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Causes, Evaluation, and Repair of Cracks in Concrete Structures

Reported by ACI Committee 224

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Causes, Evaluation, and Repair of Cracks in Concrete Structures

Reported by ACI Committee 224

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The causes of cracks in concrete structures are summarized. The procedures used to evaluate cracking in concrete and the principal techniques for the repair of cracks are presented. The key methods of crack repair are discussed, and guidance is provided for their proper application.

Keywords: alkali-silica reaction; alkali-carbonate reaction; autogenous healing; concrete; consolidation; corrosion; cracking (fracturing); drying shrinkage; epoxy resins; evaluation; grouting; heat of hydration; mass concrete; methacrylates; mixture proportion; overlay; plastic; polymer; precast concrete; prestressed concrete; reinforced concrete; repair; resins; settlement shrinkage; shrinkage; slab-on-ground; specification; thermal expansion; volume change.

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PREFACE

Cracks in concrete have many causes. They may affect appearance only, or they may indicate significant structural distress or a lack of durability. Cracks may represent the total extent of the damage, or they may point to problems of greater magnitude. Their significance depends on the type of structure, as well as the nature of the cracking. For example, cracks that are acceptable for buildings may not be acceptable in water-retaining structures.

Good crack repair techniques depend on knowing the causes and selecting appropriate repair procedures that take these causes into account; otherwise, the repair may only be temporary. Successful long-term repair procedures must address the causes of the cracks as well as the cracks themselves.

This report is intended to serve as a tool in the process of crack evaluation and repair of concrete structures.

The causes of cracks in concrete are summarized along with the principal procedures used for crack control. Both plastic and hardened concrete are considered. The importance of design, detailing, construction procedures, concrete proportioning, and material properties are discussed.

The techniques and methodology for crack evaluation are described. The need to determine the causes of cracking as a necessary prerequisite to repair is emphasized. The selection of successful repair techniques should consider the causes of cracking, whether the cracks are active or dormant, and the need for repair. Criteria for the selection of crack repair procedures are based on the desired outcome.

Twelve methods of crack repair are presented, including the techniques, advantages and disadvantages, and areas of application for each.

CHAPTER 1—CAUSES AND CONTROL OF CRACKING

1.1—Introduction

This chapter presents a brief summary of the causes of cracks and means for their control. Cracks are categorized as occurring either in plastic concrete or hardened concrete (Kelly 1981; Price 1982). In addition to the information provided herein, further details are presented in ACI 224R and articles by Carlson et al. (1979), Kelly (1981), Price (1982), and Abdun-Nur (1983). Additional references are cited throughout the chapter.

1.2—Cracking of plastic concrete

1.2.1 *Plastic shrinkage cracking*—When moisture evaporates from the surface of freshly placed concrete faster than it is replaced by bleed water, the surface concrete shrinks. Due to the restraint provided by the concrete below the drying surface layer, tensile stresses develop in the weak, stiffening plastic concrete. This results in shallow cracks of varying depths that may form a random, polygonal pattern, or be essentially parallel to one another (Fig. 1.1). These cracks may be fairly wide (as much as 1/8 in. [3 mm]) at the surface. They range from a few inches to many feet in length, and are spaced from a few inches (millimeters) to as much as 10 ft (3 m) apart. Plastic shrinkage cracks begin as shallow cracks, but can become full-depth cracks later in the life of the concrete.

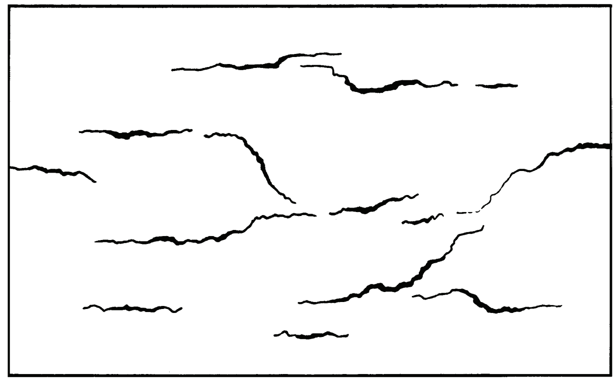


Fig. 1.1—Typical plastic shrinkage cracking (Price 1982).

Plastic shrinkage cracking is usually associated with the rapid loss of moisture caused by a combination of factors that include high air and concrete temperatures, low relative humidity, and high wind velocity at the surface of the concrete. Concrete with lower amounts of bleed water, such as those containing mineral admixtures (especially silica fume) have a greater tendency to undergo plastic shrinkage cracking than concrete with a greater tendency to bleed.

Because plastic shrinkage cracking is due to a differential volume change in the plastic concrete, successful control measures require a reduction in the relative volume change between the surface and other portions of the concrete.

Steps can be taken to prevent rapid moisture loss due to hot weather and dry winds (ACI 224R, 302.1R, and 305R). These measures include the use of fog nozzles to saturate the air above the surface and the use of plastic sheeting to cover the surface between finishing operations. Windbreaks to reduce the wind velocity and sunshades to reduce the surface temperature are also helpful. It is good practice to schedule flatwork after the windbreaks have been erected. During hot, windy weather with low humidity, it is sometimes advisable to reschedule the concrete placement or to initiate concrete operations at night.

1.2.2 *Settlement cracking*—Concrete has a tendency to continue to consolidate after initial placement, vibration, and finishing. During this period, the plastic concrete may be locally restrained by reinforcing steel, a previous concrete placement, or formwork. This local restraint may result in voids, cracks, or both, adjacent to the restraining element (Fig. 1.2). When associated with reinforcing steel, settlement cracking increases with increasing bar size, increasing slump, and decreasing cover (Dakhil et al. 1975); this is shown in Fig. 1.3 for a limited range of these variables. The degree of settlement cracking may be intensified by insufficient vibration or by the use of leaking or highly flexible forms. Suprenant and Malisch (1999) demonstrated that the addition of fibers can reduce the formation of settlement cracks.

The following items will reduce settlement cracking:

- Form design following ACI 347;
- Concrete vibration (and revibration) (ACI 309R);
- Provision of a time interval between the placement of concrete in columns or deep beams and the placement of concrete in slabs and beams (ACI 309.2R);

- Use of the lowest possible slump;
- An increase in concrete cover; and
- Addition of fibers.

1.3—Cracking of hardened concrete

1.3.1 *Drying shrinkage*—A common cause of cracking in concrete is restrained drying shrinkage. Drying shrinkage is caused by the loss of moisture from the cement paste constituent, which can shrink by as much as 1%. Fortunately, aggregate particles provide internal restraint that reduces the magnitude of this volume change to about 0.06%. On the other hand, concrete tends to expand when wetted (the volume increase can be the same order of magnitude as that observed due to shrinkage).

These moisture-induced volume changes are a characteristic of concrete. If the shrinkage of concrete could take place without restraint, the concrete would not crack. It is the combination of shrinkage and restraint (provided by another part of the structure, by the subgrade, or by the moist interior of the concrete itself) that causes tensile stresses to develop. When the tensile strength of the material is exceeded, concrete will crack. Cracks may propagate at much lower stresses than are required to cause crack initiation (ACI 446.1R).

In massive concrete elements, tensile stresses are caused by differential shrinkage between the surface and the interior concrete. The higher shrinkage at the surface causes cracks to develop that may, with time, penetrate deeper into the concrete.

The magnitude of the tensile stresses induced by volume change is influenced by a combination of factors, including the amount and rate of shrinkage, the degree of restraint, the modulus of elasticity, and the amount of creep. The amount of drying shrinkage is influenced mainly by the amount and type of aggregate and the cement paste (cement and water) content of the mixture. As the quantity of aggregate increases, the shrinkage decreases (Pickett 1956). The higher the stiffness of the aggregate, the more effective it is in reducing the shrinkage of the concrete; that is, the shrinkage of concrete containing sandstone aggregate may be more than twice that of concrete with granite, basalt, or high-quality limestone (Carlson 1938). The higher the water and cement contents, the greater the amount of drying shrinkage (U.S. Bureau of Reclamation 1975; Schmitt and Darwin 1999; Darwin et al. 2004).

Surface crazing (alligator pattern) on walls and slabs is an example of drying shrinkage on a small scale. Crazing usually occurs when the surface layer of the concrete has a higher water content than the interior concrete. The result is a series of shallow, closely spaced, fine cracks.

A procedure that will help reduce settlement cracking, as well as drying shrinkage in walls, is reducing the water content of the concrete as the wall is placed from the bottom to the top (U.S. Bureau of Reclamation 1975; ACI 304R). Using this procedure, bleed water from the lower portions of the wall will tend to equalize the water content within the wall. To be successful, this procedure needs careful control of the concrete and proper consolidation.

Shrinkage cracking can be controlled by using contraction joints and proper detailing of the reinforcement. Shrinkage cracking may also be reduced or even eliminated by using shrinkage-compensating cement or a shrinkage-compensating admixture. The reduction or elimination of subslab restraint can also be effective in reducing shrinkage cracking in slabs-on-ground (Wimsatt et al. 1987). In cases where crack control is particularly important, the minimum requirements of ACI 318 may not be adequate. These points are discussed in greater detail in ACI 224R, which describes additional construction practices designed to help control the drying shrinkage cracking that does occur, and in ACI 224.3R, which describes the use and function of joints in concrete construction.

Autogenous shrinkage "is a special case of drying shrinkage" (Mindess et al. 2003) that results from self-desiccation (internal drying) in concretes with water-cementitious material ratios (*w/cm*) below 0.42, but most often observed at a *w/cm* below 0.30. Thus, it is a problem often associated with high-strength concretes. Autogenous shrinkage occurs without the loss of moisture from the bulk concrete.

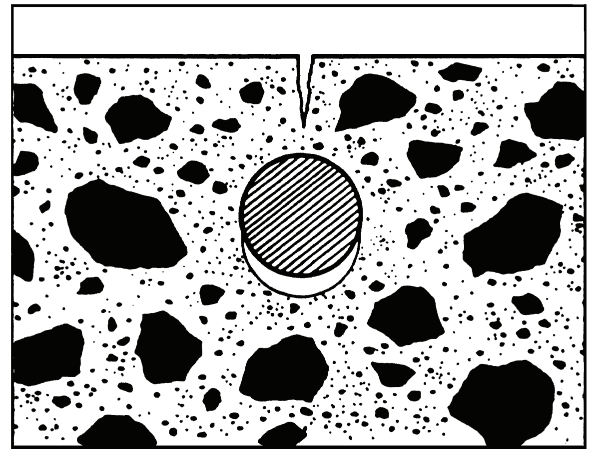


Fig. 1.2—Crack formed due to obstructed settlement (Price 1982).

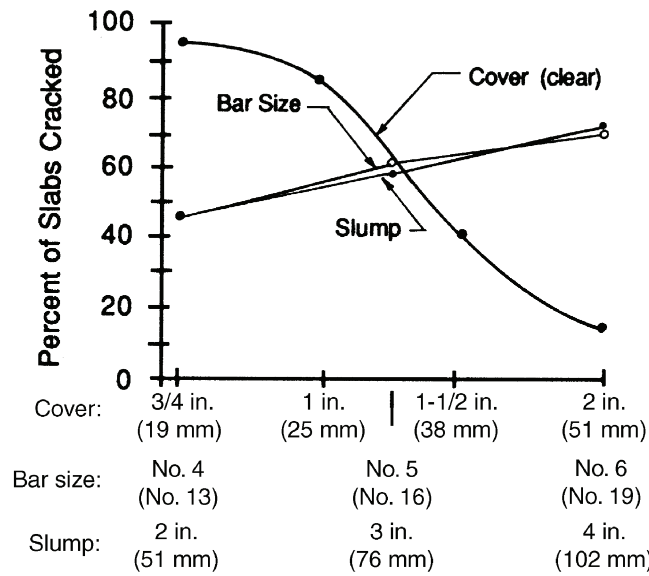


Fig. 1.3—Settlement cracking as a function of bar size, slump, and cover (Dakhil et al. 1975).

