Evaluation and Design of Concrete Repairs

Course No: C05-008
Credit: 5 PDH

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This course was adapted from the US Corps of Engineers, Publication EM 1110-2-2002, "Evaluation and Repair of Concrete Structures", Chapters 2, 3 and 4, which is in the public domain.

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	

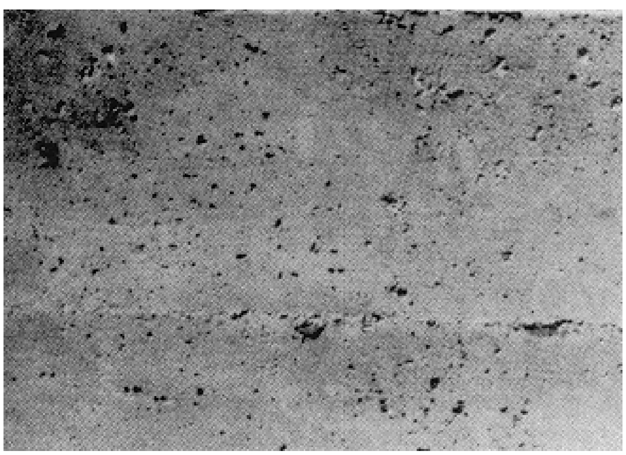

Figure 2-1. Bug holes in a vertical wall

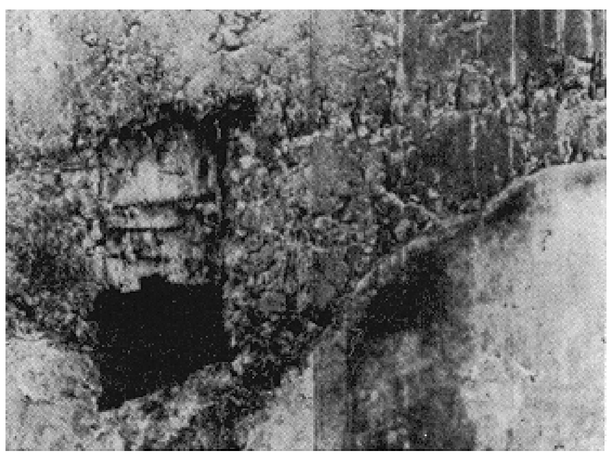

Figure 2-2. Honeycombing and cold joint

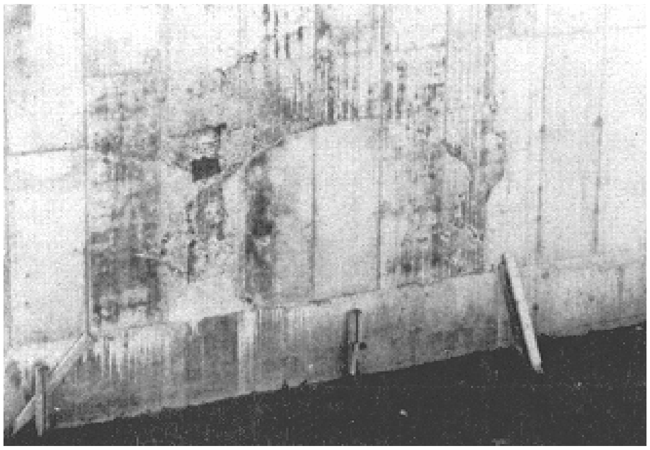

Figure 2-3. Cold joint

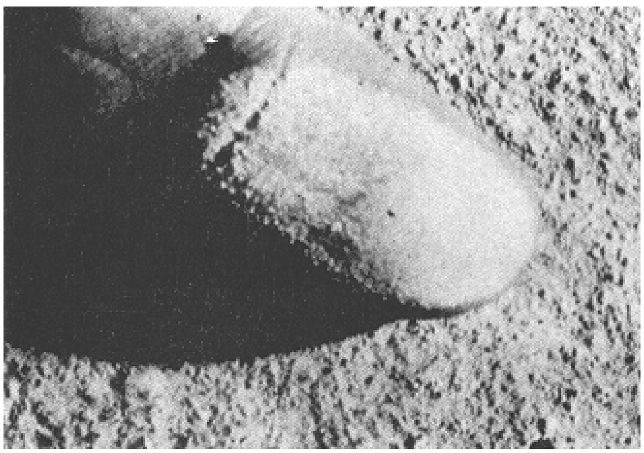

Figure 2-4. Dusting on horizontal finished surface

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.

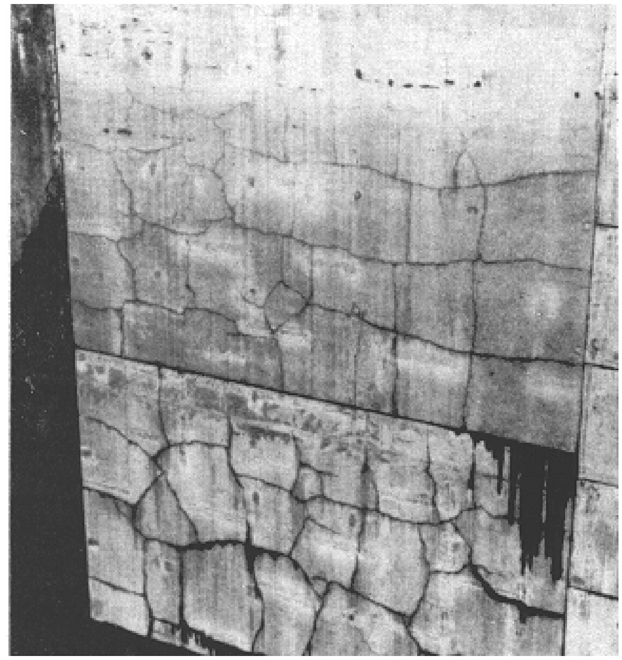

Figure 2-5. Pattern cracking caused by restrained volume changes

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).

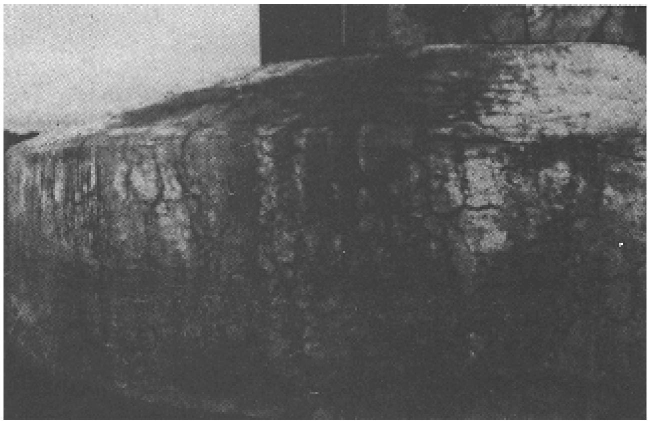

Figure 2-7. Pattern cracking caused by alkali-carbonate reaction

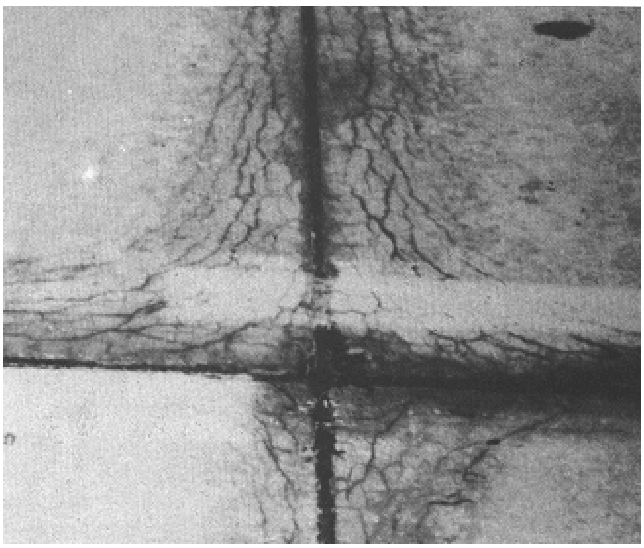

Figure 2-8. D-cracking in a concrete pavement

(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.

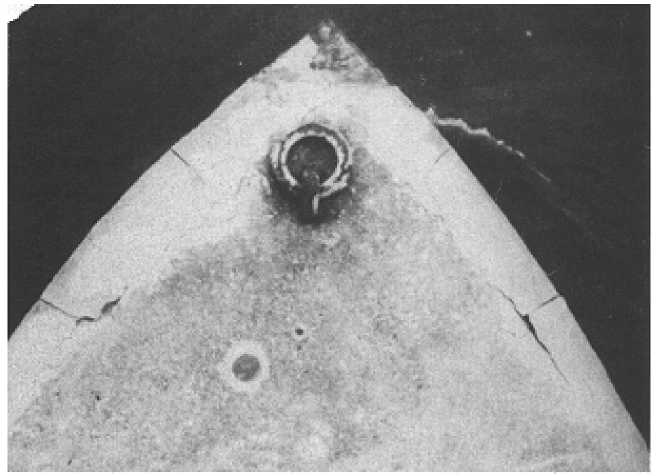

Figure 2-9. Isolated cracks as a result of restraint in the direction perpendicular to the crack

