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Joints in Concrete Construction

Reported by ACI Committee 224

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This report reviews the state of the art in design, construction, and maintenance of joints in concrete structures subjected to a wide variety of use and environmental conditions. In some cases, the option of eliminating joints is considered. Aspects of various joint sealant materials and jointing techniques are discussed. The reader is referred to ACI 504R for a more comprehensive treatment of sealant materials, and to ACI 224R for a broad discussion of the causes and control of cracking in concrete construction. Chapters in the report focus on various types of structures and structural elements with unique characteristics: buildings, bridges, slabs-on-grade, tunnel linings, canal linings, precast concrete pipe, liquid-retaining structures, walls, and mass concrete.

Keywords: bridges, buildings, canals, canal linings, concrete construction, construction joints, contraction joints, design, environmental engineering concrete structures, isolation joints, joints, parking lots, pavements, runways, slabs-on-grade, tunnels, tunnel linings, walls.

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CHAPTER 1—INTRODUCTION

1.1—Joints in concrete structures

Joints are necessary in concrete structures for a variety of reasons. Not all concrete in a given structure can be placed continuously, so there are construction joints that allow for work to be resumed after a period of time. Since concrete undergoes volume changes, principally related to shrinkage and temperature changes, it can be desirable to provide joints and thus relieve tensile or compressive stresses that would be induced in the structure. Alternately, the effect of volume changes can be considered just as other load effects are considered in building design. Various concrete structural elements are supported differently and independently, yet meet and match for functional and architectural reasons. In this case, compatibility of deformation is important, and joints may be required to isolate various members.

Many engineers view joints as artificial cracks, or as means to either avoid or control cracking in concrete structures. It is possible to create weakened planes in a structure, so cracking occurs in a location where it may be of little importance, or have little visual impact. For these reasons, ACI Committee 224—Cracking, has developed this report as an overview of the design, construction, and maintenance of joints in various types of concrete structures, expanding on the currently limited treatment in ACI 224R. While other ACI Committees deal with specific types of structures, and joints in those structures, this is the first ACI report to synthesize information on joint practices into a single document. Committee 224 hopes that this synthesis will promote continued re-evaluation of recommendations for location and spacing of joints, and the development of further rational approaches.

Diverse and sometimes conflicting guidelines are found for joint spacing. Table 1.1 reports various recommendations for contraction joints, and Table 1.2 provides a sampling of requirements for expansion joints. It is hoped that, by bringing the information together in this Committee Report, recommendations for joint spacing may become more rational, and possibly more uniform.

Aspects of construction and structural behavior are important when comparing the recommendations of Tables 1.1 and

Table 1.1—Contraction joint spacings

Author	Spacing
Merrill (1943)	20 ft (6 m) for walls with frequent openings, 25 ft (7.5 m) in solid walls.
Fintel (1974)	15 to 20 ft (4.5 to 6 m) for walls and slabs on grade. Recommends joint placement at abrupt changes in plan and at changes in building height to account for potential stress concentrations.
Wood (1981)	20 to 30 ft (6 to 9 m) for walls.
PCA (1982)	20 to 25 ft (6 to 7.5 m) for walls depending on number of openings.
ACI 302.1R	15 to 20 ft (4.5 to 6 m) recommended until 302.1R-89, then changed to 24 to 36 times slab thickness.
ACI 350R-83	30 ft (9 m) in sanitary structures.
ACI 350R	Joint spacing varies with amount and grade of shrinkage and temperature reinforcement.
ACI 224R-92	One to three times the height of the wall in solid walls.

Table 1.2—Expansion joint spacings

Author	Spacing
Lewerenz (1907)	75 ft (23 m) for walls.
Hunter (1953)	80 ft (25 m) for walls and insulated roofs, 30 to 40 ft (9 to 12 m) for uninsulated roofs.
Billig (1960)	100 ft (30 m) maximum building length without joints. Recommends joint placement at abrupt changes in plan and at changes in building height to account for potential stress concentrations.
Wood (1981)	100 to 120 ft (30 to 35 m) for walls.
Indian Standards Institution (1964)	45 m (≈ 148 ft) maximum building length between joints.
PCA (1982)	200 ft (60 m) maximum building length without joints.
ACI 350R-83	120 ft (36 m) in sanitary structures partially filled with liquid (closer spacings required when no liquid present).

1.2. These recommendations may be contrary to usual practice in some cases, but each could be correct for particular circumstances. These circumstances include, but may not be limited to: the type of concrete and placing conditions; characteristics of the structure; nature of restraint on an individual member; and the type and magnitude of environmental and service loads on the member.

1.2—Joint terminology

The lack of consistent terminology for joints has caused problems and misunderstandings that plague the construction world. In 1979 the American Concrete Institute Technical Activities Committee (TAC) adopted a consistent terminology on joints for use in reviewing ACI documents:

> *Joints will be designated by a terminology based on the following characteristics: resistance, configuration, formation, location, type of structure, and function.*

Characteristics in each category include, but are not limited to the following:

Resistance: Tied or reinforced, doweled, nondoweled, plain.

Configuration: Butt, lap, tongue, and groove.

Formation: Sawed, hand-formed, tooled, grooved, insert-formed.

Location: Transverse, longitudinal, vertical, horizontal.

Type of Structure: Bridge, pavement, slab-on-grade building.

Function: Construction, contraction, expansion, isolation, hinge.

Example: Tied, tongue and groove, hand-tooled, longitudinal pavement construction joint.

The familiar term, "**control joint**," is not included in this list of joint terminology, since it does not have a unique and universal meaning. Many people involved with construction have used the term to indicate a joint provided to "control" cracking due to volume change effects, especially shrinkage. However, improperly detailed and constructed "control" joints may not function properly, and the concrete can crack adjacent to the presumed joint. In many cases a "control joint" is really nothing more than rustication. These joints are really trying to control cracking due to shrinkage and thermal contraction. A properly detailed contraction joint is needed.

An additional problem with joint nomenclature concerns "isolation" and "expansion" joints. An isolation joint isolates the movement between members. That is, there is no steel or dowels crossing the joint. An expansion joint, by comparison, is usually doweled such that movement can be accommodated in one direction, but there is shear transfer in the other directions. Many people describe structural joints without any restraint as expansion joints.

1.3—Movement and restraint in concrete structures

Restrained movement is a major cause of cracking in concrete structures. Internal or external restraint can develop tensile stresses in a concrete member, and the tensile strength or strain capacity can be exceeded. Restrained movement of concrete structures includes the effects of settlement: compatibility of deflections and rotations where members meet, and volume changes.

Volume changes typically result from shrinkage as hardened concrete dries, and from expansion or contraction due to temperature changes.

A detailed discussion of volume change mechanisms is beyond the scope of this report. Evaluate specific cases to determine the individual contributions of temperature change and loss of moisture to the environment. The potential volume change is considered in terms of the restraint that results from geometry, as well as reinforcement.

1.3.1 *Shrinkage volume changes*—While many types of shrinkage are important and may cause cracking in concrete structures, drying shrinkage of hardened concrete is of special concern. Drying shrinkage is a complicated function of parameters related to the nature of the cement paste, plain concrete, member, or structural geometry and environment. For example, building slabs shrink about 500 x 10^{-6}, yet shrinkage of an exposed slab on grade may be less than 100 x 10^{-6}. A portion of drying shrinkage also may be reversible. A large number of empirical equations have been proposed to predict shrinkage. ACI 209R provides information on predicting shrinkage of concrete structures. If shrinkage-compensating concrete is used, it is necessary for the structural element to expand against elastic restraint from internal reinforcement before it dries and shrinks (ACI 224R).

1.3.2 *Expansion volume changes*—Where a shrinkage-compensating concrete is used, additional consideration of the expansion that will occur during the early life of the concrete is necessary. Unless a shrinkage-compensating concrete is allowed to expand, its effectiveness in compensating for shrinkage will be reduced.

1.3.3 *Thermal volume changes*—The effects of thermal volume changes can be important during construction and in service as the concrete responds to temperature changes. Two important factors to consider are the nature of the temperature change and the fundamental material properties of concrete.

The coefficient of thermal expansion for plain concrete α describes the ability of a material to expand or contract as temperatures change. For concrete, α depends on the mixture proportions and the type of aggregate used. Aggregate properties dominate the behavior, and the coefficient of linear expansion can be predicted. Mindess and Young (1981) discuss the variation of the expansion coefficient in further detail. Ideally, the coefficient of thermal expansion could be computed for the concrete in a particular structure. This is seldom done unless justified by unusual material properties or a structure of special significance. For concrete, the coefficient of thermal expansion α can be reasonably assumed to be 6 ξ 10^{-6}/F (11 x 10^{-6}/C).

During construction, the heat generated by hydrating portland cement may raise the temperature of a concrete mass higher than will be experienced in service. Contraction of the concrete as the temperature decreases while the material is relatively weak may lead to cracking. ACI 224R, ACI

207.1R, and ACI 207.2R discuss control of cracking for ordinary and mass concrete due to temperature effects during construction.

In service, thermal effects are related to long-term and nearly instantaneous temperature differentials. Long-term shrinkage has the same sense as the effect of temperature drops, so overall contraction is likely to be the most significant volume change effect for many structures.

For some components in a structure, the longer term effects are related to the difference of hottest summer and lowest winter temperature. The structure also may respond to the difference between temperature extremes and a typical temperature during construction. In most cases the larger temperature difference is most important.

Daily variations in temperature are important, too. Distortions will occur from night to day, or as sunlight heats portions of the structure differently. These distortions may be very complicated, introducing length changes, as well as curvatures into portions of the structure. An example is the effect of "sun camber" in parking structures where the roof deck surface becomes as much as 20 to 40 F (10 to 20 C) hotter than the supporting girder. This effect causes shears and moments in continuous framing.

1.4—Objectives and scope

This report reviews joint practices in concrete structures subjected to a wide variety of uses and environmental conditions. Design, construction, and maintenance of joints are discussed, and in some cases, the option of eliminating joints is considered. Chapter 2 summarizes aspects of various sealant materials and jointing techniques. However, the reader is referred to ACI 504R for a more comprehensive treatment. Chapters 3-10 focus on various types of structures and structural elements with unique characteristics: buildings, bridges, slabs-on-grade, tunnel linings, canal linings, precast concrete pipe, liquid-retaining structures, walls, and mass concrete. Many readers of this report will not be interested in all types of construction discussed in Chapters 3-10. These readers may wish to first study Chapter 2, then focus on a specific type of structure.

While not all types of concrete construction are addressed specifically in this report, the Committee feels that this broad selection of types of structures can provide guidance in other cases as well. Additional structural forms may be addressed in future versions of this report.

ACI 224R provides additional detailed discussion of both the causes of cracking and control of cracking through design and construction practice.

CHAPTER 2—SEALANT MATERIALS AND JOINTING TECHNIQUES

2.1—Introduction

A thorough discussion of joint sealant materials is found in ACI 504R. This Chapter summarizes the pertinent facts about joint sealants. The reader is cautioned that this Chapter is only an introduction.

2.2—Required properties of joint sealants

For satisfactory behavior in open surface joints the sealant should:

- Be relatively impermeable
- Deform to accommodate the movement and rate of movement occurring at the joint
- Sufficiently recover its original properties and shape after cyclical deformations
- Remain in contact with the joint faces. The sealant must bond to the joint face and not fail in adhesion, nor peel at corners or other local areas of stress concentration. An exception is preformed sealants that exert a force against the joint face
- Not rupture internally (fail in cohesion)
- Not flow because of gravity (or fluid pressure)
- Not soften to an unacceptable consistency at higher service temperatures
- Not harden or become unacceptably brittle at lower service temperatures
- Not be adversely affected by aging, weathering, or other aspects of service conditions for the expected service life under the range of temperatures and other environmental conditions that occur
- Be replaceable at the end of a reasonable service life, if it fails during the life of the structure

Seals buried in joints, such as waterstops and gaskets, require generally similar properties. The method of installation may, however, require the seal to be in a different form and, because replacement is usually impossible, exceptional durability is required.

In addition, depending on the specific service conditions, the sealant may be required to resist one or more of the following: intrusion of foreign material, wear, indentation, pickup (tendency to be drawn out of joint, as by a passing tire), and attack by chemicals present. Additional requirements may be that the sealant has a specific color, resists changes in color, and is nonstaining.

Sealant should not deteriorate when stored for a reasonable time before use. It also should be reasonably easy to handle and install, and be free of substances harmful to the user, the concrete, or other material that may come in contact.

2.3—Commercially-available materials

No material has properties perfect for all applications. Sealant materials are selected from a large range of materials that offer a sufficient number of the required properties at a reasonable cost.

Oil-based mastics, bituminous compounds, and metallic materials were the only types of sealants available for many years. However, for many applications these traditional materials do not behave well. In recent years there has been active development of many types of "elastomeric" sealants whose behavior is largely elastic rather than plastic. These newer materials are flexible, rather than stiff, at normal service temperatures. Elastomeric materials are available as field-molded and preformed sealants. Though initially more expensive, they usually have a longer service life. They can seal joints where considerable movements occur and that could not possibly be sealed by traditional materials. This latitude in properties has opened new engineering and architectural possibilities to the designer of concrete structures.

No attempt has been made here to list or discuss each attribute of every available sealant. Discussion is limited to those features considered important to the designer, specifier, and user, so that claims made for various materials can be evaluated and a suitable choice made for the particular application.

2.4—Field-molded sealants

2.4.1 *Mastics*—Mastics are composed of a viscous liquid rendered immobile by the addition of fibers and fillers. They do not usually harden, set, or cure after application, but instead form a skin on the surface exposed to the atmosphere. The vehicle in mastics may include drying or nondrying oils (including oleoresinous compounds), polybutenes, polyisobutylenes, low-melting point asphalts, or combinations of these materials. With any of these, a wide variety of fillers is used, including fibrous talc or finely divided calcareous or siliceous materials. The functional extension-compression range of these materials is about ±3 percent.

Mastics are used in buildings for general caulking and glazing where very small joint movements are anticipated and economy in first cost outweighs that of maintenance or replacement. With time, most mastics tend to harden in increasing depth as oxidation and loss of volatiles proceeds, thus reducing their serviceability. Polybutene and polyisobutylene mastics have a somewhat longer service life than do the other mastics.

2.4.2 *Thermoplastics, hot applied*—These are materials that become soft on heating and harden on cooling, usually without chemical change. They are generally black and include asphalts, rubber asphalts, pitches, coal tars, and rubber tars. They are usable over an extension-compression range of ± 5 percent. This limit is directly influenced by service temperatures and aging characteristics of specific materials. Though initially cheaper than some of the other sealants, their service life is relatively short. They tend to lose elasticity and plasticity with age, to accept rather than reject foreign materials, and to extrude from joints that close tightly or that have been overfilled. Overheating during the melting process adversely affects the properties of compounds containing rubber. Those with an asphalt base are softened by hydrocarbons, such as oil, gasoline, or jet fuel spillage. Tar-based materials are fuel and oil resistant and these are preferred for service stations, refueling and vehicle parking areas, airfield aprons, and holding pads. However, noxious fumes are given off during their placement.

Use of this class of sealants is restricted to horizontal joints, since they would run out of vertical joints when installed hot, or subsequently in warm weather. They have been widely used in pavement joints, but they are being replaced by chemically curing or thermosetting field-molded sealants or compression seals. They are also used in building roofs, particularly around openings, and in liquid-retaining structures.

2.4.3 *Thermoplastics, cold-applied, solvent, or emulsion type*—These materials are set either by the release of solvents or the breaking of emulsions on exposure to air. Sometimes they are heated up to 120 F (50 C) to simplify application, but they are usually handled at ambient temperature. Release of solvent or water can cause shrinkage and increased hardness with a resulting reduction in the permissible joint movement and in serviceability. Products in this category include acrylic, vinyl, and modified butyl types that are available in a variety of colors. Their maximum extension-compression range is ±7 percent. However, heat softening and cold hardening may reduce this figure.

These materials are restricted in use to joints with small movements. Acrylics and vinyls are used in buildings, mainly for caulking and glazing. Rubber asphalts are used in canal linings, tanks, and as crack fillers.

2.4.4 *Thermosetting, chemical curing*—Sealants in this class are either one- or two-component systems. They are applied in liquid form and cure by chemical reaction to a solid state. These include polysulfide, silicone, urethane, and epoxy-based materials. The properties that make them suitable as sealants for a wide range of uses are resistance to weathering and ozone, flexibility and resilience at both high and low temperatures, and inertness to a wide range of chemicals, including, for some, solvents and fuels. In addition, the abrasion and indentation resistance of urethane sealants is above average. Thermosetting, chemically curing sealants have an extension-compression range of up to ±25 percent, depending on the particular sealant, at temperatures from -40 to +180 F (-40 to +82 C). Silicone sealants remain flexible over an even wider temperature range. They have a wide range of uses in buildings and containers for both vertical and horizontal joints, and also in pavements. Though initially more expensive, thermosetting, chemically-curing sealants can stand greater movements than other field-molded sealants and generally have a much longer service life.

2.4.5 *Thermosetting, solvent release*—Another class of thermosetting sealants cure by the release of solvent. Chlorosulfonated polyethylene and certain butyl and neoprene materials are included in this class. Their characteristics generally resemble those of thermoplastic solvent release materials. They are, however, less sensitive to variations in temperature once they have "setup" on exposure to the atmosphere. Their maximum extension-compression range does not exceed ±7 percent. They are used mainly as sealants for caulking and joints in buildings, where both horizontal and vertical joints have small movements. Their cost is somewhat less than that of other elastomeric sealants, and their service life is likely to be satisfactory.

2.4.6 *Rigid*—Where special properties are required and movement is negligible, certain rigid materials can be used as field-molded sealants for joints and cracks. These include lead (wool or molten), sulfur, modified epoxy resins, and polymer-concrete type mortars.

2.5—Accessory materials

2.5.1 *Primers*—Where primers are required, a suitable proprietary material compatible with the sealant is usually supplied along with it. For hot poured field-molded sealants, these are usually high viscosity bitumens or tars cut back with solvent. To overcome damp surfaces, wetting agents may be included in primer formulations, or materials may be used that wet such surfaces preferentially, such as polyamide-cured coal tar-epoxies. For oleoresinous mastics, shellac can be used.

2.5.2 *Bond breakers*—Many backup materials do not adhere to sealants and thus, where these are used, no separate bond breaker is needed. Polyethylene tape, coated papers, and metal foils are often used where a separate bond breaker is needed.

2.5.3 *Backup materials*—These materials serve a variety of purposes during application of the sealant and in service. Backup materials limit the depth of the sealant; support it against sagging, indentation, and displacement by traffic or fluid pressure; and simplify tooling. They may also serve as a bond breaker to prevent the sealant from bonding to the back of the joint. The backup material should preferably be compressible so that the sealant is not forced out as the joint closes, and it should recover as the joint opens. Care is required to select the correct width and shape of material, so that after installation it is compressed to about 50 percent of its original width. Stretching, twisting, or braiding of tube or rod stock should be avoided. Backup materials and fillers containing bitumen or volatile materials should not be used with thermosetting chemical curing field-molded sealants. They may migrate to, or be absorbed at joint interfaces, and impair adhesion. In selecting a backup material to ensure compatibility, it is advisable to follow the recommendations of the sealant manufacturer.

Preformed backup materials are used for supporting and controlling the depth of field-molded sealants.