The self-desiccation occurs as the relative humidity in interior voids drops to 75 to 80%, which results in bulk shrinkage. Autogenous shrinkage strain is typically about 40 to 100×10^{-6} (Davis 1940), but has been measured as high as 2300×10^{-6} for concrete with a *w/cm* of 0.20 (Ai 2000). Houk et al. (1969) found that autogenous shrinkage increases with increasing temperature, cement content, and cement fineness.

1.3.2 *Thermal stresses*—Temperature differences within a concrete structure may be caused by portions of the structure losing heat of hydration at different rates or by the weather conditions cooling or heating one portion of the structure to a different degree or at a different rate than another portion of the structure. These temperature differences result in differential volume changes. When the tensile stresses due to the differential volume changes exceed the tensile strength, concrete will crack. Temperature differentials and the accompanying volume changes due to the dissipation of the heat of hydration of cement are normally associated with mass concrete (which can include large columns, piers, beams, and footings, as well as dams), whereas temperature differentials due to changes in the ambient temperature can affect any structure.

Cracking in mass concrete can result from a greater temperature on the interior than on the exterior. The temperature gradient may be caused by either the center of the concrete heating more than the outside due to the liberation of heat during cement hydration or more rapid cooling of the exterior relative to the interior. Both cases result in tensile stresses on the exterior and, if the tensile strength is exceeded, cracking will occur. The tensile stresses are proportional to the temperature differential, the coefficient of thermal expansion, the effective modulus of elasticity (which is reduced by creep), and the degree of restraint (Dusinberre 1945; Houghton 1972, 1976). The more massive the structure, the greater the potential for temperature differential and restraint.

Procedures to help reduce thermally induced cracking include reducing the maximum internal temperature, delaying the onset of cooling, controlling the rate at which the concrete cools, and increasing the tensile strength of the concrete. These and other methods used to reduce cracking in massive concrete are presented in ACI 207.1R, 207.2R, 207.4R, and 224R.

Hardened concrete has a coefficient of thermal expansion that may range from 3 to 7×10^{-6}/°F (6 to 13×10^{-6}/°C), with a typical value of 5.5×10^{-6}/°F (10×10^{-6}/°C). When one portion of a structure is subjected to a temperature-induced volume change, the potential for thermally induced cracking exists. Designers should give special consideration to structures in which some portions are exposed to temperature changes, while other portions of the structure are either partially or completely protected. A drop in temperature may result in cracking in the exposed element, whereas increases in temperature may cause cracking in the protected portion of the structure. Temperature gradients cause deflection and rotation in structural members; if restrained, serious stresses can result (Priestley 1978; Hoffman et al. 1983; Haynes 2001; ACI 343R). Allowing for movement by using properly designed contraction and isolation joints and correct detailing will help alleviate these problems.

1.3.3 *Chemical reaction*—Deleterious chemical reactions may cause cracking of concrete. These reactions may be due to materials used to make the concrete or materials that come into contact with the concrete after it has hardened.

Some general concepts for reducing adverse chemical reactions are presented herein, but only pretesting of the mixture or extended field experience will determine the effectiveness of a specific measure.

Concrete may crack with time as the result of slowly developing expansive reactions between aggregate containing active silica and alkalis derived from cement hydration, admixtures, or external sources (such as curing water, groundwater, deicing chemicals, and alkaline solutions stored or used in the finished structure).

When the alkalis in cement react with susceptible aggregate particles, a reaction rim of alkali-silica gel is formed around the aggregate. If this gel is exposed to moisture, it expands, causing an increase in volume of the concrete mass that will result in cracking and may eventually result in the complete deterioration of the structure. Control measures include using nonreactive aggregates, low-alkali cement, and pozzolans that consist principally of very fine, highly active silica. The first measure may preclude the problem from occurring, while the later two measures have the effect of decreasing the alkali-reactive silica ratio, resulting in the formation of a nonexpanding calcium alkali silicate hydrate.

Certain carbonate rocks participate in reactions with alkalis that, in some instances, produce detrimental expansion and cracking. These detrimental alkali-carbonate reactions are usually associated with argillaceous dolomitic limestones that have a very fine-grained (cryptocrystalline) structure (ACI 201.2R). The affected concrete is characterized by a network pattern of cracks. The reaction is distinguished from the alkali-silica reaction by the general absence of silica gel surface deposits at the crack. The problem may be minimized by avoiding reactive aggregates, dilution with nonreactive aggregates, use of a smaller maximum size aggregate, and use of low-alkali cement (ACI 201.2R).

Sulfates in soil and water are a special durability problem for concrete. When sulfate penetrates hydrated cement paste, it comes in contact with hydrated calcium aluminate. Calcium sulfoaluminate is formed, which may result in an increase in volume. The resulting expansion may cause the development of closely spaced cracks and the eventual deterioration of the concrete. ASTM C 150 Types II and V portland cement, which are low in tricalcium aluminate, will minimize the formation of calcium sulfoaluminate. Sulfate-resistant cements specified in ASTM C 595 and C 1157 are also useful in improving sulfate resistance. Pozzolans that have been tested and shown to impart additional resistance to sulfate attack are beneficial. Using concrete with a low *w/cm* is important to providing protection against severe sulfate attack.

Detrimental conditions may also occur from the application of deicing salts to the surface of hardened concrete. Chlorides from deicing chemicals can permeate concrete, reducing the ability of the concrete protect embedded reinforcement from corrosion. Because corrosion products occupy a greater volume than the original metal, corrosion can lead to delamination and cracking of the concrete. Deicing salts also make concrete more susceptible to freezing-and-thawing damage. To limit these effects, concrete subjected to water-soluble salts should be amply air entrained, have adequate cover of the reinforcing steel, and be made of high-quality, low-permeability concrete.

The effects of these and other problems relating to the durability of concrete are discussed in greater detail in ACI 201.2R.

The calcium silicate hydrate and calcium hydroxide in hydrated cement paste will combine with carbon dioxide in the air to form calcium carbonate. When this occurs, the concrete undergoes irreversible carbonation shrinkage, which may result in surface crazing. In addition, freshly placed concrete surfaces exposed (during the first 24 hours) to carbon dioxide from improperly vented combustion heaters, used to keep concrete warm during the winter months, are susceptible to dusting.

1.3.4 *Weathering*—The weathering processes that can cause cracking include freezing and thawing, wetting and drying, and heating and cooling. Damage from freezing and thawing is the most common weather-related physical deterioration. Concrete may be damaged by freezing of water in the paste, in the aggregate, or in both (Powers 1975).

Damage in hardened cement paste from freezing is caused by the movement of water to freezing sites and, for water in larger voids, by hydraulic pressure generated by the growth of ice crystals (Powers 1975).

Aggregate particles are surrounded by cement paste, which prevents the rapid escape of water. When the aggregate particles are above a critical degree of saturation, the expansion of the absorbed water during freezing may crack the surrounding cement paste or damage the aggregate itself (Callan 1952; Snowdon and Edwards 1962).

Concrete is best protected against freezing and thawing through the use of the lowest practical *w/cm* and total water content, durable aggregate, and adequate air entrainment. Adequate curing before exposure to freezing conditions is also important. Allowing the structure to dry after curing will enhance its freezing-and-thawing durability.

Other weathering processes that may cause cracking in concrete are alternate wetting and drying, and heating and cooling. Both processes produce volume changes that may cause cracking. If the volume changes are excessive, cracks may occur, as discussed in Sections 1.3.1 and 1.3.2.

1.3.5 *Corrosion of reinforcement*—Corrosion of a metal is an electrochemical process that requires an oxidizing agent, moisture, and electron flow within the metal; a series of chemical reactions takes place on and adjacent to the surface of the metal (ACI 201.2R and 222R).

The key to protecting metal from corrosion is to stop or reverse the chemical reactions. This may be done by cutting off the supplies of oxygen or moisture or by supplying excess electrons at the anodes to prevent the formation of the metal ions (cathodic protection).

Reinforcing steel usually does not corrode in concrete because a tightly adhering protective oxide coating forms in the highly alkaline environment. This is known as passive protection.

Reinforcing steel may corrode, however, if the alkalinity of the concrete is reduced through carbonation or if the passivity of this steel is destroyed by aggressive ions (usually chlorides). Corrosion of the steel produces iron oxides and hydroxides that have a volume much greater than the volume of the original metallic iron (Bentur et al. 1997). This increase in volume causes high radial bursting stresses around reinforcing bars and results in local radial cracks. These splitting cracks can propagate along the bar, resulting in the formation of longitudinal cracks (parallel to the bar) or spalling of the concrete. A broad crack may also form at a plane of bars parallel to a concrete surface, resulting in delamination, which is a well-known problem in bridge decks.

Cracks provide easy access for oxygen, moisture, and chlorides; thus, minor splitting (longitudinal) cracks can create a condition in which corrosion and cracking are accelerated. Cracks transverse to reinforcement do not usually cause continuing corrosion of the reinforcement if the concrete has low permeability. The exposed portion of a bar at a crack acts as an anode. At early ages, local corrosion occurs; the wider the crack, the greater the corrosion, because a greater portion of the bar has lost its passive protection. For continued corrosion to occur, however, oxygen and moisture must be supplied to other portions of the same bar or bars that are electrically connected by direct contact or through hardware, such as chair supports. If the combination of density and cover thickness is adequate to restrict the flow of oxygen and moisture, then the corrosion process is self-sealing (Verbeck 1975).

Corrosion can continue if a longitudinal crack forms parallel to the reinforcement because passivity is lost at many locations and oxygen and moisture are readily available along the full length of the crack.

Other causes of longitudinal cracking, such as high bond stresses, transverse tension (for example, along stirrups or along slabs with two-way tension), shrinkage, and settlement, can initiate corrosion.

For general concrete construction, the best protection against corrosion-induced splitting is the use of concrete with low permeability and adequate cover. Increased concrete cover over the reinforcement is effective in delaying the corrosion process by limiting carbonation, as well as access by oxygen, moisture, and chlorides, and also in resisting the splitting and spalling caused by corrosion or transverse tension (Gergely 1981; Beeby 1983). In the case of large bars and thick covers, it may be necessary to add small transverse reinforcement (while maintaining the minimum cover requirements) to limit splitting and to reduce the surface crack width (ACI 345R).

In very severe exposure conditions, additional protective measures may be required. A number of options are available, such as coated reinforcement, sealers, or overlays on the concrete, corrosion-inhibiting admixtures, and cathodic protection (Transportation Research Board 1979). Any procedure that effectively prevents access of oxygen and moisture to the steel surface or reverses the electron flow at the anode will protect the steel. In most cases, concrete must be allowed to breathe; that is, water must be allowed to evaporate from the concrete.

