical, or electrolytic reaction with its environment.

Cracks, active*
Those cracks for which the mechanism causing the cracking is still at work. Any crack that is still moving.

Cracks, dormant*
Those cracks not currently moving or which the movement is of such magnitude that the repair material will not be affected.

Craze cracks
Fine, random cracks or fissures in a surface of plaster, cement paste, mortar, or concrete.

Crazing
The development of craze cracks; the pattern of craze cracks existing in a surface. (See also Checking.)

Dampproofing
Treatment of concrete or mortar to retard the passage or absorption of water or water vapor, either by application of a suitable coating to exposed surfaces or by use of a suitable admixture, treated cement, or preformed films such as polyethylene sheets under slabs on grade. (See also Vapor barrier.)

D-cracking
A series of cracks in concrete near and roughly parallel to joints, edges, and structural cracks.

Delamination
A separation along a plane parallel to a surface as in the separation of a coating from a substrate or the layers of a coating from each other, or in the case of a concrete slab, a horizontal splitting, cracking, or separation of a slab in a plane roughly parallel to, and generally near, the upper surface; found most frequently in bridge decks and caused by the corrosion of reinforcing steel or freezing and thawing; similar to spalling, scaling, or peeling except that delamination affects large areas and can often be detected only by tapping.

Deterioration
Decomposition of material during testing or exposure to service. (See also Disintegration.)

Diagonal crack
In a flexural member, an inclined crack caused by shear stress, usually at about 45 deg to the neutral axis of a concrete member; a crack in a slab, not parallel to the lateral or longitudinal directions.

Discoloration
Departure of color from that which is normal or desired.

Disintegration
Reduction into small fragments and subsequently into particles.

Dry-mix shotcrete
Shotcrete in which most of the mixing water is added at the nozzle.

Drypacking
Placing of zero slump, or near zero slump, concrete, mortar, or grout by ramming it into a confined space.

Durability
The ability of concrete to resist weathering action, chemical attack, abrasion, and other conditions of service.

Dusting
The development of a powdered material at the surface of hardened concrete.

Efflorescence
A deposit of salts, usually white, formed on a surface, the substance having emerged in solution from within concrete or masonry and subsequently having been precipitated by evaporation.

Epoxy concrete
A mixture of epoxy resin, catalyst, fine aggregate, and coarse aggregate. (See also Epoxy mortar, Epoxy resin, and Polymer concrete.)

Epoxy mortar
A mixture of epoxy resin, catalyst, and fine aggregate. (See also Epoxy resin.)

Epoxy resin
A class of organic chemical bonding systems used in the preparation of special coatings or adhesives for concrete or as binders in epoxy resin mortars and concretes.

* All definitions are in accordance with ACI 116R except those denoted by an asterisk.

Erosion
Progressive disintegration of a solid by the abrasive or cavitation action of gases, fluids, or solids in motion. (See also Abrasion resistance and Cavitation damage.)

Ettringite
A mineral, high-sulfate calcium sulfoaluminate $(3CaO \cdot A1_20_3 \cdot 3CaS0_4 \cdot 32H_20)$ also written as Ca_6 $[Al(OH)_6]_2 \cdot 24H_20[(S0_4)3 \cdot (1\text{-}1/2)\ H_20]$ occurring in nature or formed by sulfate attack on mortar and concrete; the product of the principal expansion-producing reaction in expansive cements; designated as “cement bacillus” in older literature.

Evaluation*
Determining the condition, degree of damage or deterioration, or serviceability and, when appropriate, indicating the need for repair, maintenance, or rehabilitation. (See also Repair, Maintenance, and Rehabilitation.)

Exfoliation
Disintegration occurring by peeling off in successive layers; swelling up and opening into leaves or plates like a partly opened book.

Exudation
A liquid or viscous gel-like material discharge through a pore, crack, or opening in the surface of concrete.

Feather edge
Edge of a concrete or mortar patch or topping that is beveled at an acute angle.

Groove joint
A joint created by forming a groove in the surface of a pavement, floor slab, or wall to control random cracking.

Hairline cracks
Cracks in an exposed concrete surface having widths so small as to be barely preceptible.

Honeycomb
Voids left in concrete due to failure of the mortar to effectively fill the spaces among coarse aggregate particles.

Incrustation
A crust or coating, generally hard, formed on the surface of concrete or masonry construction or on aggregate particles.

Joint filler
Compressible material used to fill a joint to prevent the infiltration of debris and to provide support for sealants.

Joint sealant
Compressible material used to exclude water and solid foreign material from joints.