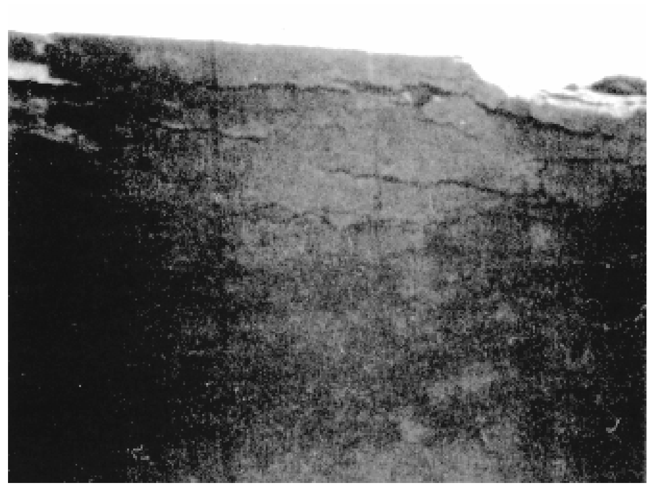

Figure 2-10. Parallel individual cracking caused by freezing and thawing

(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

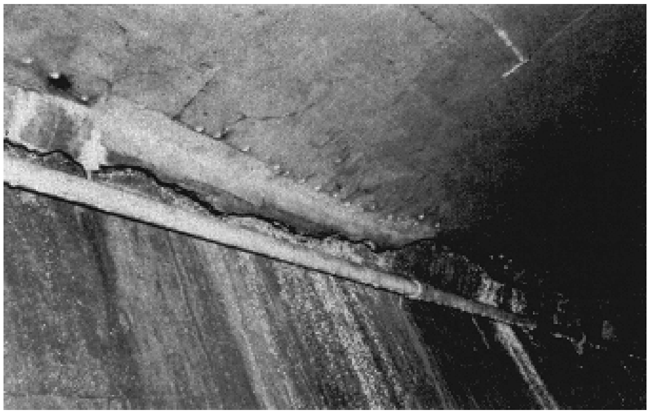

Figure 2-11. Isolated crack caused by structural overload

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.

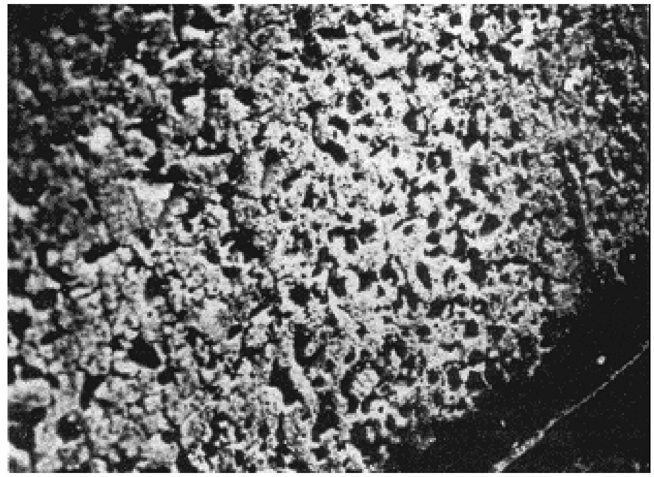

Figure 2-12. Disintegration of concrete caused by exposure to aggressive water

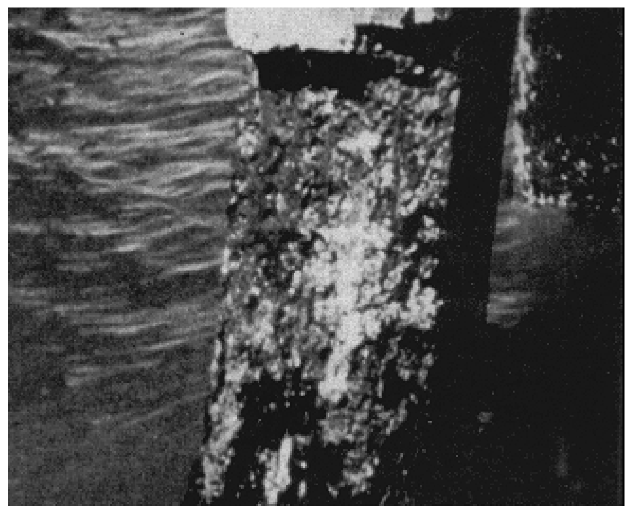

Figure 2-13. Disintegration of concrete caused by exposure to acidic water

(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

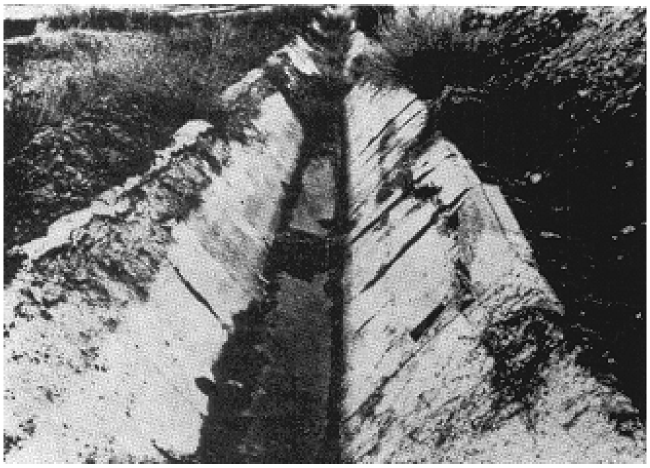

Figure 2-14. Disintegration of concrete caused by sulfate attack

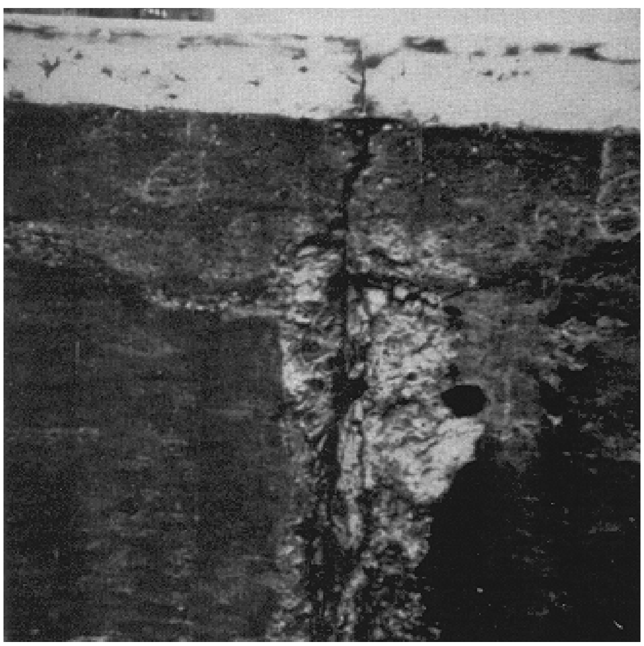

Figure 2-15. Disintegration at a monolith joint as a result of repeated cycles of freezing and thawing and barge impact