1.3.6 *Poor construction practices*—A wide variety of poor construction practices can result in cracking in concrete structures. Foremost among these is the common practice of adding water to concrete to increase workability. Added water has the effect of reducing strength, increasing settlement, and increasing drying shrinkage. When accompanied by a higher cement content to help offset the decrease in strength, an increase in water content will also mean an increase in the temperature differential between the interior and exterior portions of the structure, resulting in increased thermal stresses and possible cracking. In addition, by adding cementitious material, even if the *w/cm* remains constant, more shrinkage will occur because the paste volume is increased.

Lack of curing will increase the degree of cracking within a concrete structure. The early termination of curing will allow for increased shrinkage at a time when the concrete has low strength. The lack of hydration of the cement, due to drying, will result not only in decreased long-term strength, but also in the reduced durability of the structure.

Other construction problems that may cause cracking are inadequate formwork supports, inadequate consolidation, and placement of construction joints at points of high stress. Lack of support for forms or inadequate consolidation can result in settlement and cracking of the concrete before it has developed sufficient strength to support its own weight, while the improper location of construction joints can result in the joints opening at these points of high stress. Methods to prevent cracking due to these and other poor construction procedures are well known (ACI 224R, 224.3R, 302.1R, 304R, 305R, 308R, 309R, 345R, and 347), but require special attention to ensure their proper execution.

1.3.7 *Construction overloads*—Construction loads can often be more severe than those experienced in service. Unfortunately, these conditions may occur at early ages when the concrete is most susceptible to damage, and they often result in permanent cracks. Precast members, such as beams and panels, are most frequently subject to this abuse, but cast-in-place concrete can also be affected. A common error occurs when precast members are not properly supported during transport and erection. The use of arbitrary or convenient lifting points may cause severe damage. Lifting eyes, pins, and other attachments should be detailed or approved by the designer. When lifting pins are impractical, access to the bottom of a member must be provided so that a strap may be used. The PCI Committee on Quality Control Performance Criteria (1983, 1985) provides additional information on the causes, prevention, and repair of cracking related to fabrication and shipment of precast or prestressed beams, columns, hollow core slabs, and double tees.

Operators of lifting devices should exercise caution and be aware that damage may be caused even when the proper lifting accessories are used. A large beam or panel lowered too fast and stopped suddenly results in an impact load that may be several times the dead weight of the member. Another common construction error that should be avoided is prying up one corner of a panel to lift it off its bed or break it loose.

When considering the support of a member for shipment, the designer should be aware of loads that may be induced during transportation. Some examples that occur during shipment of large precast members via tractor and trailer are jumping curbs or tight highway corners, torsion due to differing roadway super-elevations between the trailer and the tractor, and differential acceleration of the trailer and the tractor.

Pretensioned beams can present unique cracking problems at the time of stress release—usually when the beams are less than 1 day old. Multiple strands should be detensioned following a specific pattern so as not to place unacceptable eccentric loads on the member. If all of the strands on one side of the beam are released while the strands on the other side are still stressed, cracking may occur on the side with the unreleased strands. These cracks are undesirable, but should close with the release of the balance of the strands.

In the case of a T-beam with a heavily reinforced flange and a highly prestressed thin web, cracks may develop at the web-flange junction.

Another practice that can result in cracks near beam ends is tack welding embedded bearing plates to the casting bed to hold them in place during concrete placement. The tack welds are broken only after enough prestress is induced during stress transfer to break them. Until then, the bottom of the beam is restrained while the rest of the beam is compressed. Cracks will form near the bearing plates if the welds are too strong.

Thermal shock can cause cracking of steam-cured concrete if it is treated improperly. The maximum rate of cooling frequently used is 70 °F (40 °C) per hour (ACI 517.2R; Verbeck 1958; Shideler and Toennies 1963; Kirkbride 1971b). When brittle aggregate is used and the strain capacity is low, the rate of cooling should be decreased. Even following this practice, thermally induced cracking often occurs. Temperature restrictions should apply to the entire beam, not just locations where temperatures are monitored. If the protective tarps used to contain the heat are pulled back for access to the beam ends when cutting the strands, and if the ambient temperatures are low, thermal shock may occur. Temperature recorders are seldom located in these critical areas.

Similar conditions and cracking potential exist with precast blocks, curbs, and window panels when a rapid surface temperature drop occurs.

It is believed by many (ACI 517.2R; Mansfield 1948; Nurse 1949; Higginson 1961; Jastrzebski 1961; Butt et al. 1969; Kirkbride 1971a; Concrete Institute of Australia 1972; PCI Energy Committee 1981) that rapid cooling may cause cracking only in the surface layers of very thick units and that rapid cooling is not detrimental to the strength or durability of standard precast products (PCI Energy Committee 1981). One exception is transverse cracking observed in pretensioned beams subjected to cooling before detensioning. For this reason, pretensioned members should be detensioned immediately after the steam curing has been discontinued (PCI Energy Committee 1981).

Cast-in-place concrete can be unknowingly subjected to construction loads in cold climates when heaters are used to provide an elevated working temperature within a structure. Typically, tarps are used to cover windows and door openings, and high-volume heaters are operated inside the enclosed area. If the heaters are located near exterior concrete members, especially thin walls, an unacceptably high thermal gradient can result within the members. The interior of the wall will expand in relation to the exterior. Heaters should be kept away from the exterior walls to minimize this effect. Good practice also requires that this be done to avoid localized drying shrinkage and carbonation cracking.

Storage of materials and the operation of equipment can easily result in loading conditions during construction far more severe than any load for which the structure was designed. Tight control should be maintained to avoid overloading conditions. Damage from unintentional construction overloads can be prevented only if designers provide information on load limitations for the structure and if construction personnel heed these limitations.

1.3.8 *Errors in design and detailing*—The effects of improper design or detailing range from poor appearance to lack of serviceability to catastrophic failure. These problems can be minimized only by a thorough understanding of structural behavior (meant herein in the broadest sense).

Errors in design and detailing that may result in unacceptable cracking include use of poorly detailed re-entrant corners in walls, precast members, and slabs; improper selection or detailing of reinforcement, or both; restraint of members subjected to volume changes caused by variations in temperature and moisture; lack of adequate contraction joints; and improper design of foundations, resulting in differential movement within the structure. Examples of these problems are presented by Kaminetzky (1981) and Price (1982).

Re-entrant corners provide a location for the concentration of stress and, therefore, are prime locations for the initiation of cracks. Whether the high stresses result from volume changes, in-plane loads, or bending, the designer should recognize that stresses are always high near re-entrant corners. Well-known examples are window and door openings in concrete walls and dapped-end beams, as shown in Fig. 1.4 and 1.5. Additional properly anchored diagonal reinforcement is required to keep the inevitable cracks narrow and prevent them from propagating.

The use of an inadequate amount of reinforcement may result in excessive cracking. A typical mistake is to lightly reinforce a member because it is a nonstructural member. The member (such as a wall), however, may be tied to the rest of the structure in such a manner that it is required to carry a major portion of the load once the structure begins to deform. The nonstructural element then begins to carry loads in proportion to its stiffness. Because this member is not detailed to act structurally, unsightly cracking may result even though the safety of the structure is not in question. In some cases, it may be advisable to isolate a nonstructural element from the structural system.

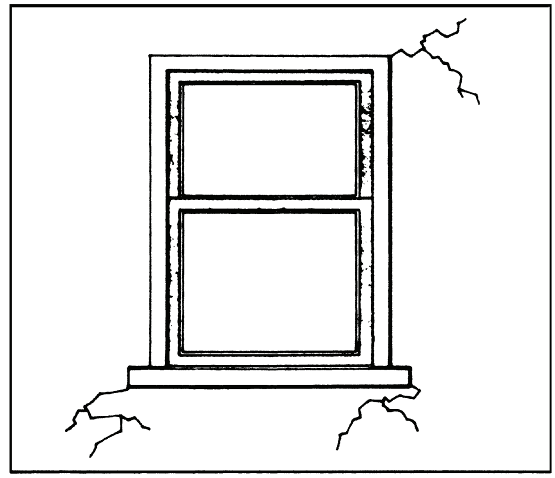


Fig. 1.4—Typical crack patterns at re-entrant corners (Price 1982).

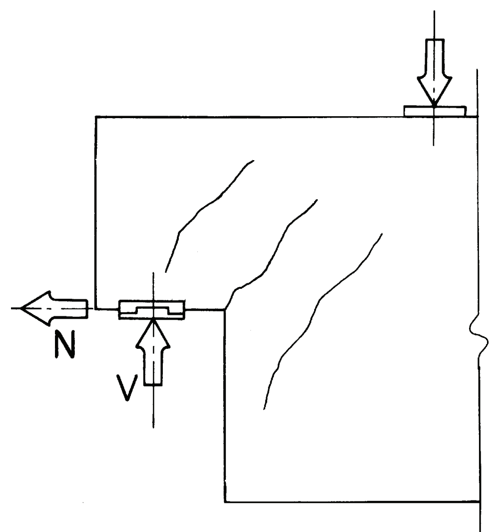


Fig. 1.5—Typical cracking pattern of dapped end beam at service load (Mattock and Chan 1979).

The restraint of members subjected to volume changes frequently results in cracks. Stresses that can occur in concrete due to restrained creep, temperature differential, and drying shrinkage can be many times the stresses that occur due to loading. A slab, wall, or a beam restrained against shortening, even if prestressed, can easily develop tensile stresses sufficient to cause cracking. Properly designed walls should have contraction joints spaced from one to three times the wall height. Beams should be allowed to move. Cast-in-place post-tensioned construction that does not permit shortening of the prestressed member is susceptible to cracking in both the member and the supporting structure (Libby 1977). The problem with restraint of structural members is especially serious in pretensioned and precast members that may be welded to the supports at both ends. When combined with other problem details (such as re-entrant corners), results may be catastrophic (Kaminetzky 1981; Mast 1981).

Improper foundation design may result in excessive differential movement within a structure. If the differential movement is relatively small, the cracking problems may be only visual in nature. If there is a major differential settlement, however, the structure may not be able to redistribute the loads rapidly enough, and a failure may occur. One of the advantages of reinforced concrete is that if the movement takes place over a long enough period of time, creep will allow at least some load redistribution to take place.

The importance of proper design and detailing will depend on the particular structure and loading involved. Special care should be taken in the design and detailing of structures in which cracking may cause a major serviceability problem. These structures also require continuous inspection during all phases of construction to supplement the careful design and detailing.

1.3.9 *Externally applied loads*—It is well known that load-induced tensile stresses result in cracks in concrete members. This point is readily acknowledged and accepted in the design of reinforced concrete structures. Current design procedures (ACI 318 and the AASHTO "Standard Specification for Highway Bridges") use reinforcing steel not only to carry the tensile forces, but to obtain both an adequate distribution of cracks and a reasonable limit on crack width.

Experimental evidence provides the basis for the following general conclusions about the variables that control cracking in flexural members: crack width increases with increasing steel stress, cover thickness, and area of concrete surrounding each reinforcing bar. Of these, steel stress is the most important variable. The bar diameter is not a major consideration. The width of a bottom crack increases with an increasing strain gradient between the steel and the tension face of the beam.

