

# 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:** autogenous healing; beams (supports); cement-aggregate reactions; concrete construction; concrete pavements; concrete slabs; concretes; consolidation; corrosion; cracking (fracturing); drilling; drying shrinkage; epoxy resins; evaluation; failure; grouting; heat of hydration; mass concrete; methacrylates; mix proportioning; plastics, polymers and resins; precast concrete; prestressed concrete; reinforced concrete; repairs; resurfacing; sealing settlement (structural); shrinkage; specifications; structural design; tension; thermal expansion; volume change.

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ACI 224.1R-93 supersedes ACI 224.1R-90 and became effective September 1, 1993.

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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.

The proper repair of cracks depends on knowing the causes and selecting the repair procedures that take these causes into account; otherwise, the repair may only be temporary. Successful long-term repair procedures must attack the causes of the cracks as well as the cracks themselves.

To aid the practitioner in pinpointing the best solution to a cracking problem, this report discusses the causes, evaluation procedures, and methods of repair of cracks in concrete. **Chapter 1** presents a summary of the causes of cracks and is designed to provide background for the evaluation of cracks. **Chapter 2** describes evaluation techniques and criteria. **Chapter 3** describes the methods of crack repair and includes a discussion of a number of techniques that are available. Many situations will require a combination of methods to fully correct the problem.

**Preface to the 1991 Revision**

Following the initial publication of ACI 224.1R in 1985, the Committee processed two minor revisions. One revision, published as ACI 224.1R-89 simply updated the format of recommended references. A second minor revision contained minor technical revisions and editorial corrections in the document, and added a new section to **Chapter 3**, regarding the use of high-molecular-weight methacrylates as sealer/healers.

During 1990 a Committee 224 Task Group reviewed the document and recommended the revisions contained herein. **Chapter 1** has been altered in only minor detail. The introduction to **Chapter 2** has been revised extensively, and additional minor revisions have been made to the rest of the Chapter. In **Chapter 3**, the section on routing and sealing has been rewritten to include flexible sealing and overbanding of cracks, and it is updated to reflect current materials and construction practices. **Section 3.2** on epoxy injection has been revised to be somewhat more general and reflect current practice. The for-

mer section on high-molecular-weight methacrylates has been moved to **Section 3.7** and retitled "Gravity Filling." This recognizes the point that "high-molecular-weight methacrylate" is a material, and not a method. References are presented in **Chapter 5**; citations throughout the text have been revised to employ the author/date format. Several new references have been added.

Additional revision of the report is ongoing. Committee 224 invites comment from the readers and users of this report on new developments, or alternate viewpoints on the *Causes, Evaluation, and Repair of Cracks in Concrete Structures*.

**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 here, 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**-"Plastic shrinkage cracking (**Fig. 1.1**) occurs...when subjected to a very rapid loss of moisture caused by a combination of factors which include air and concrete temperatures, relative humidity, and wind velocity at the surface of the concrete. These factors can combine to cause high rates of surface evaporation in either hot or cold weather."

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, resulting in shallow cracks of varying depth which

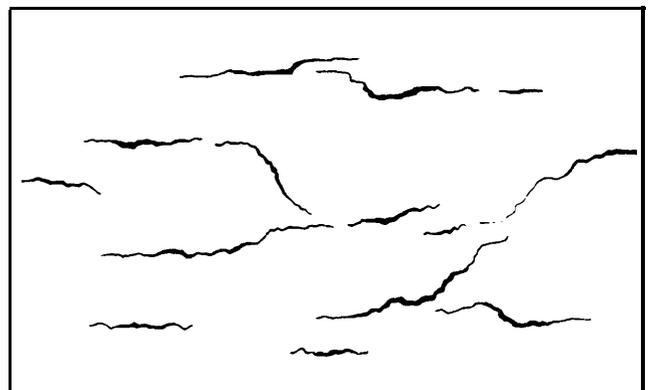


Fig. 1.1-Typical plastic shrinkage cracking (Price 1982)

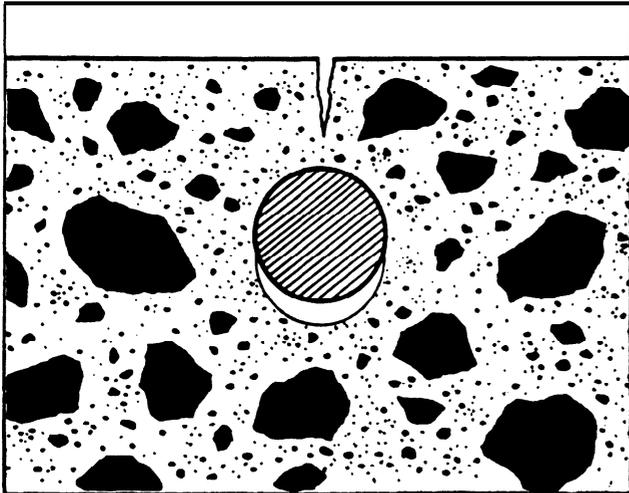


Fig. 1.2-Crack formed due to obstructed settlement (Price 1982)

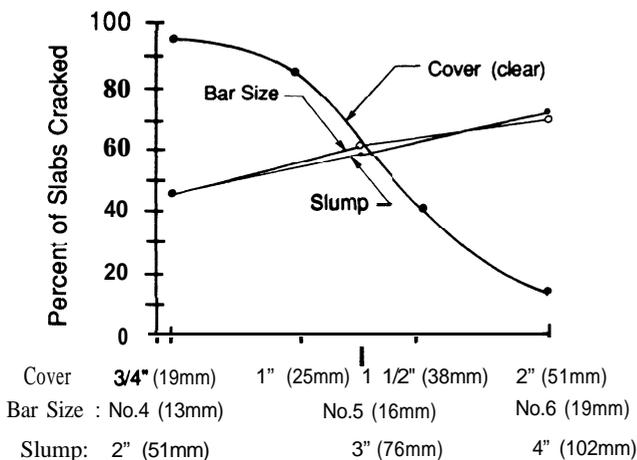


Fig. 1.3-Settlement cracking as a function of bar size, slump and cover (Dakhil et al. 1975)

may form a random, polygonal pattern, or may appear as essentially parallel to one another. These cracks are often fairly wide at the surface. They range from a few inches to many feet in length and are spaced from a few inches to as much as 10 ft (3 m) apart. Plastic shrinkage cracks begin as shallow cracks but can become full-depth cracks.