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

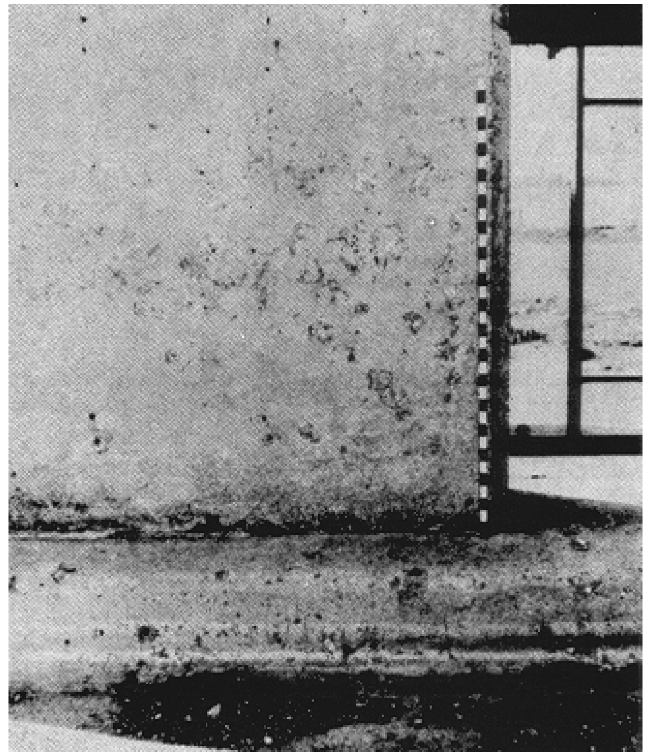

Figure 2-16. Light scaling

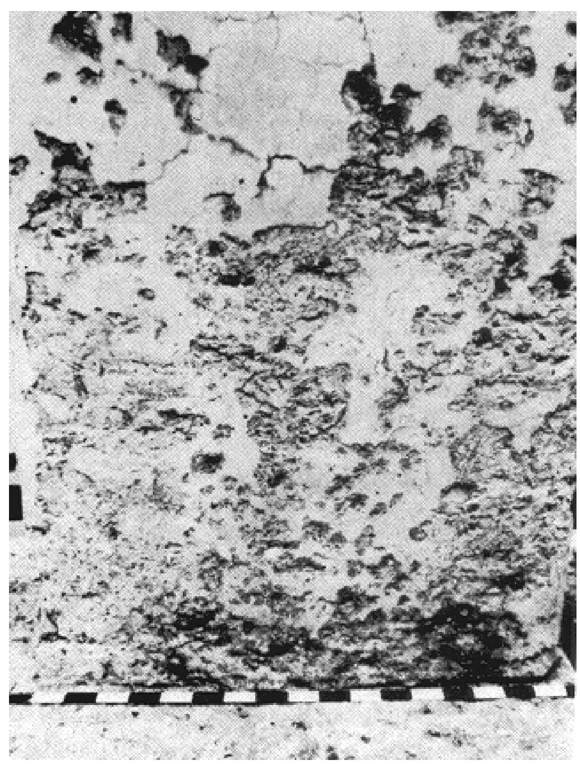

Figure 2-17. Medium scaling

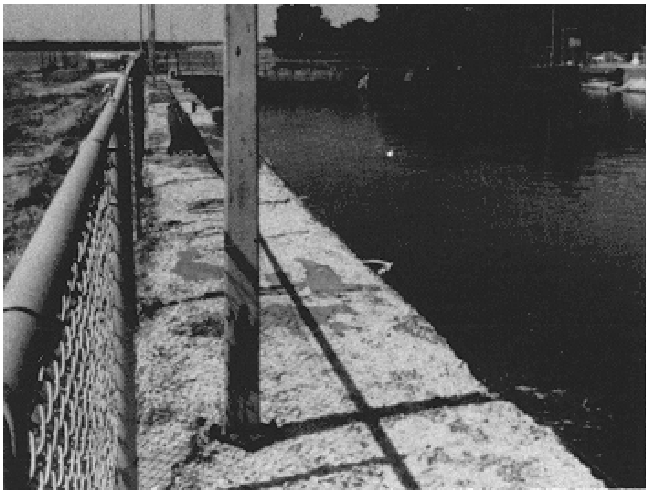

Figure 2-18. Severe scaling

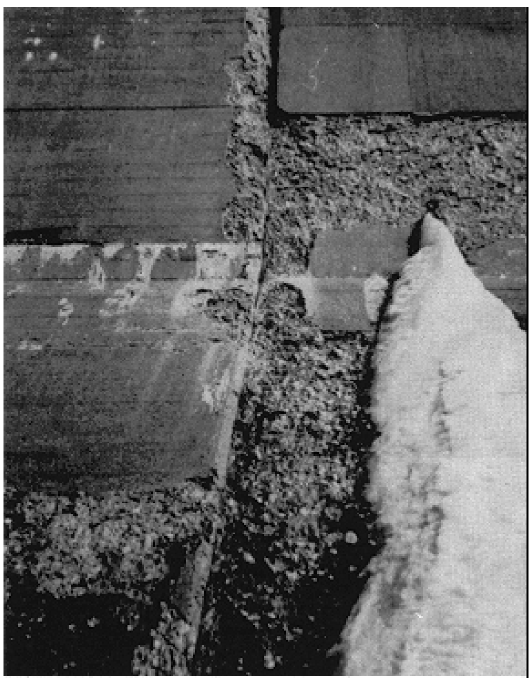

Figure 2-19. Very severe scaling

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 water-borne 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

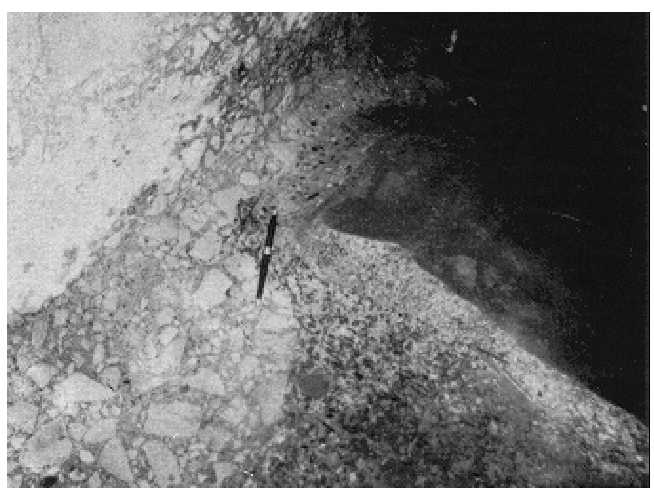

Figure 2-20. Smooth, worn, abraded concrete surface caused by abrasion of waterborne debris

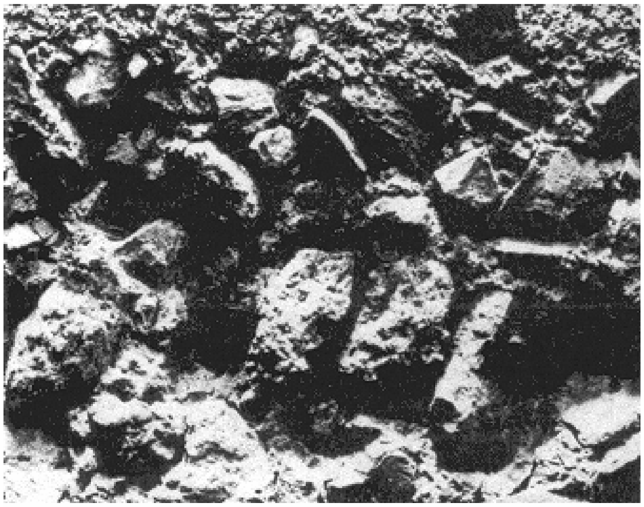

Figure 2-21. Rough, pitted concrete surface caused by cavitation

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.