The equation that is often used to predict the most probable maximum surface crack width in bending was developed by Gergely and Lutz (1968). A simplified version of this equation is

$$w = 0.076\beta f_s \sqrt[3]{d_c A} \times 10^{-3} \quad \text{(in.-lb)} \qquad (1\text{-}1)$$

$$w = 0.011\beta f_s \sqrt[3]{d_c A} \times 10^{-3} \quad \text{(SI)}$$

where

w = most probable maximum crack width, in. (mm);

β = ratio of distance between neutral axis and tension face to distance between neutral axis and centroid of reinforcing steel (taken as approximately 1.20 for typical beams in buildings);

f_s = reinforcing steel stress, ksi (MPa);

d_c = thickness of cover from tension fiber to center of the closest bar, in. (mm); and

A = area of concrete symmetric with reinforcing steel divided by number of bars, in.^{2} (mm^2).

A modification of this equation is used in the AASHTO bridge specification, which effectively limits crack widths to 0.015 or 0.012 in. (0.38 or 0.30 mm), depending on the exposure condition. Considering the information presented in Section 1.3.5 that indicates little correlation between surface crack width for cracks transverse to bars and the corrosion of reinforcement; however, these limits do not appear to be justified on the basis of corrosion control.

A re-evaluation of cracking data provided a new crack width equation based on a physical model (Frosch 1999). For the calculation of maximum crack widths, the crack width can be calculated as

$$w = 2\frac{f_s}{E_s}\beta\sqrt{d_c^2 + \left(\frac{s}{2}\right)^2} \qquad (1\text{-}2)$$

where

w = maximum crack width, in. (mm);

f_s = reinforcing steel stress, ksi (MPa);

E_s = reinforcing steel modulus of elasticity, ksi (MPa);

β = ratio of distance between neutral and tension face to distance between neutral axis and centroid of reinforcing steel (taken as approximately $1.0 + 0.08d_c$);

d_c = thickness of cover from tension face to center of closest bar, in. (mm); and

s = bar spacing, in. (mm).

Crack control is achieved in ACI 318 through the use of a spacing criterion for reinforcing steel that is based on the stress under service conditions and clear cover on the bars. This design equation was based on Eq. (1-2), considering crack widths on the order of 0.016 in. (0.40 mm).

ACI 318 also includes provisions to prevent the formation of cracks in regions away from the location of the main reinforcement through the provision of skin reinforcement in the webs of beams that are deeper than 36 in. (900 mm) and the distribution of negative moment reinforcement within the effective width of a flange in tension. ACI 318 also requires that, in cases where the primary flexural reinforcement in a slab that is acting as a T-beam flange is parallel to the beam, reinforcement perpendicular to the beam should be provided in the top of the slab. The transverse reinforcement in the slab is designed to carry the factored load on an overhanging slab width that is assumed to act as a cantilever. For isolated beams, the full width of the overhang is considered. For other T-beams, the effective width of the flange is used to design the transverse reinforcement.

There have been a number of crack width equations developed for prestressed concrete members (ACI 224R), but no single method has achieved general acceptance.

The maximum crack width in tension members is larger than that predicted by the expression for flexural members (Broms 1965; Broms and Lutz 1965). Absence of a strain gradient and compression zone in tension members is the probable reason for the larger crack widths.

On the basis of limited data, the following expression has been suggested to estimate the maximum crack width in direct tension (ACI 224R).

$$w = 0.10 f_s \sqrt[3]{d_c A} \times 10^{-3} \quad \text{(in.-lb)} \qquad (1\text{-}3)$$

$$w = 0.0145 f_s \sqrt[3]{d_c A} \times 10^{-3} \quad \text{(SI)}$$

Additional information on cracking of concrete in direct tension is provided in ACI 224.2R.

Flexural and tensile crack widths can be expected to increase with time for members subjected to either sustained or repetitive loading. Although a large degree of scatter is evident in the available data, a doubling of crack width with time can be expected (Abeles et al. 1968; Bennett and Dave 1969; Illston and Stevens 1972; Holmberg 1973; Rehm and Eligehausen 1977).

The basic principles of crack control for load-induced cracks are well understood. Well-distributed reinforcement offers the best protection against undesirable cracking. Reduced steel stress, obtained through the use of a larger amount of steel, will reduce crack width. While reduced cover will reduce the surface crack width, designers should keep in mind, as pointed out in Section 1.3.5, that cracks (and therefore, crack widths) perpendicular to reinforcing steel do not have a major effect on the corrosion of the steel, while a reduction in cover will be detrimental to the corrosion protection of the reinforcement.

CHAPTER 2—EVALUATION OF CRACKING

2.1—Introduction

When anticipating repair of cracks in concrete, it is important to first identify the location and extent of cracking. It should be determined whether the cracks are indicative of current or future structural problems, taking into consideration both present and anticipated loading conditions. The cause of the cracking should be established before repairs are specified. Drawings, specifications, and construction and maintenance records should be reviewed. If these documents, along with field observations, do not provide the needed information, a field investigation and structural analysis should be completed before proceeding with repairs.

The causes of cracks are discussed in Chapter 1. A detailed evaluation of observed cracking can determine which of the causes applies in a particular situation.

Cracks need to be repaired if they reduce the strength, stiffness, or durability of the structure to an unacceptable level, or if the function of the structure is seriously impaired. In some cases, such as cracking in water-retaining structures, the function of the structure will dictate the need for repair, even if strength, stiffness, and appearance are not seriously impaired. Cracks in pavements and slabs-on-ground may require repair to prevent edge spalls or the migration of water to the subgrade, or to maintain intended load-carrying capacity. In addition, repairs that improve the appearance of the surface of a concrete structure may be desired.

2.2—Determination of location and extent of concrete cracking

Location and extent of cracking as well as information on the general condition of concrete in a structure can be determined by both direct and indirect observations, nondestructive testing, and destructive tests such as test cores taken from the structure. Information may also be obtained from drawings and construction and maintenance records.

2.2.1 *Direct and indirect observation*—The locations and widths of cracks should be noted on a sketch of the structure.

Fig. 2.1—Comparator for measuring crack widths (courtesy of Edmound Scientific Co.).

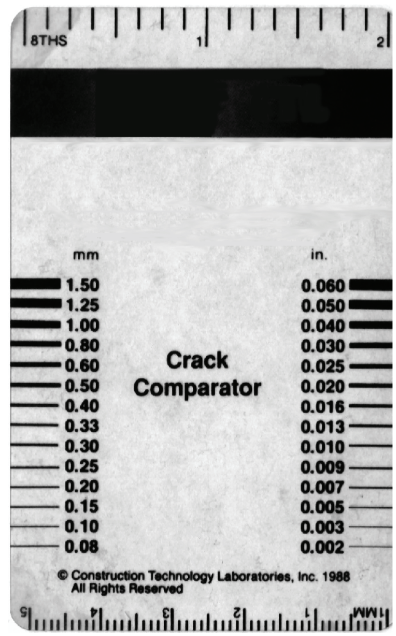


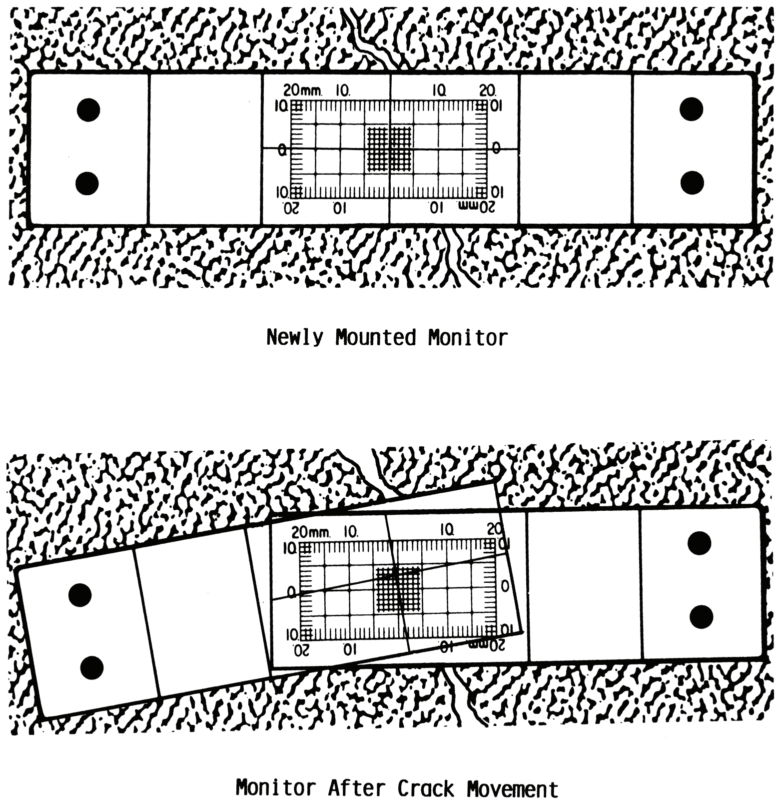
Fig. 2.2—Card used to measure crack width (courtesy of Construction Technology Laboratories).

A grid marked on the surface of the structure can be useful to accurately locate cracks on the sketch.

Crack widths can be measured to an accuracy of approximately 0.001 in. (0.025 mm) using a crack comparator, which is a small, hand-held microscope with a scale on the lens closest to the surface being viewed (Fig. 2.1). It is generally more convenient, however, to estimate crack widths using a clear card having lines of specified width marked on the card, as shown in Fig. 2.2. Any displacement of the surface (change in elevation) across the crack should also be documented. Observations such as spalling, exposed reinforcement, surface deterioration, and rust staining should be noted on the sketch. Internal conditions at specific crack locations can be observed with the use of flexible shaft fiberscopes or rigid borescopes.

Crack movement can be monitored with mechanical movement indicators of the types shown in Fig. 2.3. The indicator, or crack monitor, shown in Fig. 2.3(a) gives a direct reading of crack displacement and rotation. The indicator in Fig. 2.3(b) (Stratton et al. 1978) amplifies the crack

(a)



(b)

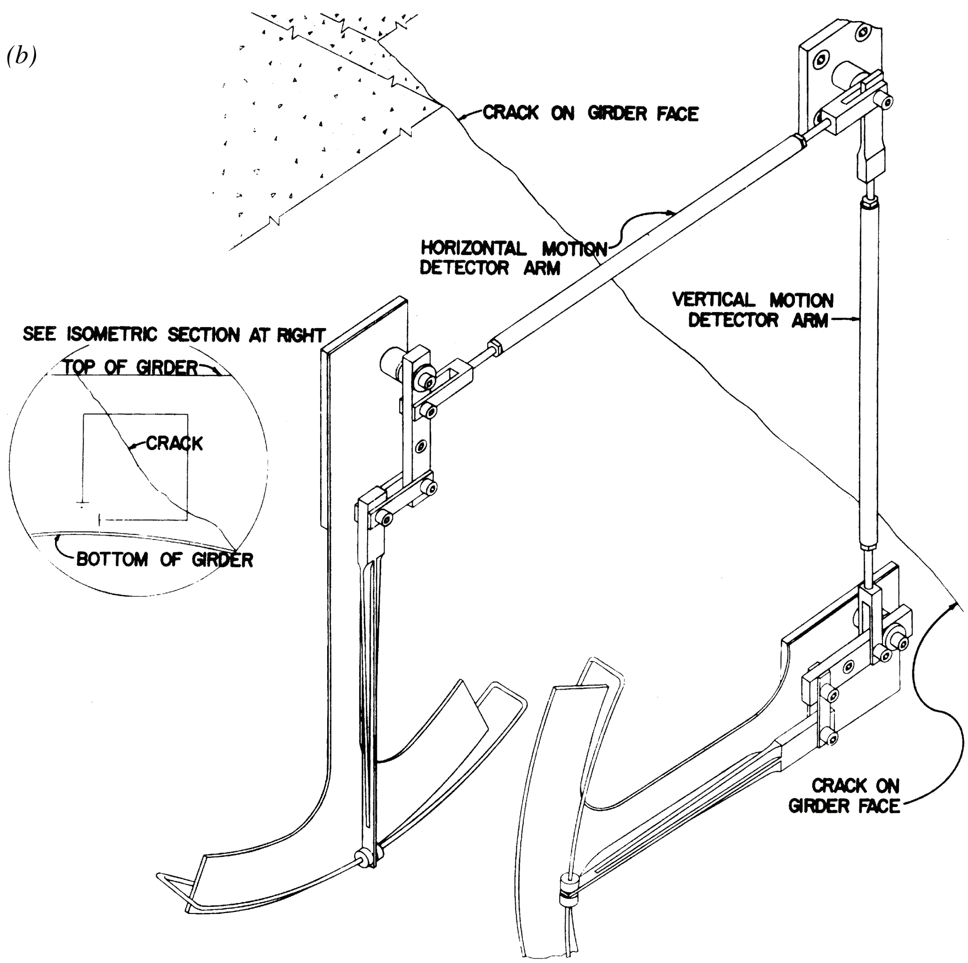


Fig. 2.3—(a) Crack monitor (courtesy of Avongard); and (b) crack movement indicator (Stratton et al. 1978).

movement (in this case, 50 times) and indicates the maximum range of movement during the measurement period. Mechanical indicators have the advantage that they do not require moisture protection. If more detailed time histories are desired, a wide range of transducers (most notably, linear variable differential transformers [LVDTs]) and data-acquisition systems (ranging from strip chart recorders to computer-based systems) are available.