Since 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 a rapid moisture loss due to hot weather and dry winds (ACI 224R, ACI 302.1R, ACI 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, and it is good practice to schedule flat work after the windbreaks have been erected.

**1.2.2 Settlement cracking** — After initial placement, vibration, and finishing, concrete has a tendency to continue to consolidate. During this period, the plastic concrete may be locally restrained by reinforcing steel, a prior concrete placement, or formwork. This local restraint may result in voids and/or cracks 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.

Form design (ACI 347R) and vibration (and revibration), 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), the use of the lowest possible slump, and an increase in concrete cover will reduce settlement cracking.

### 1.3-Cracking of hardened concrete

**1.3.1 Drying shrinkage**-A common cause of cracking in concrete is restrained drying shrinkage. Drying shrinking is caused by the loss of moisture from the cement paste constituent, which can shrink by as much as 1 percent. Fortunately, aggregate provides internal restraint that reduces the magnitude of this volume change to about 0.06 percent. On wetting, concrete tends to expand.

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 (usually provided by another part of the structure or by the subgrade) that causes tensile stresses to develop. When the tensile strength of concrete is exceeded, it will crack. Cracks may propagate at much lower stresses than are required to cause crack initiation.

In massive concrete elements, tensile stresses are caused by differential shrinkage between the surface and the interior concrete. The larger 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 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 water content of the mix. The greater the amount of aggregate, the smaller the amount of shrinkage (Pickett 1956). The higher the stiffness of the aggregate, the more effective it is in reducing the shrinkage of the concrete (*i.e.*, the shrinkage of concrete containing sandstone aggregate may be more than twice that of concrete with granite,

basalt, or limestone (Carlson 1938)). The higher the water content, the greater the amount of drying shrinkage (U.S. Bureau of Reclamation 1975).

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.

Drying shrinkage can be reduced by increasing the amount of aggregate and reducing the water content. 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. 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 steel detailing. Shrinkage cracking may also be reduced by using shrinkage-compensating cement. The reduction or elimination of slab restraint can also be effective in reducing shrinkage cracking in slabs-on-grade (Wimsatt et al. 1987). In cases where crack control is particularly important, the minimum requirements of ACI 318 are not always 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.

**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 stress capacity, concrete will crack. The effects of temperature differentials due to different rates of heat 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), while 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 up 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, ACI 207.2R, ACI 207.4R, and ACI 224R.

Hardened concrete has a coefficient of thermal expansion that may range from 4 to  $9 \times 10^{-6}$  F (7 to  $11 \times 10^{-6}$  C), with a typical value of  $5.5 \times 10^{-6}$  F ( $10 \times 10^{-6}$  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, while 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 (Priestly 1978; Hoffman et al. 1983; ACI 343R). Allowing for movement by using properly designed contraction 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 here, 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 (e.g., curing water, ground water, alkaline solutions stored or used in the finished structure.)

The alkali-silica reaction results in the formation of a swelling gel, which tends to draw water from other portions of the concrete. This causes local expansion and accompanying tensile stresses, and may eventually result in the complete deterioration of the structure. Control measures include proper selection of aggregates, use of low alkali cement, and use of pozzolans, which themselves contain very fine, highly active silicas. The first measure may preclude the problem from occurring, while the later two measures have the effect of decreasing the alkali to reactive silica ratio, resulting in the formation of a nonexpanding calcium alkali silicate.

Certain carbonate rocks participate in reactions with alkalis which, in some instances, produce detrimental expansion and cracking. These detrimental alkali-carbonate

reactions are usually associated with argillaceous dolomitic limestones which 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).

Sulfate-bearing waters 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, with a subsequently large increase in volume, resulting in high local tensile stresses that lead to cracking which causes development of closely spaced cracking and deterioration. ASTM C 150 Types II and V portland cement, which are low in tricalcium aluminate, will reduce the severity of the problem. The blended cements specified in ASTM C 595 are also useful in this regard. In severe cases, some pozzolans, known to impart additional resistance to sulfate attack, could be used after adequate testing.

Detrimental conditions may also occur from the application of deicing salts to the surface of hardened concrete. 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 hydroxide in hydrated cement paste will combine with carbon dioxide in the air to form calcium carbonate. Since calcium carbonate has a smaller volume than the calcium hydroxide, shrinkage will occur (commonly known as carbonation shrinkage). This situation may result in significant surface crazing and may be especially serious on freshly placed surfaces during the first 24 hours when improperly vented combustion heaters are used to keep concrete warm during the winter months.

With the exception of surface carbonation, very little can be done to protect or repair concrete that has been subjected to the types of chemical attack described above (ACI 201.2R).

**1.3.4 Weathering**—The weathering processes that can cause cracking include freezing and thawing, wetting and drying, and heating and cooling. Cracking of concrete due to natural weathering is usually conspicuous, and it may give the impression that the concrete is on the verge of disintegration, even though the deterioration may not have progressed much below the surface.

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 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 water-cement ratio and total water content, durable aggregate, and adequate air entrainment. Adequate curing prior to 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).

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, which have a volume much greater than the volume of the original metallic iron (Verbeck 1975). 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 (*i.e.*, 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, a well-known problem in bridge decks.

Cracks provide easy access for oxygen, moisture, and chlorides, and thus, minor splitting cracks can create a condition in which corrosion and cracking are accelerated.

Cracks transverse to reinforcement usually do not cause continuing corrosion of the reinforcement if the concrete has low permeability. This is due to the fact that the exposed portion of a bar at a crack acts as an anode.

At early ages, the wider the crack, the greater the corrosion, simply because a greater portion of the bar has lost its passive protection. However, for continued corrosion to occur, 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 reinforcing is effective in delaying the corrosion process 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 (NCHRP Synthesis 57). 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 any concrete surface treatment must allow water 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 improve 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. By adding cement, even if the water-cement ratio remains constant, more shrinkage will occur since the relative 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 (see ACI 224R, ACI 302.1R, ACI 304R, ACI 305R, ACI 308, ACI 309R, ACI 345R, and ACI 347R), but require special attention during construction to insure their proper execution.