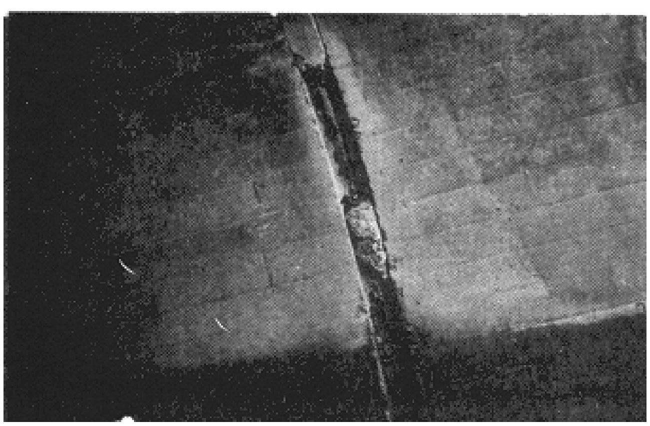

Figure 2-22. Deterioration of joint sealant

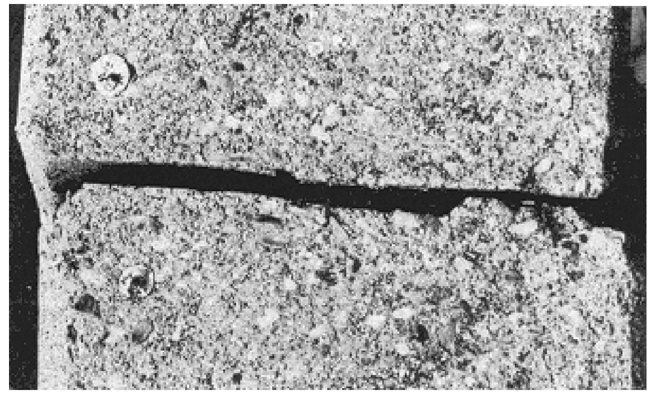

Figure 2-23. Loss of joint sealant

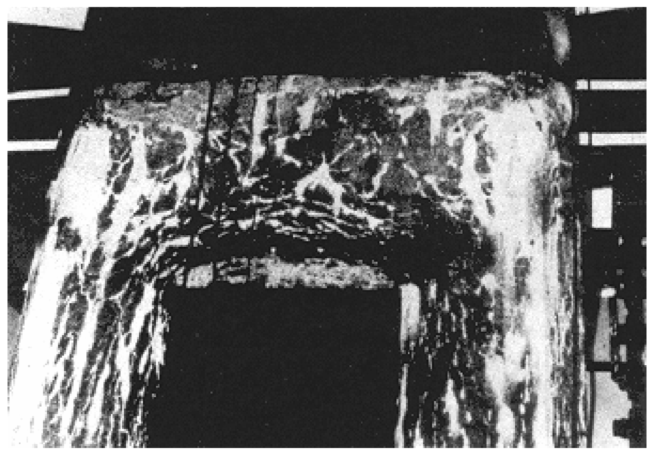

Figure 2-24. Efflorescence

(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

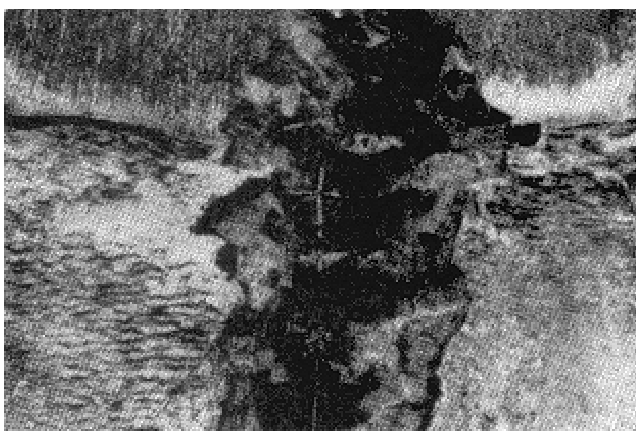

Figure 2-26. Incrustation

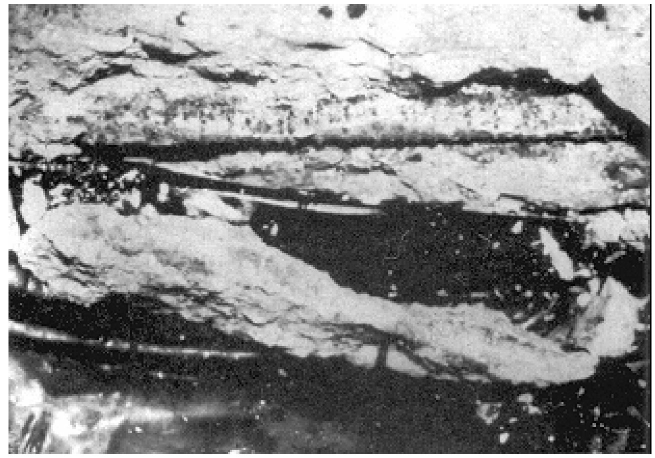

Figure 2-27. Corrosion

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

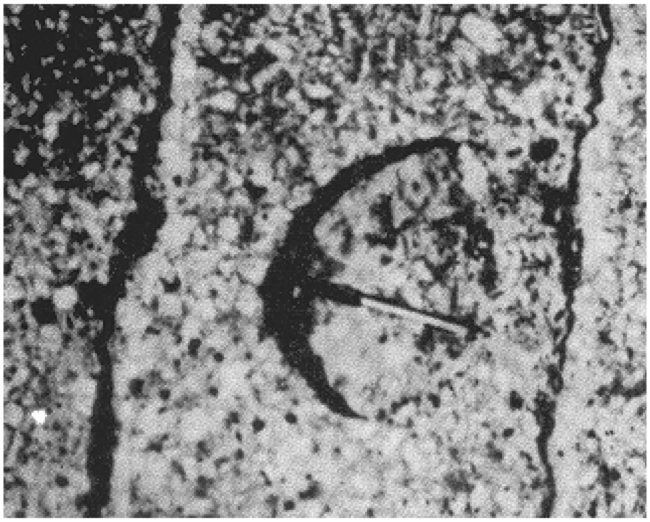

Figure 2-29. Small spall

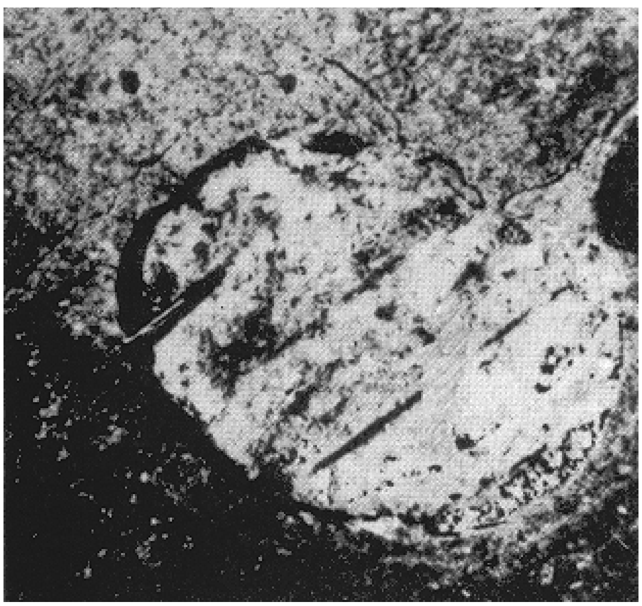

Figure 2-30. Large spall

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).

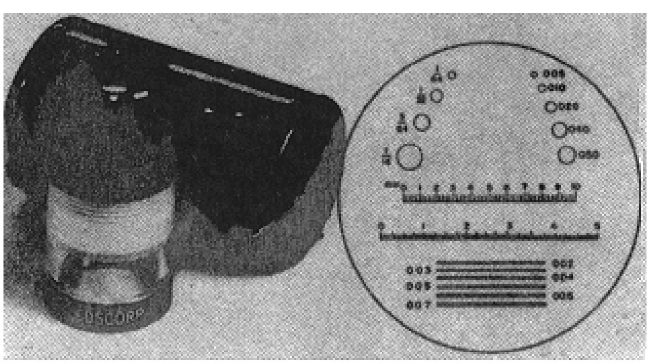

Figure 2-32. Comparator for measuring crack widths

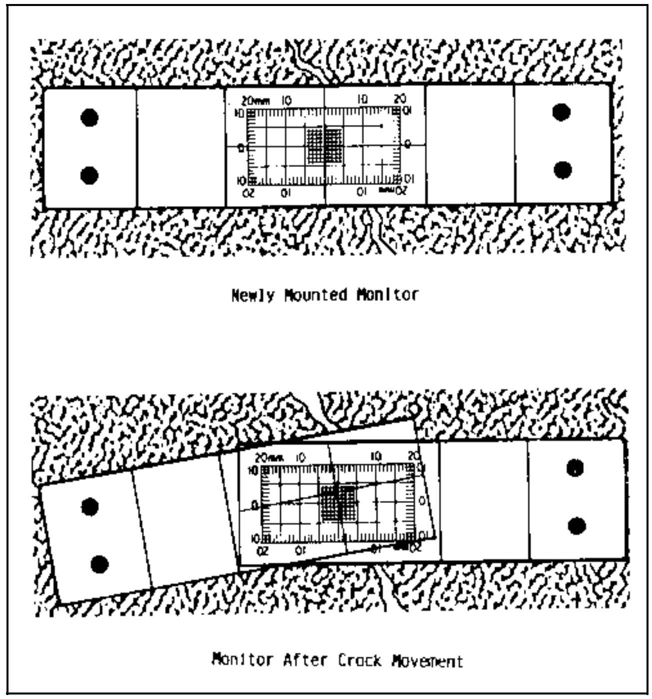

Figure 2-33. Crack monitor

(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.

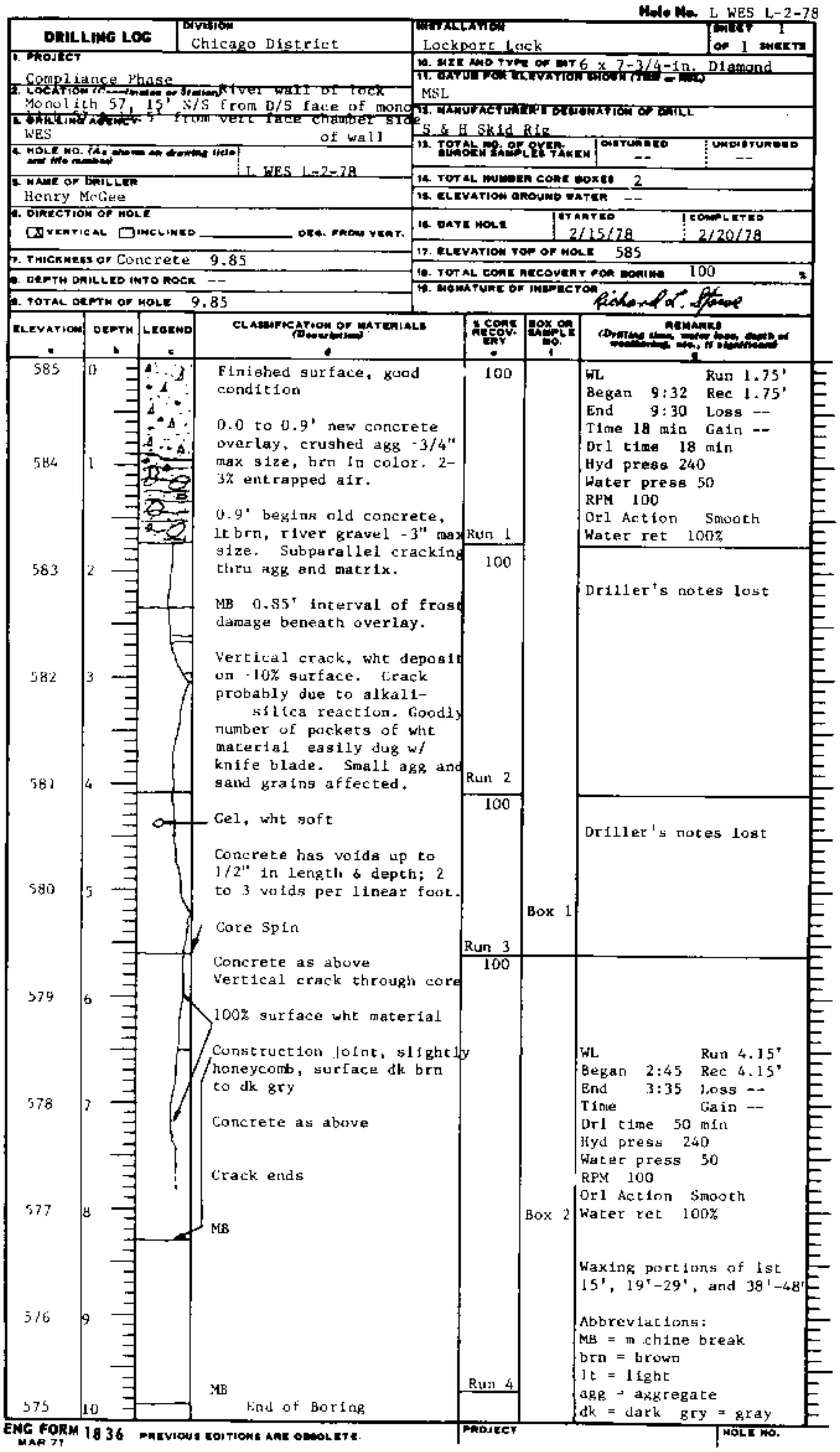

Figure 2-34. Typical information included on a drill log for concrete core

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.

