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	

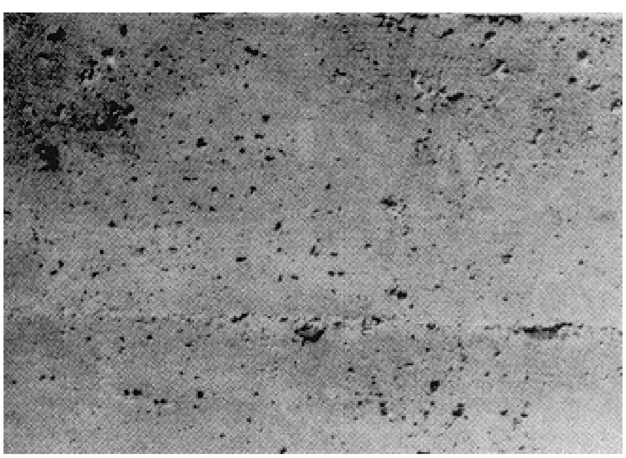

Figure 2-1. Bug holes in a vertical wall

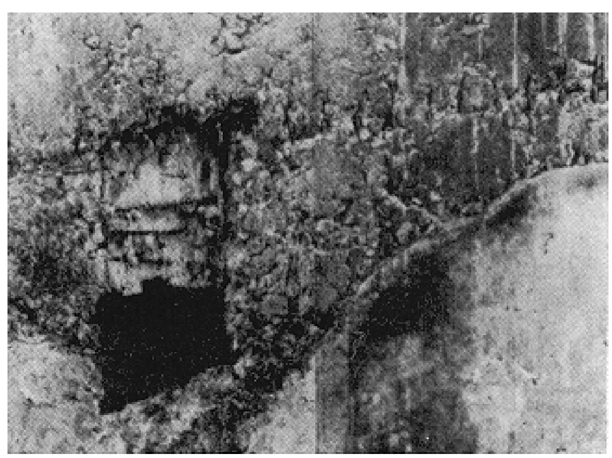

Figure 2-2. Honeycombing and cold joint

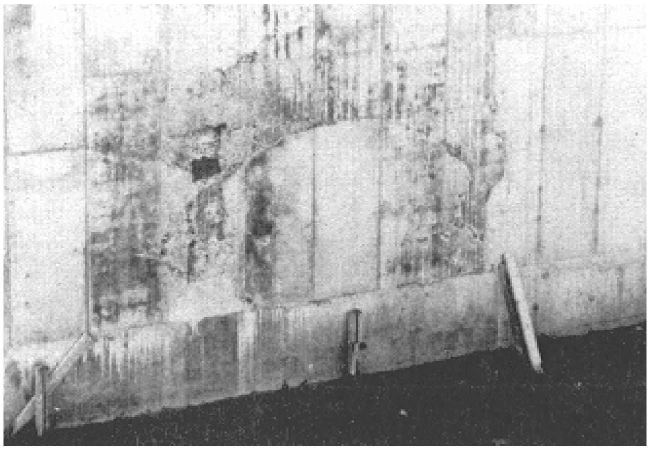

Figure 2-3. Cold joint

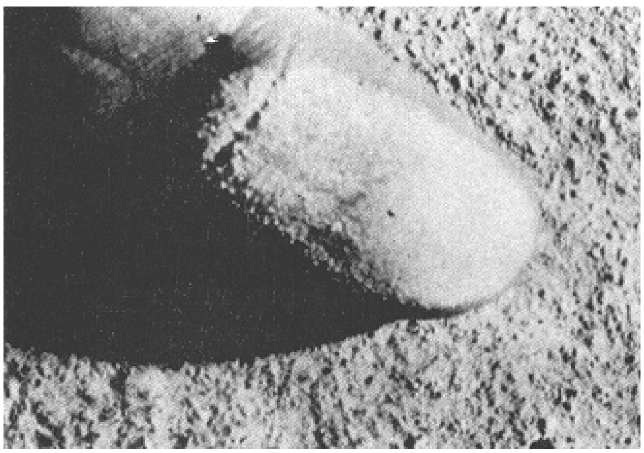

Figure 2-4. Dusting on horizontal finished surface

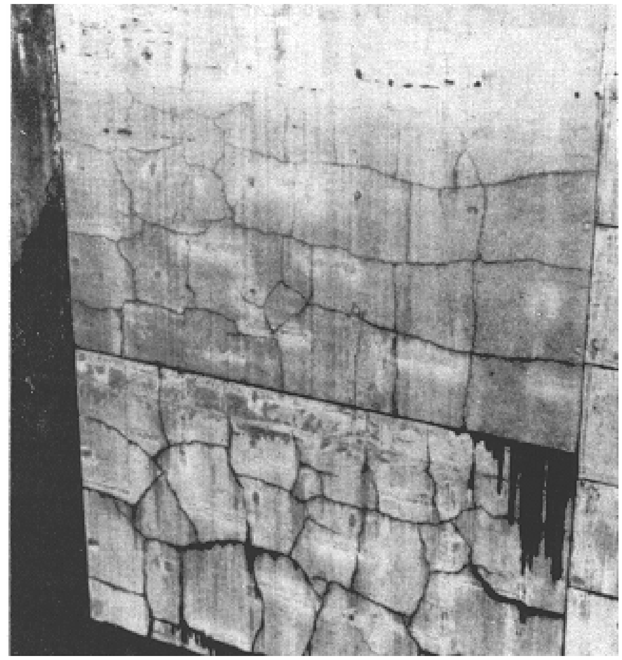

Figure 2-5. Pattern cracking caused by restrained volume changes

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