**1.3.7 Construction overloads**-Loads induced during construction can often be far 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 (1985, 1987) 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 must 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 must 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 superelevations 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 one day old. Multiple strands must 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; Jastnebski 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 prior to 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 must 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 and/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 here in the broadest sense).

Errors in design and detailing that may result in unacceptable cracking include use of poorly detailed reentrant comers in walls, precast members and slabs, improper selection and/or detailing of reinforcement, 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).

Reentrant comers provided 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 must recognize that stresses are always high near reentrant comers. 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.

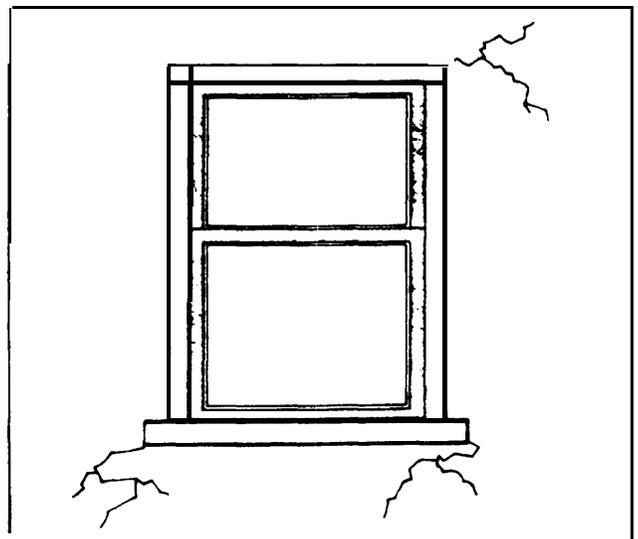


Fig. 1.4—Typical crack pattern at reentrant corners (Price 1982)

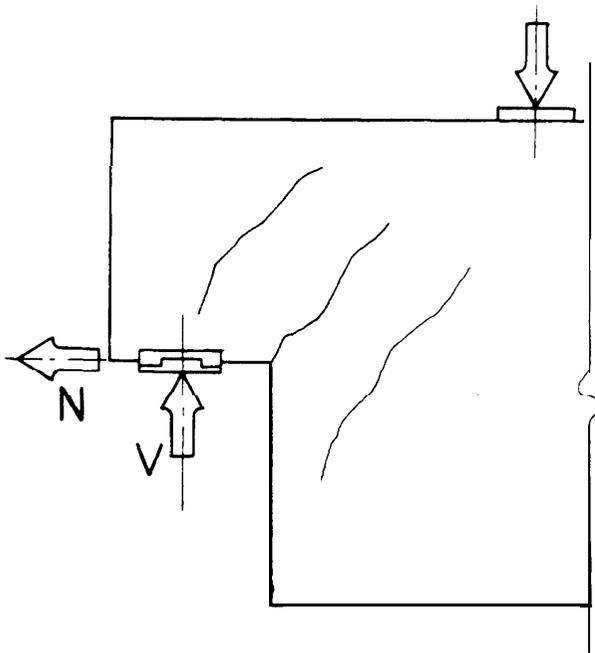


Fig. 1.5—Typical cracking pattern of dapped end at service load\*

The use of an inadequate amount of reinforcing may result in excessive cracking. A typical mistake is to lightly reinforce a member because it is a “nonstructural member.” However, the member (such as a wall) 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. Since this member is not detailed to act structurally, unsightly cracking may result even though the safety of the structure is not in question.

The restraint of members subjected to volume changes results frequently 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 reentrant comers), results may be catastrophic (Kaminetzky 1981; Mast 1981).

Improper foundation design may result in excessive differential movement within a structure. If the differ-

ential movement is relatively small, the cracking problems may be only visual in nature. However, if there is a major differential settlement, 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 must 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 concrete design. Current design procedures (ACI 318 and AASHTO) Standard Specifications 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.

Current knowledge of flexural members provides the basis for the following general conclusions about the variables that control cracking: 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 considered to best predict the most probable maximum surface crack width in bending was developed by Gergely and Lutz (1968). A simplified version of this equation is:

$$w = 0.076 \beta f_s (d_c A)^{0.33} \times 10^{-3} \quad (1.1)$$

in which  $w$  = most probable maximum crack width, in.;  $\beta$  = 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;  $d_c$  = thickness of cover from tension fiber to center of bar closest thereto, in.; and  $A$  = area of concrete symmetric with reinforcing steel divided by number of bars,  $\text{in.}^2$

A modification of this equation is used in ACI 318, which effectively limits crack widths to 0.016 in. (0.41 mm) for interior exposure and 0.013 in. (0.33 mm) for exterior exposure. However, considering the information presented in Section 1.3.5 which indicates little correlation between surface crack width for cracks transverse to bars and the corrosion of reinforcing, these limits do not appear to be justified on the basis of corrosion control.

\*From Alan H. Mattock and Timothy C. Chan (1979). “Design and Behavior of Dapped-end Beams,” *Journal, Prestressed Concrete Institute*, V. 24, No. 6, Nov.-Dec., pp. 28-45.

There have been a number of 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):

$$w = 0.10 f_s (d_c A)^{0.33} \times 10^{-3} \quad (1.2)$$

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).

Although work remains to be done, the basic principles of crack control for load-induced cracks are well understood. Well-distributed reinforcing offers the best protection against undesirable cracking. Reduced steel stress, obtained through the use of a larger amount of steel, will also reduce the amount of cracking. While reduced cover will reduce the surface crack width, designers must 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 reinforcing.

## 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 observed cracks are indicative of current or future structural problems, taking into consideration the present and anticipated future 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 those 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, or appearance are not significantly affected. Cracks in pavements and slabs-on-grade may require repair to prevent edge spalls, migration of water to the subgrade, or to transmit loads. 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 and destructive testing, and tests of cores taken from the structure. Information may also be obtained from drawings and construction and maintenance records.

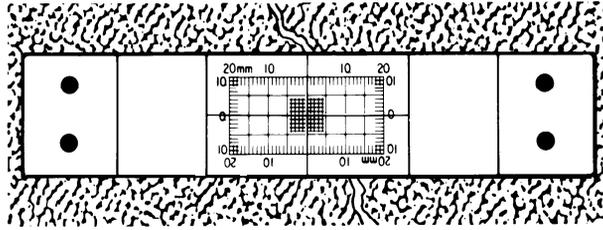
**2.2.1 Direct and indirect observation**-The locations and widths of cracks should be noted on a sketch of the structure. A grid marked on the surface of the structure can be useful to accurately locate cracks on the sketch.