2.6—Preformed sealants

Traditionally, preformed sealants have been subdivided into two classes; rigid and flexible. Most rigid preformed sealants are metallic; examples are metal water stops and flashings. Flexible sealants are usually made from natural or synthetic rubbers, polyvinyl chloride, and like materials, and are used for waterstops, gaskets, and miscellaneous sealing purposes. Preformed equivalents of certain materials, *e.g.*, rubber asphalts, usually categorized as field molded, are available as a convenience in handling and installation. Compression seals should be included with the flexible group of preformed sealants. However, their function is different. The compartmentalized neoprene type can be used in most joint sealant applications as an alternative to field-molded sealants. They are treated separately in this report.

2.6.1 *Rigid waterstops and miscellaneous seals*—Rigid waterstops are made of steel, copper, and occasionally of lead. Steel waterstops are primarily used in dams and other heavy construction projects. Ordinary steel may require additional protection against corrosion. Stainless steels are used in dam construction to overcome corrosion problems. Steel waterstops are low in carbon and stabilized with columbium or titanium to simplify welding and retain corrosion resistance after welding. Annealing is required for improved flexibility, but the stiffness of steel waterstops may lead to cracking in the adjacent concrete.

Copper waterstops are used in dams and general construction; they are highly resistant to corrosion, but require careful handling to avoid damage. For this reason, in addition to considerations of higher cost, flexible waterstops are often used instead. Copper is also used for flashings.

At one time lead was used for waterstops, flashings, or protection in industrial floor joints. Its use is now very limited. Bronze strips find wide application in dividing, rather than sealing, terrazzo and other floor toppings into smaller panels.

2.6.2 *Flexible waterstops*—The types of materials suitable and in use as flexible waterstops are butyl, neoprene, and natural rubbers. These have satisfactory extensibility and resistance to water or chemicals and may be formulated for recovery and fatigue resistance. Polyvinyl chloride (PVC) compounds are, however, probably now the most widely used. This material is not quite as elastic as the rubbers, recovers more slowly from deformation, and is susceptible to oils. However, grades with sufficient flexibility (especially important at low temperatures) can be formulated. PVC has the advantage of being thermoplastic and it can be spliced easily on the job. Special configurations can also be made for joint intersections.

Flexible waterstops are widely used as the primary sealing system in dams, tanks, monolithic pipe lines, flood walls, swimming pools, etc. They may be used in structures that either retain or exclude water. For some applications in either precast or cast-in-place construction, a flexible waterstop containing sodium bentonite may also act as an internal joint sealant. Bentonite swells when contacted by water, and forms a gel, blocking infiltration through the structure.

2.6.3 *Gaskets and miscellaneous seals*—Gaskets and tapes are widely used as sealants at glazing and frames. They are also used around window and other openings in buildings, and at joints between metal or precast concrete panels in curtain walls. Gaskets are also used extensively at joints between precast pipes and where mechanical joints are needed in service lines. The sealing action is obtained either because the sealant is compressed between the joint faces (gaskets) or because the surface of the sealant, such as of polyisobutylene, is pressure sensitive and thus adheres.

2.7—Compression seals

These are preformed compartmentalized or cellular elastomeric devices that function as sealants when in compression between the joint faces.

2.7.1 *Compartmentalized*—Neoprene (chloroprene) or EPDM (ethylene propylene diene monomer) extruded to the required configuration is now used for most compression seals. For effective sealing, sufficient contact pressure is maintained at the joint face. This requires that the seal is always compressed to some degree. For this to occur, good resistance to compression set is required (that is, the material recovers sufficiently when released). In addition, the elastomer should be crystallization-resistant at low temperatures (the resultant stiffening may make the seal temporarily ineffective though recovery will occur on warming). If during the manufacturing process the elastomer is not fully cured, the interior webs may adhere together during service (often permanently) when the seal is compressed.

To simplify installation of compression seals, liquid lubricants are used. For machine installation, additives to make the lubricant thixotropic are necessary. Special lubricant adhesives that both prime and bond have been formulated for use where improved seal-to-joint face contact is required.

Neoprene compression seals are satisfactory for a wide range of temperatures in most applications.

Individual seals should remain compressed at least 15 percent of the original width at the widest opening. The allowable movement is about 40 percent of the uncompressed seal width.

Compression seals are manufactured in widths ranging from $^1/_2$ to 6 in. (12 to 150 mm); therefore, they are excellent for use in both expansion and contraction joints with anticipated movements up to 3 in. (75 mm).

2.7.2 *Impregnated flexible foam*—Another type of compression seal material is polybutylene-impregnated foam (usually a flexible open cell polyurethane). This material has found limited application in structures such as buildings and bridges. However, its recovery at low temperature is too slow to follow joint movements. Also, when highly compressed, the impregnant exudes and stains the concrete. This generally limits application to joints where less than ±5 percent extension-compression occurs at low temperature or ±20 percent where the temperature is above 50 F (10 C). The material often is bonded to the joint face.

2.8—Jointing practice

Four primary methods are available for creating joints in concrete surfaces: forming, tooling, sawing, and placement of joint formers.

2.8.1 *Formed joints*—These are found at construction joints in concrete slabs and walls. Tongue and groove joints can be made with preformed metal or plastic strips, or built to job requirements. These strips can serve as a screed point. They need to be fastened securely so they do not become dislodged during concrete placement and consolidation.

Prefabricated circular forms are available for use at column isolation joints. They are one-piece elements that latch together in the field, and are left in place. This allows placement of concrete inside the isolation blockout when the slab concrete is placed, if desired.

2.8.2 *Tooled joints*—Contraction joints can be tooled into a concrete surface during finishing operations. A groove intended to cause a weakened plane and to control the location of cracking should be at least $^1/_4$ the thickness of the concrete. Often, tooled joints are of insufficient depth to function properly. A joint about $^1/_2$ in. (10 to 15 mm) deep is nothing more than rustication. In concrete flatwork, cracks may occur within such a groove, but they are also quite likely to occur at adjacent locations or wander across the groove. Grooving tools with blades of $1^1/_2$ to 2 in. (40 to 50 mm) deep are available.

At a tooled contraction joint, the reinforcement in the concrete element should be reduced to at least one-half the steel area or discontinued altogether. As the distance between tooled contraction joints increases, the volume of steel reinforcement should be increased to control tension stresses that are developed.

2.8.3 *Sawed joints*—Use of sawed joints reduces labor during the finishing process. Labor and power equipment are required within a short period of time after the concrete has hardened. The most favorable time for sawing joints is when the concrete temperature (raised because of heat of hydration) is greatest; this may often be outside of normal working hours. In any event, joints should be sawed as soon as practical. The concrete should have hardened enough not to ravel during cutting. If there is a delay in cutting the slab, and a significant amount of shrinkage has already occurred, a crack may jump ahead of the saw as tensile stresses accumulate and reach a rupture level. As with tooled joints, saw-cut grooves at least $^1/_4$ of the depth of the member are recommended to create a functional plane of weakness.

A variety of sawing techniques and equipment is available. Blades may be diamond-studded, or made of consumable, abrasive material. If abrasive blades are used it is important to set a limit on the wear used to determine when the blade will be replaced. If this is not done, the depth of cut will be variable, and may be insufficient to force cracking within the cut. The resulting shallow cut is ineffective as a contraction joint, just like the shallow tooled joint. Cutting may be dry, or wet, with water used to cool the blade. Equipment may be powered by air, a self-contained gasoline engine, or an electric motor. A variety of special floor-cutting saws and other frames and rollers are available, depending on the application. Air-powered saws are lighter and lessen fatigue where workers hold them off the ground. Wet cutting prolongs blade life but produces a slurry and may be unsafe with electrical equipment. Diamond blades are more expensive than abrasive blades, but can be cost-effective on large projects when considering labor time lost in changing blades.

A final drawback to the use of sawed joints is equipment clearance. In sawing a concrete slab, it is impossible with most equipment to bring the saw cut to the edge, say, where a wall bounds the slab. Where the kerf terminates 2 to 3 in. (50 to 75 mm) from the wall, an irregular crack will form in the unsawed concrete as shrinkage occurs. The depth of cutting can be increased at a wall to improve the behavior of the weakened plane at the slab edge.

2.8.4 *Joint formers*—Joint formers can be placed in the fresh concrete during placing and finishing operations. Joint formers can be used to create expansion or contraction joints. Expansion joints generally have a removable cap over expansion joint material. After the concrete has hardened, the cap is removed and the void space caulked and sealed. Joint formers may be rigid or flexible. One flexible version has a strip-off cap of the same expansion material and is useful for isolation joints and joints curved in plane. Contraction joints are made by forming a weakened plane in the concrete with a rigid plastic strip. These are generally T-shaped elements that are inserted into the fresh concrete, often with the use of a cutter bar. After the contraction joint former is inserted to the proper depth, the top or cap is pulled away before final bullfloating or troweling. If a rounded edge is desired, an edging tool can be used.

CHAPTER 3—BUILDINGS

3.1—Introduction

Volume changes caused by changes in moisture and temperature should be accounted for in the design of reinforced concrete buildings. The magnitude of the forces developed and the amount of movement caused by these volume changes are directly related to building length. Contraction and expansion joints limit the magnitude of forces and movements and cracking induced by moisture or temperature change by dividing buildings into individual segments. Joints can be planes of weakness to control the location of cracks (contraction joints), or lines of separation between segments (isolation or expansion joints).

At present, there is no universally accepted design approach to accommodate building movements caused by temperature or moisture changes. Many designers use "rules of thumb" that set limits on the maximum length between building joints.

Although widely used, rules of thumb have the drawback that they do not account for the many variables that control volume changes in reinforced concrete buildings. These include variables that influence the amount of thermally induced movement, including the percentage of reinforcement; the restraint provided at the foundation; the geometry of the structure; the magnitude of intermediate cracks; and provisions for insulation, cooling, and heating.

In addition to these variables, the amount of movement in a building is influenced by materials and construction practices. These include the type of aggregate, cement, mix proportions, admixtures, humidity, construction sequence, and curing procedures. While these variables can be addressed quantitatively, their consideration is usually beyond the scope of a typical design sequence and will not be considered here. Many of these parameters are addressed by Mann (1970).

The purpose of this chapter is to provide guidance for the placement of construction, contraction, isolation, and expansion joints in reinforced concrete buildings. Joints in slabs on grade within the buildings are covered in Chapter 5. Additional information on joints in buildings is available in an annotated bibliography by Gray and Darwin (1984), and reports by PCA (1982) and Pfeiffer and Darwin (1987).

Once joint locations are selected, the joint should be constructed so that it will act as intended. The weakened section at a contraction joint may be formed or sawed, either with no reinforcement or a portion of the total reinforcement passing through the joint. The expansion or isolation joint is a discontinuity in both reinforcement and concrete; therefore, an expansion joint is effective for both shrinkage and temperature variations. Both joints can be used as construction joints, as described in the following section.

3.2—Construction joints

For many structures, it is impractical to place concrete in a continuous operation. Construction joints are needed to accommodate the construction sequence for placing the concrete. The amount of concrete that can be placed at one time is governed by batching and mixing capacity, crew size, and the amount of time available. Correctly located and properly executed construction joints provide limits for successive concrete placements, without adversely affecting the structure.

For monolithic concrete, a good construction joint might be a bonded interface that provides a watertight surface, and allows for flexural and shear continuity through the interface. Without this continuity, a weakened region results that may serve as a contraction or expansion joint. A contraction joint is formed by creating a plane of weakness. Some, or all, of the reinforcement may be terminated on either side of the plane. Some contraction joints, termed "partial contraction joints," allow a portion of the steel to pass through the joint. These joints, however, are used primarily in water-retaining structures. An expansion joint is formed by leaving a gap in the structure of sufficient width to remain open under extreme high temperature conditions. If possible, construction joints should coincide with contraction, isolation, or expansion joints. The balance of this section is devoted to construction joints in regions of monolithic concrete. Additional considerations for contraction, isolation, or expansion joints are discussed in the sections that follow.

3.2.1 *Joint construction*—To achieve a well-bonded watertight interface, a few conditions should be met before the placement of fresh concrete. The hardened concrete is usually specified to be clean and free of laitance (ACI 311.1R). If only a few hours elapse between successive placements, a visual check is needed to be sure that loose particles, dirt, and laitance are removed. The new concrete will be adequately bonded to the hardened green concrete, provided that the new concrete is vibrated thoroughly.

Older joints need additional surface preparation. Cleaning by an air-water jet or wire brooming can be done when the concrete is still soft enough that laitance can be removed, but hard enough to prevent aggregate from loosening. Concrete that has set should be prepared using a wet sand blast or ultra-high pressure water jet (ACI 311.1R).

ACI 318 states that existing concrete should be moistened thoroughly before placement of fresh concrete. Concrete that has been placed recently will not require additional water, but concrete that has dried out may require saturation for a day or more. Pools of water should not be left standing on the wetted surface at the time of placement; the surface should just be damp. Free surface water will increase the water-cement ratio of new concrete at the interface and weaken the bond strength. Other methods may also be useful for preparing a construction joint for new concrete.

Form construction plays an important role in the quality of a joint. It is essential to minimize the leakage of grout from under bulkheads (Hunter, 1953). If the placement is deeper than 6 in. (150 mm), the possibility of leakage increases due to the greater pressure head of the wet concrete. Grout that escapes under a bulkhead will form a thin wedge of material, which must be cut away before the next placement. If not removed, this wedge will not adhere to the fresh concrete, and, under load, deflection in the element will cause this joint to open.

3.2.2 *Joint location*—Careful consideration should be given to selecting the location of the construction joint. Construction joints should be located where they will least affect the structural integrity of the element under consideration, and be compatible with the building's appearance. Placement of joints varies, depending on the type of element under construction and construction capacity. For this reason, beams and slabs will be addressed separately from columns and walls. When shrinkage-compensating concrete is used, joint location allows for adequate expansion to take place. Details are given in ACI 223.

3.2.2.1 *Beams and slabs*—Desirable locations for joints placed perpendicular to the main reinforcement are at points of minimum shear or points of contraflexure. Joints are usually located at midspan or in the middle third of the span, but locations should be verified by the engineer before placement is shown on the drawings. Where a beam intersects a girder, ACI 318 requires that the construction joint in the girder should be offset a distance equal to twice the width of the incident beam.

Horizontal construction joints in beams and girders are usually not recommended. Common practice is to place beams and girders monolithically with the slab. For beam and girder construction where the members are of considerable depth, Hunter (1953) recommends placing concrete in the beam section up to the slab soffit, then placing the slab in a separate operation. The reasoning behind this is that cracking of the interface may result because of vertical shrinkage in a deep member if the beam and slab concrete are placed monolithically. With this procedure, there is a possibility that the two surfaces will slip due to horizontal shear in the member. ACI 318 requires that adequate shear transfer be provided.

The main concern in joint placement is to provide adequate shear transfer and flexural continuity through the joint. Flexural continuity is achieved by continuing the reinforcement through the joint with sufficient length past the joint to ensure an adequate splice length for the reinforcement. Shear transfer is provided by shear friction between the old and new concrete, or dowel action in the reinforcement through the joint. Shear keys are usually undesirable (Fintel 1974), since keyways are possible locations for spalling of the concrete. The bond between the old and new concrete, and the reinforcement crossing the joint, are adequate to provide the necessary shear transfer if proper concreting procedures are followed.

3.2.2.2 *Columns and walls*—Although placements with a depth of 30 ft (10 m) have been made with conventional formwork, it is general practice to limit concrete placements to a height of one story. Construction joints in columns and bearing walls should be located at the undersides of floor slabs and beams. Construction joints are provided at the top of floor slabs for columns continuing to the next floor; column capitals, haunches, drop panels, and brackets should be placed monolithically with the slab. Depending on the architecture of the structure, the construction joint may be used as an architectural detail, or located to blend in without being noticeable. Quality form construction is of the highest importance in providing the visual detail required (PCA 1982).

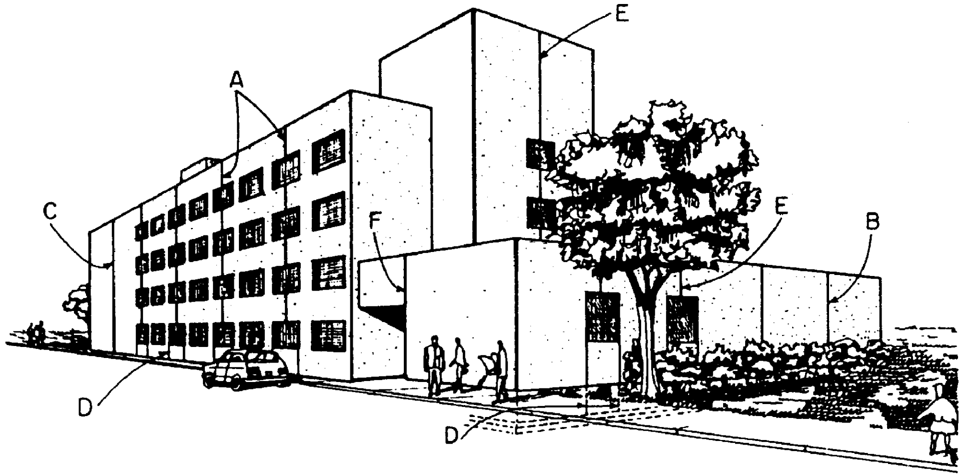
The placement of fresh concrete on a horizontal surface can affect structural integrity of the joint. Although it is not always necessary, common practice has been to provide a bedding layer of mortar, of the same proportions as that in the concrete, before placement of new concrete above the joint. ACI 311.1R recommends using a bedding layer of concrete with somewhat more cement, sand, and water than the design mix for the structure. Aggregate less than $^3/_4$ in. (20 mm) can be left in the bedding layer, but larger aggregate should be removed. This mixture should be placed 4 to 6 in. (100 to 150 mm) deep and vibrated thoroughly with the regular mixture placed above.

The concrete in the columns and walls should be allowed to stand for at least two hours before placement of subsequent floors. This will help to avoid settlement cracks in slabs and beams due to vertical shrinkage of previously placed columns and walls.

The location of vertical construction joints in walls needs to be compatible with the appearance of the structure. Construction joints are often located near re-entrant corners of walls, beside columns, or other locations where they become an architectural feature of the structure. If the building architecture does not dictate joint location, construction requirements govern. These include production capacity of the crew and requirements for reuse of formwork. These criteria will usually limit the maximum horizontal length to 40 ft (12 m) between joints in most buildings (PCA 1982). Because of the critical nature of building corners, it is best to avoid vertical construction joints at or near a corner, so that the corner will be tied together adequately.

Shear transfer and bending at joints in walls and columns should be addressed in much the same way it is for beams and slabs. The reinforcement should continue through the joint, with adequate length to ensure a complete splice. If the joint is subject to lateral shear, load transfer by shear friction or dowel action is added. Section 8.5 provides additional information on construction joints in walls.

3.2.3 *Summary*—Construction joints are necessary in most reinforced concrete construction. Due to their critical nature, they should be located by the designer, and indicated on the design drawings to ensure adequate force transfer and aesthetic acceptability at the joint. If concrete placement is stopped for longer than the initial setting time, the joint should be treated as a construction joint. Advance input is required from the designer on any additional requirements needed to ensure the structural integrity of the element being placed.



A. 20 ft (6m) apart in walls with frequent openings.
B. Never more than 20 ft (6m) apart, walls with no openings.
C. Within 10 to 15 ft (3 to 5m) of a corner, if possible.
D. In line with each jamb at first–story level.
E. Above first story at centerline of opening
F. Jamb lines are preferable.