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.

(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).

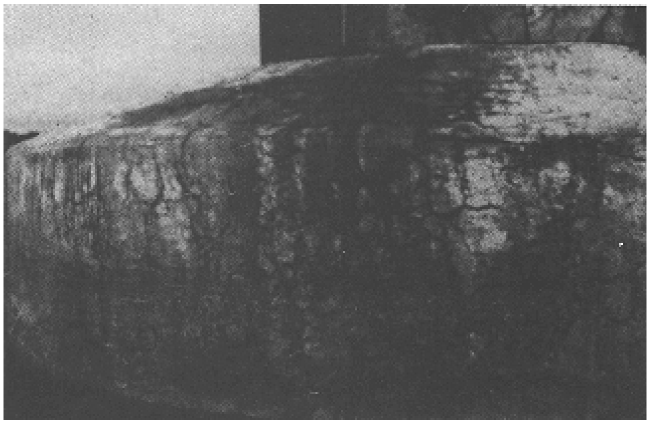

Figure 2-7. Pattern cracking caused by alkali-carbonate reaction

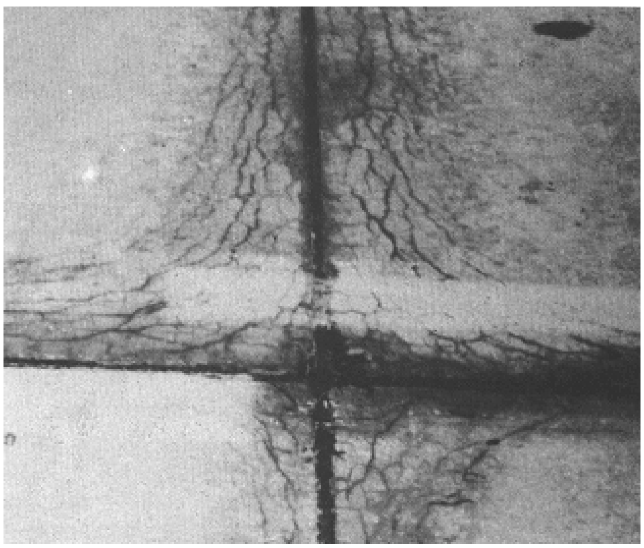

Figure 2-8. D-cracking in a concrete pavement

(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.

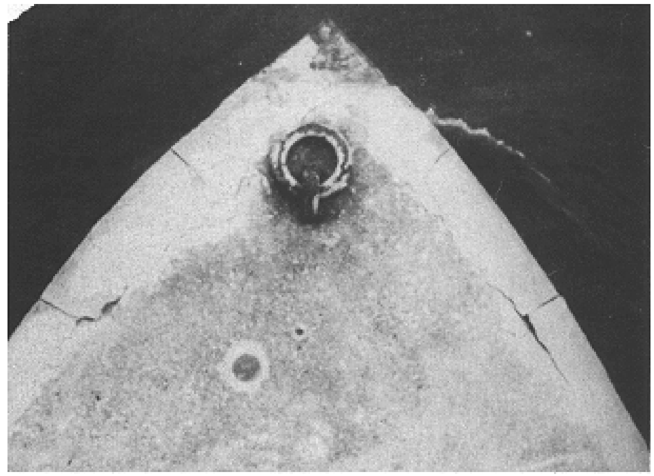

Figure 2-9. Isolated cracks as a result of restraint in the direction perpendicular to the crack