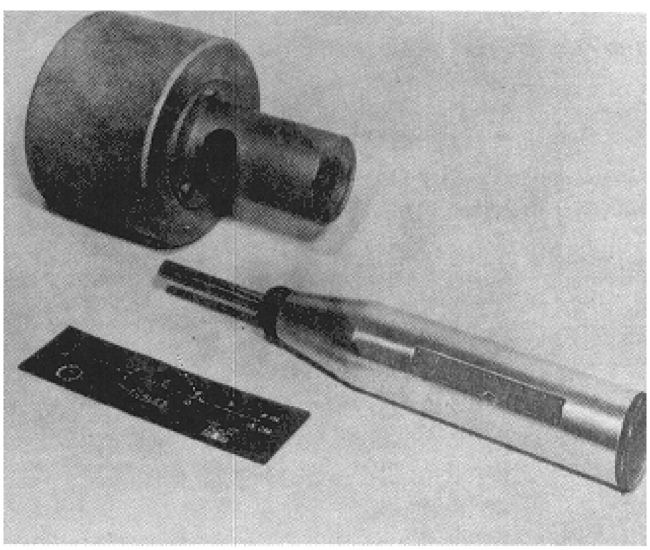

Figure 2-35. Rebound hammer

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

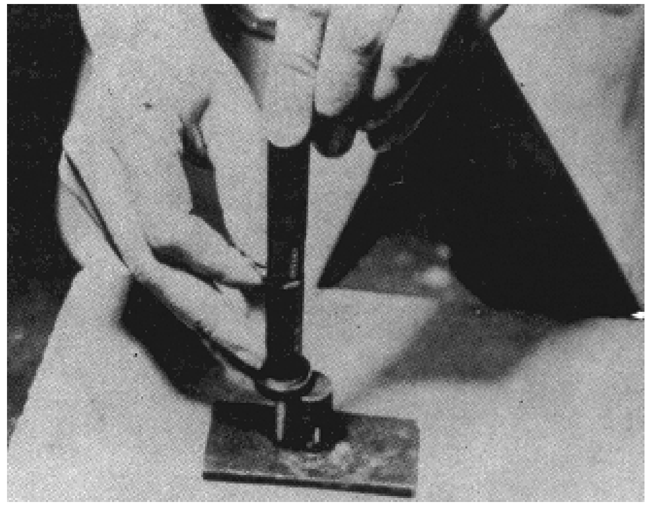

Figure 2-38. Device for measuring length of probe extending from surface of concrete

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.

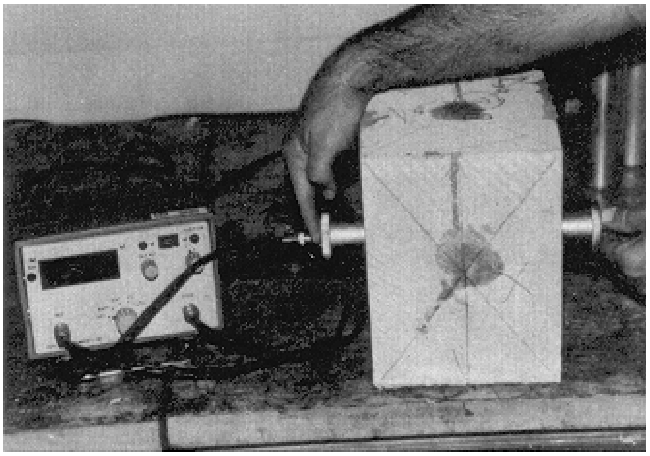

Figure 2-39. Ultrasonic pulse-velocity apparatus

(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.

Table 3-3
Relating Symptoms to Causes of Distress and Deterioration of Concrete

	Symptoms							
Causes	Construction Faults	Cracking	Disintegration	Distortion/ Movement	Erosion	Joint Failures	Seepage	Spalling
Accidental Loadings		X						X
Chemical Reactions		X	X				X	
Construction Errors	X	X				X	X	X
Corrosion		X						X
Design Errors		X				X	X	X
Erosion			X		X			
Freezing and Thawing		X	X					X
Settlement and Movement		X		X		X		X
Shrinkage	X	X		X				
Temperature Changes		X				X		X

Chapter 4
Planning and Design of Concrete Repairs

4-1. General Considerations

To achieve durable repairs it is necessary to consider the factors affecting the design and selection of repair systems as parts of a composite system. Selection of a repair material is one of the many interrelated steps; equally important are surface preparation, the method of application, construction practices, and inspection. The critical factors that largely govern the durability of concrete repairs in practice are shown in Figure 4-1. These factors must be considered in the design process so that a repair material compatible with the existing concrete substrate can be selected. Compatibility is defined as the balance of physical, chemical, and electrochemical properties and dimensions between the repair material and the concrete substrate. This balance is necessary if the repair system is to withstand all anticipated stresses induced by volume changes and chemical and electrochemical effects without distress or deterioration in a specified environment over a designated period of time. For detailed discussions of compatibility issues and the need for a rational approach to durable concrete repairs, see Emmons, Vaysburd, and McDonald (1993 and 1994).

Dimensional compatibility is one of the most critical components of concrete repair. Restrained contraction of repair materials, the restraint being provided through bond to the existing concrete substrate, significantly increases the complexity of repair projects as compared to new construction. Cracking and debonding of the repair material are often the result of restrained contractions caused by volume changes. Therefore, the specified repair material must be dimensionally compatible with the existing concrete substrate to minimize the potential for failure. Those material properties that influence dimensional

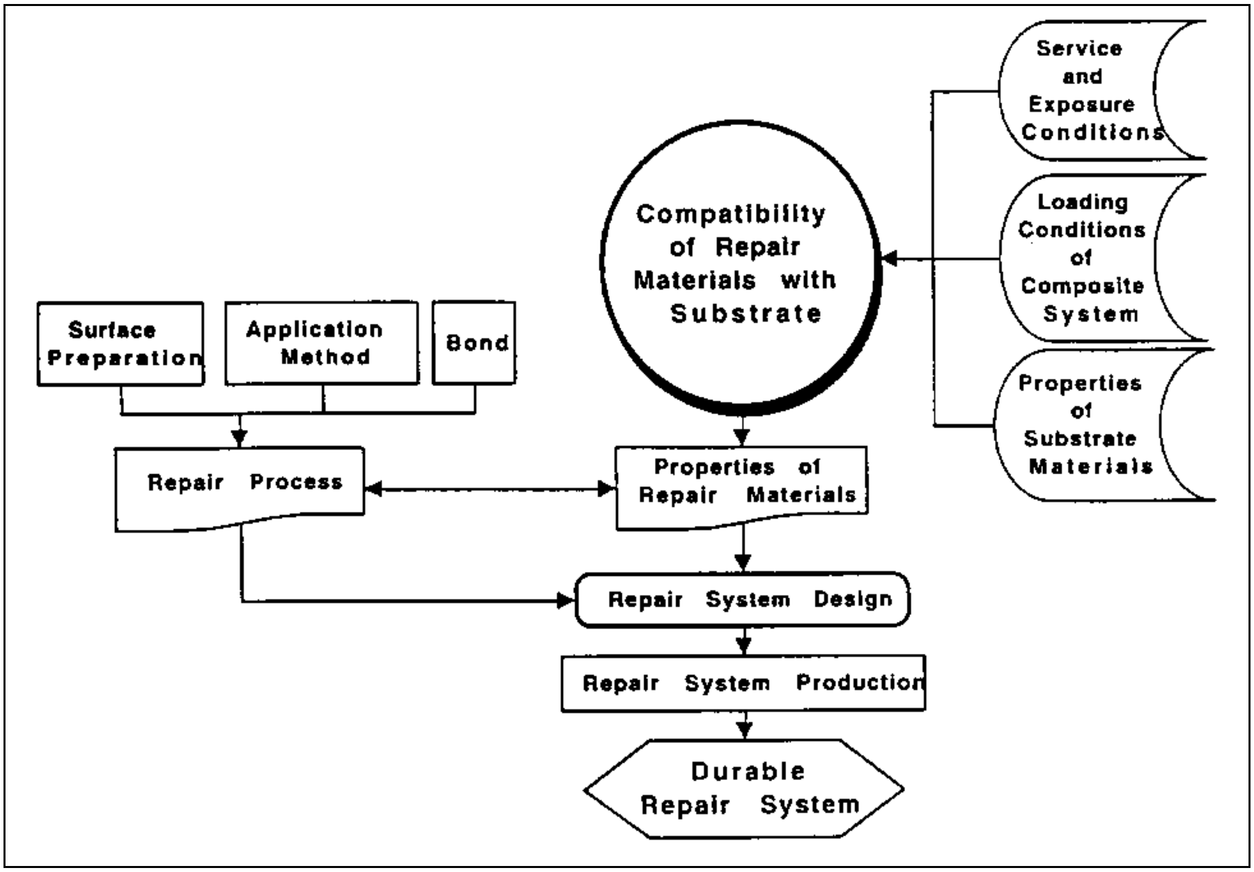

Figure 4-1. Factors affecting the durability of concrete repair systems (Emmons and Vaysburd 1995)

compatibility include drying shrinkage, thermal expansion, modulus of elasticity, and creep.