Laitance
A layer of weak and nondurable material containing cement and fines from aggregates, brought by bleeding water to the top of overwet concrete, the amount of which is generally increased by overworking or over-manipulating concrete at the surface by improper finishing or by job traffic.

Latex
A water emulsion of a high molecular-weight polymer used especially in coatings, adhesives, and leveling and patching compounds.

Maintenance*
Taking periodic actions that will prevent or delay damage or deterioration or both. (See also Repair.)

Map cracking
See Crazing.

Microcracks
Microscopic cracks within concrete.

Monomer
An organic liquid of relatively low molecular weight that creates a solid polymer by reacting with itself or other compounds of low molecular weight or with both.

Overlay
A layer of concrete or mortar, seldom thinner than 25 mm (1 in.), placed on and usually bonded onto the worn or cracked surface of a concrete slab to restore or improve the function of the previous surface.

Pattern cracking
Intersecting cracks that extend below the surface of hardened concrete; caused by shrinkage of the drying surface which is restrained by concrete at greater depth where little or no shrinkage occurs; vary in width and depth from fine and barely visible to open and well defined.

---

* All definitions are in accordance with ACI 116R except those denoted by an asterisk.

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.

Pitting
Development of relatively small cavities in a surface caused by phenomena such as corrosion or cavitation, or in concrete localized disintegration such as a popout.

Plastic cracking
Cracking that occurs in the surface of fresh concrete soon after it is placed and while it is still plastic.

Plastic shrinkage cracks
See Plastic cracking.

Polyester
One of a large group of synthetic resins, mainly produced by reaction of dibasic acids with dihydroxy alcohols, commonly prepared for application by mixing with a vinyl-group monomer and free-radical catalyst at ambient temperatures and used as binders for resin mortars and concretes, fiber laminates (mainly glass), adhesives, and the like. (See also Polymer concrete.)

Polyethylene
A thermoplastic high-molecular-weight organic compound used in formulating protective coatings; in sheet form, used as a protective cover for concrete surfaces during the curing period, or to provide a temporary enclosure for construction operations.

Polymer
The product of polymerization; more commonly, a rubber or resin consisting of large molecules formed by polymerization.

Polymer concrete
Concrete in which an organic polymer serves as the binder; also known as resin concrete; sometimes erroneously employed to designate hydraulic-cement mortars or concretes in which part or all of the mixing water is replaced by an aqueous dispersion of a thermoplastic copolymer.

Polymer-cement concrete
A mixture of water, hydraulic cement, aggregate, and a monomer or polymer polymerized in place when a monomer is used.

Polymerization
The reaction in which two or more molecules of the same substance combine to form a compound containing the same elements in the same proportions, but of higher molecular weight, from which the original substance can be generated, in some cases only with extreme difficulty.

Polystyrene resin
Synthetic resins varying in color from colorless to yellow formed by the polymerization of styrene, or heated, with or without catalysts, that may be used in paints for concrete or for making sculptured molds or as insulation.

Polysulfide coating
A protective coating system prepared by polymerizing a chlorinated alkylpolyether with an inorganic polysulfide.

Polyurethane
Reaction product of an isocyanate with any of a wide variety of other compounds containing an active hydrogen group; used to formulate tough, abrasion-resistant coatings.

Polyvinyl acetate
Colorless, permanently thermoplastic resin, usually supplied as an emulsion or water-dispersible powder characterized by flexibility, stability toward light, transparency to ultraviolet rays, high dielectric strength, toughness, and hardness; the higher the degree of polymerization, the higher the softening temperature; may be used in paints for concrete.

Polyvinyl chloride
A synthetic resin prepared by the polymerization of vinyl chloride; used in the manufacture of nonmetallic waterstops for concrete.

Popout
The breaking away of small portions of concrete surface due to internal pressure, which leaves a shallow, typically conical, depression.

Pot life
Time interval after preparation during which a liquid or plastic mixture is usable.

Reactive aggregate
Aggregate containing substances capable of reacting chemically with the products of solution or hydration of the portland cement in concrete or mortar under ordinary

conditions of exposure, resulting in some cases in harmful expansion, cracking, or staining.

Rebound hammer
An apparatus that provides a rapid indication of the mechanical properties of concrete based on the distance of rebound of a spring-driven missile.

Rehabilitation
The process of repairing or modifying a structure to a desired useful condition.

Repair
Replace or correct deteriorated, damaged, or faulty materials, components, or elements of a structure.

Resin
A natural or synthetic, solid or semisolid organic material of indefinite and often high molecular weight having a tendency to flow under stress that usually has a softening or melting range and usually fractures conchoidally.