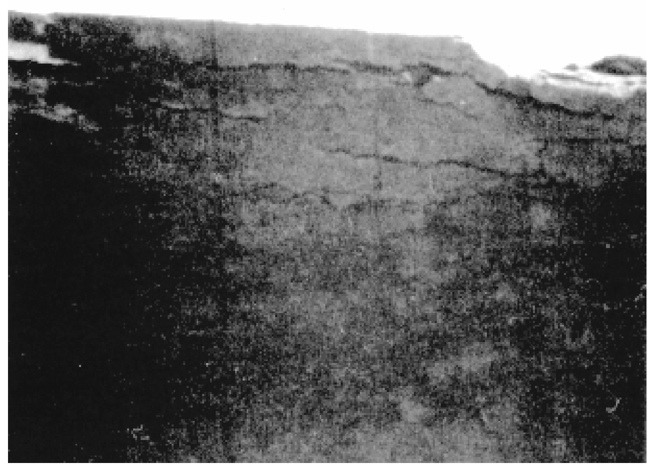

Figure 2-10. Parallel individual cracking caused by freezing and thawing

(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

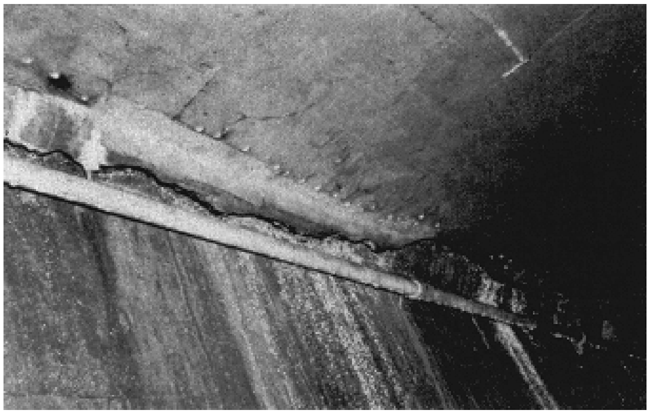

Figure 2-11. Isolated crack caused by structural overload

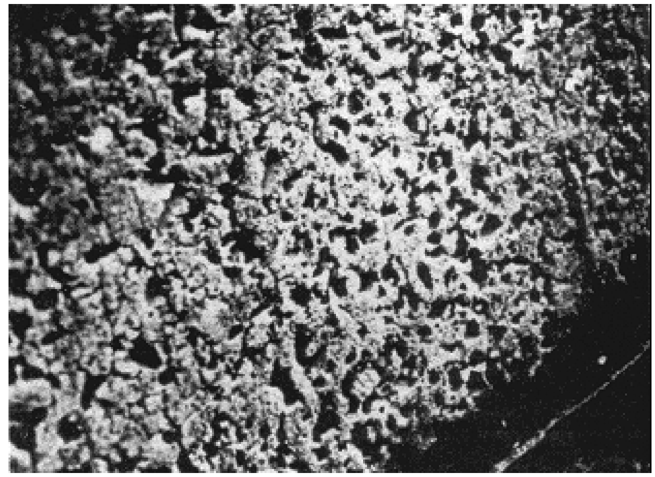

Figure 2-12. Disintegration of concrete caused by exposure to aggressive water

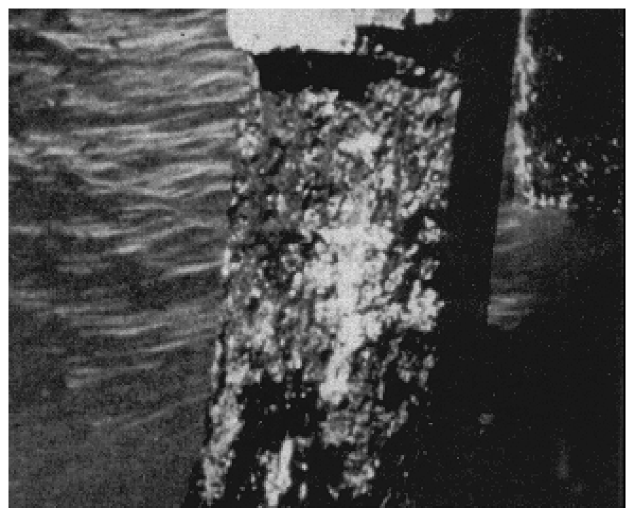

Figure 2-13. Disintegration of concrete caused by exposure to acidic water

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.