4-2. Properties of Repair Materials

In addition to conventional portland-cement concrete and mortar, there are hundreds of proprietary repair materials on the market, and new materials are continually being introduced. This wide variety of both specialty and conventional repair materials provides a greater opportunity to match material properties with specific project requirements; however, it can also increase the chances of selecting an inappropriate material. No matter how carefully a repair is made, use of the wrong material will likely lead to early repair failure (Warner 1984). Some of the material properties and their relative importance to durable repairs are discussed in the following text. These properties should be considered before any material is selected for use on a repair or rehabilitation project.

a. Compressive strength. Although there is some controversy over the required structural performance for many repairs, it is generally accepted that the repair material should have a compressive strength similar to that of the existing concrete substrate. Assuming the need for repair is not necessitated by inadequate strength, there is usually little advantage to be gained from repair materials with compressive strengths greater than that of the concrete substrate. In fact, significantly higher strengths of cementitious materials may indicate an excessive cement content which can contribute to higher heat of hydration and increased drying shrinkage. Repair of erosion-damaged concrete is one area in which increased strength (and corresponding higher erosion resistance) of the repair material is desirable.

b. Modulus of elasticity. Modulus of elasticity is a measure of stiffness with higher modulus materials exhibiting less deformation under load compared to low modulus materials. In simple engineering terms, the modulus of elasticity of a repair material should be similar to that of the concrete substrate to achieve uniform load transfer across the repaired section. A repair material with a lower modulus of elasticity will exhibit lower internal stresses thus reducing the potential for cracking and delamination of the repair.

c. Coefficient of thermal expansion. All materials expand and contract with changes in temperature. For a given change in temperature, the amount of expansion or contraction depends on the coefficient of thermal expansion for the material. Although the coefficient of expansion of conventional concrete will vary somewhat, depending on the type of aggregate, it is usually assumed to be about 10.8 millionths per degree C (6 millionths per degree F). Using repair materials such as polymers, with higher coefficients of expansion, will often result in cracking, spalling, or delamination of the repair.

(1) Depending on the type of polymer, the coefficient of expansion for unfilled polymers is 6 to 14 times greater than that for concrete. Adding fillers or aggregate to polymers will improve the situation, but the coefficient of expansion for the polymer-aggregate combinations will still be one and one-half to five times that of concrete. As a result, the polymer repair material attempts to expand or contract more than the concrete substrate. This movement, when restrained through bond to the existing concrete, induces stresses that can cause cracking as the repair material attempts to contract or buckling and spalling when the repair material attempts to expand.

(2) While thermal compatibility is most important in environments that are frequently subject to large temperature changes, it should also be considered in environments in which temperature changes are not as frequent. Also, thermal compatibility is especially important in large repairs and/or overlays.

d. Adhesion/bond. In most cases, good bond between the repair material and the existing concrete substrate is a primary requirement for a successful repair. Bond strengths determined by slant-shear tests (ASTM C 882) are often reported by material suppliers. However, these values are highly dependent on the compressive strength of the substrate portion of the test cylinder. The test procedure requires only a minimum compressive strength of 31 MPa (4,500 psi) with no maximum strength. Therefore, these values have little or no value in comparing alternate materials unless the tests were conducted with equal substrate strengths.

(1) Bond is best specified as a surface preparation requirement. The direct tensile bond test described in ACI 503R is an excellent technique for evaluating materials, surface preparation, and placement procedures. A properly prepared, sound concrete substrate will almost always provide sufficient bond strength. In many cases, bond failures between repair materials and a properly prepared concrete substrate are a result of differential thermal strains or drying shrinkage and are not a result of inadequate bond strengths.

(2) According to ACI 503.5R, polymer adhesives provide a better bond of plastic concrete to hardened concrete than can be obtained with a cement slurry or the

plastic concrete alone. However, experience indicates that the improvement in bond is less than 25 percent as compared to properly prepared concrete surfaces without adhesives.

e. Drying shrinkage. Since most repairs are made on older structures where the concrete will exhibit minimal, if any, additional drying shrinkage, the repair material must also be essentially shrinkage-free or be able to shrink without losing bond. Shrinkage of cementitious repair materials can be reduced by using mixtures with very low w/c or by using construction procedures that minimize the shrinkage potential. Examples include dry-pack and preplaced-aggregate concrete. However, proprietary materials are being used in many repairs, often with undesirable results.

(1) A random survey of data sheets for cement-based repair materials produced in this country showed that drying shrinkage data was not even reported by some manufacturers. In those cases where data was reported, manufacturers tended to use a variety of tests and standards to evaluate the performance of their products. This arbitrary application and modification of test methods has resulted in controversy and confusion in the selection and specification of repair materials. Consequently, a study was initiated, as part of the REMR research program, to select a reliable drying shrinkage test and to develop performance criteria for selecting cement-based repair materials (Emmons and Vaysburd 1995).

(a) Three test methods are currently being evaluated in laboratory and field studies: ASTM C 157 (Modified); Shrinkage Cracking Ring; and German Angle Method. The ASTM test method, with modified curing conditions and times for length change measurements, has been used to develop preliminary performance criteria for drying shrinkage. In the modified procedure, materials are mixed and cured for 24 hr in accordance with manufacturer's recommendations. When no curing is recommended, specimens are cured in air at 50 percent relative humidity. When damp curing is recommended, specimens are placed in a moist curing room. No curing compounds are used. Following the 24 hr curing period, specimens are stored in air at 50 percent relative humidity with length change measurements at 1, 3, 28, and 60 days after casting.

(b) The ASTM C 157 (Modified) test method has been used to evaluate the drying shrinkage of 46 commercially available patching materials (Gurjar and Carter 1987). Test results at 28 days were sorted and categorized by Emmons and Vaysburd (in preparation) as shown in Figure 4-2. Shrinkage of conventional concrete

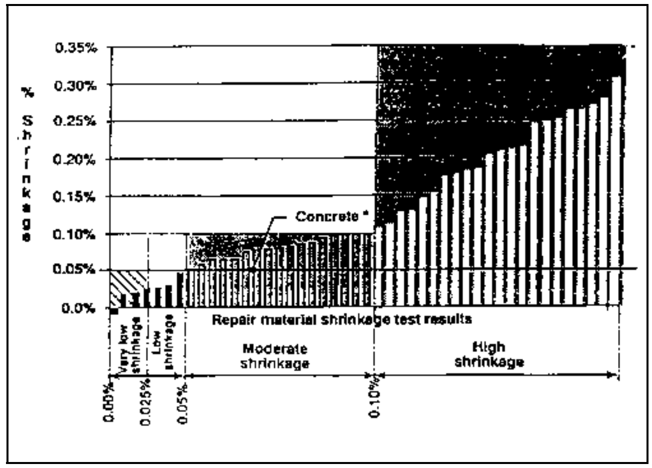

Figure 4-2. Classification of repair materials based on drying shrinkage (Emmons and Vaysburd 1995)

(0.05 percent at 28 days) was selected as a benchmark. Eighty-five percent of the materials tested had a higher shrinkage than that of concrete.

(2) Based on this work, a maximum shrinkage of 0.04 percent at 28 days (ASTM C 157 (modified)) has been proposed as preliminary performance criteria for dimensionally compatible repair materials. Final performance criteria will be selected upon completion of current large-scale laboratory and field tests to establish a correlation between laboratory test results and field performance.

f. Creep. In structural repairs, creep of the repair material should be similar to that of the concrete substrate, whereas in protective repairs higher creep can be an advantage. In the latter case, stress relaxation through tensile creep reduces the potential for cracking. It is unfortunate that most manufacturers make no mention of creep in their literature and are unable to supply basic values or to advise on environmental effects. Current tensile and compressive creep tests on selected repair materials should provide some insight into the role of creep in the overall repair system.

g. Permeability. Good quality concrete is relatively impermeable to liquids, but when moisture evaporates at a surface, replacement liquid is pulled to the evaporating surface by diffusion. If impermeable materials are used for large patches, overlays, or coatings, moisture that diffuses through the base concrete can be trapped between the substrate and the impermeable repair material. The entrapped moisture can cause failure at the bond line or critically saturate the substrate and, in the case of nonair-entrained concrete, can cause the substrate to fail if it is subjected to repeated cycles of freezing and thawing. Entrapped moisture can be a particularly troublesome problem with Corps hydraulic structures that are subject to freezing and thawing. Materials with low water absorption and high water vapor transmission characteristics are desirable for most repairs.

4-3. Application and Service Conditions

The conditions under which the repair material will be placed and the anticipated service or exposure conditions can have a major impact on design of a repair and selection of the repair material. The following factors should be considered in planning a repair strategy (Warner 1984).

a. Application conditions.

(1) Geometry. The depth and orientation of a repair section can influence selection of the repair material. In thick sections, heat generated during curing of some repair materials can result in unacceptable thermal stresses. Also, some materials shrink excessively when placed in thick layers. Some materials, particularly cementitious materials, will spall if placed in very thin layers. In contrast, some polymer-based materials can be placed in very thin sections. The maximum size of aggregate that can be used will be dictated by the minimum thickness of the repair. The repair material must be capable of adhering to the substrate without sagging when placed on vertical or overhead surfaces without forming.

(2) Temperature. Portland-cement hydration ceases at or near freezing temperatures, and latex emulsions will not coalesce to form films at temperature below about 7 ºC (45 °F). Other materials may be used at temperatures well below freezing, although setting times may be increased. High temperatures will make many repair materials set faster, decrease their working life, or preclude their use entirely.

(3) Moisture. A condition peculiar to hydraulic structures is the presence of moisture or flowing water in the repair area. Generally, flowing water must be stopped by grouting, external waterproofing techniques, or drainage systems prior to repair. Some epoxy and polymer materials will not cure properly in the presence of moisture while others are moisture insensitive. Materials suitable for spall repair of wet concrete surfaces have been identified by Best and McDonald (1990a).

(4) Location. Limited access to the repair site may restrict the type of equipment, and thus the type of material that can be used for repair. Also, components of some repair materials are odorous, toxic, or combustible. Obviously, such materials should not be used in poorly ventilated areas or in areas where flammable materials aren't permitted.

b. Service conditions.

(1) Downtime. Materials with rapid strength gain characteristics that can be easily placed with minimal waste should be used when the repaired structure must be returned to service in a short period of time. Several types of rapid-hardening cements and patching materials are described in REMR Technical Note CS-MR-7.3 (USAEWES 1985g).

(2) Traffic. If the repair will be subject to heavy vehicular traffic, a high-strength material with good abrasion and skid resistance is necessary.