Sketches can be supplemented by photographs documenting the condition of the structure at the time of investigation. Guidance for making a condition survey of concrete in service is given in ACI 201.1R, 201.3R, 207.3R, 345.1R, and 546.1R.

2.2.2 *Nondestructive testing*—Nondestructive tests can be made to determine the presence of internal cracks and voids and the depth of penetration of cracks visible at the surface.

Tapping the surface with a hammer or using a chain drag are simple techniques to identify laminar cracking near the surface. A hollow sound indicates one or more cracks below and parallel to the surface. Infrared imaging equipment, though expensive, has also proven to be useful in identifying regions in which concrete has delaminated.

The presence of reinforcement can be determined using a pachometer (Fig. 2.4). A number of pachometers are available that range in capability from merely indicating the presence of steel to those that may be calibrated to allow the experienced user a closer determination of depth and the size of reinforcing steel. In some cases, however, it may be necessary to remove the concrete cover (often by drilling or chipping) to identify the bar sizes or to calibrate cover measurements, especially in areas of congested reinforcement. Newer devices use magnetic fields and computer algorithms to provide a visual picture of the reinforcing bar layout in the scanned area. This device can be used to detect reinforcement, measure concrete cover, and determine the reinforcement size and position (ACI 228.2R).

If corrosion is a suspected cause of cracking, the easiest approach to investigate for corrosion entails the removal of a portion of the concrete to directly observe the steel. Corrosion potential can be detected by electrical potential measurements using a suitable reference half-cell. The most commonly used is a copper-copper sulfate half-cell (ASTM C 876); its use also requires access to a portion of the reinforcing steel. The half-cell technique, however, is sensitive to the moisture condition of the concrete, and if readings are taken when the concrete is dry, half-cell potentials may indicate no corrosion activity when, in fact, the steel is undergoing corrosion.

With properly trained personnel and careful evaluation, it is possible to detect cracks using ultrasonic nondestructive test equipment (ASTM C 597). The most common technique is through-transmission testing using commercially available equipment (Malhotra and Carino 2004; Knab et al. 1983). A mechanical pulse is transmitted to one face of the concrete member and received at the opposite face, as shown in Fig. 2.5. The time taken for the pulse to pass through the member is measured electronically. If the distance between the transmitting and receiving transducers is known, the pulse velocity can be calculated.

Fig. 2.4—Pachometer (reinforcing bar indicator) (courtesy of James Instruments).

When access is not available to opposite faces, transducers may be located on the same face (Fig. 2.5(a)). While this technique is possible, the interpretation of results is not straightforward.

A significant change in measured pulse velocity can occur if an internal discontinuity results in an increase in path length for the signal. Generally, the higher the pulse velocity, the higher the quality of the concrete. The interpretation of pulse velocity test results is significantly improved with the use of an oscilloscope that provides a visual representation of the received signal (Fig. 2.5(b)).

Some equipment provides only a digital readout of the pulse travel time with no oscilloscope display. If no signal arrives at the receiving transducer, a significant internal discontinuity, such as a crack or void, is indicated. An indication of the extent of the discontinuity can be obtained by taking readings at a series of positions on the member.

Ultrasonic equipment should be operated by trained personnel, and the results should be evaluated cautiously by an experienced individual because moisture, reinforcing steel, and embedded items may affect the results. For example, with fully saturated cracks, ultrasonic testing will generally be ineffective, and in some cases, it is difficult to discern between a group of narrow cracks and a single large crack.

An alternative to through-transmission testing is the pulse-echo technique in which a simple transducer is used to send and receive ultrasonic waves. It has, however, been difficult to develop a practical pulse-echo test for concrete. Pitch-catch systems have been developed that use separate transmitting and receiving transducers (Alexander 1980). More

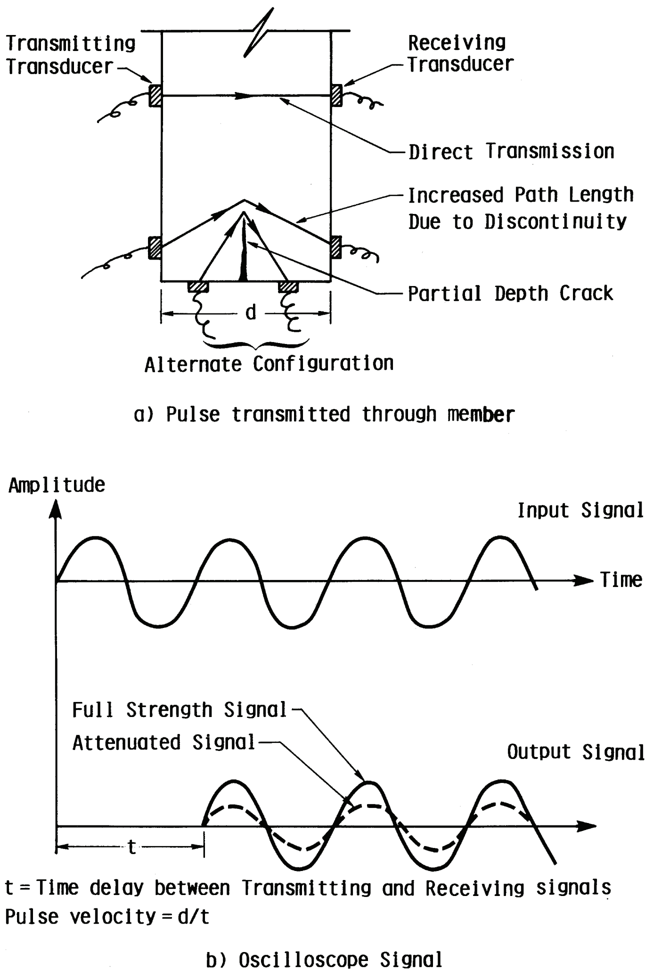


Fig. 2.5—Ultrasonic testing: through-transmission technique.

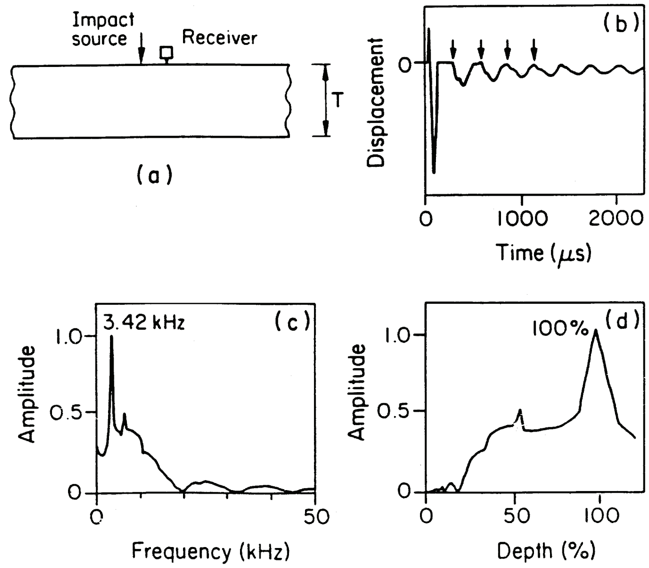


Fig. 2.6—Impact-echo response of a solid plate: (a) schematic of test configuration; (b) displacement waveform; (c) amplitude spectrum; and (d) normalized amplitude spectrum.

detailed information on pulse-echo and other wave propagation methods is provided by Malhotra and Carino (2004).

Significant advances in use of wave propagation techniques for flaw detection in concrete by the impact-echo technique have been made (Sansalone and Carino 1988, 1989; Sansalone 1997). A mechanical pulse is generated by impact on one face of the member, as illustrated in Fig. 2.6. The wave propagates through the member, reflects from a defect or other surface of the member, and is received by a displacement transducer placed near the impact point. Figure 2.6(b) shows a surface time-domain waveform received by the transducer. A resonance condition is set up in the member between the member boundaries or boundary and defect. By analyzing the frequency content of the time-domain waveform (Fig. 2.6(c)), the frequency associated with the resonance appears as a peak amplitude. In the case of Fig. 2.6(a), the peak is that associated with the thickness frequency (Fig. 2.6(d)). If an internal flaw exists, then a significant amplitude peak from the reflections from the flaw depth will be observed at the associated flaw depth frequency.

Radiography can also be used to detect internal discontinuities. Both x-ray and gamma-ray equipment is available (Malhotra and Carino 2004; Bungey 1990). The procedures are best suited for detecting crack planes parallel to the direction of radiation; it is difficult to discern crack planes perpendicular to the radiation. Gamma-ray equipment is less expensive and relatively portable compared with x-ray equipment and, therefore, appears to be more suitable for field testing.

Ground-penetrating radar (GPR), also called impulse radar, uses electromagnetic waves to measure discontinuities below a concrete surface. The technique has been used to locate cracks, voids, reinforcing steel, and to measure thickness. “Due to the large number of physical properties affecting the received signal,” however, “current application to structural concrete (other than measurements of the spatial configuration of concrete and reinforcement) is largely restricted to qualitative comparative uses” (ACI 228.2R).

An important use of nondestructive testing is finding those portions of the structure that require a more detailed investigation, which may include core tests.

2.2.3 *Tests on concrete cores*—Significant information can be obtained from cores taken from selected locations within the structure. Cores and core holes afford the opportunity to accurately measure the width and depth of cracks. In addition, an indication of concrete quality can be obtained from compressive strength tests; however, cores that contain cracks should not be used to determine concrete strength.

Petrographic examinations (ASTM C 856) of cracked concrete can identify material causes of cracking, such as alkali reactivity, cyclic freezing damage, D-cracking, expansive aggregate particles, fire-related damage, shrinkage, and corrosion. Petrography can also identify other factors that may be related to cracking, such as the *w/cm*, relative paste volume, and distribution of concrete components. Petrography can frequently determine the relative age of cracks and can identify secondary deposits on fracture surfaces, which have an influence on repair schemes.