(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

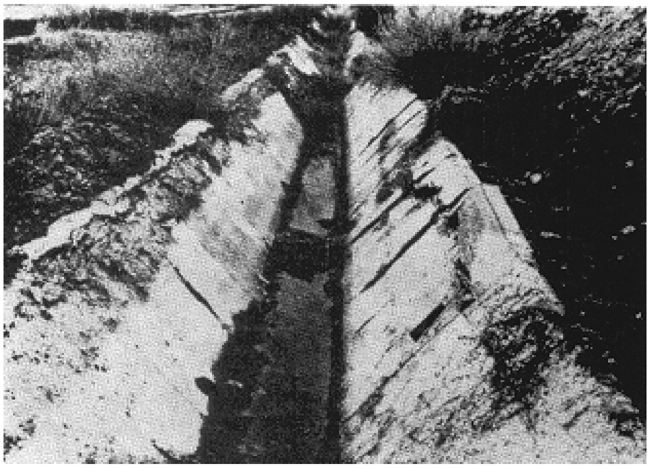

Figure 2-14. Disintegration of concrete caused by sulfate attack

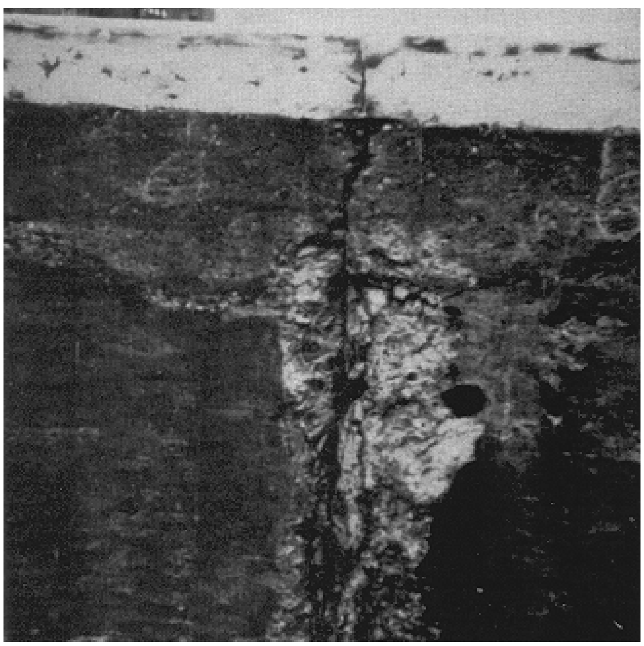

Figure 2-15. Disintegration at a monolith joint as a result of repeated cycles of freezing and thawing and barge impact