Fig. 3.1—Locations for contraction joints in buildings as recommended by the Portland Cement Association (1982)

3.3—Contraction joints

Drying shrinkage and temperature drops cause tensile stress in concrete if the material is restrained. Cracks will occur when the tensile stress reaches the tensile strength of the concrete. Because of the relatively low tensile strength of concrete [$f_t' \sim 4.0\sqrt{f_c'}$] for normal weight concrete, f_c' and f_t' in psi (ACI 209R)], cracking is likely to occur. Contraction joints provide planes of weakness for cracks to form. With the use of architectural details, these joints can be located so that cracks will occur in less conspicuous locations. Sometimes they can be eliminated from view (Fig. 3.1). Contraction joints are used primarily in walls, addressed in this chapter, and in slabs-on-grade, discussed in Chapter 5.

For walls, restraint is provided by the foundation. Structural forces due to volume changes increase as the distance between contraction joints increases. To resist these forces and minimize the amount of crack opening in the concrete, reinforcement is increased as the distance between joints and the degree of restraint increases. Increased reinforcement generally results in more, but finer, cracks.

3.3.1 *Joint configuration*—Contraction joints consist of a region with a reduced concrete cross section and reduced reinforcement. The concrete cross section should be reduced by a minimum of 25 percent to ensure that the section is weak enough for a crack to form. In terms of reinforcement, there are two types of contraction joints now in use, "full" and "partial" contraction joints (ACI 350R). Full contraction joints, preferred for most building construction, are constructed with a complete break in reinforcement at the joint. Reinforcement is stopped about 2 in. (50 mm) from the joint and a bond breaker placed between successive placements at construction joints. A portion of the reinforcement passes through the joint in partial contraction joints. Partial contraction joints are also used in liquid containment structures and are discussed in more detail in Section 9.2. Waterstops can be used to ensure watertightness in full and partial contraction joints.

3.3.2 *Joint location*—Once the decision is made to use contraction joints, the question remains: What spacing is needed to limit the amount of cracking between the joints? Table 1.1 shows recommendations for contraction joint spacing. Recommended spacings vary from 15 to 30 ft (4.6 to 9.2 m) and from one to three times the wall height. The Portland Cement Association (1982) recommends that contraction joints be placed at openings in walls, as illustrated in Fig. 3.1. Sometimes this may not be possible.

Contraction and expansion joints within a structure should pass through the entire structure in one plane (Wood 1981). If the joints are not aligned, movement at a joint may induce cracking in an unjointed portion of the structure until the crack intercepts another joint.

3.4—Isolation or expansion joints

All buildings are restrained to some degree; this restraint will induce stresses with temperature changes. Temperature-induced stresses are proportional to the temperature change. Large temperature variations can result in substantial stresses to account for in design. Small temperature changes may result in negligible stresses.

Temperature-induced stresses are the direct result of volume changes between restrained points in a structure. An estimate of the elongation or contraction caused by temperature change is obtained by multiplying the coefficient of expansion of concrete α [about 5.5 x 10^{-6}/F (9.9 x 10^{-6}/C)] by the length of the structure and the temperature change. A 200-ft- (61-m-) long building subjected to a temperature increase of 25 F (14 C) would elongate about $^3/_8$ in. (10 mm) if unrestrained.

Expansion joints are used to limit member forces caused by thermally-induced volume changes. Expansion joints permit separate segments of a building to expand or contract without adversely affecting structural integrity or serviceability. Expansion joints also isolate building segments and provide relief from cracking because of contraction of the structure.

Joint width should be sufficient to prevent portions of the building on either side of the joint from coming in contact. The maximum expected temperature rise should be used in determining joint size. Joints vary in width from 1 to 6 in. (25 to 150 mm) or more, with 2 in. (50 mm) being typical. Wider joints are used to accommodate additional differential building movement that may be caused by settlement or seismic loading. Joints should pass through the entire structure above the level of the foundation. Expansion joints should be covered (Fig. 3.2) and may be empty or filled (Fig. 3.3). Filled joints are required for fire-rated structures.

Expansion joint spacing is dictated by the amount of movement that can be tolerated, and the permissible stresses or capacity of the members. As with contraction joints, rules of thumb have been developed (Table 1.2). These rules are generally quite conservative and range from 30 to 200 ft (9 to 60 m) depending on the type of structure. In practice, spacing of expansion joints is rarely less than 100 ft (30 m). As an alternative to the rules of thumb, analytical methods may be used to calculate expansion joint spacing. This section presents two of these methods (Martin and Acosta 1970, National Academy of Sciences 1974).

Pfeiffer and Darwin (1987) used those two procedures along with a third by Varyani and Radhaji (1978) to obtain expansion joint spacings for two reinforced concrete frames. Pfeiffer and Darwin include sample calculations and a discussion of the relative merits of the methods. The methods of Martin and Acosta and the National Academy of Sciences are not rational, but are easy to use and produce realistic joint spacings. The method of Varyani and Radhaji has a rational basis, but gives unrealistic results.

3.4.1 *Single-story buildings: Martin and Acosta*—Martin and Acosta (1970) presented a method for calculating the maximum spacing of expansion joints in one-story frames with nearly equal spans. The method assumes that with adequate joint spacing, the load factors for gravity loads will provide an adequate margin of safety for the effects of temperature change. Martin and Acosta developed a single expression for expansion joint spacing L_j in terms of the stiffness properties of a frame and the design temperature change ΔT. This expression was developed after studying

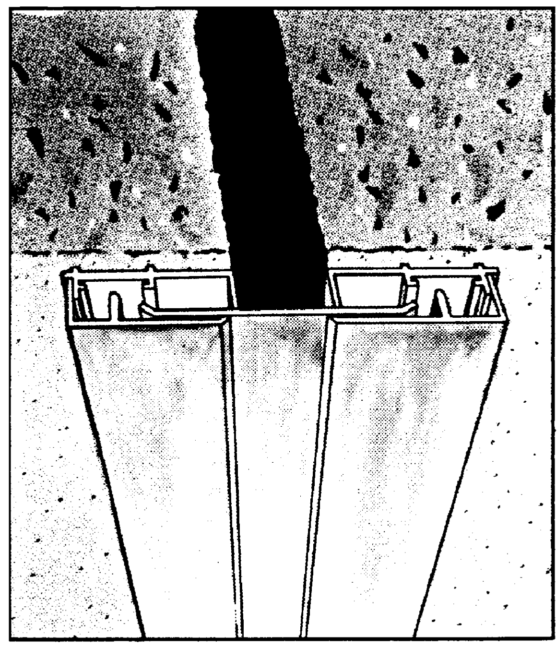


Fig. 3.2—Wall expansion joint cover (courtesy Architectural Art Mfg., Inc.)

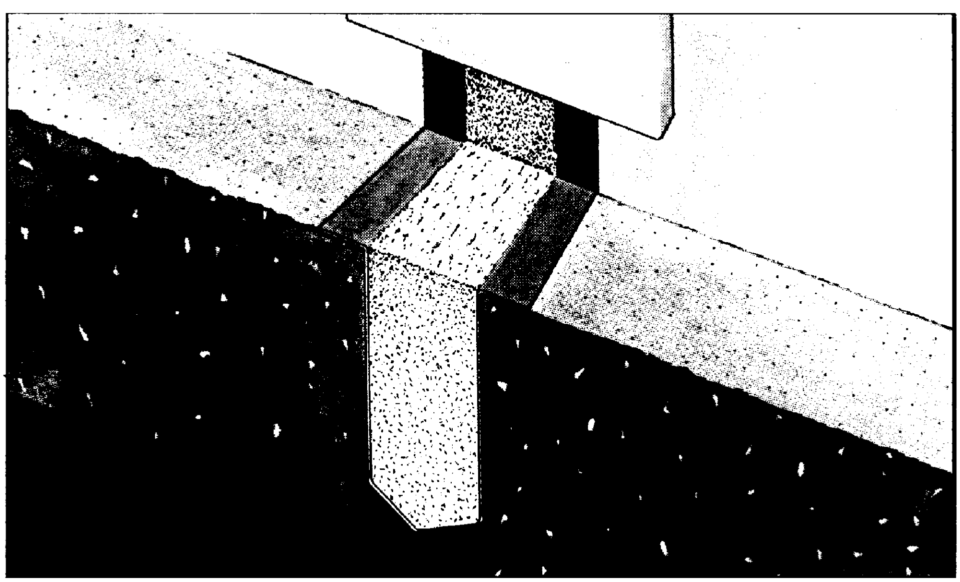


Fig. 3.3—Fire rated filled expansion joint (courtesy Architectural Art Mfg., Inc.)

Table 3.1—Maximum and minimum daily temperatures for selected locations (Martin and Acosta 1970)

Location	Normal daily temperature, F	
	Maximum	Minimum
Anchorage, AK	66.0	4.3
Atlanta, GA	87.0	37.1
Boston, MA	81.9	23.0
Chicago, IL	84.1	19.0
Dallas, TX	95.0	36.0
Denver, CO	88.4	14.8
Detroit, MI	84.7	19.1
Honolulu, HI	85.2	65.8
Jacksonville, FL	92.0	45.0
Los Angeles, CA	75.9	45.0
Miami, FL	89.7	57.9
Milwaukee, WI	78.9	12.8
New Orleans, LA	90.7	44.8
New York, NY	85.3	26.4
Phoenix, AZ	104.6	35.3
Pittsburgh, PA	83.3	20.7
San Francisco, CA	73.8	41.7
San Juan, PR	85.5	70.0
Seattle, WA	75.6	33.0
St. Louis, MO	89.2	23.5
Tulsa, OK	93.1	26.5

Note: C = 5/9 (F-32).

frame structures designed with ACI 318-63. The expansion joint spacing is

$$L_j = \frac{112,000}{R\Delta T} \text{ in., } \Delta T \text{ in F}$$

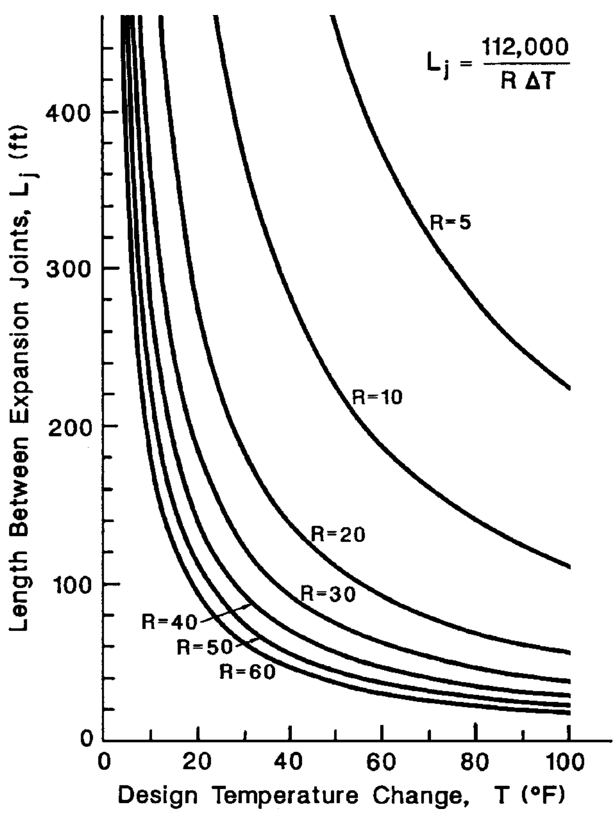


Fig. 3.4—Length between expansion joints versus design temperature change, ΔT *(Martin & Acosta 1970)*

or

$$L_j = \frac{12.24}{R\Delta T} \text{ m, } \Delta T \text{ in CM} \tag{3-1}$$

In the above expressions:

$$R = 144\frac{I_c}{h^2}\frac{(1+r)}{(1+2r)} \tag{3-2}$$

where:

r = ratio of stiffness factor of column to stiffness factor of beam = K_c/K_b;

$$\Delta T = \frac{2}{3}(T_{max} - T_{min}) + T_s \tag{3-3}$$

K_c = column stiffness factor = I_c/h, in.^{3} (m^3)
K_b = beam stiffness factor = I_b/L, in.^{3} (m^3)
h = column height, in. (m)
L = beam length, in. (m)
I_c = moment of inertia of the column, in.^{4} (m^4)
I_b = moment of inertia of the beam, in.^{4} (m^4)
T_s = 30 F (17 C)

Values for T_{max} and T_{min} can be obtained from the Environmental Data Service for a particular location (see Table 3.1 for a partial listing). The design temperature change ΔT is based on the difference between the extreme values of the normal daily maximum and minimum temperatures. An additional drop in temperature of about 30 F (17 C) is then added to account for drying shrinkage. Martin (1970) provides site-specific values of shrinkage-equivalent temperature drop. Because of the additional volume change due to drying shrinkage, joint spacing is governed by contraction instead of expansion. L_j from Eq. (3-1) is plotted in Fig. 3.4 for typical values of R.

Martin and Acosta proposed an additional criterion for L_j to limit the maximum allowable lateral deflection, δ to $h/180$ so as to avoid damage to exterior walls. The maximum lateral deflection imposed on a column is taken as

$$\delta = \frac{1}{2}\alpha L_j \Delta T \tag{3-4}$$

where α is the coefficient of linear expansion of concrete (about 5.5 x 10^{-6}/F or 9.9 x 10^{-6}/C).

Eq. (3-4) is based on the assumption that the lateral deflection of a floor system caused by a temperature change is not significantly restrained by the columns. This assumption is realistic since the in-plane stiffness of a floor system is generally much greater than the lateral stiffness of the supporting columns. Thus, the columns have little effect on δ.

This leads to the limitation on L_j of

$$L_j \le \frac{2000h}{\Delta T} \text{ ; } \Delta T \text{ in F}$$

or

$$L_j \le \frac{1111h}{\Delta T}\text{; }\Delta T\text{ in C} \qquad (3\text{-}5)$$

Martin and Acosta state that Eq. (3-1) yields conservative results (adequately low values of L_j) in these cases, but is very conservative for very rigid structures. Because of changes in ACI 318 since 1963, expansion joint spacings determined from Eq. (3-1) are somewhat lower than would be obtained had later versions of ACI 318 been used.

3.4.2 *Single and multi-story buildings: National Academy of Sciences criteria*—The lack of nationally recognized design procedures for locating expansion joints prompted the Federal Construction Council to develop more definitive criteria. The Council directed its Standing Committee on Structural Engineering (SCSE) to develop a procedure for expansion joint design to be used by federal agencies. The SCSE criteria were published by the National Academy of Sciences (1974).

As part of the SCSE investigation, the theoretical influence of temperature change on two-dimensional elastic frames was compared to the actual movements recorded during a one-year study by the Public Buildings Administration (1943-1944).

Prior to that time, most federal agencies relied on rules (Fig. 3.5) that provided maximum building dimensions for heated and unheated buildings as a function of the change in the exterior temperature. However, no significant quantitative data was found to support these criteria. The criteria illustrated in Fig. 3.5 reflect two assumptions. First, the maximum allowable building length between joints decreases as the maximum difference between the mean annual temperature and the maximum/minimum temperature increases. Second, the distance between joints can be increased for heated structures. Here, the severity of the outside temperature change is reduced through building temperature control. The lower and upper bounds of 200 and 600 ft (60 and 200 m) were a consensus, but have no experimental or theoretical justification.

An unpublished report by structural engineers of the Public Buildings Administration (1943-1944) documents the expansion joint movement in nine federal buildings over a period of one year. Based on this report, the SCSE drew a series of conclusions that were included in their design recommendations:

- A considerable time lag (2 to 12 hr) exists between the maximum dimensional change and the peak temperature associated with this change. This time lag is due to three factors: the temperature gradient between the outside and inside temperatures, the resistance to heat transfer because of insulation, and the duration of the ambient temperature at its extreme levels.
- The effective coefficient of thermal expansion of the first floor level is about one-third to two-thirds that of the upper floors. The dimensional changes in the upper levels of buildings correspond to a coefficient of thermal expansion between 2 and 5 x 10^{-6}/F (3.6 to 9 x 10^{-6}/C). The upper building levels undergo dimensional changes corresponding to the coefficient of thermal expansion of the primary construction material.

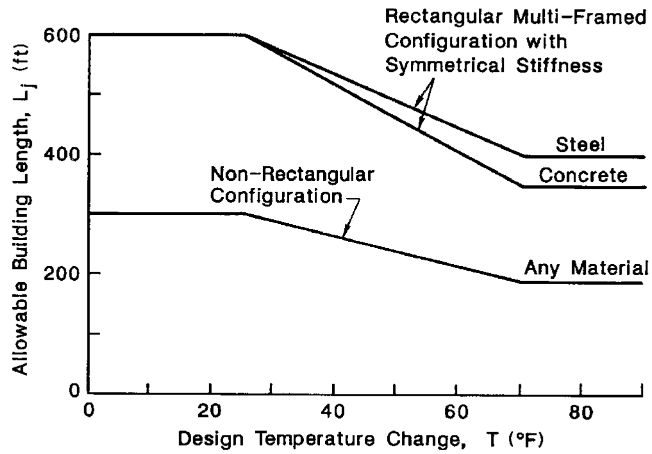


Fig. 3.5—Expansion joint criteria of one federal agency (National Academy of Sciences 1974)

The SCSE also analyzed typical two-dimensional frames subjected to uniform temperature changes. The conclusions of that analysis were:

- The intensity of the horizontal shear in first-story columns is greatest at the ends of the frame and approaches zero at the center. The beams near the center of a frame are subjected to maximum axial forces. Columns at the ends of a frame are subjected to maximum bending moments and shears at the beam-column joint.
- Shears, axial forces, and bending moments at critical sections within the lowest story are almost twice as high for fixed-column buildings compared to hinged-column buildings.
- The horizontal displacement of one side of the upper floors of a building is about equal to the assumed displacement that would occur in an unrestrained frame if both ends of the frame were equally free to displace about $^1/_2\, \alpha L_j \Delta T$ [Eq. (3-4)].
- The horizontal displacement of a frame that is restricted from side displacement at one end results in a total horizontal displacement of the other end of about $\alpha L_j \Delta T$.
- An increase in the relative cross-sectional area of the beams (without a simultaneous increase in the moment of inertia of the beams), results in a considerable increase in the controlling design forces. This occurs because the magnitude of the thermally induced force is proportional to the cross-sectional area of the element.
- Hinges placed at the top and bottom of exterior columns of a frame result in a reduction of the maximum stresses that develop. These hinges, however, allow an increase in the horizontal expansion of the first floor.

As a result, the SCSE developed Fig. 3.6. The SCSE rationalized that the step function shown in Fig. 3.5 could not represent the behavior of physical phenomena such as thermal effects. A linearly varying function for a 30 to 70 F (20 to

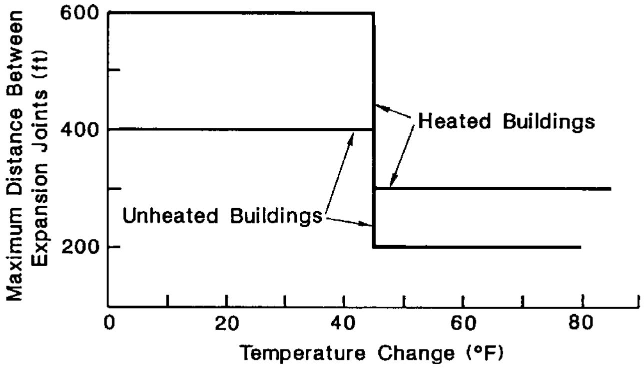


Fig. 3.6—Expansion joint criteria of the Federal Construction Council (National Academy of Sciences 1974)

40 C) temperature change was assumed. The upper and lower bounds are based on Fig. 3.5.

The relationships shown in Fig. 3.6 are directly applicable to beam-column frames with columns hinged at the base and heated interiors. Modifications that reflect building stiffness and configuration, heating and cooling, and the type of column connection to the foundation are provided. The graph is adaptable to a wide range of buildings.