Crack widths can be measured to an accuracy of about 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](#)). Crack widths may also be estimated using a clear comparator card having lines of specified width marked on the card. 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 fiber-scopes or rigid borescopes.

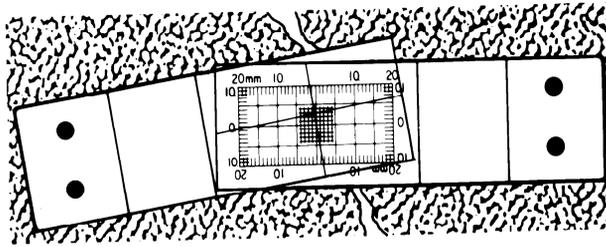
Crack movement can be monitored with mechanical movement indicators of the types shown in [Fig. 2.2](#). The indicator, or crack monitor, shown in [Fig. 2.2 \(a\)](#) gives a direct reading of crack displacement and rotation. The indicator in [Fig. 2.2 \(b\)](#) (Stratton et al. 1978) amplifies the crack movement (in this case, 50 times) and indicates the maximum range of movement during the measurement period. Mechanical indicators have the advantage



*Fig. 2.1-Comparator for measuring crack widths (courtesy of Edmund Scientific Co.)*

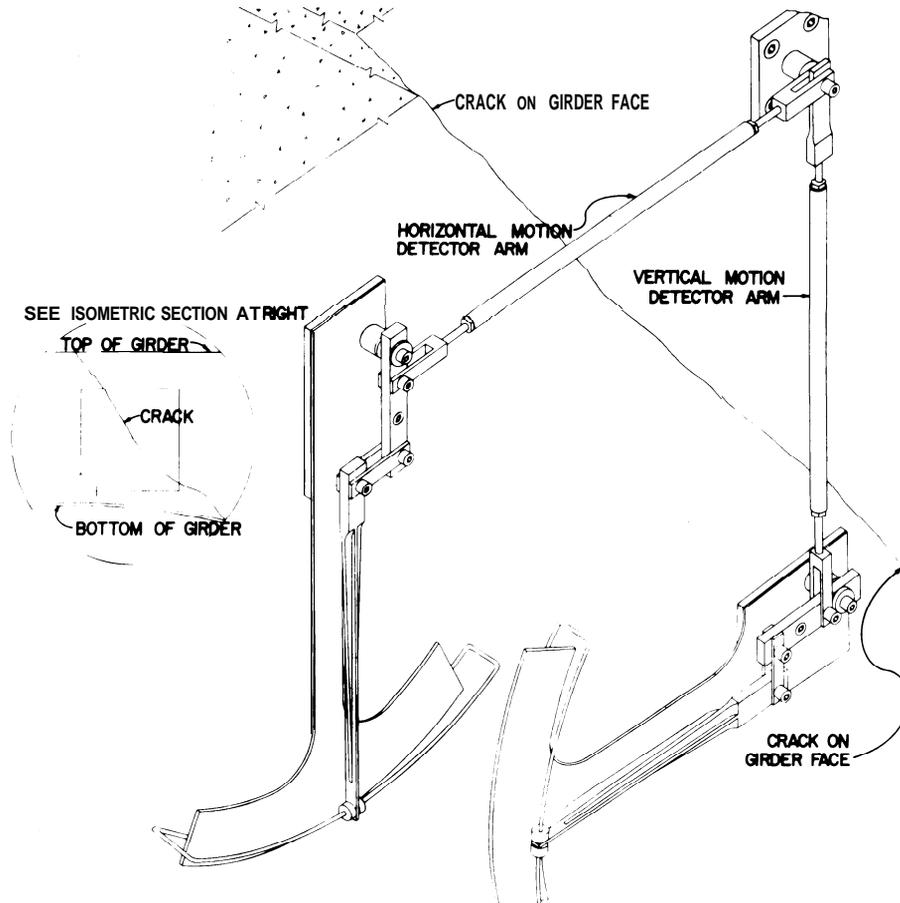


Newly Mounted Monitor



Monitor After Crack Movement

(a)-Crack monitor (courtesy of Avongard)



(b)-Crack movement indicator (Stratton et al. 1978)

Figure 2.2



Fig. 2.3-Pachometer (reinforcing bar locator) (courtesy of James Instruments)

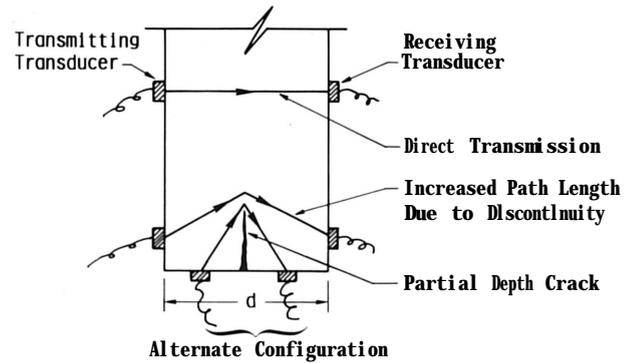
that they do not require moisture protection. If more detailed time histories are desired, a wide range of transducers (most notably linear variable differential transformers or LVDTs) and data acquisition systems (ranging from strip chart recorders to computer-based systems) are available.

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, ACI 201.3R, ACI 207.3R, ACI 345.1R, and ACI 546.1R.

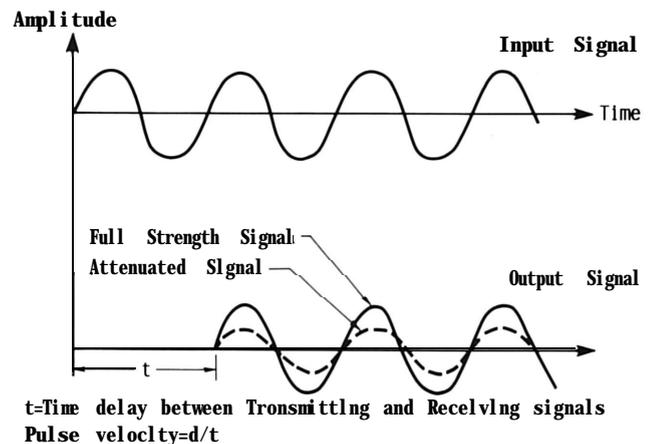
**2.2.2 Nondestructive testing-**Nondestructive tests can be made to determine the presence of internal cracks and voids and the depth of penetration of cracks visible at the surface.