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

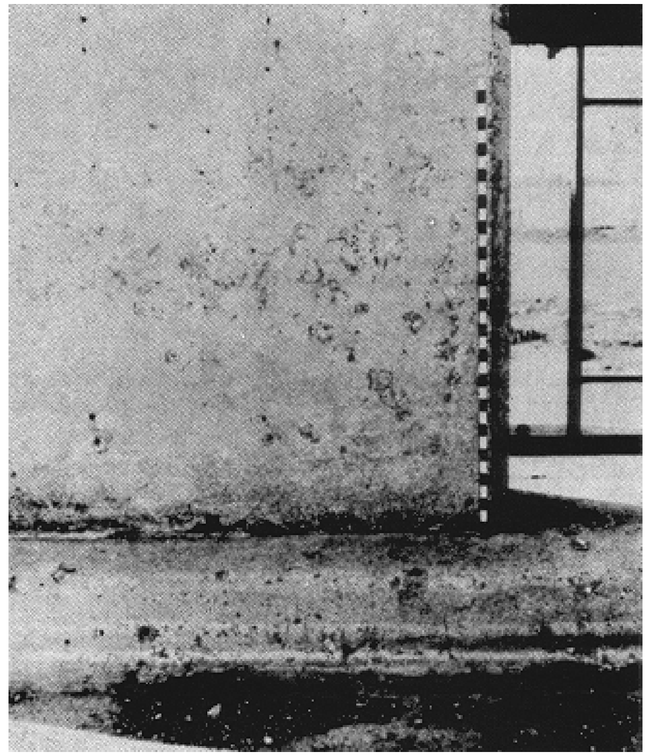

Figure 2-16. Light scaling

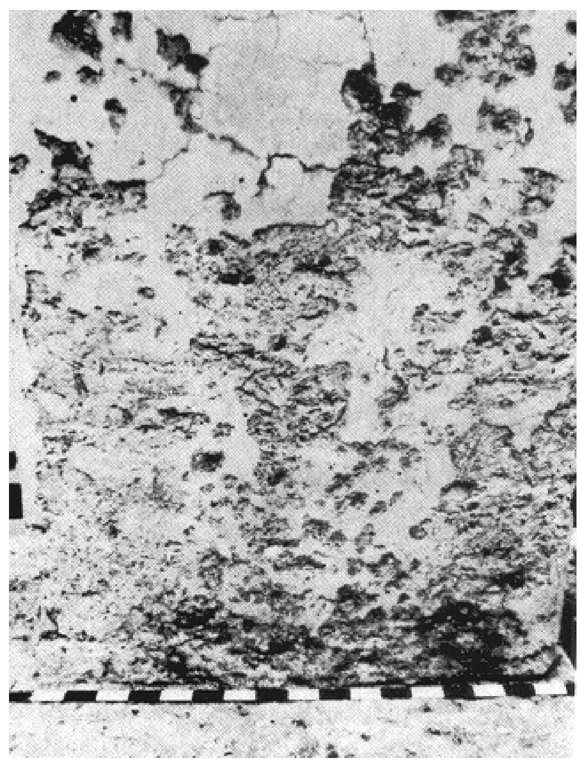

Figure 2-17. Medium scaling

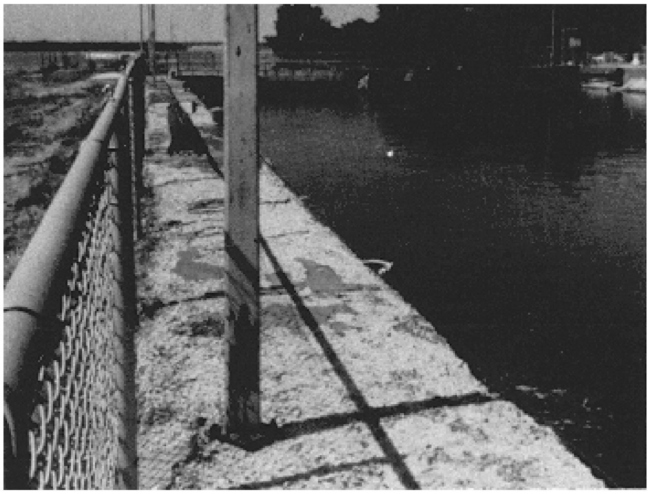

Figure 2-18. Severe scaling

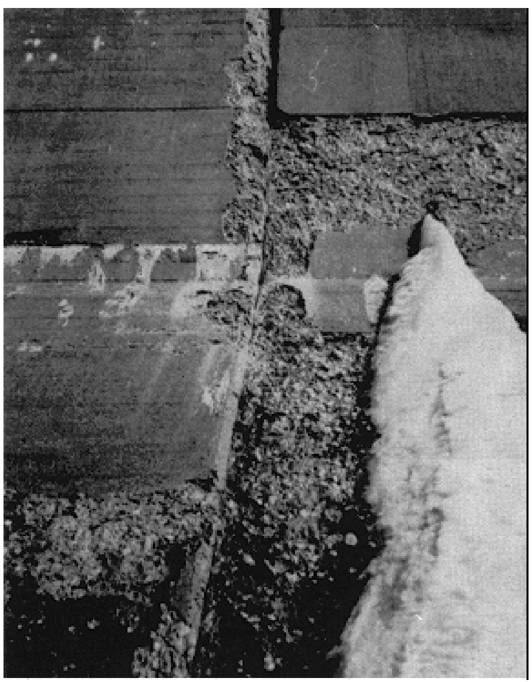

Figure 2-19. Very severe scaling

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

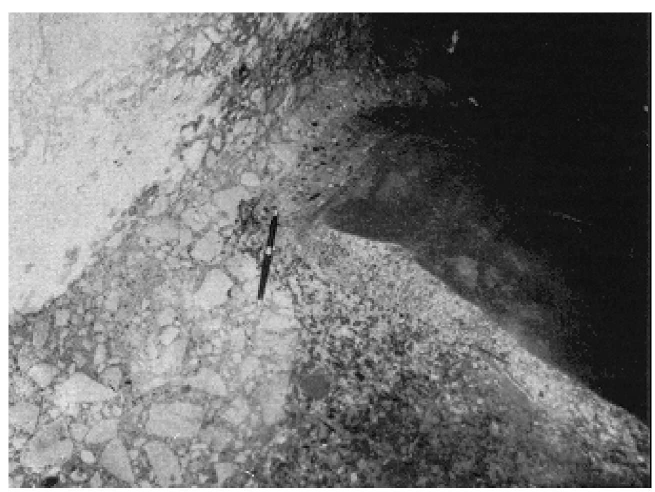

Figure 2-20. Smooth, worn, abraded concrete surface caused by abrasion of waterborne debris