(3) Temperature. A material with a coefficient of thermal expansion similar to that of the concrete substrate should be used for repairs subject to wide fluctuations in temperature. High-service temperatures may adversely affect the performance of some polymer materials. Resistance to cycles of freezing and thawing will be very important in many applications.

(4) Chemical attack. Acids and sulfates will cause deterioration in cement-based materials while polymers are resistant to such chemical attack. However, strong solvents may attack some polymers. Soft water is corrosive to portland-cement materials.

(5) Appearance. If it is necessary to match the color and texture of the original concrete, many, if not most, of the available repair materials will be unsuitable. Portland-cement mixtures with materials and proportions similar to those used in the original construction are necessary where appearance is a major consideration. Procedures for repair of architectural concrete are described by Dobrowolski and Scanlon (1984).

(6) Service life. The function and remaining service life of the structure requiring repair should be considered in selection of a repair material. An extended service life requirement may dictate the choice of repair material regardless of cost. On the other hand, perhaps a lower cost, less durable, or more easily applied material can be used if the repair is only temporary.

4-4. Material Selection

Most repair projects will have unique conditions and special requirements that must be thoroughly examined before the final repair material criteria can be established. Once the criteria for a dimensionally compatible repair have been established, materials with the properties necessary to meet these criteria should be identified. A variety of repair materials have been formulated to provide a wide range of properties. Since these properties will affect the performance of a repair, selecting the correct material for a specific application requires careful study. Properties of the materials under consideration for a given repair may be obtained from manufacturer's data sheets, the REMR Repair Materials Database and *The REMR Notebook* (USAEWES 1985), evaluation reports, contact with suppliers, or by conducting tests.

a. Material properties.

(1) Manufacturer's data. Values for compressive strength, tensile strength, slant-shear bond, and modulus of elasticity are frequently reported in material data sheets provided by suppliers. However, other material properties of equal or greater importance, such as drying shrinkage, tensile bond strength, creep, absorption, and water vapor transmission, may not be reported.

(a) Experience indicates that the material properties reported in manufacturer's data sheets are generally accurate for the conditions under which they were determined. However, the designer should beware of those situations in which data on a pertinent material property is not reported. Unfavorable material characteristics are seldom reported.

(b) Material properties pertinent to a given repair should be requested from manufacturers if they are not included in the data sheets provided. General descriptions of materials, such as compatible, nonshrink, low shrinkage, etc., should be disregarded unless they are supported by data determined in accordance with standardized test methods. Material properties determined in accordance with "modified" standard tests should be viewed with caution, particularly if the modifications are not described.

(2) Repair Material Database. The REMR Repair Materials Database is described in Section 4-5. The computerized database provides rapid access to the results of tests conducted by the Corps and others; however, less than 25 percent of the available repair materials have been evaluated to date. *The REMR Notebook* currently contains 128 Material Data Sheets that include material descriptions, uses and limitations, available specifications, manufacturer's test results, and Corps test results. In addition, *The REMR Notebook* contains a number of Technical Notes that describe materials and procedures that can be used for maintenance and repair of concrete.

(3) Material suppliers. Reputable material suppliers can assist in identifying those materials and associated properties that have proven successful in previous repairs provided they are made aware of the conditions under which the material will be applied and the anticipated service conditions.

(4) Conduct tests. The formulations for commercially available materials are subject to frequent modifications for a number of reasons including changes in ownership, changes in raw materials, and new technology. Sometimes these modifications result in changes in material properties without corresponding changes to the manufacturer's data sheets or notification by the material supplier. Consequently, testing of the repair material is recommended to ensure compliance with design criteria if durability of the repair is of major importance, or the volume of repair is large (Krauss 1994).

b. Selection considerations. Concrete repair materials have been formulated to provide a wide range of properties; therefore, it is likely that more than one type of material will satisfy the design criteria for durable repair of a specific structure. In this case, other factors such as ease of application, cost, and available labor skills and equipment should be considered in selection of the repair material. To match the properties of the concrete substrate as closely as possible, portland-cement concrete or similar cementitious materials are frequently the best choices for repair. There are some obvious exceptions such as repairs that must be resistant to chemical attack. However, an arbitrary decision to repair like with like will not necessarily ensure a durable repair: The new repair material must be dimensionally compatible with the existing substrate, which has often been in place for many years.

4-5. Repair Materials Database

The Corps of Engineers Repair Material Database was developed to provide technology transfer of results from evaluations of commercial repair products performed under the REMR Research Program. The database contains manufacturer's information on uses, application procedures, limitations and technical data for approximately 1,860 commercially available repair products. In addition, Corps of Engineers test results are included for

280 products and test results from other sources for 120 products. Results of material evaluations performed by the Corps of Engineers are added to the database as reports are published. Database organization and access procedures are described in detail by Stowe and Campbell (1989) and summarized in the following.

a. Access. The database is maintained on a host computer that can be accessed by telephone via a modem using the following communication parameters:

Baud Rate:	1,200 to 9,600	Emulation:	VT-100
Data Bits:	8	Stop Bits:	1
Phone No.:	(601) 634-4223	Parity:	None

b. Operation.

(1) The database is menu driven and has help windows to facilitate its use. The products in the database are identified as either end-use or additive. An end-use product is a material that is used as purchased to make a repair, whereas an additive product is a material used in combination with other materials to produce an end-use product. The end-use products portion of the database contains products for maintenance and repair of concrete, steel, or both. The additive products portion of the database contains products that are portland-cement admixtures, binders, fibers, or special filler materials.

(2) For end-use products, product categories identify the basic type of material of which the product is composed, and for additive products, the type of end-use product for which the product is an ingredient or additive. The product uses identify the type use(s) for which the product is applicable. Keywords for searching category and use fields can be listed through the program help screens along with their definitions. Once the user selects the end-use or additive database, searches can be made by manufacturer's name, product name, product category, product use, or both category and use.

c. Assistance. For assistance or additional information regarding the database contact:

CEWES-SC-CA
3909 Halls Ferry Road
Vicksburg, MS 39180-6199
Phone: (601) 634-2814

4-6. General Categorization of Repair Approach

For ease of selecting repair methods and materials, it is helpful to divide the possible approaches into two general categories: those more suited for cracking or those more suited for spalling and disintegration. This categorization requires that some of the symptoms that were listed in Table 2-1 be regrouped as follows to facilitate selection of a repair approach:

Cracking Repair Approaches	Spalling and Disintegration Repair Approaches
Construction faults (some)	Construction faults (some)
Cracking	Disintegration
Seepage	Erosion
	Spalling

Note that distortion or movement and joint sealant failures, which were listed in Table 2-1, are not included in these categories. These are special cases that must be handled outside the process to be outlined in this chapter. Joint repair and maintenance are covered in Chapter 7. Distortion and movement are usually indications of settlement or of chemical reactions causing expansion of concrete such as severe alkali-aggregate reaction. Repairs for these conditions are beyond the scope of this manual. Materials and methods more suited for crack repairs are described in Section 4-7, while those more suited for spalling and disintegration repairs are described in Section 4-8.

4-7. Repair of Cracking

The wide variety of types of cracking described in Chapter 2 suggests that there is no single repair method that will work in all instances. A repair method that is appropriate in one instance may be ineffective or even detrimental in another. For example, if a cracked section requires tensile reinforcement or posttensioning to be able to carry imposed loads, routing and sealing the cracks with a sealer would be ineffective. On the other hand, if a concrete section has cracked because of incorrect spacing of contraction joints, filling the cracks with a high-strength material such as epoxy will only cause new cracking to occur as the concrete goes through its next contraction cycle.

a. Considerations in selecting materials and methods. Prior to the selection of the appropriate material and method for repair of cracking, the following questions should be answered (Johnson 1965):

(1) What is the nature of the cracking? Are the cracks in pattern or isolated? What is the depth of the cracking? Are the cracks open or closed? What is the extent of the cracking?

(2) What was the cause of the cracking?

(3) What was the exact mechanism of the cracking? This question requires that an analysis beyond the simple identification of the cause be conducted. For example, if the cause of the cracking has been determined to be drying shrinkage, it should then be determined whether the occurrence is the result of unusual restraint conditions or excess water content in the concrete. Understanding the mechanism will help to ensure that the same mistake is not repeated.

(4) Is the mechanism expected to remain active? Whether the causal mechanism is or is not expected to remain active will play a major role in the process to select a repair material and method.

(5) Is repair feasible? Repair of cracking caused by severe alkali-aggregate reaction may not be feasible.

(6) Should the repair be treated as spalling rather than cracking? If the damage is such that future loss of concrete mass is probable, treatment of the cracks may not be adequate. For example, cracking caused by corrosion of embedded metal or by freezing and thawing would be best treated by removal and replacement of concrete rather than by one of the crack repair methods.

(7) What will be the future movement of the crack? Is the crack active or dormant? The repair materials and techniques for active cracks are much different from the repair materials and techniques for dormant cracks. Many cracks which are still active have been "welded" together with injected epoxies only to have the crack reoccur alongside the original crack.

(8) Is strengthening across the crack required? Is the crack structural in nature? Has a structural analysis been performed as a part of the repair program?

(9) What is the moisture environment of the crack?

(10) What will be the degree of restraint for the repair material?

b. Materials and methods to consider. Once these questions have been answered, potential repair materials and methods may be selected with the procedures shown in Figures 4-3 and 4-4. All of the materials and methods listed in these figures are described in Chapter 6. In most cases, more than one material or method will be applicable. Final selection of the repair material and method must take into account the considerations discussed in Sections 4-1 through 4-4 and other pertinent project-specific conditions.