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.

The presence of reinforcement can be determined using a pachometer (Fig. 2.3) (Malhotra 1976). 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.



a) Pulse transmitted through member



t=Time delay between Transmitting and Receiving signals  
Pulse velocity= $d/t$

b) Oscilloscope Signal

Fig. 2.4-Ultrasonic testing through-transmission technique

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; Clear and Hay 1973); its use also requires access to a portion of the reinforcing steel.

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 1991; Knab et al. 1983). A mechanical pulse is transmitted to one face of the concrete member and received at the opposite face, as shown Fig. 2.4. 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.

When access is not available to opposite faces, transducers may be located on the same face [Fig. 2.4(a)]. While this technique is possible, the interpretation of results is not straightforward.

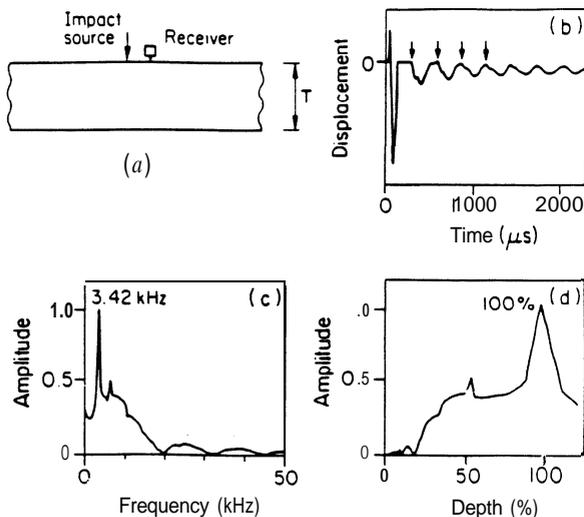


Fig. 2.5-Impact-echo response of a solid plate: a) schematic of test configuration; b) displacement waveform; c) amplitude spectrum; and d) normalized amplitude spectrum

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. 2.4(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 a trained person, and the results should be evaluated cautiously by an experienced person, because moisture, reinforcing steel, and embedded items may affect the results. For example, with fully saturated cracks, ultrasonic testing will generally be ineffective. In some cases, it is difficult to discern between a group of close 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 been difficult to develop a practical pulse-echo test for concrete. Pitch-catch systems have been developed which use separate transmitting and receiving transducers (Alexander 1980). More detailed information on pulse-echo and other wave propagation methods is provided by Malhotra and Carino (1991).

Significant advances in use of wave propagation techniques for flaw detection in concrete by the impact-echo technique have been made by Sansalone and Carino (1988, 1989). A mechanical pulse is generated by impact on one face of the member as illustrated in Fig. 2.5. 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. Fig. 2.5(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. 2.5(c)] the frequency associated with the resonance appears as a peak amplitude. In the case of Fig. 2.5(a), the peak is that associated with the thickness frequency [see Fig. 2.5(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 are available (Malhotra and Carino 1991; 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 to x-ray equipment and therefore appears to be more suitable for field testing.

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 of cracked concrete can identify material causes of cracking, such as alkali reactivities, 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 water-to-cement ratio, 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 and reinforcement placing or other shop drawings should be reviewed to confirm that the concrete thickness and quality, along with installed reinforcing, meets or exceeds strength and serviceability requirements noted in the governing building code(s). A detailed review of actual applied loading compared to design loads should get special consideration. Concrete configurations, restraint conditions, and the presence of construction and other joints should be considered in calculating the tensile stresses induced by concrete

deformation (creep, shrinkage, temperature, etc.). Special consideration should be given to cracks that develop parallel to one-way reinforced slabs primarily supported on beams, but also bear on the girders that support those beams.

### 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 and increase strength;
2. Restore and increase stiffness;
3. Improve functional performance;
4. Provide watertightness;
5. Improve appearance of the concrete surface;
6. Improve durability; and/or
7. Prevent development of 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. However, it may be necessary to provide additional strength by adding reinforcement or using post-tensioning. Epoxy injection alone can be used to restore flexural stiffness if further cracking is not anticipated (ACI 503R).

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 (See [section 3.14](#)). 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. However, the crack locations may still be visible and it is likely that 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 the cracked structure, a suitable repair procedure can be selected. Successful repair procedures take into account the cause(s) of the cracking. For example, if the cracking was 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 a continuing 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

the 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 546.1R and ACI Compilation No. 5 (1980), 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. 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). However, unless the cause of the cracking has been corrected, it will probably recur near the original crack. If the cause of the cracks cannot be removed, then two options are available. One is to rout and seal the crack, thus treating it as a joint, or, establish a joint that will accommodate the movement and then inject the crack with epoxy or other suitable material. 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, but contaminants in the cracks (including silt and water) can reduce the effectiveness of the epoxy to structurally repair the cracks.

The use of a low-modulus, flexible adhesive in a crack will not allow significant movement of the concrete structure. The effective modulus of elasticity of a flexible adhesive in a crack is substantially the same as that of a rigid adhesive (Adams et al. 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 application of the technique may be limited by the ambient temperature. The general procedures involved in epoxy injection are as follows (ACI 503R):

- Clean the cracks. The first step is to clean the 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 specially 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.

- Seal 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-grade 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 must therefore 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 appearance of the completed work is important. If extremely high injection pressures are needed, the crack can be cut out to a depth of 1/2 in. (13 mm) and width of about 3/4 in. (20 mm) in a V-shape, filled with an epoxy, and struck off flush with the surface.
- Install the entry and venting ports. Three methods are in general use:
  - a. Fittings inserted into drilled holes. This method was the first to be used, and is often used in conjunction with V-grooving of the cracks. The 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 watercooled corebit, is useful in preventing the cracks from being plugged with drilling dust.
  - b. Bonded flush fitting. When the cracks are not V-grooved, a method 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.
  - c. 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.
- Mix the epoxy. This is done either by batch or continuous methods. In batch mixing, the adhesive components are premixed according to the manu-

facturer's instructions, usually with the use of a mechanical stirrer, like a paint mixing paddle. Care must be taken to mix only the amount of adhesive that can be used prior to 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 prior to passing through an automatic mixing head. The continuous mixing system allows the use of fast setting adhesives that have a short working life.