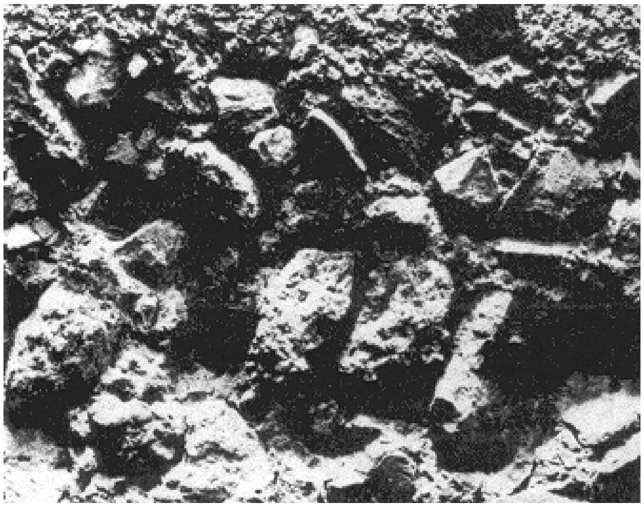

Figure 2-21. Rough, pitted concrete surface caused by cavitation

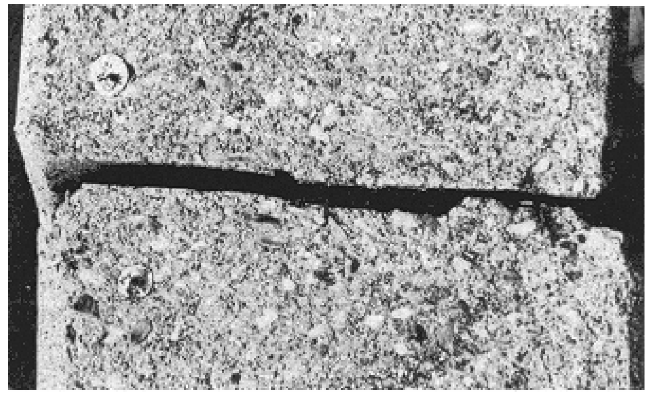

Figure 2-23. Loss of joint sealant

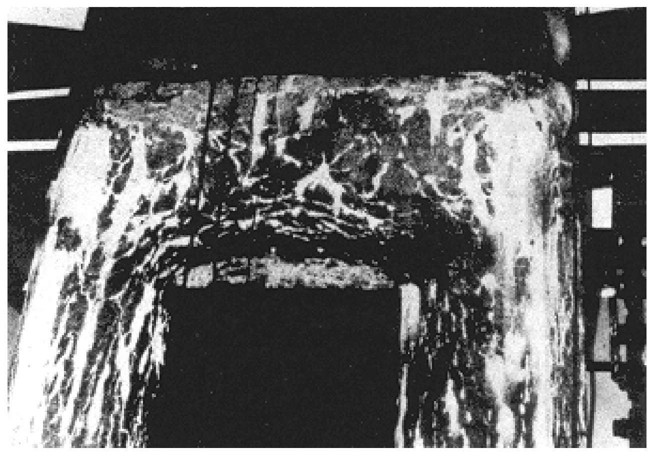

Figure 2-24. Efflorescence

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.