4-8. Repair of Spalling and Disintegration

Spalling and disintegration are only symptoms of many types of concrete distress. There is no single repair method that will always apply. For example, placing an air-entrained concrete over the entire surface of concrete

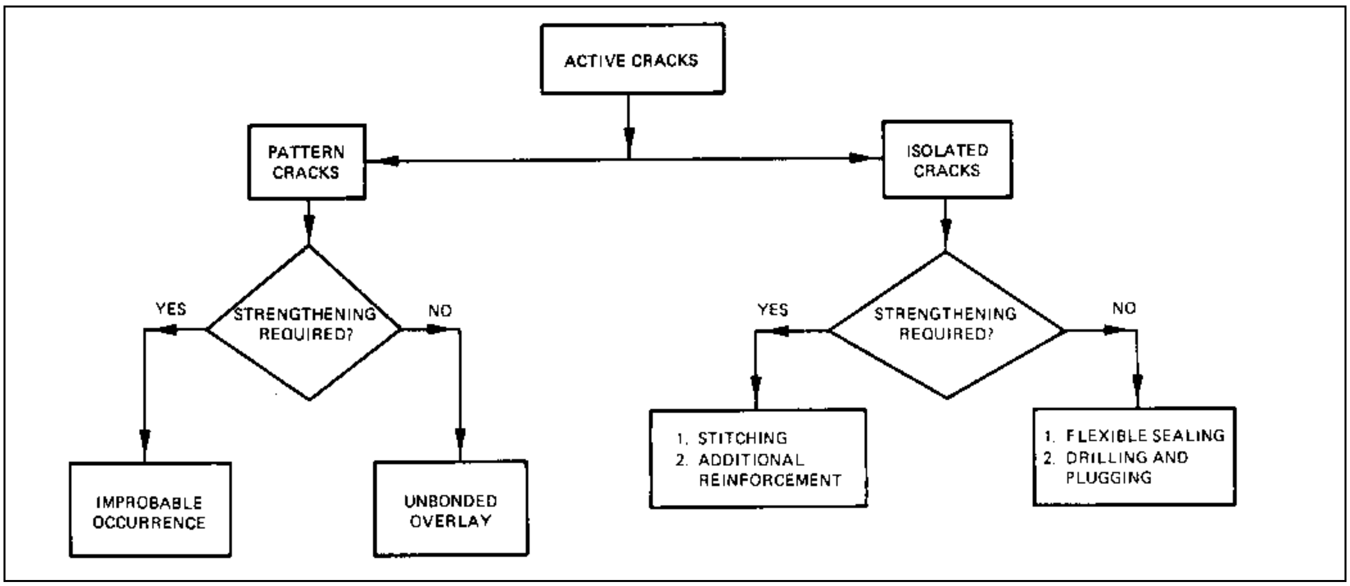

Figure 4-3. Selection of repair method for active cracks (after Johnson 1965)

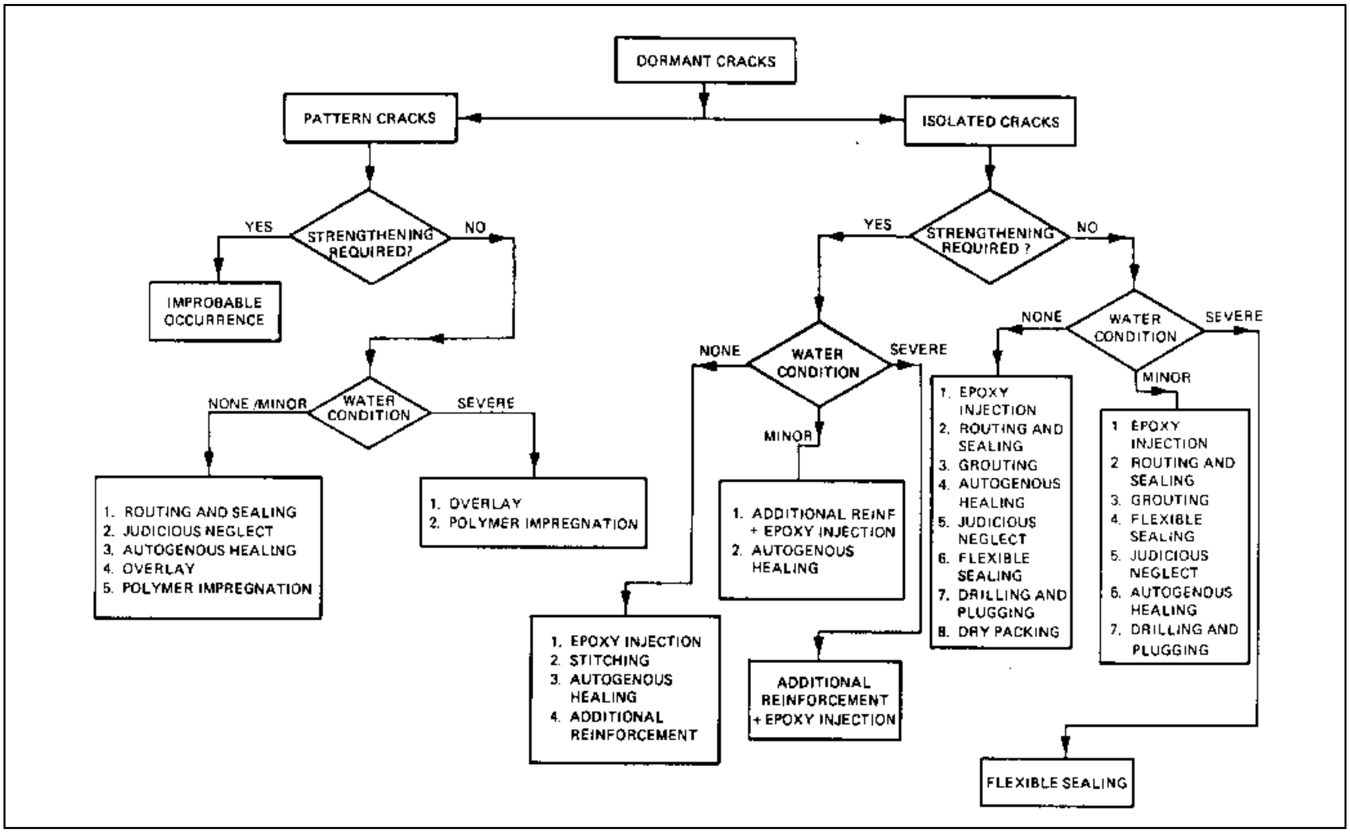

Figure 4-4. Selection of repair method for dormant cracks (after Johnson 1965)

that is deteriorating because of freezing and thawing may be a sound repair method. Use of the same technique on concrete deteriorating from strong acid attack may not be effective.

a. Considerations in selecting materials and methods. Selection of a method for repairing spalling or disintegration involves answering the following questions:

(1) What is the nature of the damage?

(2) What was the cause of the damage?

(3) Is the cause of the damage likely to remain active? If the answer to this question is yes, procedures for eliminating the factors contributing to the cause of damage should be considered. For example, if poor design details have contributed to freezing and thawing damage by allowing water to pond on a structure, drainage may be improved as part of the repair. Similarly, if attack by acid water has caused disintegration of a concrete surface, elimination of the source of the acid may eliminate acid attack as a cause of future problems. Knowledge of the future activity of a causative factor is essential in the selection of a repair method. In the example just cited, elimination of the source of acidity might make possible a satisfactory repair with portland-cement-based material rather than a more expensive coating.

(4) What is the extent of the damage? Is the damage limited to isolated areas or is there major spalling or disintegration? The answer to this question will assist in the selection of a repair material or method that is economical and appropriate for the problem at hand.

b. General repair approach. Once these questions have been answered, a general repair approach can be selected from Table 4-1, which presents a comparison of the possible causes of spalling and disintegration symptoms and the general repair approaches that may be appropriate for each case. Table 4-2 relates the repair approaches shown in Table 4-1 to specific repair methods that are described in Chapter 6. As is true for repairing cracks, there will usually be several possible methods. The final selection must take into account the general considerations discussed in Sections 4-1 through 4-4 along with other pertinent project-specific considerations.

Table 4-1
Causes and Repair Approaches for Spalling and Disintegration

Cause	Deterioration Likely to Continue: Yes	Deterioration Likely to Continue: No	Repair Approach
1. Erosion (abrasion, cavitation)	X		Partial replacement Surface coatings
2. Accidental loading (impact, earthquake)		X	Partial replacement
3. Chemical reactions			
Internal	X		No action Total replacement
External	X	X	Partial replacement Surface coatings
4. Construction errors (compaction, curing, finishing)	X		Partial replacement Surface coatings No action
5. Corrosion	X		Partial replacement
6. Design errors	X	X	Partial or total replacement based on future activity
7. Temperature changes (excessive expansion caused by elevated temperature and inadequate expansion joints)	X		Redesign to include adequate joints and partial replacement
8. Freezing and thawing	X		Partial replacement No action

NOTE: This table is intended to serve as a general guide only. It should be recognized that there are probably exceptions to all of the items listed.

Table 4-2
Repair Methods for Spalling and Disintegration

Repair Approach	Repair Method
1. No action	Judicious neglect
2. Partial replacement (replacement of only damaged concrete)	Conventional concrete placement Drypacking Jacketing Preplaced-aggregate concrete Polymer impregnation Overlay Shotcrete Underwater placement High-strength concrete
3. Surface coating	Coatings Overlays
4. Total replacement of structure	Remove and replace

NOTE: Individual repair methods are discussed in Chapter 6, except those for surface coatings which are discussed in Chapter 7.