Resin mortar (or concrete)
See Polymer concrete.

Restraint (of concrete)
Restriction of free movement of fresh or hardened concrete following completion of placement in formwork or molds or within an otherwise confined space; restraint can be internal or external and may act in one or more directions.

Rock pocket
A porous, mortar-deficient portion of hardened concrete consisting primarily of coarse aggregate and open voids, caused by leakage of mortar from form, separation (segregation) during placement, or insufficient consolidation. (See also Honeycombing.)

Sandblasting
A system of cutting or abrading a surface such as concrete by a stream of sand ejected from a nozzle at high speed by compressed air; often used for cleanup of horizontal construction joints or for exposure of aggregate in architectural concrete.

Sand streak
A streak of exposed fine aggregate in the surface of formed concrete that is caused by bleeding.

Scaling
Local flaking or peeling away of the near-surface portion of hardened concrete or mortar; also of a layer from metal. (See also Peeling and Spalling.) (Note: Light scaling of concrete does not expose coarse aggregate; medium scaling involves loss of surface mortar to 5 to 10 mm in depth and exposure of coarse aggregate; severe scaling involves loss of surface mortar to 5 to 10 mm in depth with some loss of mortar surrounding aggregate particles 10 to 20 mm in depth; very severe scaling involves loss of coarse-aggregate particles as well as mortar generally to a depth greater than 20 mm.)

Shotcrete
Mortar or concrete pneumatically projected at high velocity onto a surface; also known as air-blown mortar; also pneumatically applied mortar or concrete, sprayed mortar, and gunned concrete. (See also Dry-mix shotcrete and Wet-mix shotcrete.)

Shrinkage
Volume decrease caused by drying and chemical changes; a function of time but not temperature or of stress caused by external load.

Shrinkage crack
Crack due to restraint of shrinkage.

Shrinkage cracking
Cracking of a structure or member from failure in tension caused by external or internal restraints as reduction in moisture content develops or as carbonation occurs, or both.

Spall
A fragment, usually in the shape of a flake, detached from a larger mass by a blow, action of weather, pressure, or expansion within the larger mass; a small spall involves a roughly circular depression not greater than 20 mm in depth nor 150 mm in any dimension; a large spall may be roughly circular or oval or, in some cases, elongated more than 20 mm in depth and 150 mm in greatest dimension.

Stalactite
A downward-pointing deposit formed as an accretion of mineral matter produced by evaporation of dripping water from the surface of concrete, commonly shaped like an icicle.

Stalagmite
An upward-pointing deposit formed as an accretion of mineral matter produced by evaporation of dripping water, projecting from the surface of concrete, and commonly conical in shape.

Spalling
The development of spalls.

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 causing deterioration.

Sulfate resistance
Ability of concrete or mortar to withstand sulfate attack. (See also Sulfate attack.)

Swiss hammer
See Rebound hammer.

Temperature cracking
Cracking as a result of tensile failure caused by temperature drop in members subjected to external restraints or temperature differential in members subjected to internal restraints.

Thermal shock
The subjection of newly hardened concrete to a rapid change in temperature which may be expected to have a potentially deleterious effect.

Thermoplastic
Becoming soft when heated and hard when cooled.

Thermosetting
Becoming rigid by chemical reaction and not remeltable.

Transverse cracks
Cracks that develop at right angles to the long direction of a member.

Tremie
A pipe or tube through which concrete is deposited underwater, having at its upper end a hopper for filling and a bail for moving the assemblage.

Tremie concrete
Subaqueous concrete placed by means of a tremie.

Tremie seal
The depth to which the discharge end of the tremie pipe is kept embedded in the fresh concrete that is being placed; a layer of tremie concrete placed in a cofferdam for the purpose of preventing the intrusion of water when the cofferdam is dewatered.

Vapor barrier
A membrane placed under concrete floor slabs that are placed on grade and intended to retard transmission of water vapor.

Waterstop
A thin sheet of metal, rubber, plastic, or other material inserted across a joint to obstruct seepage of water through the joint.

Water void
Void along the underside of an aggregate particle or reinforcing steel which formed during the bleeding period and initially filled with bleed water.

Weathering
Changes in color, texture, strength, chemical composition, or other properties of a natural or artificial material caused by the action of the weather.

Wet-mix shotcrete
Shotcrete in which the ingredients, including mixing water, are mixed before introduction into the delivery hose; accelerator if used, is normally added at the nozzle.