- Inject the epoxy. Hydraulic pumps, paint pressure pots, or air-actuated caulking guns may be used. The pressure used for injection must 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 the 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 the same manner. The crack is full if the pressure can be maintained. If the pressure can not be maintained, the epoxy is still flowing into unfilled portions or leaking out of the crack.

- Remove the surface seal. After the injected epoxy has cured, the surface seal should be removed by grinding or other means as appropriate.
- Alternative procedure. For massive structures, an alternate procedure consists of drilling a series of holes [usually 7/8 to 4-in. (20 to 100-mm) diameter] that intercepts the crack at a number of locations. Typically, holes are spaced at 5-ft (1.5-m) intervals.

Another method recently being used is a vacuum or vacuum assist method. There are two techniques: one is to entirely enclose the cracked member with a bag and introduce the liquid adhesive at the bottom and to 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. As reported by Stratton and McCollum the first, second, and sixth steps are omitted and the process is terminated at a specific location when epoxy exits from the crack at some distance from the injection ports. This procedure does not arrest ongoing corrosion. The procedure can also be

attempted for other applications, and is available as an option, although not accepted universally. Success of the repair depends on the absence of bond-inhibiting contaminants from the crack plane. 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 seven years after injection. Smithson and Whiting describe epoxy injection as a method to rebond delaminated bridge deck overlays. Committee 224 is developing additional information on this application for inclusion in a future revision of this Report.

### 3.3-Routing and sealing

Routing and sealing of cracks can be used in conditions requiring remedial repair and where structural repair is not necessary. This method involves enlarging the crack along its exposed face and filling and sealing it with a suitable joint sealant (Fig. 3.1). This is a common technique for crack treatment and is relatively simple in comparison to the procedures and the training required for epoxy injection. The procedure is most applicable to approximately flat horizontal surfaces such as floors and pavements. However, routing and sealing can be accomplished on vertical surfaces (with a non-sag sealant) as well as on curved surfaces (pipes, piles and pole).

Routing and sealing is used to treat both fine pattern cracks and larger, isolated 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. Satisfactory sealants should be able to withstand cyclic deformations and should not be brittle.

The procedure consists of preparing a groove at the surface ranging in depth, typically, from 1/4 to 1 in. (6 to 25 mm). A concrete saw, hand tools or pneumatic tools may be used. The groove is then cleaned by air blasting, sandblasting, or waterblasting, and dried. A sealant is placed into the dry groove and allowed to cure.

A bond breaker may be provided at the bottom of the groove to allow the sealant to change shape, without a concentration of stress on the bottom (Fig. 3.2). The bond breaker may be a polyethylene strip or tape which will not bond to the sealant.

Careful attention should be applied when detailing the joint so that its width to depth aspect ratio will accommodate anticipated movement (ACI 504R).

In some cases overbanding (strip coating) is used in-

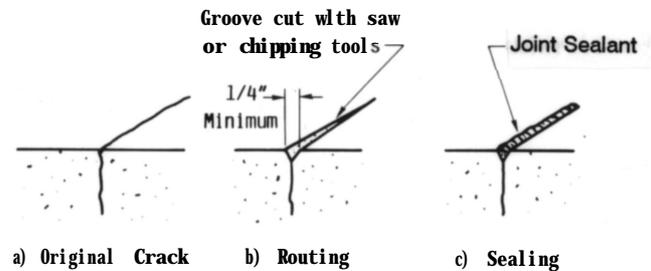


Fig. 3.1-Repair of crack by routing and sealing (Johnson 1965)

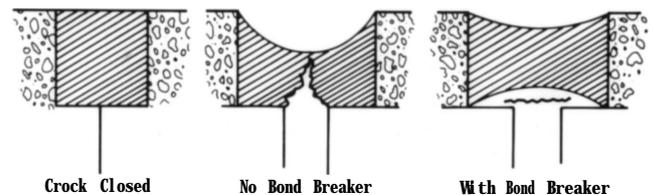


Fig. 3.2-Effect of bond breaker

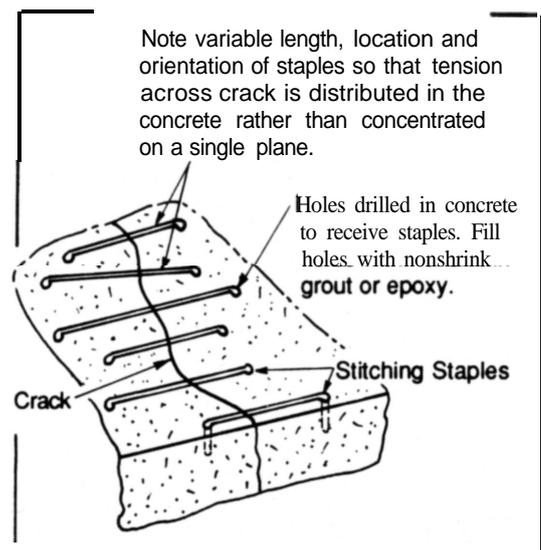


Fig. 3.3-Repair of crack by stitching (Johnson 1965)

dependently 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 sandblast-

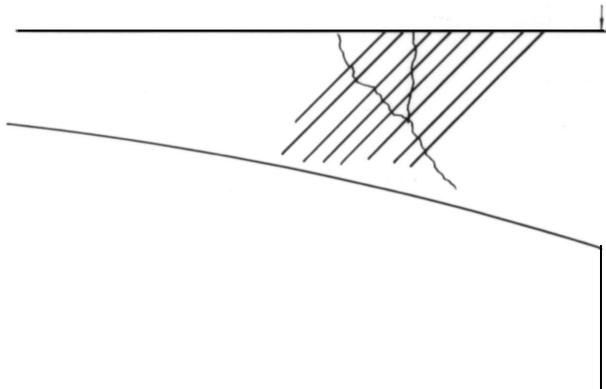


Fig. 3.4-Reinforcing bar orientation used to effect the repair (Stratton et al. 1978)

ing or other means of surface preparation, and apply 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 non-traffic 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 must be used in conjunction with overbanding to ensure a waterproof repair.