To apply the method, the design temperature change ΔT is calculated for a specific site as the larger of

$$\Delta T = T_w - T_m$$

or

$$\Delta T = T_m - T_c \tag{3-6}$$

in which

T_m = temperature during the normal construction season in the locality of the building, assumed to be the continuous period in a year during which the minimum daily temperature equals or exceeds 32 F (0 C)

T_w = temperature exceeded, on average, only 1 percent of the time during the summer months of June through September

T_c = temperature equaled or exceeded, on average, 99 percent of the time during the winter months of December, January, and February

Values for T_m, T_w, and T_c for selected locations throughout the United States are given in Appendix A. The temperature data are taken from the SCSE report (National Academy of Sciences 1974). The information also can be derived from information now available in ASHRAE (1981).

As stated above, the limits prescribed in Fig. 3.6 are directly applicable to buildings of beam-column construction (including structures with interior shear walls or perimeter base walls), hinged at the foundation, and heated. For other conditions, the following modifications should be applied to the joint spacings obtained from Fig. 3.6.

- If the building will be heated, but not air-conditioned, and has hinged column bases, use the length specified.
- If the building will be heated and air-conditioned, increase the allowable length by 15 percent.
- If the building will not be heated, decrease the allowable length by 33 percent.
- If the building will have fixed column bases, decrease the allowable length by 15 percent.
- If the building will have substantially greater stiffness against lateral displacement at one end of the structure, decrease the allowable length by 25 percent.

When more than one of these conditions occur, the total modification factor is the algebraic sum of the individual adjustment factors that apply.

The SCSE did not recommend this procedure for all situations. For a unique structure or when the empirical approach provides a solution that professional judgement suggests is too conservative, they recommended a more detailed analysis. This analysis should recognize the amount of lateral deformation that can be tolerated. The structure should then be designed so that this limit is not exceeded.

CHAPTER 4—BRIDGES

4.1—Introduction

Joints are used in bridges for two reasons. The primary reason is to accommodate movements caused by thermal expansion and contraction. Movements of 4 in. (100 mm) or greater can be expected in longer span bridges. The secondary reason is for construction purposes. Here, joints serve as a convenient separation between previously placed concrete and fresh concrete.

Transverse construction joints may be coincident with expansion joints, particularly for shorter span bridges. However, often construction joints are not coincident with expansion joints. Construction joints are provided between the deck and the base of parapets. Longitudinal joints may be used when bridges exceed a width that can be placed with common type construction equipment. Transverse construction joints are used when the volume of concrete deck to be placed is too great. Construction joints are also necessary in the webs of concrete box girders and around embedded items such as large expansion joints.

The two major classifications of expansion joints in bridges are open joints and sealed joints. The popularity of watertight or sealed joints is growing although they have been in use since the 1930s. There are many more open than sealed expansion joints in service. However, it is now quite common to specify at least one proprietary type of sealed expansion joint system for new construction or rehabilitation projects.

There has been a recent trend to design bridges without intermediate transverse joints in the decks except for construction joints (Loveall 1985). The structure is designed to accommodate the movements induced by temperature changes. This trend toward jointless bridge designs has developed because of poor expansion joint behavior and structural deterioration caused by leaking and frozen joints. The result of poor joint performance has been costly maintenance and frequent replacement of joints. The extremities of a jointless bridge will have large movements that must be accommodated.

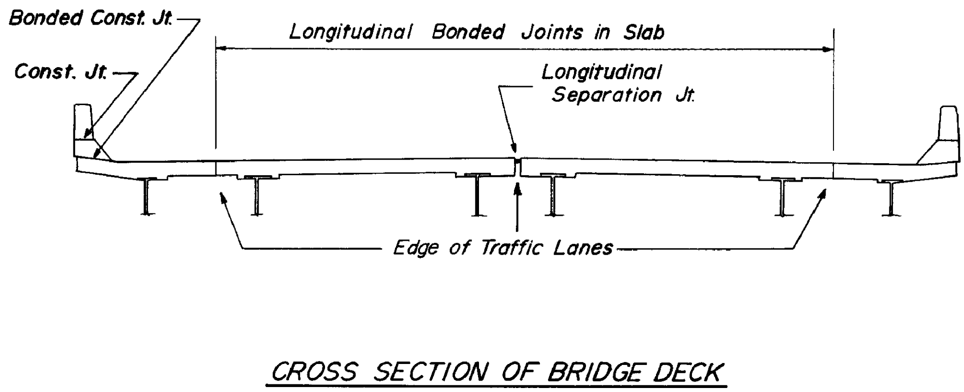


Fig. 4.1—Types of joints in bridge decks

This Chapter discusses the types of joints in bridges and provides general guidance for their use. Bridges without intermediate expansion joints are discussed to identify the relative advantages and disadvantages of this type of structure, compared to conventional bridge structures with joints. Joints in segmental bridges are not covered specifically.

4.2—Construction joints

The use of construction joints in a bridge deck such as those seen in Fig. 4.1 are inevitable. Construction joints may be required in the parapet, sidewalk, and bridge deck. In the bridge deck slab, transverse and longitudinal construction joints may be required.

Longitudinal construction joints as seen in Fig. 4.1 may be used, but only at certain locations. These joints are normally placed towards the outside and, when possible, should line up with the edges of the approach pavements. These joints should not be located inside the outer edges of the approach pavement except on extremely wide decks where the longitudinal bonded construction joint is at the edge of an intermediate traffic lane. In addition, a longitudinal bonded construction joint should not cross a beam line. Special consideration should be given to placement of the longitudinal slab reinforcement in relation to a longitudinal construction joint.

When the width of the bridge deck is very wide [greater than 90 ft (27.4 m)], the deck may need to be split by means of an open joint as seen in Fig. 4.1. This joint is typically sealed with an epoxy sealant and rubber rod.

Transverse construction joints are used when the volume of concrete is too great to conveniently cast and finish. In this case, concrete is first placed in the positive moment regions. Then after several days, concrete is cast in the negative moment areas. A transverse construction joint should be placed near the point of dead load contraflexure with a given day's concrete casting terminating at the end of the positive moment region.

4.3—Bridges with expansion joints

Bridge expansion joints are designed to accommodate superstructure movements and carry high impact wheel loads while being exposed to prevailing weather conditions. Expansion joints are contaminated with water, dirt, and debris that collect on the roadway surface and in many localities are also subjected to deicing salts that can lead to corrosion.

The primary purpose of joints in bridge decks is to accommodate horizontal movements generally caused by temperature changes, and those caused by end rotations at simple supports. Thermal movements can be several inches (hundreds of millimeters) for longer span bridges. Joints are also provided to accommodate shortening due to prestress. Safety considerations in ensuring vehicle tires do not drop into the joint, particularly when a joint is skewed, dictate a practical limit of about 4 in. (100 mm). For expected movements greater than 4 in. (100 mm), additional joints may be required. However, there have been joint systems designed to accommodate as much as 26 in. (660 mm) of movement at a single joint (*Better Roads* 1986).

Until the mid-1970s, it was common practice to accommodate movements between 2 and 4 in. (50 and 100 mm) with the use of open joints. However, experience has shown that open joints often lead to deterioration of the structure beneath the openings. Runoff from top deck surfaces mixes with deicing salt and forms an aggressive brine solution. This can lead to steel corrosion in areas that are difficult to inspect and maintain. With time, the aggressive salt solution penetrates concrete surfaces of supporting girders, piers, and abutments that eventually lead to severe deterioration. The use of open joints in a bridge deck requires a dedicated maintenance program to remove debris on a regular basis that could prevent deck movement, to clean and paint steel surfaces that have rusted, and to repair deteriorated concrete.

Because of shortcomings with an open joint bridge deck design, current practice leans toward watertight expansion devices. Sealed deck joints assume that it is easier to dispose of deck drainage beyond the abutments, or with scuppers, than underneath open joints.

4.3.1 *Open joints*—The use of open joints, assuming a dedicated maintenance program, may be the economic choice for some bridges, particularly in southern states. Open joints in decks are located where moments are negligible. For simple span bridge structures, this is generally at locations of abutments and piers.

Open joints are generally designed for maximum movements of 4 in. (100 mm) or less. An open joint is formed by placing a suitable material in the deck before concrete is cast,

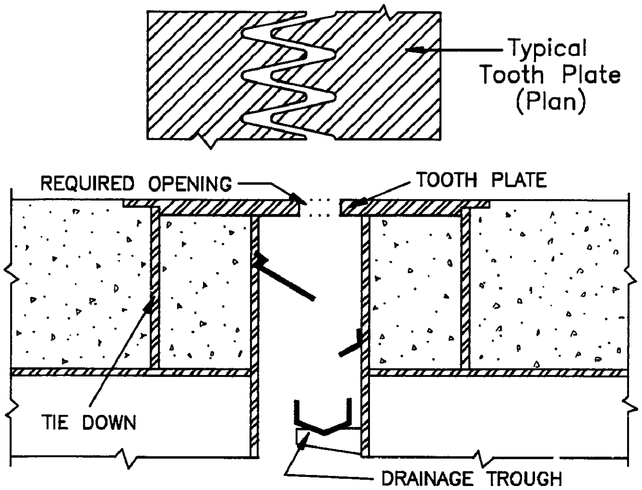


*Fig. 4.2—Open finger joint with drainage trough (*Better Roads *1986a)*

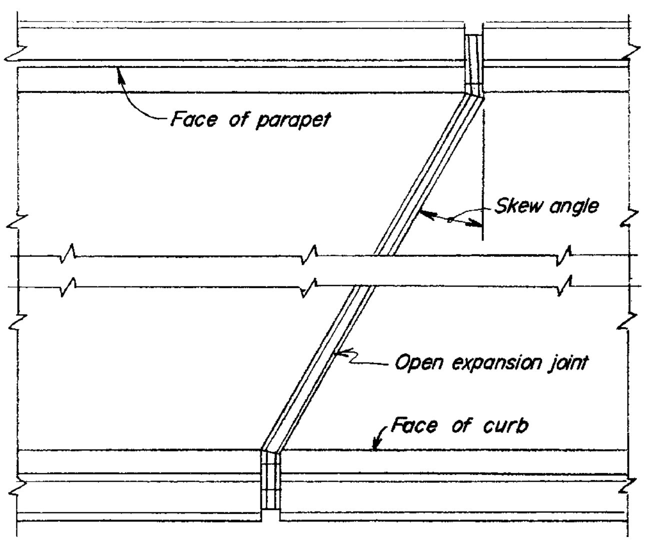


Fig. 4.3—Open expansion joint in a skew bridge

and then removing the material after the concrete hardens. To avoid damage from vehicular impact loads, deck edges on each side of an open joint are often protected by sliding steel plates or steel fingers.

Joints that use a premolded neoprene compression seal are used at locations where no movement is desired, such as at a construction joint, or when less than 1 in. (25 mm) of movement is anticipated. The placement and behavior of compression seals is enhanced if the joint is armored with steel angles and the seal is installed with a lubricant adhesive. If an open joint is desired, but substructure deterioration is of concern, a supplementary device such as a drainage trough (as shown in the steel finger joint of Fig. 4.2) is used to carry away runoff passing through the deck.

To adjust for the expected movement in a bridge deck when the structure is skewed, it is common practice to increase the calculated joint movement for an equal length non-skew bridge. The expansion device is oversized to account for racking. Thus, a 45-deg skew bridge would have more expected total joint movement than an equal span 15-deg skew bridge or a nonskew bridge. An approximation for the total movement is estimated by calculating the movement for a nonskew bridge of equal span length and dividing by the cosine of the skew angle. An example of the layout of an open joint at an abutment in a skew bridge is shown in Fig. 4.3.

Table 4.1—Definitions — Joint sealing systems (NCHRP 204 1979)

Term	Description
Joint seal/gland	Device or part of a device spanning gap of an open deck joint.
Rubber/neoprene	Any elastomer of natural or synthetic rubber used in fabrication of joint assemblies.
Gland/trough	Seal constructed as a thin pad of rubber/neoprene [about $^1/_8$ in. (5 mm) thick], generally bent or U-shaped in the central unsupported portion of joint and flat or knob-formed along winged edges, depending on manner of anchorage.
Cushion/pad	Seal, retainer, or portion of an assembly constructed as a thick rubber/neoprene pad, typically $1^1/_2$ to $2^1/_2$ in. (30 to 40 mm) thick.
Retainer/extrusion	Device on each side of joint gap that grips knob-formed edges of gland seals. The winged flat edges of gland seals clamp to the deck by bolted anchorages (edges of thick rubber/neoprene material manufactured monolithically with a thin gland. These are not considered as retainers, but are part of the joint seal design.).
Blockout	Formed recess in the ends of the concrete decks that receive the joint-sealing assembly. Certain kinds of retainers/extrusions can be cast into final position before deck slab construction and therefore do not require a blockout.
Armor	Steel plates or angles used to provide a uniform opening for rubber/neoprene compression seals and protect the edge of the concrete.
Seat	Horizontal surface of a blockout.
Shoulder	Vertical surface of a blockout.

More specific requirements for open joints and joints filled with caulking materials are provided in Section 23 of the AASHTO Specifications.

4.3.2 *Sealed joints*—Sealed joints are used in bridge decks when bridge substructure deterioration is particularly likely because of aggressive environmental conditions. Although watertight joints are initially more costly than open joints, less maintenance is required. Another functional objective of an expansion joint seal is to prevent the accumulation of debris within the joint and keep the joint moving freely. Many proprietary watertight expansion devices are designed to accommodate debris or are flush with the deck surface to inhibit debris accumulation.

Joint-sealing terminology is provided in Table 4.1. (NCHRP 204 1979). Some watertight seals consist of a thin collapsible rubber neoprene membrane or part of a thick cushion or pad. Thin membrane seals are often reinforced with several plies of fabric. Thick cushion seals are often reinforced by thin metal plates or loose metal rods free to move within the cushion.

There are various types of watertight expansion sealing systems that have evolved over the years. These include systems composed of neoprene troughs or glands, sliding plates with elastomeric compounds poured in, armored joints with compression seals, foam strips and others. However, most expansion devices can be placed in one of three categories: compression seals, strip seals, and steel reinforced modular seals. There are many joint-sealing systems available, some of which are proprietary. Fig. 4.4 illustrates some major classifications of watertight joint-sealing systems.

Neoprene strip seal glands [see Fig. 4.4(c)] are generally supplied as one continuous strip for the entire length of the deck joint. Strip seals that are made monolithic with thick rubber cushion pads are supplied in only certain specified lengths. Rubber cushions and all retainers, whether rubber or metal, are supplied in discrete size sections and spliced together either in the shop or field. Rubber pads and steel extrusion retainers are generally produced in segment lengths from 4 to 6 ft (1.2 to 1.8 m) and 12 to 20 ft (3.6 to 6 m), respectively. Segment splicing should be done by butting the ends together with an adhesive. Metal retainer seals are joined by welding.

Blockouts and shoulders for joint-sealing systems are sometimes formed by metalwork cast in the deck to ensure plane surfaces and accurate tolerances for the seal. However, more often than not, blockout and shoulder surfaces are formed without benefit of embedded armor. Armor is recommended on new structures detailed with compression seals.

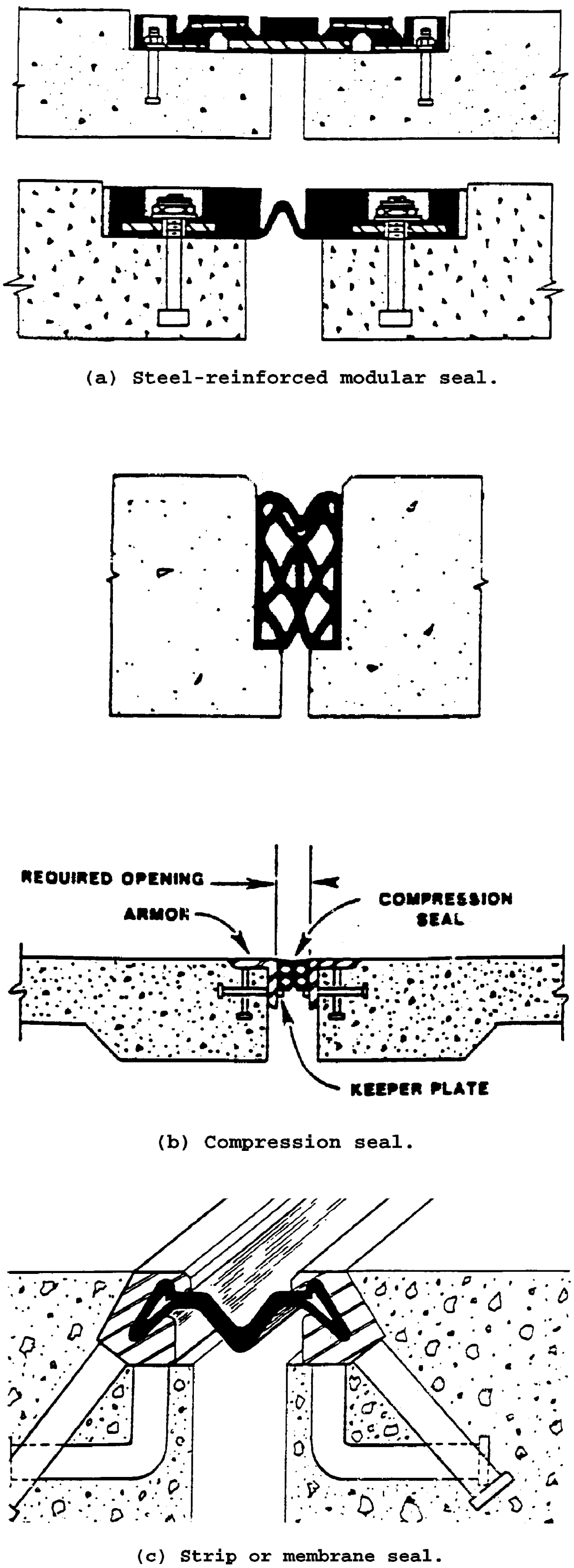
Many techniques are used to secure the edges of the sealing device or retainer to the deck. Common methods include long anchor bolts cast in the concrete slab and projecting above the blockout seat, and bolt studs or sloped reinforcing bars welded to metal retainers or armor angles in the seat.

Strip seal systems [Fig. 4.4(c)] are classified as low-stress systems because there is generally only a small amount of flexure and compression in the membrane when installed. Later superstructure movements cause very little stress, except in cases where the joint is severely skewed. Extreme contraction of the joint may produce some tension in the membrane. The glands can be replaced at a nominal cost if they are punctured or pushed out.

In contrast to strip seal systems [Fig. 4.4(c)], steel reinforced modular seals as shown in Fig. 4.4(a) generally are in a moderate state of stress. At the midpoint of the temperature range for which a steel reinforced modular system has been designed, no strain theoretically exists in the seal. However, at all other temperatures, a moderate amount of compression or tension in the joint assembly exists because of movements in the superstructure. Installation of this type of system is preferred at the midpoint temperature, since no artificial compressing or stretching is required. However, this is not always possible.