Chemical tests for the presence of excessive chlorides indicate the potential for corrosion of embedded reinforcement.

2.2.4 *Review of drawings and construction data*—The original structural design, reinforcement placing, and other construction drawings should be reviewed to confirm that the concrete thickness and quality, along with the reinforcement, meets or exceeds strength and serviceability requirements noted in the governing building code(s). The actual loading should be compared with design loads. Concrete configurations, restraint conditions, and the presence of construction and other joints should be considered when calculating the tensile stresses induced by concrete deformation (creep, shrinkage, temperature). Consideration should be given to cracks that develop parallel to a slab-girder joint in one-way reinforced concrete slabs in which the main slab reinforcement is parallel to the girder as the slab spans between beams supported by the girder.

2.3—Selection of repair procedures

Based on the careful evaluation of the extent and cause of cracking, procedures can be selected to accomplish one or more of the following objectives:

1. Restore or increase strength;
2. Restore or increase stiffness;
3. Improve functional performance;
4. Provide watertightness;
5. Improve appearance of the concrete surface;
6. Improve durability; and
7. Prevent development of a corrosive environment at reinforcement.

Depending on the nature of the damage, one or more repair methods may be selected. For example, tensile strength may be restored across a crack by injecting it with epoxy or other high-strength bonding agent, if further cracking is not anticipated (ACI 503R). It may be necessary, however, to provide additional strength by adding reinforcement or using post-tensioning.

Cracks causing leaks in water-retaining or other storage structures should be repaired unless the leakage is considered minor or there is an indication that the crack is being sealed by autogenous healing (Section 3.13). Repairs to stop leaks may be complicated by a need to make the repairs while the structures are in service.

Cosmetic considerations may require the repair of cracks in concrete. The crack locations, however, may still be visible and, in fact, may even be more apparent due to the repair. Depending on circumstances, some form of coating over the entire surface may be required.

To minimize future deterioration due to the corrosion of reinforcement, cracks exposed to a moist or corrosive environment should be sealed.

The key methods of crack repair available to accomplish the objectives outlined are described in Chapter 3.

CHAPTER 3—METHODS OF CRACK REPAIR

3.1—Introduction

Following the evaluation of a cracked structure, a suitable repair procedure can be selected. Successful repair procedures account for the cause(s) of cracking. For example, if cracking is primarily due to drying shrinkage, then it is likely that, after a period of time, the cracks will stabilize. On the other hand, if the cracks are due to ongoing foundation settlement, repair will be of no use until the settlement problem is corrected.

This chapter provides a survey of crack repair methods, including a summary of the characteristics of cracks that may be repaired with each procedure, the types of structures that have been repaired, and a summary of the procedures that are used. Readers are also directed to ACI 546R, 546.1R, 546.2R, RAP-1, Emmons and Emmons (1994), Trout (1997), and ICRI Guideline No. 03734, which specifically address the subject of concrete repair.

3.2—Epoxy injection

Cracks as narrow as 0.002 in. (0.05 mm) can be bonded by the injection of epoxy ("Crack Repair Method: Epoxy Injection" 1985). The technique generally consists of establishing entry and venting ports at close intervals along the cracks, sealing the crack on exposed surfaces, and injecting the epoxy under pressure.

Epoxy injection has been successfully used in the repair of cracks in buildings, bridges, dams, and other types of concrete structures (ACI 503R). Unless the cause of the cracking has been corrected, however, new cracks will probably form near the original crack. If the cause of the cracks cannot be removed, then three options are available: 1) rout and seal the crack, thus treating it as a joint; 2) establish a joint that will accommodate the movement and then inject the crack with epoxy or other suitable material; and 3) install additional support or reinforcement at the crack location to minimize movement. Epoxy materials used for structural repairs should conform to ASTM C 881 (Type IV). ACI 504R describes practices for sealing joints, including joint design, available materials, and methods of application.

With the exception of certain moisture-tolerant epoxies, this technique is not applicable if the cracks are actively leaking and cannot be dried out. Wet cracks can be injected using moisture-tolerant materials that will cure and bond in the presence of moisture, but contaminants in the cracks (including silt and water) can reduce the effectiveness of the epoxy to structurally repair the cracks (Barlow 1993).

The use of a low-modulus, flexible adhesive in a crack will not allow significant movement of the concrete structure (Gaul 1993). The effective modulus of elasticity of a flexible adhesive in a crack is substantially the same as that of a rigid adhesive (Adams and Wake 1984) because of the thin layer of material and high lateral restraint imposed by the surrounding concrete.

Epoxy injection requires a high degree of skill for satisfactory execution, and use of the technique may be limited by the ambient temperature. The general procedures involved in epoxy injection are as follows (ACI 503R and RAP-1):

- **Cleaning the cracks.** The first step is to clean any cracks that have been contaminated (to the extent this is possible and practical). Contaminants, such as oil, grease, dirt, or fine particles of concrete, prevent epoxy penetration and bonding, and reduce the effectiveness of repairs. Preferably, contamination should be removed by vacuuming or flushing with water or other effective cleaning solutions. The solution is then flushed out using compressed air and a neutralizing agent or adequate time is provided for air drying. It is important, however, to recognize the practical limitations of accomplishing complete crack cleaning. A reasonable evaluation should be made of the extent, and necessity, of cleaning. Trial cleaning may be required;

- **Sealing the surfaces.** Surface cracks should be sealed to keep the epoxy from leaking out before it has gelled. Where the crack face cannot be reached, but where there is backfill or where a slab-on-ground is being repaired, the backfill material or subbase material is sometimes an adequate seal; however, such a condition can rarely be determined in advance, and uncontrolled injection can cause damage such as plugging a drainage system. Extreme caution should be exercised when injecting cracks that are not visible on all surfaces. A surface can be sealed by applying an epoxy, polyester, or other appropriate sealing material to the surface of the crack and allowing it to harden. If a permanent glossy appearance along the crack is objectionable and if high injection pressure is not required, a strippable plastic surface sealer may be applied along the face of the crack. When the job is completed, the surface sealer can be stripped away to expose the gloss-free surface. Cementitious seals can also be used where the appearance of the completed work is important. If extremely high injection pressures are needed, alternate procedures should be followed;
- **Installing the entry and venting ports.** Three methods are in general use:

  1. **Fittings inserted into drilled holes.** This method entails drilling a hole into the crack, approximately 3/4 in. (20 mm) in diameter and 1/2 to 1 in. (13 to 25 mm) below the apex of the V-grooved section, into which a fitting such as a pipe nipple or tire valve stem is usually bonded with an epoxy adhesive. A vacuum chuck and bit, or a water-cooled core bit, is useful in preventing the cracks from being plugged with drilling dust;
  2. **Bonded flush fitting.** An alternative method that is frequently used to provide an entry port is to bond a fitting flush with the concrete face over the crack. The flush fitting has an opening at the top for the adhesive to enter and a flange at the bottom that is bonded to the concrete; and
  3. **Interruption in seal.** Another system of providing entry is to omit the seal from a portion of the crack. This method can be used when special gasket devices are available that cover the unsealed portion of the crack and allow injection of the adhesive directly into the crack without leaking.
- **Mixing the epoxy.** Mixing is done by batch or continuous methods. In batch mixing, the adhesive components are premixed according to the manufacturer's instructions, usually with the use of a mechanical stirrer, like a paint-mixing paddle. Care should be taken to mix only the amount of adhesive that can be used before commencement of gelling of the material. When the adhesive material begins to gel, its flow characteristics begin to change, and pressure injection becomes more and more difficult. In the continuous mixing system, the two liquid adhesive components pass through metering and driving pumps before passing through an automatic mixing head. The continuous mixing system applies for all epoxies, including fast-setting adhesives that have a short working life;
- **Injecting the epoxy.** Hydraulic pumps, paint pressure pots, or air-actuated caulking guns may be used. The pressure used for injection should be selected carefully. Increased pressure often does little to accelerate the rate of injection. In fact, the use of excessive pressure can propagate the existing cracks, causing additional damage.

  If the crack is vertical or inclined, the injection process should begin by pumping epoxy into the entry port at the lowest elevation until the epoxy level reaches a predetermined entry port above. The lower injection port is then capped, and the process is repeated until the crack has been completely filled and all ports have been capped.

  For horizontal cracks, the injection should proceed from one end of the crack to the other in a similar manner. It may be required to repeat this process several times until the crack is sealed. The crack is full if the pressure can be maintained. If the pressure cannot be maintained, the epoxy is still flowing into unfilled portions or leaking out of the crack.

  A low-pressure injection system is available that uses discrete capsules containing a premixed epoxy. The capsules are mounted on injection ports. A spring inside each capsule maintains a constant low pressure to dispense a low-viscosity, long-pot-life epoxy resin into a crack. Capillary action moves the epoxy into the cracks; and
- **Removing the surface seal.** After the injected epoxy has cured, the surface seal should be removed by grinding or other means as appropriate.

Another method involves the use of a vacuum or vacuum assist. There are two techniques. One technique is to entirely enclose the cracked member with a bag, introduce the liquid adhesive at the bottom, and apply a vacuum at the top. The other technique is to inject the cracks from one side and pull a vacuum from the other. Typically, epoxies are used; however, acrylics and polyesters have proven successful.

Stratton and McCollum (1974) describe the use of epoxy injection as an effective intermediate-term repair procedure for delaminated bridge decks. Epoxy resins and injection procedures should be carefully selected when attempting to inject delaminations. Unless there is sufficient depth or anchorage to surrounding concrete, the injection process can be unsuccessful or increase the extent of delamination. Smith (1992) provides information on bridge decks observed for up to 7 years after injection. Smithson and Whiting (1992) describe epoxy injection as a method to rebond delaminated bridge deck overlays.

3.3—Routing and sealing

Routing and sealing of cracks can be used in conditions requiring repair where structural repair is not necessary. This

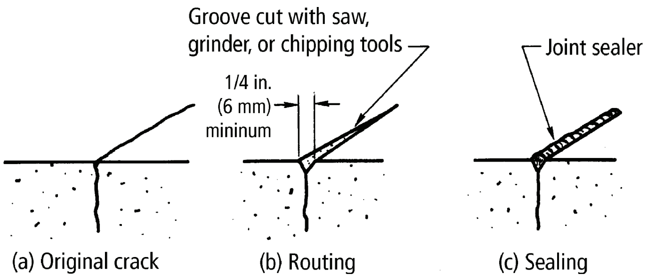


Fig. 3.1—Repair of crack by routing and sealing (Johnson 1965).

method involves enlarging the crack along its exposed face and filling and sealing it with a suitable joint sealant. Figure 3.1 shows the procedure for repairing a crack. This is a common technique for crack treatment and is relatively simple compared with the procedures and the training required for epoxy injection. The procedure is most applicable to approximately flat horizontal surfaces such as floors and pavements. Routing and sealing can be accomplished on vertical surfaces (with a nonsag sealant) as well as on curved surfaces (pipes, piles, and poles).

Routing and sealing is used to treat both narrow and wide cracks. A common and effective use is for waterproofing by sealing cracks on the concrete surface where water stands or where hydrostatic pressure is applied. This treatment reduces the ability of moisture to reach the reinforcing steel or pass through the concrete, causing surface stains or other problems.