### 3.4-Stitching

Stitching involves drilling holes on both sides of the crack and grouting in U-shaped metal units with short legs (staples or stitching dogs) that span the crack as shown in Fig 3.3 (Johnson 1965). Stitching may be used when tensile strength must be reestablished across major cracks (Hoskins 1991). Stitching a crack tends to stiffen the structure, and the stiffening may increase the overall structural restraint, causing the concrete to crack elsewhere. Therefore, it may be necessary to strengthen the adjacent section or sections using technically corrected-reinforcing methods. Because stresses are often concentrated, using this method in conjunction with other methods may be necessary.

The stitching procedure consists of drilling holes on both sides of the crack, cleaning the holes, and anchoring the legs of the staples in the holes, with either a non-shrink grout or an epoxy resin-based bonding system. The staples should be variable in length, orientation, or both, and they should be located so that the tension transmitted across the crack is not applied to a single plane within the section but is spread over an area.

### 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, 1982; Stratton 1980).

This technique consists of sealing the crack, drilling holes that intersect the crack plane at approximately 90 deg (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 (10 M or 15 M) 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 very 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 must be provided for the prestressing steel, and care is 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).

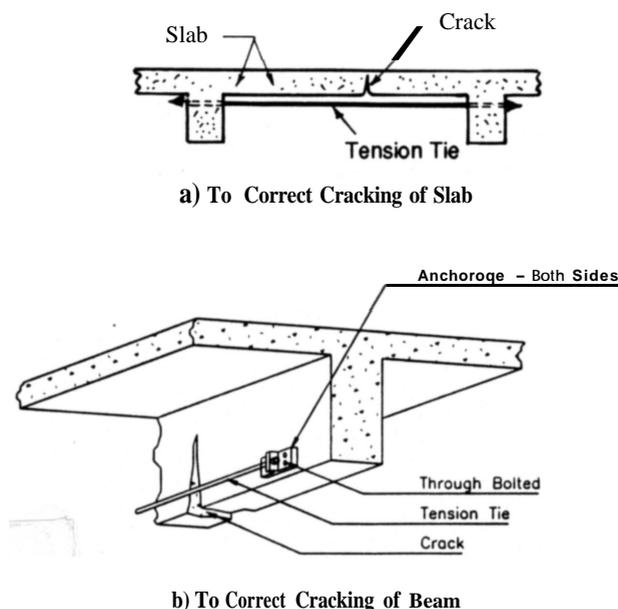


Fig 3.5-Examples of external prestressing (Johnson 1965)

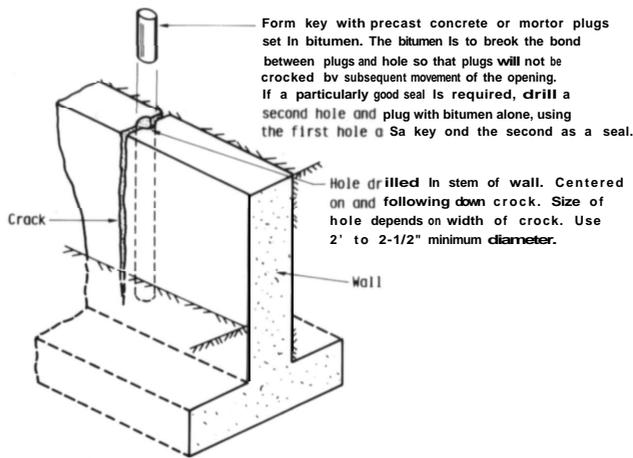


Fig. 3.6-Repair of crack by drilling and plugging

This technique is only applicable when cracks run in reasonable straight lines and are accessible at one end. This method is most often used to repair vertical cracks in retaining walls.

A hole [typically 2 to 3 in. (50 to 75 mm) in diameter] should be drilled, centered on and following the crack. The hole must 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 water-tightness is essential and structural load transfer is not, the drilled hole should be filled with a resilient material of low modulus in lieu of grout. If the keying effect is essential, the resilient material can be placed in a second hole, 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). 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 air blasting and/or waterblasting. Wet surfaces should be permitted to dry 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 since the monomer or resin recedes slowly into the cracks. 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 taken at cracks can be used to evaluate the effectiveness of the crack filling. The depth of penetration of the sealant can be measured. Shear (or tension) tests can be performed with the load applied in a direction parallel to the repaired cracks (as long as reinforcing steel is not present in the core in or near the failure area). 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 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. However, the water-cement ratio 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 should be used. After the crack is filled, the pressure should be maintained for several minutes to insure good penetration.

**3.8.2 Chemical grouting**-Chemical grouts consist of solutions of two or more chemicals (such as urethanes, sodium silicates, and acrylamides) that combine to form a gel, a solid precipitate, or a foam, as opposed to cement grouts that consist of suspensions of solid particles in a fluid. 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 their 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 1978). Because of the low water-cement ratio 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

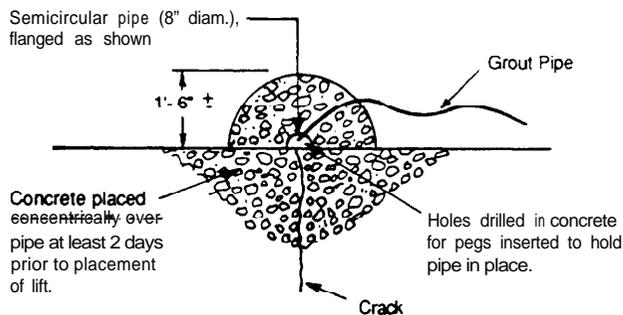


Fig. 3.7-Crack arrest method of crack repair

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 dry pack 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 grey portland cement and white portland cement may be used. Normally, about one-third 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 should be re-mixed prior to 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 should be scratched to facilitate bonding with the next layer. There need be no time delays between layers.

The mortar may be finished by laying the flat side of a hardwood piece against it and striking it several times with a hammer. Surface appearance may be improved by a few light strokes with a rag or sponge float. 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 semicircular 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-gage pipe and bending it to a semicircular section with about a 3-in. (75-mm) flange on each side; (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, thereby restoring all or a portion of the structural continuity.