Compression seal systems [Fig. 4.4(b)] are generally only effective when the seal is in compression. Consequently, it is imperative that the maximum expected joint opening be accurately determined so that the appropriate width compression seal be installed to ensure residual compression at this expected joint opening. The compression seal is preferably installed at the lower end of the expected temperature range when the joint opening is the greatest. However, it is possible to install a compression seal at higher temperatures when the joint opening is smaller by following proper procedures for installation of a precompressed seal.

In situations where the expected superstructure movement is $^1/_2$ in. (13 mm) or less, a joint may be filled by a sealant instead of using a compression or cushion type seal (California



(a) Steel-reinforced modular seal.

(b) Compression seal.

(c) Strip or membrane seal.

*Fig. 4.4—Joint sealing systems (*Better Roads *1986a)*

DOT 1984). A sealed joint of this type consists of a groove in the concrete that is filled with a watertight, field-mixed and placed polyurethane sealant. In this case, the joint is generally formed by cutting a groove within $^{1}/_{8}$ in. (3 mm) of the expected movement and with a bottom width within 1/16 in. (1.5 mm) of the desired top width (California DOT 1984). Both sides of the groove should be cut simultaneously with a minimum first pass depth of 2 in. (50 mm). A primer is applied to the sides of the joint before placement of the sealant to ensure good bond.

For small joint movements, compression and cushion-type seals may also be used. Economics may dictate the use of pourable sealants, but considerations of maintenance, life, and durability may dictate the more expensive compression or cushion-type seals.

4.3.3 *Good practices in expansion joint design*—One of the most common problems with expansion joints is failure of the anchoring system, whether it be bolts or epoxy (Shanafelt 1985). The sudden, heavy, and repetitive nature of the loading causes high localized stresses on connections. The locations of the connections and concrete integrity adjacent to the anchorage system are important.

The expansion device capacity should always be greater than the calculated or expected thermal movement. The result of prestress shortening must be considered when determining the size of joints. The joint assembly should be designed to carry wheel loads with no appreciable deflection. Steel armoring should also be provided to protect the edges of concrete at the joint system/concrete interface. Anchors should be placed within the deck reinforcement to minimize any looseness or "working" of the anchorage system. Top anchor studs should be located no higher than 3 in. (75 mm) from the top deck surface.

For a joint to be watertight, the seal should be continuous across the entire deck surface. Moreover, the contact surfaces between the expansion device and adjoining concrete also must be watertight. Fabrication and installation require the highest quality-control procedures to ensure a watertight expansion joint. When open joints are used, substructure concrete should be protected by epoxy coatings or chemical sealers. Usually, open joints are no longer recommended.

In sealed systems, the rubber or neoprene material used should not be directly affected by wheel loads. Additionally, the design should minimize the accumulation of debris that can damage the seal and inhibit movement. One important design aspect is to insure that no parts of the expansion joint protrude above the deck surface where they can be damaged by snowplows.

Expansion joints should be designed for minimum maintenance. To limit maintenance, joints should have a life expectancy at least equal to that of the deck. It should be possible to replace individual seals without removing supporting elements of the expansion joint, if damage results from vehicles or snow plowing.

4.4—Bridges without expansion joints

In recent years, there has been a movement toward limiting expansion joints in bridge structures. Joints are only detailed if a structure is very long, and then only at abutments. The reasons for this trend are that joints can be costly to purchase and install, and expensive to maintain. Joints may allow water and deicing salt to leak onto the superstructure, pier caps, and foundations below, resulting in structural deterioration. Elimination of joints in the superstructure deck may be the only choice in some structural bridge systems such as cable-stayed bridges.

The "no-joint" approach became more feasible with the development of computers and structural analysis programs to carry out laborious calculations necessary for continuous bridge design. Elimination of joints may be accomplished by designing for continuity and taking advantage of the flexibility of the structural system. Precast girder bridges should be designed to be continuous for live load to reduce the number of joints in the bridge. Many precast girder bridges have been constructed with up to 500 ft (150 m) between expansion joints (Loveall 1985, Shanafelt 1985).

Many state highway departments routinely design bridges in both steel and concrete with joints only at the abutments (Wolde-Tinsae, et al. 1988) In Tennessee, the longest bridge without intermediate joints is a 2650-ft (795-m), dual 29-span prestressed concrete composite deck box-beam bridge designed to be continuous for live load (*Concrete Today* 1986). It is important to note that Tennessee has a moderate temperature range. The design of longer bridge structures without intermediate expansion joints is achieved more easily than in colder climates.

As a general rule, bridges should be continuous from end to end. There should be no intermediate joints introduced in the bridge deck other than construction joints. This applies to both longitudinal and transverse joints.

Jointless bridges should be designed to accommodate the movements and stresses caused by thermal expansion and contraction. These movements should not be accommodated by unnecessary bridge deck expansion joints and expansion bearings. This solution creates more problems than it solves. Structural deterioration due to leaking expansion joints and frozen expansion bearings leads to major bridge maintenance problems. To eliminate these problems, design and construct bridges with continuous superstructures, with fixed and integral connections to substructures, and no bridge deck expansion joints unless absolutely necessary. When expansion joints are necessary, they should only be provided at abutments. This philosophy is a good policy as long as the temperature-induced deformations are accommodated.

The Federal Highway Administration (FHWA 1980) recommended the following limits on length of integral abutment, no-joint bridges:

Steel: 300 ft (91.4 m)
Cast-in-place concrete: 500 ft (152.4 m)
Prestressed concrete: 600 ft (182.9 m)

However, FHWA further states that these lengths may be

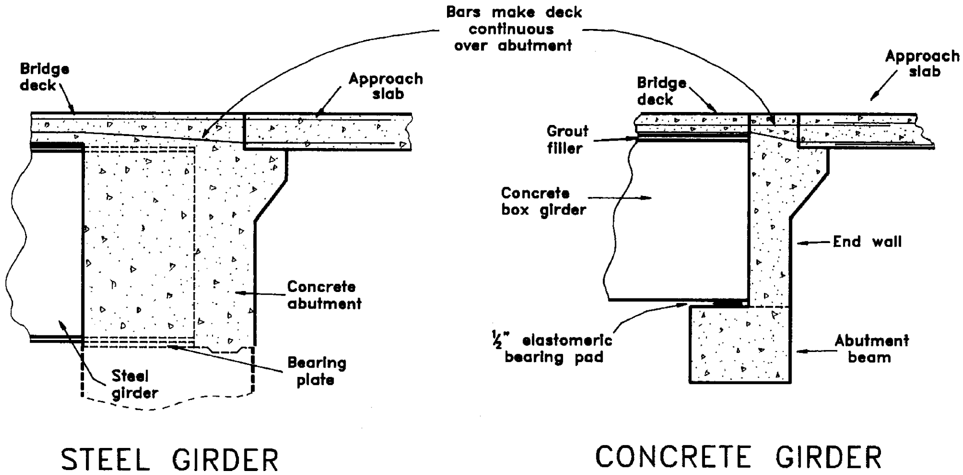


Fig. 4.5—Typical abutment/deck details for bridge deck without joints (Loveall,1985)

increased based on successful past experience. These recommendations have been exceeded by some highway agencies, notably Tennessee and Missouri (Wolde-Tinsae, et al. 1988).

Drainage is an important consideration when no joints are used, especially at the abutments. This is particularly critical when large thermal movements are expected. Washouts can occur with drainage flowing over an abutment paving notch or between the shoulder and the wingwall.

Special attention should be given to the abutment in order to design a bridge without joints. This requires knowledge of the total expected movement of the superstructure over a specified temperature range, and the Tennessee DOT designs concrete bridges for a temperature range from 20 to 90 F (-5 to +30 C). Steel superstructure bridges are designed for a temperature range from 0 to 120 F (-20 to +50 C) (Loveall 1985).

For the indicated temperature ranges and expansion coefficients of 6.0 x 10^{-6}/F for concrete and 6.5 x 10^{-6}/F for steel (10.8 x 10^{-6}/C and 11.7 x 10^{-6}/C, respectively), the expected thermal movement is about $^1/_2$ in. per 100 ft (40 mm per 100 m) of span for concrete and 1 in. per 100 ft (80 mm per 100 m) of span for steel. A concrete bridge 400 ft (120 m) long or a steel superstructure bridge 200 ft (60 m) in length must accommodate about 2 in. (50 mm) of thermal movement. If no joints are included in the deck at the abutments, as shown in Fig. 4.5, then the abutments must be designed to be flexible enough to accommodate this movement. Abutments with details such as shown in Fig. 4.6 are required. If this type abutment detail is not provided, larger thermal cracks can be expected in the deck.

If piers are not designed to be flexible enough and movement is restrained, destructive forces may occur in bridge components. The forces developed by restraint from stiff piers can cause damaging bridge movement, jamming of expansion joints at abutments, displacement of bearings, shearing of anchor bolts, damage to pier caps and piles, damage to rail and curb sections, damage to abutments, and possible damage to girders and stringers. Bridge repair will be significantly reduced by ensuring flexibility and ample bridge movement.

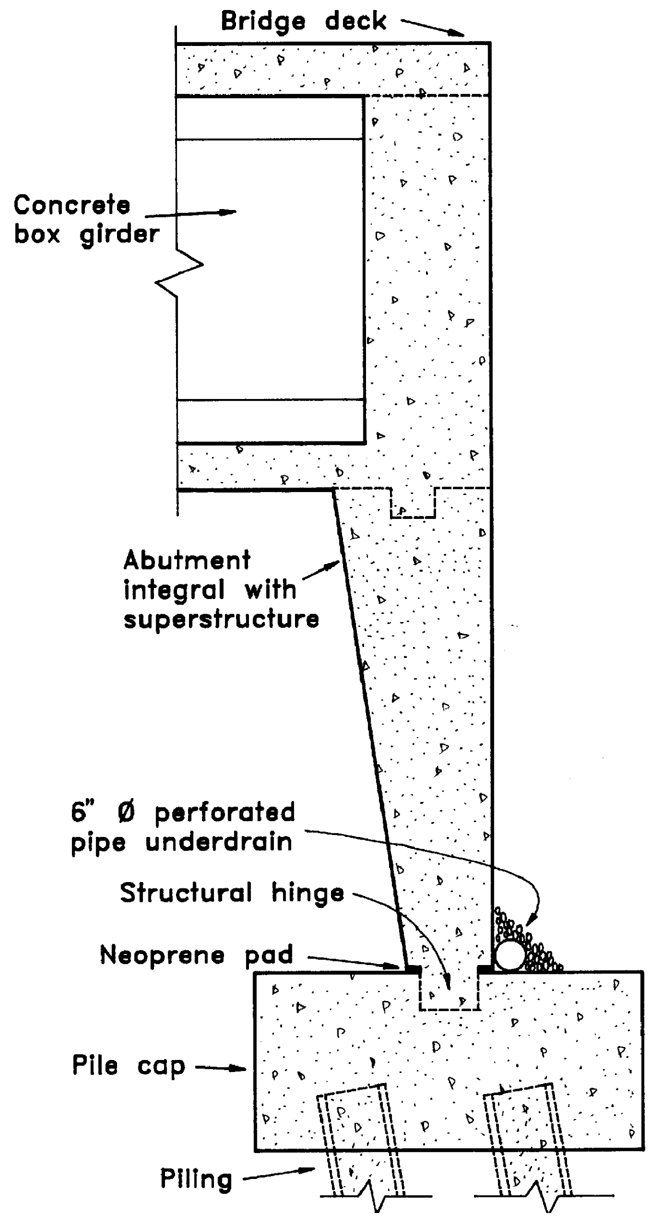


Fig. 4.6—Typical abutment hinge detail for bridge with no joints

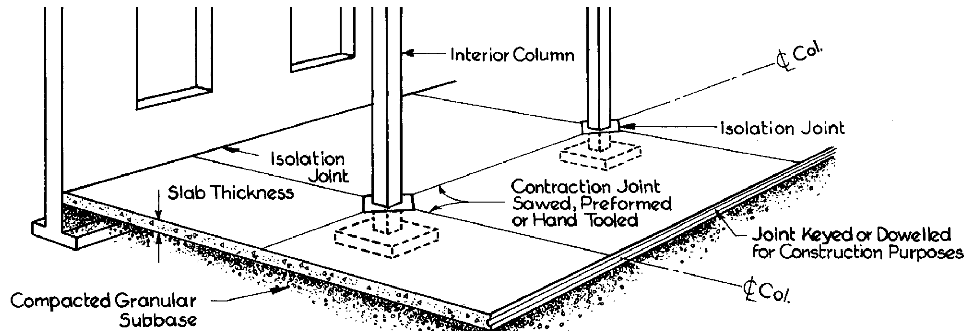


Fig. 5.1—Location and types of joints (ACI 302.1R)

CHAPTER 5—SLABS ON GRADE

5.1—Introduction

Joints in concrete slabs on grade are constructed to allow the concrete slab to move slightly, and, to a degree, provide a crack-free appearance for the slab. Slab movements are caused primarily by

- Shrinkage of the concrete, a volume change due drying
- Temperature changes
- Direct or flexural stress from applied loads
- Settlement of the slab

If movement is restrained, the slab will crack when the tensile strength of the concrete is exceeded. These cracks may appear anytime and at any location. Joints are needed so that cracks are more likely to form at preselected locations.

The slab on grade with least cost of initial construction is unreinforced with relatively closely spaced joints. Unreinforced concrete may not always be the most economical if the required slab thickness is large. Joint construction and joint maintenance increase cost. The relationship between recurring costs and the cost of initial construction, including slab reinforcement, use of shrinkage-compensating concrete, post-tensioning, and special use considerations of the finished slab, can be considered.

Typical joints and locations are illustrated in Fig. 5.1, and discussed in the following sections. This chapter describes applications related primarily to building construction. ACI 360R provides additional information. Chapter 6 discusses pavements.

5.2—Contraction joints

5.2.1 *General*—Contraction joints should be provided in the slab to accommodate shrinkage and to relieve internal stresses. A concrete slab on grade does not dry uniformly throughout its thickness, since environmental conditions are different at the top and bottom surfaces. The top portion of the slab will dry faster than the bottom and, as a result, the slab will warp at the edges. Similar effects result from temperature changes. The amount of deformation can be controlled with contraction joint spacing. Deformation also can be controlled, or at least reduced, by the use of dowelled joints, properly distributed reinforcement, and thickened slab edges. Joints that are properly placed and constructed should reduce random cracking. Preplanned contraction joints are also easier to seal and maintain than random cracks.

Contraction joints should be provided in slabs on grade at areas where differences in subgrade and slab support may cause cracks, such as above large underground utility trenches.

5.2.2 *Joint layout and spacing*—It is common practice to locate contraction joints along column lines, but usually additional joints are needed. Joints should be spaced so that the slab on grade is divided into small rectangular areas. Squares are preferred, but the slab geometry may dictate otherwise. As a general rule, ratios of the long to short side should not exceed 1.25 to 1.5. ACI 302.1R states that cracking may become excessive for ratios greater than 1.5. However, some feel that this is too great, based on observations of field performance. Odd shapes should be avoided, but if they cannot be avoided, re-entrant corners should be reinforced to limit the cracking at these locations.

ACI 302 recommends that contraction joints be provided at 24 to 36 times the slab thickness in both directions, unless intermediate cracks are acceptable. PCA (1983) recommended adjustments of the multiplier, depending on the likely shrinkage, as represented by the amount of mix water in the concrete and the aggregate size. For relatively high-slump concrete with the maximum aggregate size less than $^3/_4$ in. (20 mm), spacings should be at the low end of the range. Greater spacings can be used for low-slump concrete with larger aggregate. These recommendations are for normal construction practices, typical concrete mix proportions, and average concrete properties. Detailed analysis and local or specific materials may justify much larger or smaller joint spacings.

5.2.3 *Types of joints*—Contraction joints can be formed by means described in Chapter 2. Fig. 5.2 shows a variety of contraction joints.

5.2.3.1 *Sawed joints*—One of the most common methods of making contraction joints in slabs on grade is saw cutting the hardened concrete. The joints are usually sawed in the sequence as the slab was cast (ACI 302.1R). However, hot weather, winds or other special conditions affecting shrinkage may dictate the sequence of sawing.

5.2.3.2 *Hand-tooled or preformed joints*—Other methods of forming contraction joints are by hand-tooling to the required depth, or by inserting plastic or hardboard strips into the concrete before finishing. When floor slabs are thick, such that the insertion of a preformed strip or hand-tooling is cumbersome, a premolded insert can be placed on the bottom of the slab. The combined depth of the top and bottom inserts should still exceed $^1/_4$ the slab depth.

In cases where load transfer by a keyed joint is planned, a full-depth premolded joint can be placed in the slab. This is usually required if the movement between segments exceeds that recommended for adequate load transfer through aggregate interlock.

5.2.4 *Load transfer*—Because contraction joints subdivide the entire slab into smaller slabs, it is expected that the contraction joint should be capable of transferring vertical loads from one segment to the other. Load transfer is accomplished through aggregate interlock, through a preformed key, or by the use of a dowelled joint.

5.2.4.1 *Aggregate interlock*—The effectiveness of aggregate interlock in transferring load depends on several factors such as crack width, the presence of reinforcing extending across the crack, slab thickness, loading conditions, aggregate shape, and subgrade support. Crack widths should be less than 0.035 in. (0.9 mm) for good load transfer and durability. PCA (1992a) recommends that joint spacing not exceed 15 ft (4.5 m) when load transfer depends on aggregate interlock.

The magnitude and type of load is important in considering the effectiveness of aggregate interlock in load transfer. Repeated loads may cause fracturing of the aggregate, and eventual loss of load transfer effectiveness. Light loads of 5000 lb (20 kN) or less have been found to cause little or no joint deterioration.

Subgrade support is very important in contraction joint effectiveness. Soils such as some silts and clays have low support values, and repetitive loadings will cause a loss of aggregate interlock faster than for slabs supported on sandy soils.

Crushed aggregate is more effective in transferring load than natural gravel, and coarse aggregate is more effective than fine aggregate.

5.2.4.2 *Keyed joints*—Load transfer also can be accomplished by the use of a keyed contraction joint. This joint can be formed by the insertion of a full-depth preformed key at the time of concrete placement. A keyed contraction joint is formed by the use of keyed bulkheads so that the slab will have a tongue-and-groove joint once the concrete has been cast on both sides of the joint. The keyway can be formed by beveled wood strips, with a premolded key, or by preformed metal forms. ACI 302.1R provides typical details for keys and recommends that keyed contraction joints not be used for slabs less than 6 in. (150 mm) thick. Contraction joints are usually sawcut or edged after the concrete is cast. This allows sealing of the joint and provides a better appearance. Keyed contraction joints permit horizontal movement and transfer of vertical loads. Due to the bevel of the joint, load transfer is dependent on relatively small movements at the joint. The joint strength and load transfer requirements should be checked, accounting for the effects of the joint opening.