The sealants may be any of several materials, including epoxies, urethanes, silicones, polysulfides, asphaltic materials, or polymer mortars. Cement grouts should be avoided due to the likelihood of cracking.

For floors, the sealant should be sufficiently rigid to support the anticipated traffic. Load transfer at the floor crack should be provided by aggregate interlock or dowels; otherwise, traffic loads moving across the crack may cause rigid sealants to debond.

Routing and sealing consists of preparing a vertical walled groove at the surface typically ranging in depth from 1/4 to 1 in. (6 to 25 mm). A concrete saw or right-angle grinder may be used. The groove is then cleaned by airblasting, sandblasting, or waterblasting, and dried. A sealant is placed into the dry groove (Fig. 3.1) and allowed to cure.

A variation of this procedure is to provide load transfer by having epoxy fill or partially fill the crack by gravity. The vertical-walled groove is used as a reservoir for epoxy that is selected for an appropriate viscosity. The final step is to fill the vertical-walled groove with a high-viscosity, rigid epoxy.

Active cracks should be repaired using a bond breaker at the base of the routed channel (Fig. 3.2). A flexible sealant is then placed in the routed channel. It is important that the width-to-depth ratio of the channel is usually 2 or more. This permits the sealant to respond to movement of the crack with high extensibility.

In some cases, overbanding (strip coating) is used independently of or in conjunction with routing and sealing. This method is used to enhance protection from edge spalling and, for aesthetic reasons, to create a more uniform-appearing treatment. A typical procedure for overbanding is to prepare an area approximately 1 to 3 in. (25 to 75 mm) on each side of the crack by sandblasting or other means of surface preparation, and applying a coating (such as urethane) 0.04 to 0.08 in. (1 to 2 mm) thick in a band over the crack. Before overbanding in nontraffic areas, a bond breaker is sometimes used over a crack that has not been routed or over a crack previously routed and sealed. In traffic areas, a bond breaker is not recommended. Cracks subject to minimal movement may be overbanded, but if significant movement can take place, routing and sealing should be used in conjunction with overbanding to ensure a waterproof repair.

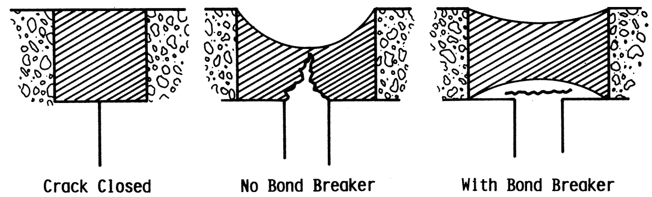


Fig. 3.2—Effect of bond breaker.

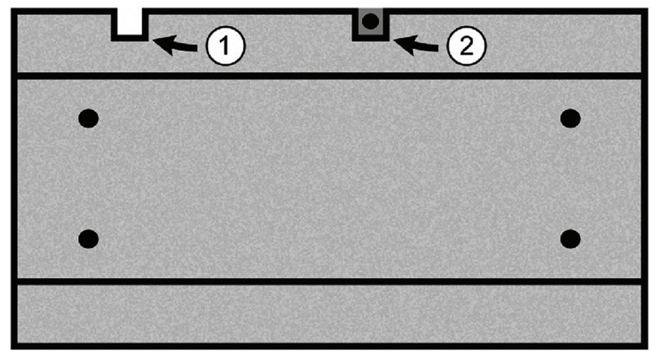


Fig. 3.3—Repair using near-surface-mounted reinforcing: (1) sawcut 1/8 to 1/4 in. (3 to 6 mm) larger than bar diameter; and (2) deformed bar or FRP bar bedded in epoxy resin.

3.4—Near-surface reinforcing and pinning

Near-surface reinforcing (NSR) is a method used to add tensile reinforcement perpendicular to the plane of the crack. As shown in Fig. 3.3, a slot is sawcut across the crack, and the slot is then cleaned. Typically, an epoxy resin is placed in the slot to act as a bonding agent and protective barrier to the bar that is subsequently placed. Both deformed steel reinforcing bars and precured fiber-reinforced polymer (FRP) bars are placed in the slot that is cut to approximately 0.125 in. (3 mm) wider and deeper than the diameter of the reinforcement to be installed. The reinforcing needs to be designed to increase the capacity beyond the tensile forces at the crack location.

3.5—Additional reinforcement

3.5.1 *Conventional reinforcement*—Cracked reinforced concrete bridge girders have been successfully repaired by inserting reinforcing bars and bonding them in place with epoxy (Stratton et al. 1978; Stratton 1980; "Crack Repair Method: Conventional Reinforcement" 1985). This technique consists of sealing the crack, drilling holes that intersect the

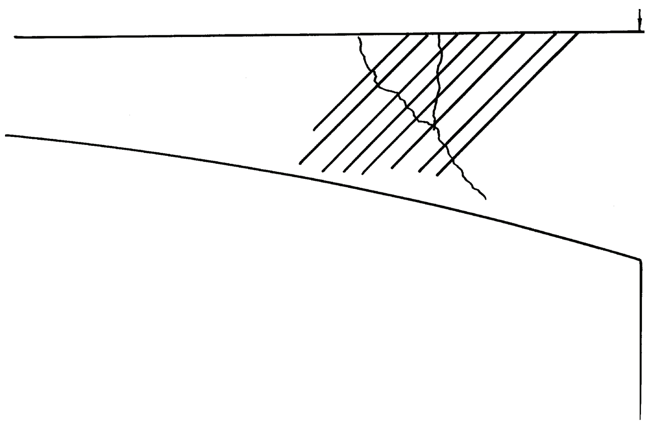
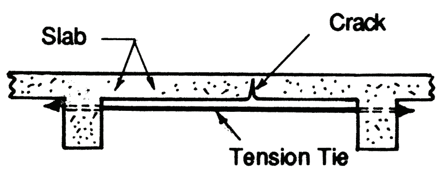
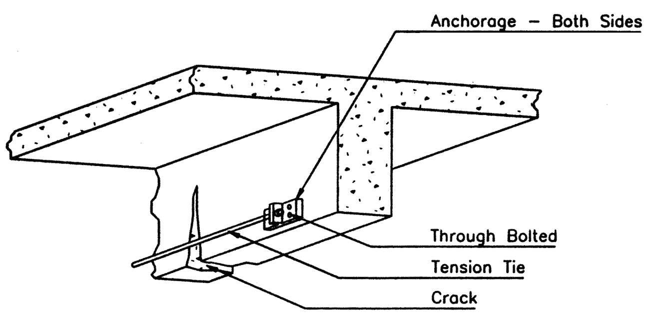


Fig. 3.4—Reinforcing bar orientation used to affect the repair (Stratton et al. 1978).



(a) To correct cracking of slab



(b) To correct cracking of beam

Fig. 3.5—Examples of external prestressing (Johnson 1965).

crack plane at approximately 90 degrees (Fig. 3.4), filling the hole and crack with injected epoxy, and placing a reinforcing bar into the drilled hole. Typically, No. 4 or 5 (No. 13 or 16) bars are used, extending at least 18 in. (0.5 m) each side of the crack. The reinforcing bars can be spaced to suit the needs of the repair. They can be placed in any desired pattern, depending on the design criteria and the location of the in-place reinforcement. The epoxy bonds the bar to the walls of the hole, fills the crack plane, bonds the cracked concrete surfaces back together in one monolithic form, and thus reinforces the section. The epoxy used to rebond the crack should have a low viscosity and conform to ASTM C 881 Type IV.

3.5.2 *Prestressing steel*—Post-tensioning is often the desirable solution when a major portion of a member must be strengthened or when the cracks that have formed must be closed (Fig. 3.5). This technique uses prestressing strands or bars to apply a compressive force. Adequate anchorage should be provided for the prestressing steel, and care is

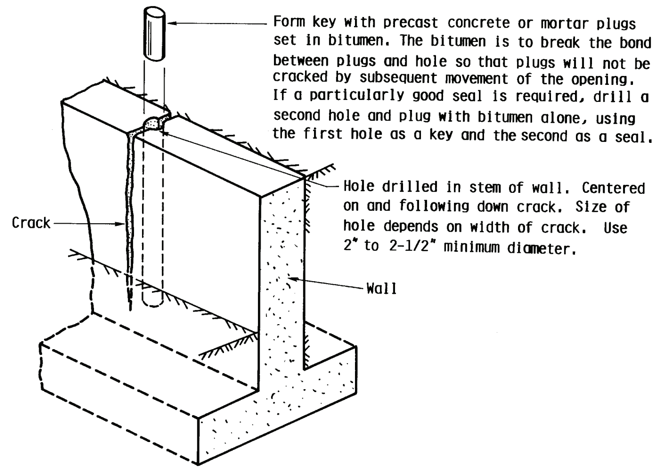


Fig. 3.6—Repair of crack by drilling and plugging.

needed so that the problem will not merely migrate to another part of the structure. The effects of the tensioning force (including eccentricity) on the stress within the structure should be carefully analyzed. For indeterminate structures post-tensioned using this procedure, the effects of secondary moments and induced reactions should be considered (Nilson 1987; Lin and Burns 1981).

3.6—Drilling and plugging

Drilling and plugging a crack consists of drilling down the length of the crack and grouting it to form a key (Fig. 3.6).

This technique is only applicable when cracks run in reasonably straight lines and are accessible at one end. This method is most often used to repair vertical cracks in retaining walls (“Selection of a Crack Repair Method” 1985).

A hole (typically 2 to 3 in. [50 to 75 mm] in diameter) should be drilled, centered on and following the crack. The hole should be large enough to intersect the crack along its full length and provide enough repair material to structurally take the loads exerted on the key. The drilled hole should then be cleaned, made tight, and filled with grout. The grout key prevents transverse movements of the sections of concrete adjacent to the crack. The key will also reduce heavy leakage through the crack and loss of soil from behind a leaking wall.

If watertightness is essential and structural load transfer is not, the drilled hole should be filled with a resilient material of low modulus instead of grout. If the keying effect is essential, the resilient material can be placed in a second hole, with the first being grouted.

3.7—Gravity filling

Low-viscosity monomers and resins can be used to seal cracks with surface widths of 0.001 to 0.08 in. (0.03 to 2 mm) by gravity filling (Rodler et al. 1989; ACI RAP-2). High-molecular-weight methacrylates, urethanes, and some low-viscosity epoxies have been used successfully. The lower the viscosity, the finer the cracks that can be filled.

The typical procedure is to clean the surface by airblasting, waterblasting, or both. Wet surfaces should be permitted to dry for several days to obtain the best crack filling. The monomer or resin can be poured onto the surface and spread with brooms, rollers, or squeegees. The material should be worked back and forth over the cracks to obtain maximum filling because the monomer or resin recedes slowly into the cracks. The use of this method on elevated slabs will require sealing of the cracks on the bottom of the slab to contain material from leaking through the crack. Excess material should be broomed off the surface to prevent slick, shining areas after curing. If surface friction is important, sand should be broadcast over the surface before the monomer or resin cures.

If the cracks contain significant amounts of silt, moisture, or other contaminants, the sealant cannot fill them. Water-blasting followed by a drying time may be effective in cleaning and preparing these cracks. Cores may be taken to verify crack filling and the depth of penetration measured. Caution should be employed to avoid cutting existing reinforcement during the coring process. Cores can be tested to give an indication of the effectiveness of the repair method. The accuracy of the results may be limited, however, as a function of the crack orientation or due to the presence of reinforcing steel in the core. For some polymers, the failure crack will occur outside the repaired crack.