### 3.11-Polymer impregnation

Monomer systems can be used for effective repair of some cracks. A monomer system is a liquid consisting of monomers which 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, filling the cracks, much as water does. 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. However, if the cracks contain moisture, the monomer will not soak into the concrete at each crack face, and 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).

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 polymers is given in ACI 548R.

### 3.12-Overlay and surface treatments

Fine surface cracks in structural slabs and pavements may be repaired using either a bonded overlay or surface treatment if there will not be further significant move-

ment across the cracks. Unbonded overlays may be used to cover, but not necessarily repair a slab. Overlays and surface treatments can be appropriate for cracks caused by one-time occurrences and which do not completely penetrate the slab. These techniques are not appropriate for repair of progressive cracking, such as that induced by reactive aggregates, and D-cracking.

Slabs-on-grade in freezing climates should not be repaired by an overlay or surface treatment that is a vapor barrier. An impervious barrier will cause condensation of moisture passing from the subgrade, leading to critical saturation of the concrete and rapid disintegration during cycles of freezing and thawing.

**3.12.1 Surface treatments**—Low solids and low-viscosity resin-based systems have been used to seal the concrete surfaces, including treatment of very fine cracks. They are most suited for surfaces not subject to significant wear.

Bridge decks and parking structure slabs, as well as other interior slabs may be coated effectively after cracks are treated by injecting with epoxy or by routing and sealing. Materials such as urethanes, epoxies, polyesters, and acrylics have been applied in thickness of 0.04 to 2.0 in. (1 to 50 mm), depending on the material and purpose of the treatment. Skid-resistant aggregates are often mixed into the material or broadcast onto the surface to improve traction.

**3.12.2 Overlays**—Slabs containing dormant cracks can be repaired by applying an overlay, such as polymer-modified portland cement mortar or concrete, or by silica fume concrete. Slabs with working cracks can be overlaid if joints are placed in the overlay directly over the working cracks. In highway bridge applications, an overlay thickness as low as 1-1/4 in. (30 mm) has been used successfully (NCHRP Synthesis 57). Suitable polymers include styrene butadiene or acrylic latexes. The resin solids should be at least 15 percent by weight of the portland cement, with 20 percent usually being optimum (Clear and Chollar 1978).

The surface to be overlaid should be cleaned to remove laitance, carbonated or otherwise weak material, or contaminants, such as grease or oil. A bond coat consisting of the mortar fraction broom-applied, or an epoxy adhesive should be applied immediately before placing the overlay. Since polymer-modified concretes normally solidify rapidly, continuous batching and mixing equipment should be used. Polymer-modified overlays should be mixed, placed and finished rapidly (within 15 min in warm weather). A 24-hr moist curing is typical for these overlays.

### 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 may be found in mass concrete structures.

Healing occurs through the continued cement hydra-

tion and the carbonation of calcium hydroxide in the cement paste by carbon dioxide, which is present in the surrounding air and water. Calcium carbonate and calcium hydroxide crystals precipitate, accumulate, and grow within the cracks. The crystals interlace and twine, producing a mechanical bonding effect, which is supplemented by a chemical bonding between adjacent crystals and between the crystals and the surfaces of the paste and the aggregate. As a result, some of the tensile strength of the concrete is restored across the cracked section, and the crack may become sealed.

Healing will not occur if the crack is active and is subjected to movement during the healing period. Healing will also not occur if there is a positive flow of water through the crack, which dissolves and washes away the lime deposit, unless the flow of water is so slow that complete evaporation occurs at the exposed face causing redeposition of the dissolved salts.

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 must 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. Healing should be commenced as soon as possible after the crack appears. Delayed healing results in less restoration of strength than does immediate correction.

## CHAPTER 4-SUMMARY

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. Both analytical and field requirements are discussed. 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 repairs. Criteria for the selection of crack repair procedures are based on the desired outcome of the repairs.

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

## ACKNOWLEDGMENT

ACI Committee 224 — Cracking, gratefully acknow-

ledges the assistance of Robert Gaul, Paul Krauss, and James Warner, non-members of the Committee, for their suggestions and review of the revisions to this document. The Committee would also like to recognize the contributions of Raymond J. Schutz, former Committee Member Paul H. Karr and deceased Committee Members Donald L. Houghton and Robert E. Philleo who were Contributing Authors of the original version of ACI 224.1R.

## CHAPTER 5-REFERENCES

### 5.1 Recommended references

The Documents of the various standards-producing organizations referred to in this document are listed below with their serial designation.

#### *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
- 207.1R Mass Concrete
- 207.2R Effect of Restraint, Volume Change, and Reinforcement on Cracking of Mass Concrete
- 207.4R Cooling and Insulating Systems for Mass Concrete
- 224R Control of Cracking in Concrete Structures
- 224.2R Cracking of Concrete Members in Direct Tension
- 224.3R Joints in Concrete Construction
- 302.1R Guide to Concrete Floor and Slab Construction
- 304R Guide for Measuring, Mixing Transporting and Placing Concrete
- 305R Hot Weather Concreting
- 308 Standard Practice for Curing Concrete
- 309R Guide for Consolidation of Concrete
- 309.2R Identification and Control of Consolidation-Related Surface Defects in Formed Concrete
- 318 Building Code Requirements for Reinforced Concrete
- 343R Analysis and Design of Reinforced Concrete Bridge Structures
- 345R Guide for Concrete Highway Bridge Deck Construction
- 347R Guide to Concrete Formwork
- 350R Environmental Engineering of Concrete Structures
- 503R Use of Epoxy Compounds with Concrete
- 504R Guide to Sealing Joints in Concrete Structures
- 517.2R Accelerated Curing of Concrete at Atmospheric Pressure--State of the Art
- 546.1R Guide for Repair of Concrete Bridge Superstructures
- 548R Polymers in Concrete

#### *American Society for Testing and Materials*

- C 150 Standard Specification for Portland Cement
- C 595 Standard Specification for Blended Hydraulic Cements
- C 597 Standard Test Method for Pulse Velocity through Concrete
- C 876 Standard Test Method for Half Cell Potentials of Reinforcing Steel in Concrete
- C 881 Standard Specification for Epoxy-Resin-Base Bonding Systems for Concrete
- C 1059 Standard Specification for Latex Agents for Bonding Fresh to Hardened Concrete

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 19150  
Detroit MI 48219

American Society for Testing and Materials  
1916 Race Street  
Philadelphia PA 19103

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