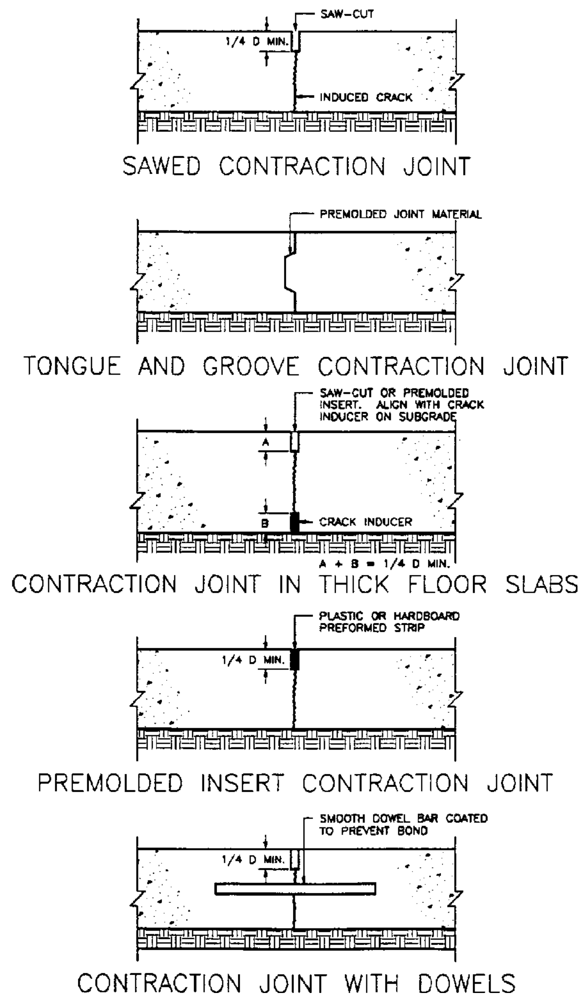


Fig. 5.2—Contraction joint types (ACI 302.1R)

5.2.4.3 *Dowelled joints*—For heavily loaded slabs with a high percentage of reinforcement for loads and crack control, contraction joints may be opened up too much for adequate load transfer through aggregate interlock. Load transfer at these joints can be accomplished with dowels. A combination of shear and bending action by the dowels will allow for load transfer between slabs. If the joint is not formed full depth, the joint should still be made on the top surface. In order to function properly, the dowels should be level and parallel to one another, and parallel to the length of the slab. The dowels should be centered on the joint. To permit horizontal movement, the dowels must not bond to the concrete on at least one side of the joint. Only smooth bars should be used. Bonding can be prevented by coating or greasing the dowels or by wrapping the dowels with a bond-breaking plastic. When placed at an expansion joint, an expansion cap is needed at dowel ends.

Fig. 5.3 shows a prefabricated dowel assembly. Its preassembly and rigid nature make alignment and positioning easier than when individual dowels are used. Table 5.1 shows dowel spacings recommended in ACI 302.1R.

5.2.4.4 *Joint sealing*—Sawed or formed joints in slabs may be sealed to improve joint performance. Sealed joints will also prevent water from entering the joint and causing

Table 5.1—Dowels for floor slabs (ACI 302.1R)

Slab thickness		Dowel diameter		Total dowel length*	
in.	mm	in.	mm	in.	mm
5	(125)	$^3/_4$	(20)	16	(400)
6	(150)	$^3/_4$	(20)	16	(450)
7	(175)	1	(25)	18	(450)
8	(200)	1	(25)	18	(450)
9	(225)	$1^1/_4$	(30)	18	(450)
10	(250)	$1^1/_4$	(30)	18	(450)
11	(275)	$1^3/_8$	(30)	18	(450)

* Allowance made for joint openings and minor errors in positioning of dowels.
Note: Recommended dowel spacing is 12 in. (300 mm), on center. Dowels must be carefully aligned and supported during concreting operations. Misaligned dowels cause cracking.

damage to the joint by freezing, corroding the reinforcement, or damage to the subgrade. A sealant will also prevent dirt and debris from collecting in the joint, making floor cleaning easier.

ACI 302.1R recommends that joints in industrial floors subject to small hard-wheeled traffic be filled with a material such as epoxy that gives adequate support to the joint and has sufficient resistance to wear. These joint materials should have a minimum Shore A hardness of 50 (ASTM D 2240), and elongation of 6 percent. These materials should be used where only minimal further movement is expected and should be applied from 3 to 6 months after construction.

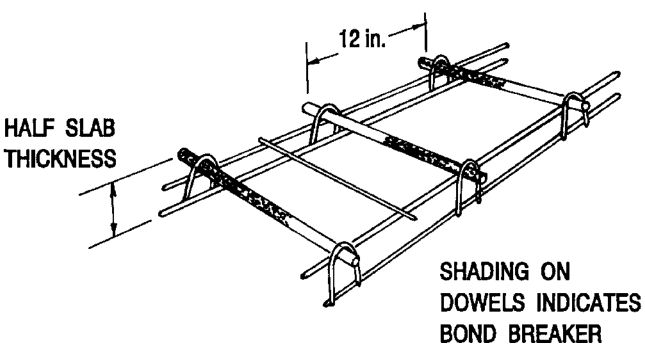


Fig. 5.3—Dowel bar assembly (Gustaferro 1980)

Field-molded or preformed elastic sealants are used only where they will not be subject to the traffic of small hard wheels.

5.3—Expansion or isolation joints

The purpose of isolation joints in slabs on grade is to allow horizontal and vertical movement between the slab and adjoining structures such as walls, columns, footings, or specially loaded areas (i.e., machinery bases). The movements of these structural elements are likely different than those of a slab on grade due to differences in support conditions, loading, and environment. If the slab were rigidly connected to the columns or walls, cracking would be likely because the differences in movement could not be accommodated. Isolation joints allow these differences in movement because there is no bond, reinforcement, mechanical connection, or keyway across the joint. A typical slab/wall isolation joint is shown in Fig. 5.4.

Isolation joints in slabs on grade also may be expansion joints. However, the expansion of concrete slabs on grade is generally less than the initial shrinkage, and provision for expansion is seldom required.

The isolation material filling the joint between the slab on grade and the adjoining structural element should be wide enough to permit both vertical and horizontal movements. For lightly loaded slabs with relatively small movements,

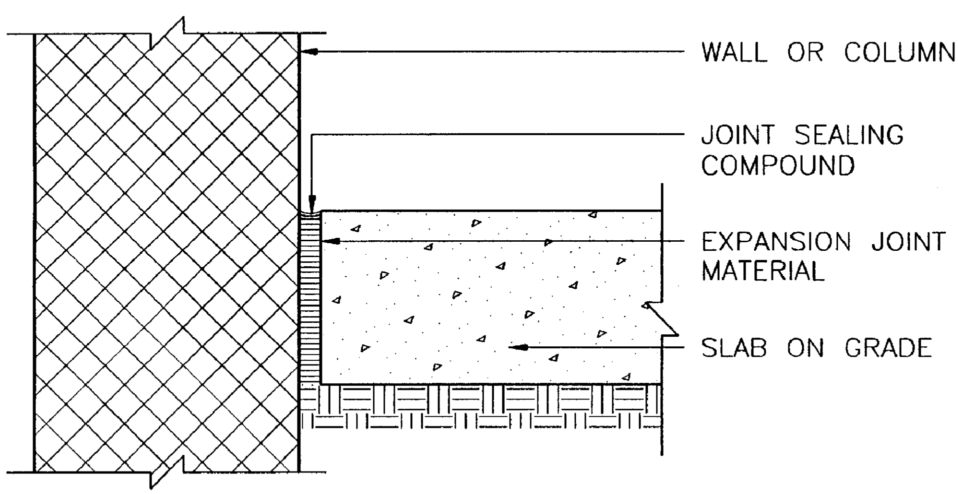


Fig. 5.4—Isolation joint (PCA 1985)

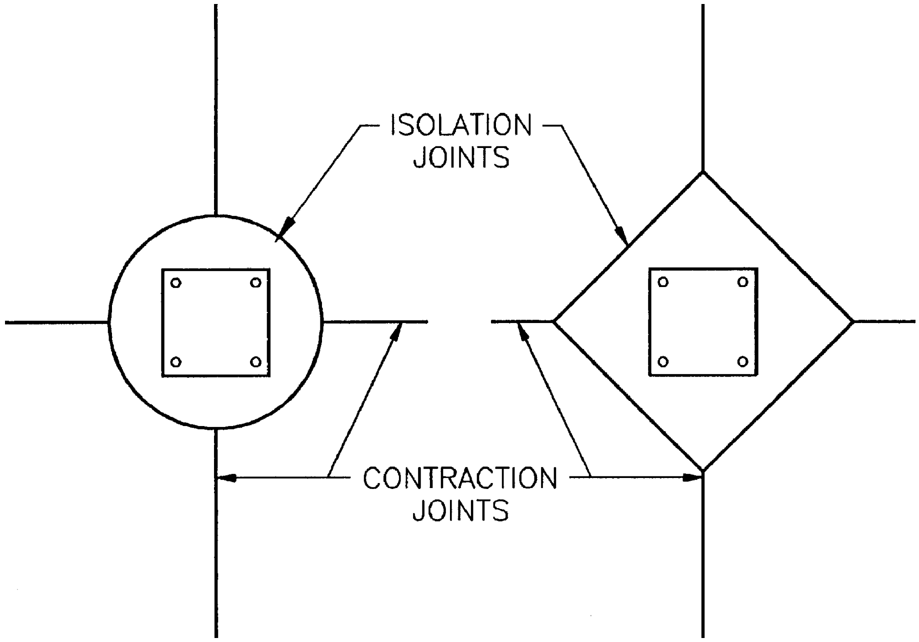


Fig. 5.5—Isolation joints at columns (ACI 302.1R)

two or more layers of asphalt-impregnated roofing felt (or similar material) can be used.

Large differential movements may be unacceptable for heavily loaded slabs. Special design and detailing practices may be required to limit the differential movements. The slab and wall may also have to be adequately reinforced to resist any induced internal forces caused by the restriction in relative movements.

Isolation joints at columns can be either circular or square, as illustrated in Fig. 5.5. A circular joint avoids re-entrant corners where stress concentrations may occur. Square isolation joints at columns are usually rotated (forming a diamond shape). If the square pattern is not rotated, radial cracks may propagate from the corners. If this layout is used, additional reinforcing is needed at corners to restrain crack development.

5.4—Construction joints

5.4.1 *General*—Construction joints are placed in the slab where the concreting operations are terminated. The practice of checkerboard placements of slab segments utilized in the past is no longer recommended by ACI 302.1R. It was once thought that a checkerboard placement would allow most of the expected shrinkage to occur prior to placement of the adjoining slab segments. However, it has been found that long-term effects of shrinkage must still be accounted for in joint design, and the additional expense of checkerboard placements is seldom worthwhile.

The construction joint type and layout should be determined before concrete placement, so that it will coincide with the isolation and contraction joints. The building use in service should also be considered.

5.4.2 *Types of joints*—Construction joints can coincide with contraction joints. These joints could then be keyed and dowelled joints discussed earlier. Bonded or butt joints may be used when construction and contraction joints do not coincide.

5.4.2.1 *Bonded joints*—Bonded construction joints should be used if the concreting operations are interrupted long enough to permit the concrete to harden. A bonded construction joint with tie bars crossing the joint to limit joint opening is adequate for an unreinforced slab-on-grade. PCA (1983) recommends the use of 30 in. (750 mm) long tie bars, spaced at 30 in. (750 mm). These should be #4 (13 mm) bars for slabs 5 to 8 in. (125 to 200 mm) thick and #5 (16 mm) for 9 and 10 in. (225 and 250 mm) slabs.

5.4.2.2 *Butt joints*—When construction joints do not coincide with isolation or contraction joints, butt joints may be used in thin, lightly loaded slabs. Load transfer is not a major factor in the design. However, for thicker slabs, or slabs with heavy loads, load transfer across the joint should be provided with a key or dowels. Otherwise, thickened edges can be designed to reduce slab edge deflections.

5.5—Special considerations

5.5.1 *Post-tensioning*—Post-tensioning can be used to control the amount of cracking and reduce the number of joints. Post-tensioned slabs on grade are used to construct large floor areas without joints and where soils are particularly expansive or compressible. The compression in the slab from post-tensioning keeps cracks closed tightly, and only very long slabs require contraction joints. Construction joints in post-tensioned slabs-on-grade will open somewhat more than joints in conventionally reinforced slabs due to the elastic shortening of the slab that results from the post-tensioning (Ytterberg 1987). PTI (1980) provides additional information on the design and construction of post-tensioned slabs-on-ground.

5.5.2 *Shrinkage-compensating concrete*

5.5.2.1 *General*—Concrete made with shrinkage-compensating cement can significantly improve the performance of the joints in slabs on grade by offsetting shrinkage with expansion (ACI 223). Shrinkage compensating concretes are used to construct large floor areas without intermediate joints.

Gulyas (1984) reports that the addition of shrinkage-compensating cement causes greater expansion in the top half of the slab than in the bottom half. This results in less curling than for slabs constructed with ordinary portland cement. The reason for the differences in shrinkage between the top and bottom surfaces have been attributed to the difference in constraints of the two surfaces. The subgrade restraint on the bottom surface can be significantly different than the "restraint" on the free top surface.

The reduced curling of the slab at the joints leads to better performance of the floor due to the potentially smaller differential movements between the adjoining slab sections.

5.5.2.2 *Special considerations in joint detailing*—It is important to alter the characteristics of the major joint types when using shrinkage-compensating concrete:

<u>Contraction joints</u>—The number of contraction joints is significantly reduced, and in many cases eliminated, with the use of shrinkage-compensating concrete. If contraction joints are planned, they should be constructed in the same way as they are for slabs constructed with portland cement concrete. However, the designer should carefully evaluate the joint type to assure that adequate load transfer is obtained. Larger movements at the joint from temperature effects will change the load transfer capabilities of the joint. Joint sealing and long-term maintenance also may be affected by the larger joint movements, and proper detailing is important for satisfactory joint performance (ACI 223; Gulyas 1984; Ytterberg 1987).

<u>Isolation joints</u>—These joints are detailed to accommodate the initial expansion of the concrete, and to allow the expansive strain to elongate the internal reinforcement. The thickness of the compressible material should be based on the expected slab expansion computed as described in Appendix A of ACI 223.

Rigid exterior restraint should be avoided to permit the expansion of the concrete, thereby preventing an associated force buildup on the restraining members. Laboratory tests have shown that rigid restraints can cause a stress buildup as high as 170 psi (1.2 MPa), that could produce forces large enough to damage the restraining structures.

The number of isolation joints around internal columns may be reduced or eliminated if a properly designed and installed compressible material is provided. If the expected net shrinkage at the columns is negligible, the joint around the column may simply be a compressible bond breaker wrapped around the column. This "joint" will permit the initial expansion of the concrete to occur, and permit vertical movement due to differences in support stiffness. However, due to potential stress buildup in the slab at this location, it is recommended that the reinforcing be increased to restrict the widths of any cracks that occur.

<u>Construction joints</u>—One of the greatest benefits from using shrinkage compensating cement is that slab placement patterns may be enlarged. Slabs located inside enclosed structures, or where temperature changes are small, may be placed in areas as large as 16,000 ft^2 (1500 m^2) without joints. In areas where temperature changes are larger, or where slabs are not under enclosed structures, slab placements are normally reduced to 7000 to 12,000 ft^2 (650 to 1100 m^2). However, the area should not be larger than a work crew can place and finish in a day.

ACI 223 recommends that slab sections be placed in areas as square as possible and that length to width ratio should not exceed 1.5 to 1.

CHAPTER 6—PAVEMENTS

6.1—Introduction

Proper jointing systems for concrete pavements insure structural capacity and riding quality. This chapter synthesizes recommendations for joints in concrete pavements, including particularly parking lots 5 to 9 in. (125 to 225 mm) thick, city streets 5 to 8 in. (125 to 200 mm) thick, and highway pavements 8 to 14 in. (200 to 350 mm) thick.

A concrete pavement slab is restrained by the subbase and its own weight. Cracks may occur in concrete pavements as a result of restrained drying shrinkage, or a temperature drop. These may occur during the first few days of curing. Cracking also can occur due to traffic loadings. Occasionally, excessive expansion of a pavement slab due to a rise in temperature may cause "blowups" to occur. This happens when the original joints are filled with debris and/or the pavement has grown in length.

A temperature gradient (differential) through the slab depth will cause the slab to curl. Maximum positive temperature differentials for slabs 6 to 9 in. (150 to 225 mm) thick approach 2.5 to 3 F/in. (0.05 C/mm) of slab thickness. In the hot afternoon, the slab is warm at the top and cool at the bottom, so the edges of the slab will tend to curl downward. The weight of concrete will tend to hold the slab in its original position with the result that tensile flexural stresses are induced at the bottom of the slab. Application of an external load at this stage will result in additional tensile flexural stresses. These tensile stresses may cause transverse and longitudinal cracks that start at the bottom of the slab. Longitudinal cracks typically are due to curling, warping, temperature change, and moisture loss.

Changes in moisture and resulting volumetric changes in the subgrade or base course as well as pumping may cause additional stresses in the slab that enhance cracking. To minimize and control cracking in rigid pavements, transverse and longitudinal joints are used in reinforced and unreinforced slabs. These joints must be capable of opening and closing and transferring load between adjacent slabs.

Joints should be properly constructed, sealed, and maintained. Improper construction, such as late sawing or inadequate depth of cut, can cause shrinkage cracking longitudinally and transverse to the pavement at locations other than the contraction joint. Inadequate joint sealant de-

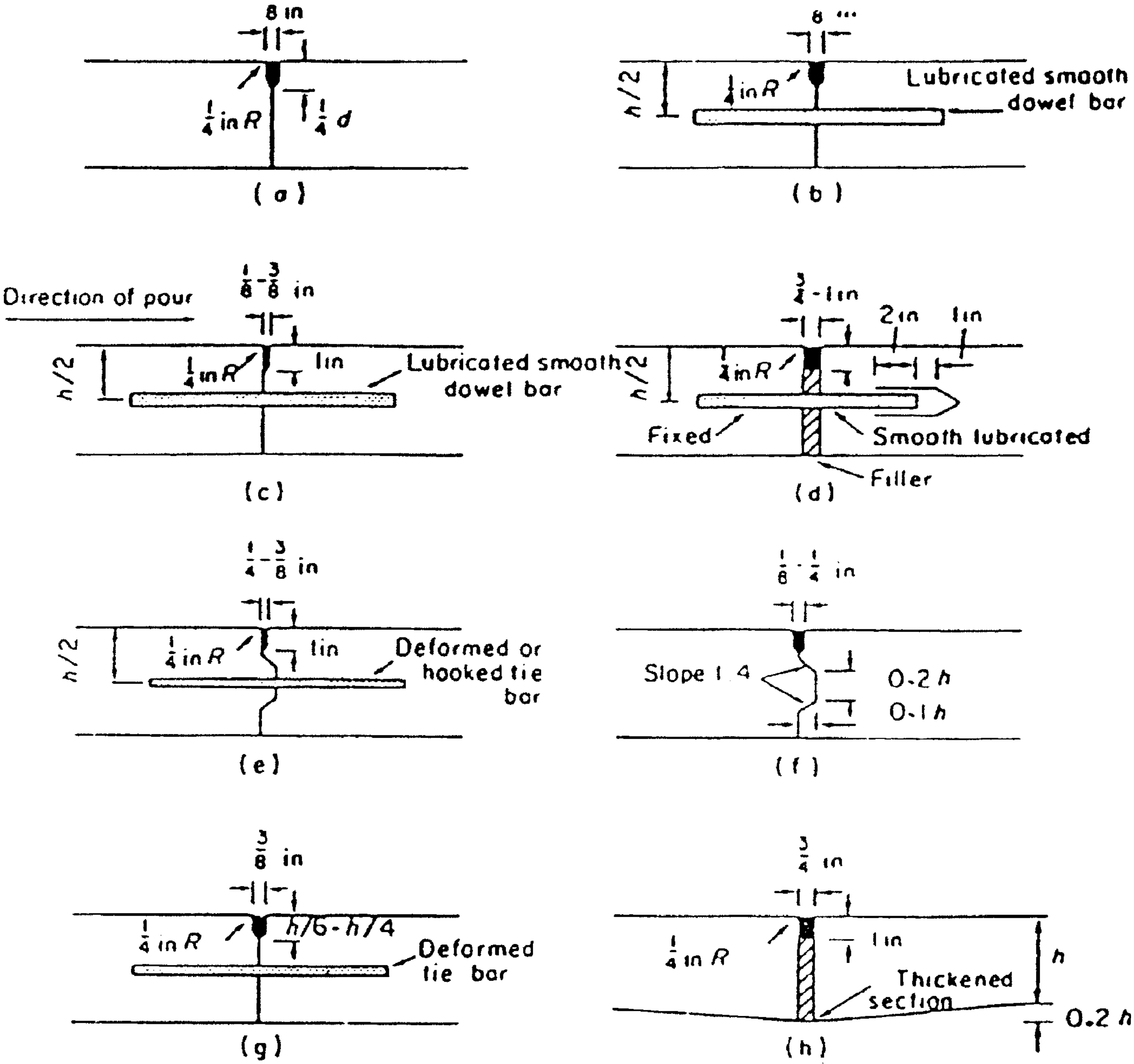


Fig. 6.1—Pavement joints

sign will allow infiltration of water and incompressibles that result in pumping, erosion, and loss of support in the subbase. Pavement failure can then result.