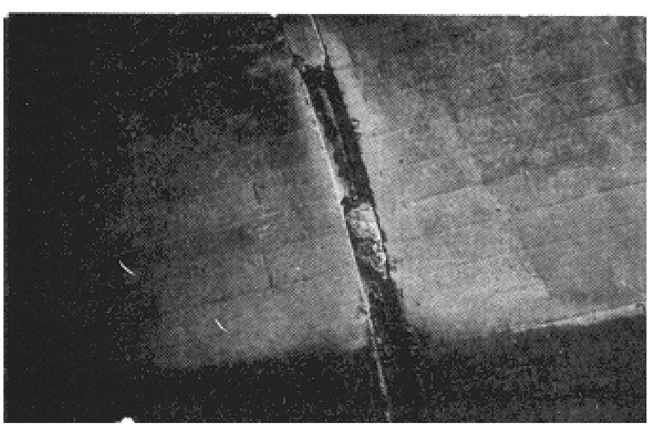

Figure 2-22. Deterioration of joint sealant

(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

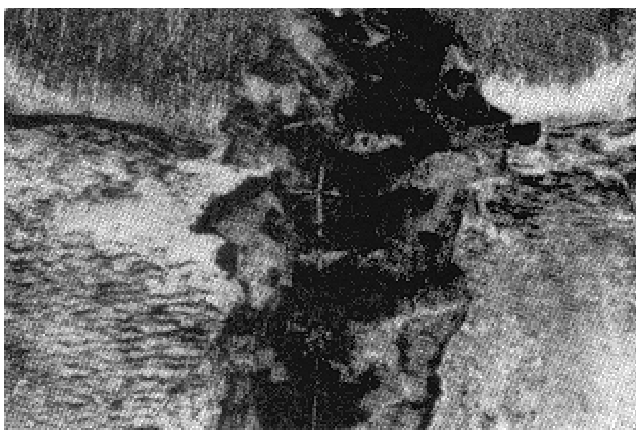

Figure 2-26. Incrustation

(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

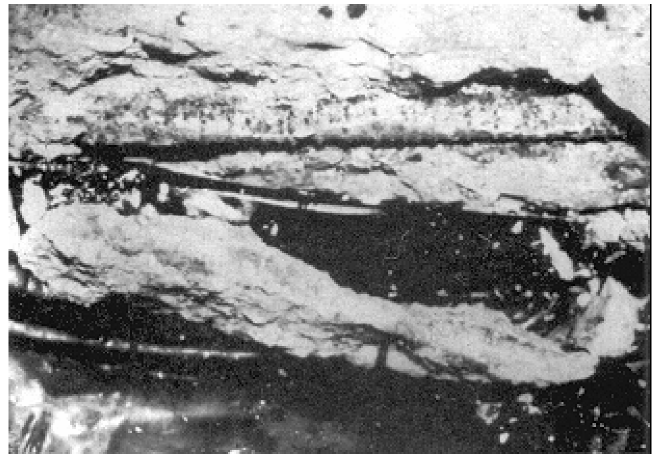

Figure 2-27. Corrosion

Figure 2-28. Water seepage through joint

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

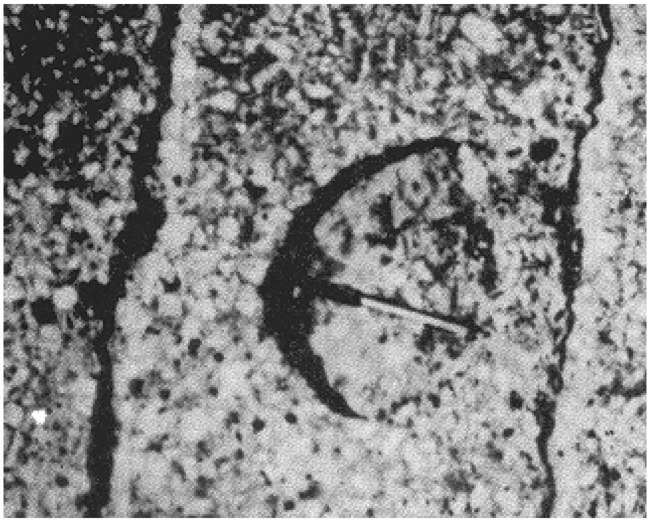

Figure 2-29. Small spall

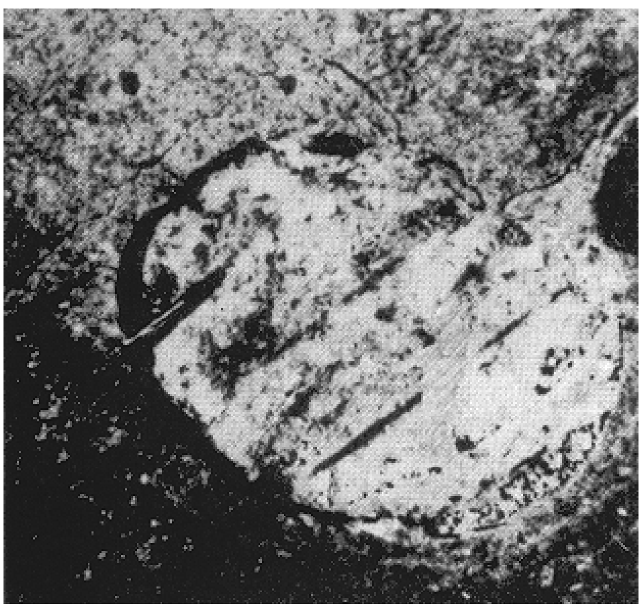

Figure 2-30. Large spall

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).