Appendix C
Abbreviations

ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
AWA	antiwashout admixture
CDMS	Continuous Deformation Monitoring System
CE	Corps of Engineers
CERC	Coastal Engineering Research Center
CEWES-SC	U.S. Army Engineer Waterways Experiment Station, Structures Laboratory, Concrete Technology Division
CMU	concrete masonry units
CRD	Concrete Research Division, Handbook for Concrete and Cement
EM	Engineer Manual
EP	Engineer Pamplet
ER	Engineer Regulation
FHWA	Federal Highway Administration
GPS	Global Positioning System
HAC	high alumina cement
HMWM	high molecular weight methacrylate
HQUSACE	Headquarters, U.S. Army Corps of Engineers
HRWRA	high-range water-reducing admixture
ICOLD	International Commission on Large Dams
MPC	magnesium phosphate cement
MSA	maximum size aggregate
MSDS	Manufacturer's Safety Data Sheet
NACE	National Association of Corrosion Engineers
NAVSTAR	Navigation Satellite Timing and Ranging
NCHRP	National Cooperative Highway Research Program
NDT	nondestructive testing
OCE	Office, Chief of Engineers
PC	polymer concrete
PCA	Portland Cement Association
PIC	polymer-impregnated concrete
PPCC	polymer portland-cement concrete
PMF	Probable Maximum Flood
PVC	polyvinyl chloride
R-values	rebound readings
RCC	roller-compacted concrete
REMR	Repair, Evaluation, Maintenance, and Rehabilitation Research Program
ROV	remotely operated vehicle
TM	Technical Manual
TOA	time of arrival
UPE	ultrasonic pulse-echo
UV	ultraviolet
w/c	water-cement ratio
WES	Waterways Experiment Station
WRA	water-reducing admixture

Evaluation and Repair of Concrete Structures Part I
Updated on: 01/31/2017

1. When the condition of a structure indicates that major repair or rehabilitation is probably necessary, a comprehensive evaluation of the structure should be conducted to determine the scope of the work required.

    a) true
    b) false

2. An evaluation of the structure should include:

    a) a review of the available design and construction documentation
    b) a visual examination of the condition of the concrete in the structure
    c) an evaluation of the structure by nondestructive testing means
    d) all of the above

3. As part of the visual inspection, which needs to be checked:

    a) disintegration
    b) construction faults
    c) cracking
    d) all of the above

4. Erosion of concrete may be categorized as one of two general types, they include:

    a) abrasion
    b) cavitation
    c) none of the above
    d) both A and B

5. __________is defined in ACI 207.3R as "the movement of water or other fluids through pores or interstices."

    a) seepage
    b) efflorescence
    c) spalling
    d) perspiration

6. Infrared thermography is a useful method of detecting delaminations in bridge checks.

    a) true
    b) false

7. ___________is a parallel procedure to a cracking survey in which deterioration of the surface concrete is located and described.

   a) Thermal surveying
   b) Joint survey
   c) Core drilling
   d) Surface mapping

8. Which of the following is not a method of underwater inspection:

   a) Visual inspection by divers
   b) Ultrasonic pulse velocity
   c) Ultrasonic pulse-echo system.
   d) Radioactive dye penetration

9. The purpose of __________ is to determine the various relative properties of concrete such as strength, modulus of elasticity, homogeneity, and integrity, as well as conditions of strain and stress, without damaging the structure.

   a) nondestructive testing
   b) destructive testing
   c) introspection
   d) none of the above

10. The _______ number is obtained by the use of a hammer that consists of a steel mass and a tension spring in a tubular frame

   a) rebound
   b) penetration
   c) hardness
   d) spalling

11. Some of the causes of distress and deterioration include all of the following except for:

   a) accidental loadings
   b) chemical reactions
   c) construction errors
   d) too much steel in concrete

12. Under most circumstances, Portland cement concrete provides good protection to the embedded reinforcing steel.

   a) True
   b) False

13. While a structure may be adequately designed to meet loadings and other overall requirements, poor ________ may result in localized concentrations of high stresses in otherwise satisfactory concrete.

   a) detailing
   b) workmanship
   c) materials
   d) none of the above

14. Which of the following preventative measures are NOT recommended by ACI 201.2R for concrete that will be exposed to freezing and thawing while saturated:

   a) using a concrete with a low w/c.
   b) designing the structure to minimize the exposure to moisture.
   c) using a concrete with a high w/c.
   d) providing adequate curing before concrete is exposed to freezing

15. ________ shrinkage is the long-term change in volume of concrete caused by the loss of moisture.
   a) cracking
   b) temperature
   c) wet
   d) drying

16. If there is evidence of swelling of the concrete, a possible cause is:

   a) chemical reactions
   b) temperature changes
   c) both A and B
   d) none of the above