Joints for concrete pavements can be divided according to their desired function into the usual three basic groups: contraction joints, isolation, or expansion joints, and construction joints. Longitudinal joints as special cases of contraction/construction joints also are recognized in pavement construction. They allow a hinge-like action and control the effects of warping. Fig. 6.1 shows typical details of pavement joints. ACI 325.7R discusses post-tensioned highway slabs.

6.2—Contraction joints

Contraction joints are transverse joints whose function is to relieve tensile stresses resulting from contraction and curling of the concrete. They are not intended to relieve the effects of large expansions. However, pavements are unlikely to expand to a volume greater than when the concrete is placed, so these joints can accommodate the required movements. In conventional groove contraction joints [Fig. 6.1(a) and Fig. 6.1(b)] a groove at least $^1/_4$ of the slab thickness (or at least $^1/_3$ slab thickness for pavements built on stabilized base, according to PCA 1992a) is cut or formed at the pavement surface to force cracks to occur at the joint location. For cut grooves, concrete is sawed as soon as possible after placing the concrete. Sawing is delayed until the concrete does not ravel (see Chapter 2). Transverse grooves are cut first. According to the American Concrete Pavement Association (ACPaA 1991), contraction joints at 60- to 80-ft (20- to 25-m) spacings may be cut first in some cases. The initial cut should be at least $^1/_8$-in. (3-mm) wide to create a functioning joint; joints may be recut and widened later to fashion a prop-

Table 6.1—Dowels for concrete parking lots, city streets, and highways

Pavement	Slab thickness		Dowel diameter		Dowel embedment*		Total dowel length†	
	in.	mm	in.	mm	in.	mm	in.	mm
Parking lots (ACI 330R)	5‡	125	5/8	15	5	125	12	300
	6‡	150	3/4	20	6	150	14	350
	7‡	175	7/8	25	6	150	14	350
	8	200	1	25	6	150	14	350
	9	225	1 1/8	30	7	180	16	400
City streets (PCA 1992) Jointed reinforced pavements with joint spacings greater than 20 ft (6 m)	6‡	150	3/4	20	5	125	14	360
	6.5‡	165	7/8	22	5	125	14	360
	7‡	180	1	25	6	150	16	400
	7.5	190	1 1/8	28	7	180	16	400
	8	225	1 1/4	32	8	200	17	430
Highways (ACPaA 1991)	< 10	250	1 1/4	30	7 1/2	190	18§	450§
	≥ 10	300	1 1/2	40	9	225	20§	500§

* On each side of joint, 6 times dowel diameter for load transfer.
† Includes allowance for joint opening and minor errors in positioning — 2 times embedment length + 2 to 3 in. (50-75 mm).
‡ Dowels may be impractical in thinner pavements.
§ Computed from recommendations in ACPaA (1991), which also states: Most agencies specify 18-in.- (450-mm-) long dowels for typical highway pavement.
Note: Dowel spacing 12 in. (300 mm).

er sealant reservoir. For formed grooves, a metal or fiber strip is placed during casting of the concrete and removed as initial set has taken place. The groove is usually sealed later.

Observations of performance of existing plain concrete pavements have shown that the spacing of these joints should not be much more than up to 24 times the slab thickness (typically 21 times for stabilized bases and 24 times for nonstabilized), and not farther than 20 ft (6 m) apart, regardless of thickness. ACPaA (1991) recommends that contraction joint spacing for reinforced pavements should not exceed 30 ft (9 m).

Load transfer in the contraction joint shown in Fig. 6.1(a) is accomplished by aggregate interlock of the cracked lower portion of the slab. To assure aggregate interlock, joint maintenance is necessary to keep debris out of the joint. PCA (1992a) recommends that transverse joint spacing for plain concrete pavements not exceed 15 ft (4.5 m) when aggregate interlock provides load transfer at joints. Load transfer from aggregate interlock is not always efficient. For heavier loadings and more than 80 to 120 truck semi-trailers per lane per day, dowel bars are placed across the joint to improve load transfer as shown in Fig. 6.1(b).

Dowel bars are generally of a standard design that varies with local or regional practice. They are usually spaced at 12 in. (300 mm) and placed at mid-depth of the slab. Since the slabs move in relation to one another, perpendicular to the joint, it is necessary that the bars be properly aligned [within ± 1/4 in./ft of dowel length (± 8 mm for a 400-mm dowel)], and smooth, and coated at least on one side of the joint, to permit freedom of movement. Bonding, bends, or misalignment in the bars result in restraint, hinder the proper function of the joint, and may result in premature cracking or spalling. During construction, prefabricated baskets (Fig. 5.3) position the bars and hold them in place during concrete placement and contain the coated end of the bar in mortar-tight sleeves; dowel bar inserters also are used in slipform paving.

Usual dowels in current practice are plain round steel bars. Various sizes and shapes have been used in the past. Square bars (Fig. 6.2) have been re-examined. Another suggested approach is the use of a continuous metal plate or strip in lieu of discrete dowels. The Committee does not have documented information on field performance of strips or square dowels in recent construction. Square bars could allow for total load transfer on the top and bottom faces if compressible material is place at the sides. This would reduce restraint of shrinkage and thermal movement between adjacent slabs (Fig. 6.3) that would result from dowels, or more typically, tie bars.

Dowels can be coated with paraffin-based lubricant, asphalt emulsions, form oil, or grease to prevent bonding to the concrete.

Hardware should have a life equal to that for the pavement. Many highway departments require epoxy-coated dowel bars and dowel baskets. Some use dowels with a stainless cladding. In the past, dowels have varied in diameter from 1 to 2 in. (25 to 50 mm), and for highways 1 1/4- to 2-in.- (30- to 40-mm-) diameter dowels used for slabs thicker than 8 1/2 in. (215 mm). PCA (1992a) recommendations for dowels in city streets and highway pavements are described in Table 6.1.

PCA (1992a, 1992b) also recommends that contraction joints be skewed, or panel spacing adjusted, so that these joints don't fall within 5 ft (1.5 m) of catch basins or manholes. Skewed contraction joints also are used in some highway pavements to reduce load transfer stresses because each wheel on an axle crosses the joint independently. For street paving, failure to continue pavement transverse contraction joints through the curb or curb and gutter is likely to cause cracking in the curb (and gutter) adjacent to the pavement joint.

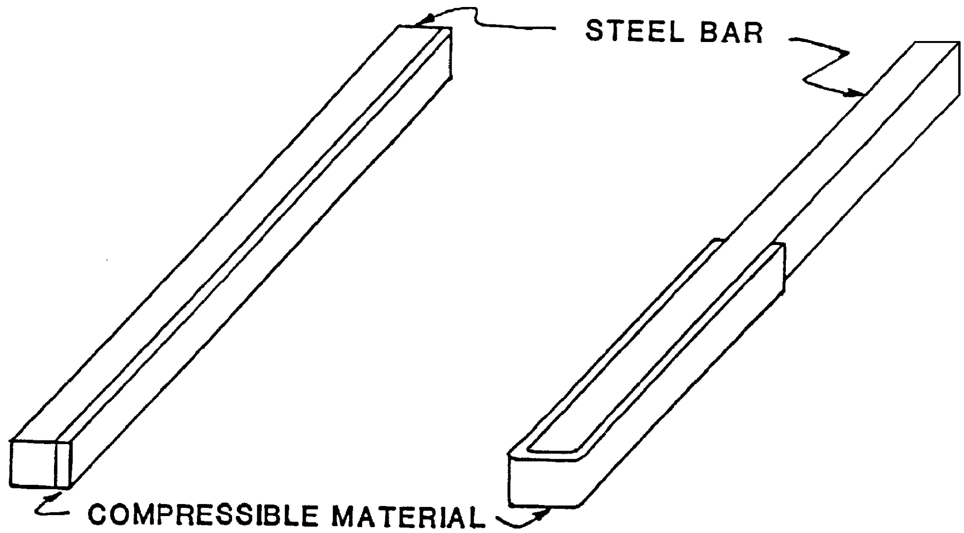


Fig. 6.2—Square dowels with compressible side material

6.3—Expansion or isolation joints

Expansion or isolation joints are constructed with a clean break throughout the depth of the slab to permit movement (Fig. 6.1d). Expansion joints are no longer used in mainline pavements,* except that expansion joints with dowels for load transfer are used at bridges. Isolation joints are used at fixed structures like manholes and drainage inlets, and at T- or other nonsymmetric intersections. The clear distance across the joint is often maintained at about $^3/_4$ in. (20 mm), although openings of $^1/_2$ in. (12.5 mm) and 1 in. (25 mm) are also used. Since the joint has no aggregate interlock, it is necessary to provide some type of load transfer. Thickened edges [Fig. 6.1(h)] have been used at expansion joints to reduce or eliminate the need for dowel bars. When thickened edges are specified, consider cost, constructibility, and the restraint it may provide to slab contraction.

The structural adequacy of an expansion joint is determined to a large extent by its load transfer device. If adequate load transfer is provided, deflection of the slabs is minimized, and pumping action is reduced. It is necessary to maintain the joints, periodically, and in some cases to replace the filler material in the joint.

Common types of fillers include fibrous and bituminous materials and cork. It is essential to seal the joints periodically to prevent infiltration of surface water. Resealing of joints is best accomplished during a cool period, when the joint has opened, thus permitting placement of a sealant. Expansion joints also may gradually close up in pavements that have unsealed contraction joints that can fill with incompressible material. This is a very undesirable condition that should be avoided by proper design, construction, and maintenance.

Nearly all states have discontinued the use of expansion joints, except at fixed structures, because they appear to be unnecessary and are difficult to construct in a slip-form paving train. However, one state successfully uses expansion joints in lieu of contraction joints. A few states use contraction joints with every third or fourth joint being an expansion joint; this system causes the adjacent joints to open and fail.

For airport pavements, isolation joints should be placed between new and old concrete slabs and between different pavement features, such as ramp-to-taxiway and taxiway-to-runway.

6.4—Construction joints

Dowelled butt construction joints are most common with load transfer across the joint [Fig. 6.1(c)]. Construction joints are used at planned or unplanned interruptions in construction, such as at the end of the day's placement.

In some cases, keyed construction joints, as indicated in Fig. 6.1(e) and 6.1(f), are used. The tied butt type shown in Fig. 6.1(c) is perhaps the most common for transverse construction joints in highway work. If keyed longitudinal construction joints are used, it is common practice to place concrete in alternate lanes. The key is slipformed or formed by special metal plates or wood strips fastened to stationary forms.

Untied keyways [Fig. 6.1(f)] have caused many joint spalling failures where key dimensions were unsuitable. They are not recommended for use on highway or airport pavements. They do not actually provide much load transfer when open, and, thus, do not provide any benefit to the pavement. Once they begin to shear off a progressive breakdown is likely and maintenance is extremely difficult.

6.5—Longitudinal joints

Longitudinal joints are used on highways to control cracking along the pavement centerline. The joint type depends primarily upon the method of placing the concrete slabs. If

*At one time, blowups were a major consideration for joints in highway pavements. These typically occurred when incompressible materials entered unsealed joints, often in the winter when joint widths were greatest. In summer, the pavement expanded in responde to daily and seasonal temperature changes. For a joint containing incompressible material, compressive stresses developed that lead to failure in some cases. Properly designed pavements with sealed and maintained joints are not susceptible to blow-ups. True expansion joints in pavements are needed only in very unusual conditions of construction or with unusual materials. See PCA (1992b).

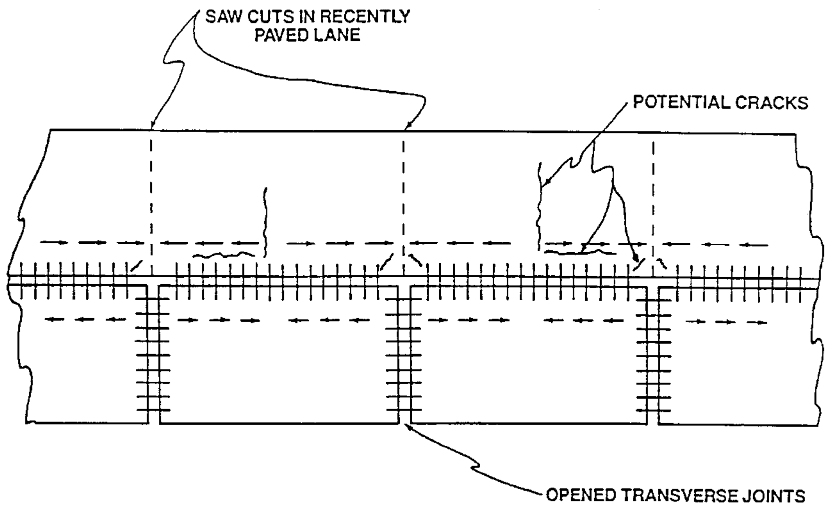


Fig. 6.3—Cumulative effect of dowel restraint

lane-at-a-time construction is used, keyed joints are generally built with ties to ensure load transfer [Fig. 6.1(e)].

In two-lane construction the grooved longitudinal joint is most convenient. Tie bars are spaced about 3 ft (1 m) apart to maintain aggregate interlock. See ACPaA (1991) for specific recommendations on tie bar spacing. Tie bars should be firmly anchored to prevent movement. Some states use hooked tie bars in longitudinal joints. Tie bars can be placed into the plastic concrete before the final finishing and placement of the joint groove. In some cases, joints have been formed by placing impregnated fibrous material along the center line. This material is left in the concrete and forms an integral part of the warping joint. However, this type of joint forming has resulted in heavy spalling and cracking problems and consequently is not recommended.

6.6—Parking lots

For purposes of this report, parking lots will be treated as a special case of concrete pavements. Parking lots and hardstands perform a function similar to pavements, in terms of providing an all-weather surface that protects the underlying soil and distributes loads to the soil. There are some important differences, however. Concrete parking lots are often constructed to serve a particular type of vehicle, rather than a broad spectrum of traffic. Facilities designed for heavy vehicles may use reasonably accurate predictions of the size and number of vehicles using the area. Facilities for light vehicles may be governed by practical requirements of placing concrete and environmental exposure. ACI 330R, the primary source for information presented in this section, provides a thorough discussion of pavement design, materials, construction, inspection and testing, and maintenance and repair of concrete parking lots.

Proper joint spacing in parking lots depends on pavement thickness, concrete strength, aggregate properties, climatic conditions, and whether reinforcement is used. Closely spaced joints usually result in smaller joint openings, and increased aggregate interlock between the panels. In some cases, jointing can be used to delineate driving lanes and parking stalls.

6.6.1 *Contraction joints*—ACI 330R recommends that contraction joints be spaced at 10 to 12 ft (3 to 4 m) in both directions for unreinforced 4-in. (100-mm) slabs and at 15- to 20-ft (5- to 6.3-m) intervals for 8-in. (200-mm) slabs. Joint spacing may vary with subgrade conditions, aggregate, concrete strength, and curing conditions. The contraction joint pattern should divide the pavement into panels that are approximately square; the length of a panel may be 25 percent greater than the width. ACI 330R recommends a maximum contraction joint spacing of 30 times the thickness for unreinforced slabs.

6.6.2 *Expansion joints*—True expansion joints are generally not needed in parking lots. Expansion joints may be required for parking lots with widely spaced contraction joints, and when concrete will be placed when its temperature is below 40 F (5 C). They may also be required for concrete with unusual expansion and contraction characteristics. Where needed, expansion joint materials are sheets or strips of bituminous mastic, bituminous impregnated cellulose or cork, sponge rubber, or resin-bound cork.

6.6.3 *Isolation joints*—Concrete slabs should be separated from fixed objects within or adjacent to the paved area. Isolation joints are frequently encountered at light standard foundations, planters, area drains, curb inlets, sidewalks, and buildings or other structures. These joints are produced by inserting premolded joint fillers before or during construction. Joint forming material extends to the subbase and should not protrude above the pavement. If vehicles pass over the isolation joint, the slab edges should be treated as construction joints. The pavement edge should be thickened by about 20 to 25 percent and tapered to the required nominal thickness over a distance of at least 3 ft (1 m).

6.6.4 *Construction joints*—Construction joints in concrete parking lots may be keyed or butt-type, and may be tied. Butt-type joints do not provide load transfer, but this is not required for lots that serve light vehicles. Load transfer devices are needed for heavier traffic. Appendix C of ACI 330R recommends construction joint details.

Transverse construction joints are designed for interruptions in paving operations, such as at the end of a day, weather delay, or equipment breakdown. Where possible, transverse construction joints should coincide with planned locations for contraction joints. Butt joints should be thickened by 20 percent if dowels are not used for load transfer. A construction joint can be made in the middle third of a panel if deformed tie bars are used. This prevents joint movement that would cause reflective cracking in the adjacent lanes.

Load transfer at longitudinal construction joints between paving lanes also should be considered. Keyed joints may be formed or slipformed. Longitudinal construction joints along the periphery of a parking area may be tied with deformed bars if tightness of the joints is critical and heavy vehicles are expected. It is usually sufficient to tie only the first joint in from the exterior of the lot. Tying additional joints may unduly restrict movement and lead to undesirable cracks.

Where slabs of different thicknesses come together, the subgrade under the thinner pavement should be shaped to provide a gradual transition in a distance of 6 to 10 times the thickness, or at least 3 ft (1 m).

6.6.5 *Reinforcement*—Reinforcement in parking lot pavements may be needed where joint spacing is large, or in irregular or odd-shaped panels. ACI 330R recommends that reinforcement be considered for rectangular panels where the length-to-width ratio exceeds 3:1, nonrectangular panels of any aspect ratio, and panels where the slab tapers to a sharp angle. The amount of reinforcement for these cases can be calculated as discussed in ACI 330R.

Distributed reinforcement may be used to control intermediate cracks. This is done when joint spacings are too great to control shrinkage cracking or when subbase conditions cannot be modified to provide uniform support. Distributed reinforcement does not increase the slab capacity. ACI 330R provides a means of estimating the amount of reinforcement. Distributed steel is needed where transverse joints are spaced in excess of 30 times the slab thickness. Distributed steel is interrupted at joints; truck pavements with wide joint spacing may require dowels for load transfer.