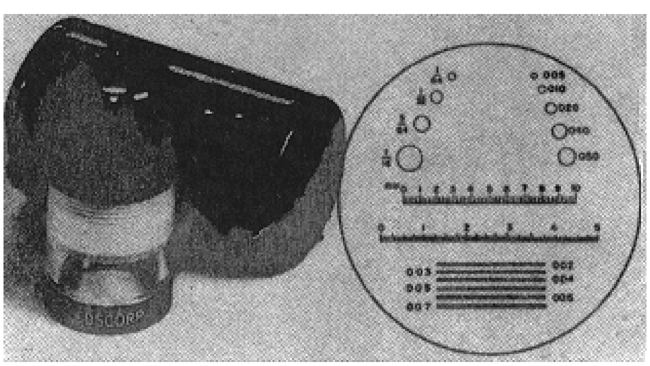

Figure 2-32. Comparator for measuring crack widths

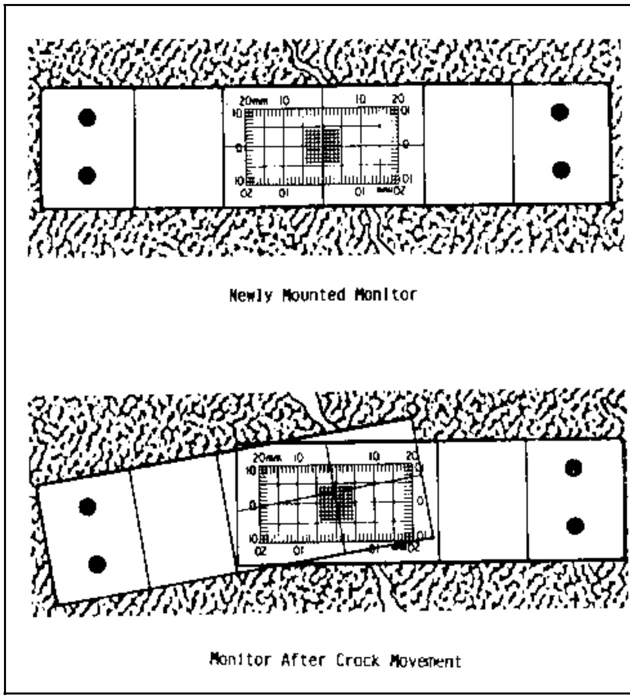

Figure 2-33. Crack monitor

(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	100		WL Run 1.75' Began 9:32 Rec 1.75' End 9:30 Loss -- Time 18 min Gain --
			0.0 to 0.9' new concrete overlay, crushed agg -3/4" max size, brn in color. 2-3% entrapped air.			Drl time 18 min Hyd press 240 Water press 50 RPM 100
584	1		0.9' begins old concrete, lt brn, river gravel -3" max size. Subparallel cracking thru agg and matrix.	Run 1		Drl Action Smooth Water ret 100%
583	2		MB 0.85' interval of frost damage beneath overlay.	100		Driller's notes lost
582	3		Vertical crack, wht deposit on -10% surface. Crack probably due to alkali-silica reaction. Goodly number of pockets of wht material easily dug w/ knife blade. Small agg and sand grains affected.			
581	4		Gel, wht soft	Run 2		
				100		Driller's notes lost
580	5		Concrete has voids up to 1/2" in length & depth; 2 to 3 voids per linear foot.		Box 1	
			Core Spin	Run 3		
579	6		Concrete as above Vertical crack through core	100		
			100% surface wht material			
			Construction joint, slightly honeycomb, surface dk brn to dk gry			WL Run 4.15' Began 2:45 Rec 4.15' End 3:35 Loss -- Time Gain --
578	7		Concrete as above			Drl time 50 min Hyd press 240 Water press 50 RPM 100
			Crack ends			Drl Action Smooth
577	8		MB		Box 2	Water ret 100%
						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.

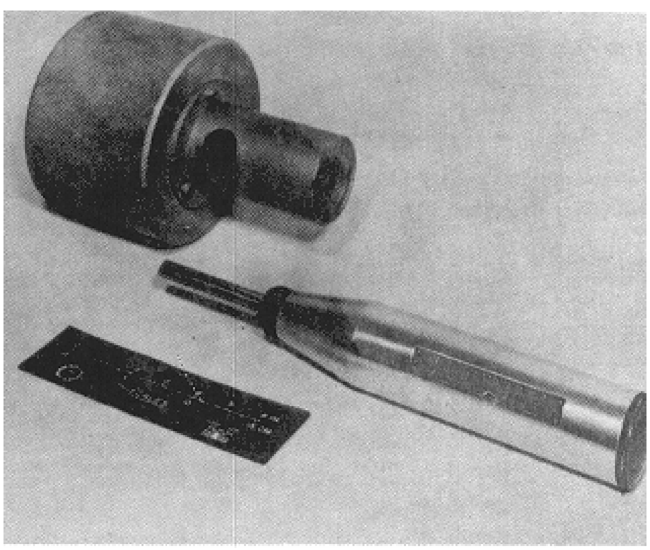

Figure 2-35. Rebound hammer

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

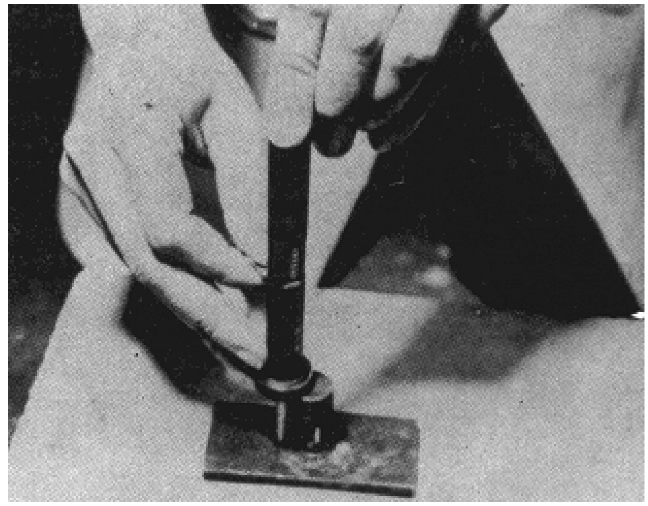

Figure 2-38. Device for measuring length of probe extending from surface of concrete

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.

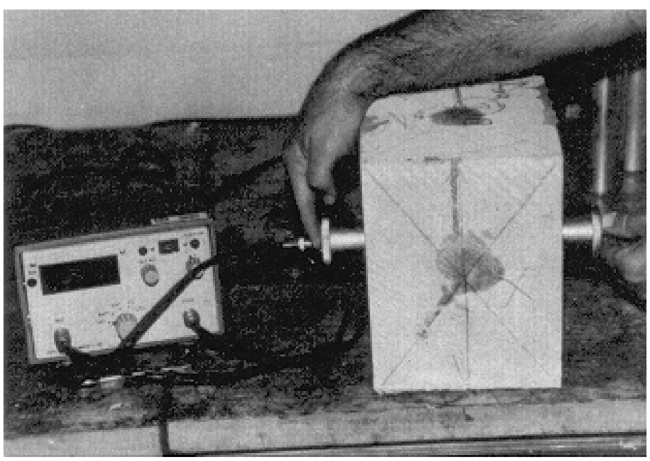

Figure 2-39. Ultrasonic pulse-velocity apparatus

(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.