3.8—Grouting

3.8.1 *Portland-cement grouting*—Wide cracks, particularly in gravity dams (Warner 2004) and thick concrete walls, may be repaired by filling with portland-cement grout. This method is effective in stopping water leaks, but it will not structurally bond cracked sections. The procedure consists of cleaning the concrete along the crack; installing built-up seats (grout nipples) at intervals astride the crack (to provide a pressure-tight connection with the injection apparatus); sealing the crack between the seats with a cement paint, sealant, or grout; flushing the crack to clean it and test the seal; and then grouting the whole area. Grout mixtures may contain cement and water or cement plus sand and water, depending on the width of the crack. The *w/cm*, however, should be kept as low as practical to maximize the strength and minimize shrinkage. Water reducers or other admixtures may be used to improve the properties of the grout. For small volumes, a manual injection gun may be used; for larger volumes, a pump may be used. After the crack is filled, the pressure should be maintained for several minutes to ensure good penetration.

3.8.2 *Chemical grouting*—Chemical grouts, such as urethanes and acrylomides, are activated by catalysts or water to form a gel, a solid precipitate, or foam that will fill void space within concrete. The materials are primarily used for sealing cracks from water penetration. Bond strengths are typically low, so structural repairs are not made with chemical grouts. Cracks in concrete as narrow as 0.002 in. (0.05 mm) have been filled with chemical grout.

The advantages of chemical grouts include applicability in moist environments (excess moisture available), wide limits of control of gel time, and their ability to be applied in very fine fractures. Disadvantages are the high degree of skill needed for satisfactory use, and lack of strength.

3.9—Drypacking

Drypacking is the hand placement of a low water content mortar followed by tamping or ramming of the mortar into place, producing intimate contact between the mortar and the existing concrete (U.S. Bureau of Reclamation 1975; "Crack Repair Method: Drypacking" 1985). Because of the low *w/cm* of the material, there is little shrinkage, and the patch remains tight and can have good quality with respect to durability, strength, and watertightness.

Drypack can be used for filling narrow slots cut for the repair of dormant cracks. The use of drypack is not advisable for filling or repairing active cracks.

Before a crack is repaired by drypacking, the portion adjacent to the surface should be widened to a slot about 1 in. (25 mm) wide and 1 in. (25 mm) deep. The slot should be undercut so that the base width is slightly greater than the surface width.

After the slot is thoroughly cleaned and dried, a bond coat, consisting of cement slurry or equal quantities of cement and fine sand mixed with water to a fluid paste consistency, or an appropriate latex bonding compound (ASTM C 1059), should be applied. Placing of the drypack mortar should begin immediately. The mortar consists of one part cement, one to three parts sand passing a No. 16 (1.18 mm) sieve, and just enough water so that the mortar will stick together when molded into a ball by hand.

If the patch must match the color of the surrounding concrete, a blend of gray portland cement and white portland cement may be used. Normally, about 1/3 white cement is adequate, but the precise proportions can be determined only by trial.

To minimize shrinkage in place, the mortar should stand for 1/2 hour after mixing, and then be remixed before use. The mortar should be placed in layers about 3/8 in. (10 mm) thick. Each layer should be thoroughly compacted over the surface using a blunt stick or hammer and each underlying layer scratched to facilitate bonding with the next layer. There need be no time delays between layers.

The repair should be cured by using either water or a curing compound. The simplest method of moist curing is to support a strip of folded wet burlap along the length of the crack.

3.10—Crack arrest

During construction of massive concrete structures, cracks due to surface cooling or other causes may develop and propagate into new concrete as construction progresses. Such cracks may be arrested by blocking the crack and spreading the tensile stress over a larger area (U.S. Army Corps of Engineers 1945).

A piece of bond-breaking membrane or a grid of steel mat may be placed over the crack as concreting continues. A semicircular pipe placed over the crack may also be used (Fig. 3.7). A description of installation procedures for semi-circular pipes used during the construction of a massive concrete structure follows:

1. The semicircular pipe is made by splitting an 8 in. (200 mm), 16 gauge pipe and bending it to a semicircular section with an approximately 3 in. (75 mm) flange on each side;

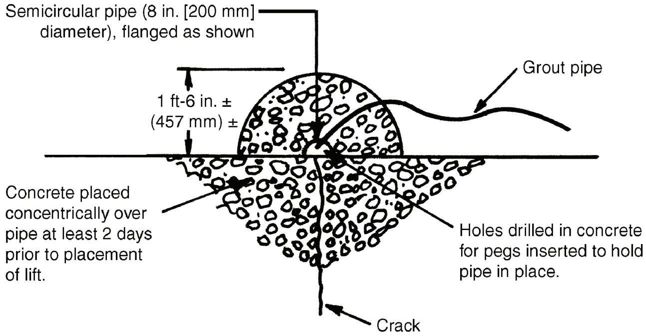


Fig. 3.7—Crack arrest method of crack repair.

2. The area in the vicinity of the crack is cleaned;
3. The pipe is placed in sections so as to remain centered on the crack;
4. The sections are then welded together;
5. Holes are cut in the top of the pipe to receive grout pipes; and
6. After setting the grout pipes, the installation is covered with concrete placed concentrically over the pipe by hand. The installed grout pipes are used for grouting the crack at a later date.

3.11—Polymer impregnation

This method is used to repair cracks by impregnation with a very low-viscosity monomer that is then polymerized in place. A monomer system is a liquid consisting of monomers that will polymerize into a solid. Suitable monomers have varying degrees of volatility, toxicity, and flammability, and they do not mix with water. They are very low in viscosity, and will soak into dry concrete and fill the cracks. The most common monomer used for this purpose is methyl methacrylate.

Monomer systems used for impregnation contain a catalyst or initiator plus the basic monomer (or combination of monomers). They may also contain a cross-linking agent. When heated, the monomers join together, or polymerize, creating a tough, strong, durable plastic that greatly enhances a number of concrete properties.

If a cracked concrete surface is dried, flooded with the monomer, and polymerized in place, some of the cracks will be filled and structurally repaired. If the cracks contain moisture, however, the monomer will not soak into the concrete at each crack face; consequently, the repair will be unsatisfactory. If a volatile monomer evaporates before polymerization, it will be ineffective. Polymer impregnation has not been used successfully to repair fine cracks. Polymer impregnation has primarily been used to provide more durable, impermeable surfaces (Webster et al. 1978; Hallin 1978; "Crack Repair Method: Polymer Impregnation" 1985).

Badly fractured beams have been repaired using polymer impregnation. The procedure consists of drying the fracture, temporarily encasing it in a watertight (monomer-proof) band of sheet metal, soaking the fractures with monomer, and polymerizing the monomer. Large voids or broken areas in compression zones can be filled with fine and coarse aggregate before being flooded with monomer, providing a polymer concrete repair. A more detailed discussion of polymer impregnation is given in ACI 548.1R.

3.12—Overlay and surface treatments

Cracks in structural slabs and pavements may be covered by using either a bonded or unbonded overlay or surface treatment. These methods do not repair cracks, but rather hide or obscure the cracks, unless a sufficiently reinforced overlay is installed that can handle the design loads and not allow reflective cracking.

3.13—Autogenous healing

A natural process of crack repair known as autogenous healing can occur in concrete in the presence of moisture and the absence of tensile stress (Lauer and Slate 1956). It has practical application for closing dormant cracks in a moist environment, such as in mass concrete structures.

Healing occurs by formation of calcium carbonate within cracks. Calcium carbonate forms by exposure of calcium hydroxide, which is a by-product of cement hydration, to carbon dioxide in air and water (Edvardsen 1999). The calcium carbonate that slowly precipitates is a hard, strong material. It not only fills void space within cracks, but also bonds crack surfaces and restores strength to the concrete.

Healing will not occur if the crack is subjected to movement during the healing period. Healing will also not occur if there is a positive flow of water through the crack.

Saturation of the crack and the adjacent concrete with water during the healing process is essential for developing any substantial strength. Submergence of the cracked section is desirable. Alternatively, water may be ponded on the concrete surface so that the crack is saturated. The saturation should be continuous for the entire period of healing. A single cycle of drying and reimmersion will produce a drastic reduction in the amount of healing strength. Procedures to facilitate healing should begin as soon as possible after the crack appears. Delayed healing results in less restoration of strength than does immediate correction.

CHAPTER 4—REFERENCES

4.1—Referenced standards and reports

The standards and reports listed below were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

American Association of State Highway and Transportation Officials
Standard Specification for Highway Bridges

American Concrete Institute
201.1R Guide for Making a Condition Survey of Concrete in Service
201.2R Guide to Durable Concrete
201.3R Guide for Making a Condition Survey of Concrete Pavements
207.1R Guide to Mass Concrete

207.2R Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete
207.3R Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions
207.4R Cooling and Insulating Systems for Mass Concrete
222R Protection in Metals in Concrete Against Corrosion
224R Control of Cracking in Concrete Structures
224.2R Cracking of Concrete Members in Direct Tension
224.3R Joints in Concrete Construction
228.2R Nondestructive Test Methods for Evaluation of Concrete in Structures
302.1R Guide for Concrete Floor and Slab Construction
304R Guide for Measuring, Mixing, Transporting, and Placing Concrete
305R Hot Weather Concreting
308R Guide to Curing Concrete
309R Guide for Consolidation of Concrete
309.2R Identification and Control of Visual Effects of Consolidation Formed Concrete Surfaces
318 Building Code Requirements for Structural Concrete
343R Analysis and Design of Reinforced Concrete Bridge Structures
345R Guide for Concrete Highway Bridge Deck Construction
345.1R Guide for Maintenance of Concrete Bridge Members
347 Guide to Formwork for Concrete
446.1R Fracture Mechanics of Concrete: Concepts, Models, and Determination of Material Properties
503R Use of Epoxy Compounds with Concrete
504R Guide to Sealing Joints in Concrete Structures
517.2R Accelerated Curing of Concrete at Atmospheric Pressure
546R Concrete Repair Guide
546.1R Guide for Repair of Concrete Bridge Superstructures
546.2R Guide to Underwater Repair of Concrete
548.1R Guide for the Use of Polymers in Concrete
RAP-1 Structural Crack Repair by Epoxy Injection, http://www.concrete.org/general/RAP-1.pdf
RAP-2 Crack Repair by Gravity Feed by Resin, http://www.concrete.org/general/RAP-2.pdf

ASTM International
C 150 Specification for Portland Cement
C 595 Specification for Blended Hydraulic Cements
C 597 Test Method for Pulse Velocity through Concrete
C 856 Practice for Petrographic Examination of Hardened Concrete
C 876 Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete
C 881 Specification for Epoxy-Resin-Base Bonding Systems for Concrete
C 1059 Specification for Latex Agents for Bonding Fresh to Hardened Concrete
C 1157 Performance Specification for Hydraulic Cement

International Concrete Repair Institute
ICRI Guideline No. 03734, "Guide for Verifying Field Performance of Epoxy Injection of Concrete Cracks"

The above publications may be obtained from the following organizations:

American Association of State Highway and Transportation Officials
444 N Capitol Street NW
Suite 224
Washington, D.C. 20001

American Concrete Institute
P.O. Box 9094
Farmington Hills, MI 48333-9094

ASTM International
100 Barr Harbor Drive
West Conshohocken, PA 19428-2959

International Concrete Repair Institute
3166 S. River Road, Suite 132
Des Plaines, IL 60018

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