Dowels or other load transfer devices are not needed for most parking lots. They can be justified for cases of poor subgrade or heavy truck traffic. Smooth dowels across contraction joints in pavements with wide joint spacing provide load transfer while permitting joints to move. Correct alignment and coating are essential for proper performance; see Section 6.2. Epoxy-coated dowels should be used where deicers can be deposited on the pavement. ACI 330R recommends dowels spacing as shown in Table 6.1. Dowels may be impractical for pavements thinner than 7 in. (175 mm).

Tie bars should be used on centerline joints if there are no curbs. They also are used along the first longitudinal joint in from the pavement edge to keep the outside slab from separating from the pavement. Tie bars are not required in the interior joints of parking lots and other wide paved areas, since these are confined by the surrounding slabs.

6.7—Shrinkage-compensating concrete

Shrinkage-compensating pavement slabs have been placed with joint spacings ranging from 75 to 150 ft (23 to 46 m). Provision should be made to accommodate the initial expansion of the pavement and possible wider joint openings after shrinkage has occurred. Further details concerning the use of shrinkage-compensating concrete are given in ACI 223.

CHAPTER 7—TUNNELS, CANAL LININGS, AND PIPES

7.1—Introduction

This chapter discusses transverse and longitudinal joints, and the selection of appropriate sealants for cast-in-place and precast tunnel linings, concrete canal linings, and precast concrete pipe.

7.2—Concrete tunnel lining

Concrete tunnel lining is required to support or resist external structural and hydrostatic loads, and to improve hydraulic flow characteristics. There are two methods for lining tunnels with concrete; the cast-in-place method and the precast concrete method. Joints and sealing systems are critical details in the construction of concrete linings.

7.2.1 *Cast-in-place tunnel lining*

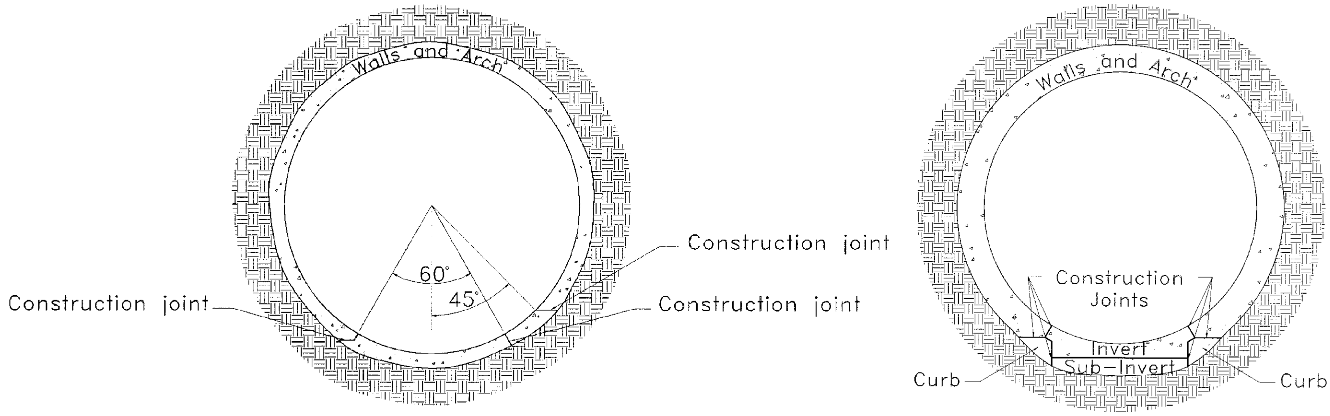
7.2.1.1 *Transverse joints*—In cast-in-place concrete tunnel linings, it is common practice to provide transverse construction joints to simplify construction and reduce shrinkage cracking. The location and spacing of the transverse joints are normally based on the limitations of concrete placement procedures and forming system. The practical length of form that provides the best combination of ease of handling and economy is important. Typical spacing is between 20 to 40 ft (6 and 12 m). Transverse joints are generally provided with waterstops. Information on waterstops is found in ACI 504R.

Continuously placed concrete lining without transverse construction joints may be satisfactory where leakage would not be objectionable or detrimental.

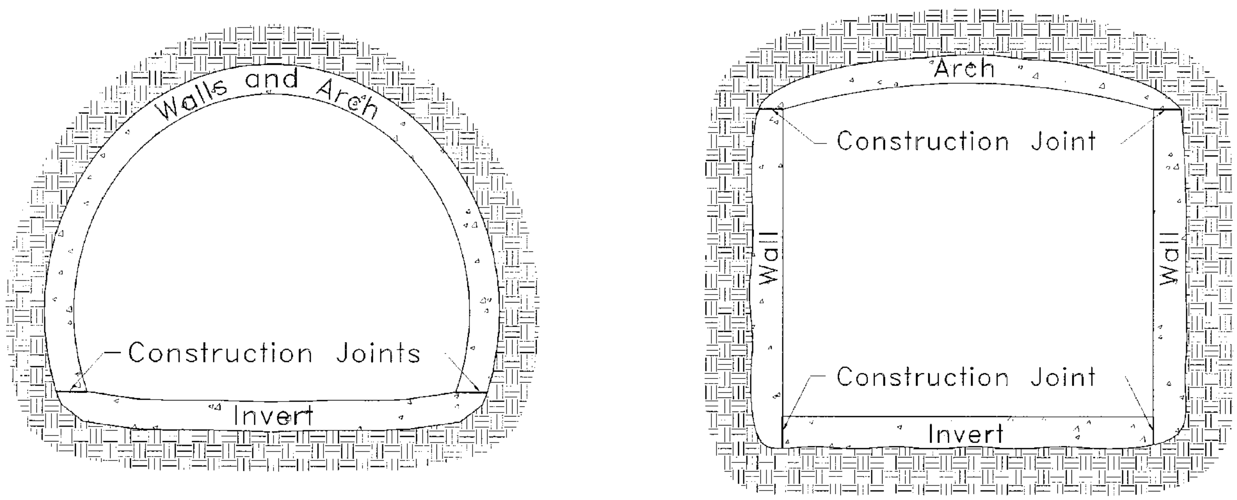
Transverse contraction or expansion joints are not normally needed because the seasonal temperature changes are small in tunnels, except at the portals.

7.2.1.2 *Longitudinal joints*—Longitudinal joints divide the tunnel lining cross section into parts designated as inverts, walls, and arch. The location of the longitudinal joints depends on the tunnel cross sections and the concrete placing sequence. Typical longitudinal joints for circular and horseshoe tunnels are shown in Fig. 7.1 (ACI 504.1R). Sealing of the longitudinal joints is usually achieved by concrete-to-concrete bond. When high hydrostatic pressures exist and watertightness is required, waterstops should be placed between the invert and walls.

7.2.2 *Precast tunnel lining*—Joint designs for precast concrete tunnel liners vary with each project, depending on geologic conditions, construction procedures, and types of tunneling machine to be used. An excellent review of the



Circular tunnels



Horseshoe tunnels

Fig. 7.1—Longitudinal joints for circular and horseshoe tunnels

state-of-the-art on precast concrete tunnel liners can be found in ACI 504.1R and Birkmyer (1975).

7.2.2.1 *Segment size*—The size of a precast concrete segment and number of segments in a ring depend primarily on the type and design of the tunnel boring machine and support equipment. This includes muck-handling systems and vertical hoists within the tunnel and access shafts (Birkmyer 1975). A tunnel boring machine that can provide full and constant support to the excavated face may permit the use of a wide segment. Less sophisticated equipment in similar ground may dictate narrow segments. The type and arrangement of muck handling and other tunneling equipment should be considered in making a decision on the number of segments in a ring. The space needed for this equipment in the tunnel places constraints on the size, particularly the length of the segment that can be handled (Birkmyer 1975).

7.2.2.2 *Joints*—The types of longitudinal joints that have been used are lap, knuckle, flush, tongue-and-groove, and convex-to-convex. For transverse joints, the lap, flush, and tongue-and-groove types are being used. The advantages and disadvantages of each joint type are discussed by Lamond (1981). The knuckle type or lap type appear to have the most advantages for longitudinal and transverse joints, respectively.

Both bolted and unbolted segmental concrete tunnel liners are used. The bolted system should be used where groundwater exists, as it is the only system that can be made watertight.

7.2.2.3 *Sealant*—A $^1/_2$-in. (13-mm) joint width is considered satisfactory to provide construction tolerance (Lamond 1981). The sealants commonly used are bituminous mastics, elastomeric materials, wood strips, gasket-caulking, epoxy materials, or a combination of these materials. An appropriate sealant type may be selected from ACI 504R.

7.3—Concrete canal linings

By their very nature concrete canal linings are long strips of paving between the various canal structures. They may be 8 or 10 ft (2.5 to 3 m) wide for small canals and laterals and considerably wider than 100 ft (30 m) for large main canals. These linings are rarely less than 2 in. (50 mm) thick in small canals and $4^1/_2$ in. (110 mm) thick for the large ones. Since the early 1950s, the use of reinforcement in canal lining concrete has not been considered worthwhile.

It might be argued that with canals in service carrying water, there can be no shrinkage from drying. The temperature range is often modest, and unlikely to drop below about 30 F

(0 C). There is little tendency for cracking and need for joints in the lining concrete. This is true when there is water in the canal. There are times before the canal goes into service, when it is drained for cleaning or repair, or if the canal is not flowing full, when a portion of the lining will be exposed to natural drying and cooling.

Due to the restraint of the subgrade, these conditions will produce tension that can result in cracking if joints are not used. Most cracks will form in a generally transverse direction, but in wide canals, longitudinal cracks and cracks with random direction may appear. Since any such cracking may well cause a loss of water, and will be at least unsightly, joints should be placed in the canal lining. Joints in most larger canals are equipped with waterstops or mastic sealing compounds to minimize the loss of water, and saturation of the embankment or foundation. Where waterstops are not needed unsealed joints are used to control the location of cracks.

Transverse joints, whether sealed or not, are spaced from 12 to 15 ft (4 to 5 m) for $4^{1}/_{2}$-in. (110 mm) linings, down to $7^{1}/_{2}$ to 10 ft (2.3 to 3 m) for 2-in. (100 mm) linings. In larger canals, longitudinal joints at similar spacings are required if the bottom slab is wide. They are used also at the bottom of the side slopes if the slopes and bottom are placed separately. Longitudinal joints on side slopes should include one at the line between the cut excavation and the embankment fill, where a crack often appears.

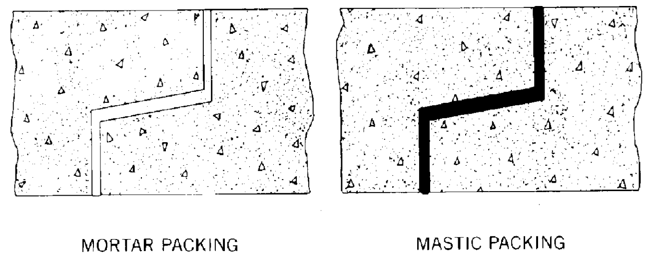
Methods and materials for sealing contraction joints were developed during the construction of the California Aqueduct system in the 1960s. Vendors of PVC waterstops and polysulfide sealants, and manufacturers of canal lining machines contributed to these developments. The methods developed were soon used and improved by the U.S. Bureau of Reclamation, and are described in the Section 108(d) and Fig. 157, 158, and 159 of the USBR *Concrete Manual* (1975).

7.4—Concrete pipe

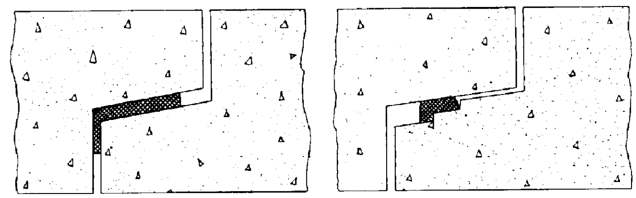
This discussion is limited to precast concrete pipe. Additional information is available from the American Concrete Pipe Association (1988). Pipe joints are required for various functions, depending on the type of pipe and application. Several factors are important in selecting appropriate joints:

- Resistance to infiltration of groundwater or backfill material
- Resistance to exfiltration of sewage or storm water
- Control of leakage from internal or external heads
- Flexibility to accommodate lateral deflection or longitudinal movement without creating leakage problems
- Resistance to shear stresses between adjacent pipe sections without creating leakage problems
- Hydraulic continuity and smooth line flow
- Controlled infiltration of groundwater for subsurface drainage
- Ease of installation

In comparing the economy of various joint types, it may be necessary to compare the installed cost of the joint with the



(a) Joints with mortar or mastic packing



(b) Compression-type rubber gasket joint

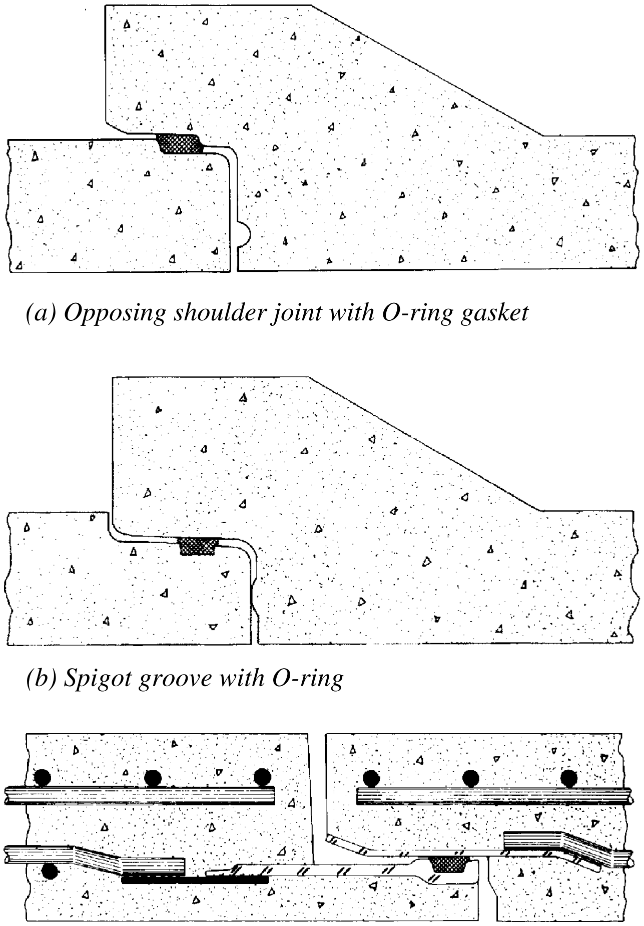
Fig. 7.2—Typical sections of joints with packing or gasket in precast concrete pipe (courtesy of American Concrete Pipe Association)

costs of pumping and treatment that result from increased or decreased infiltration. The joints discussed in this section are beneficial for specific applications. They are shown in Fig. 7.2 and 7.3. Field construction practice and conditions of service are variable. For a particular application, local manufacturers of concrete pipe should be consulted to determine the availability and cost of various joints.

7.4.1 *Joints with mortar or mastic packing*—Concrete surfaces with either bell-and-spigot or tongue-and-groove configurations can be packed with cement-based mortar, preformed mastic, or a trowel-applied mastic. Joints with mortar fillers are rigid and may crack and leak following deflection or movement after installation. A properly applied mastic filler will provide a degree of flexibility and retain watertightness. These joints are not recommended for conditions with internal or external head if leakage is important. A similar system for use with this joint is an external sealing band type rubber gasket that conforms to ASTM C 877. This jointing system performs well in resisting external heads when used in straight wall and modified tongue-and-groove configurations.

7.4.2 *Joints with compression-type rubber gasket joints*—A compression-type gasket may be used to seal concrete surfaces with or without shoulders on the tongue or the groove. Joint dimensions and gasket cross section vary widely, but most are manufactured in conformity with ASTM C 443. This joint may be used with either bell and spigot or tongue-and-groove pipe manufactured to meet the requirements of ASTM C 14 or ASTM C 76.

7.4.3 *Joints with O-ring gaskets*—Circular cross section, rubber gaskets (O-rings) are used where specified infiltration or exfiltration requirements are specified, or when the line resists internal or external pressures.



(a) *Opposing shoulder joint with O-ring gasket*

(b) *Spigot groove with O-ring*

(c) *Steel end ring joint with spigot groove and O-ring gasket*

Fig. 7.3—Typical O-ring gasket joints in precast concrete pipe (courtesy American Concrete Pipe Association)

7.4.3.1 *Opposing shoulder type*—This joint configuration has concrete surfaces with opposing shoulders on both the bell and spigot. These joints are designed for low pressure capability, such as irrigation lines, water lines, sewer force mains, and gravity or low-head sewer lines where infiltration or exfiltration is important. These joints meet ASTM C 443 and are also used with joints meeting the requirements of ASTM C 361.

7.4.3.2 *Spigot groove type*—Also described as a confined O-ring joint, there is a groove only on the concrete spigot in this joint. The applications are similar to the opposing shoulder type joint.

7.4.3.3 *Steel end ring with spigot groove*—This joint is recommended for both low- and high-pressure applications, such as in water transmission and distribution lines. Other applications are irrigation lines, sewer force mains, and sewers where infiltration or exfiltration is considered. Steel rings are cast in both the bell and spigot of the pipe; a groove for the O-ring gasket is made in the steel spigot ring. This joint meets the requirements of ASTM C 361 and ASTM C 443, as well as AWWA standards C 300 through C 303. The joint has great shear strength, excellent watertightness and flexibility, and is the least subject to damage during installation.

CHAPTER 8—WALLS

8.1—Introduction

Joints are used in reinforced concrete walls to simplify construction, provide relief from constrained movement, and control cracking. Usually it is not possible to place a wall in one operation, so construction joints are required. These construction joints do not serve a structural purpose. Concrete walls are subject to changes in length, alignment, or volume that result in movements of the structure. These movements may be caused by creep, shrinkage, temperature gradients, differential settlement, or loads.

If there are no joints, forces may develop in the wall that may cause cracking if the tensile capacity of the concrete is exceeded. Cracking may be minimized by reducing the restraint of free movement of the wall. This normally is accomplished by dividing a wall into suitable lengths separated by joints that allow movement. These joints function to provide stress relief. Cracking due to base restraint in a long concrete wall is shown in Fig. 8.1. The wall is restrained at the bottom by the footing and the top by the floor system. Restraints produce forces in the concrete that exceed the tensile capacity and cause cracking.

Reinforcement resists tensile stresses that develop in the wall. Cracking cannot be prevented by reinforcing the wall, but widths of cracks that do form can be controlled. A wall need not be crack-free if the locations and width of cracks are controlled to minimize the effects on strength, function, or appearance of the wall, as required. This can be done effectively and economically by the proper use of joints.

Shrinkage-compensating concrete can be used to reduce the number of joints and cracks in walls. See ACI 223 for special details required to accommodate the expansions at the wall base.

8.2—Types of joints in concrete walls

Contraction, isolation or expansion, and construction joints are used in concrete walls. There are other kinds of wall joints, but these are usually some slight variation or combination of one of the three main types. A discussion of the purpose of each main type of joint and its use in walls is provided in the following discussion.

8.3—Contraction joints

The contraction joint is an intentionally created plane of weakness in the wall made by reducing the wall thickness, reinforcement, or both. The cracking may then occur at this weakened plane rather than at random locations in the wall. Contraction joints locate cracks in places selected for purposes of appearance or structural integrity.

Contraction joints can be inexpensive and relatively simple to construct in walls. They are often made using wooden, rubber, plastic, or metal strips attached inside the forms. These strips leave narrow vertical grooves in the concrete on the inside and the outside of the wall. The total depth of the grooves should be at least $^1/_4$ of the wall thickness, as shown in Fig. 8.2 (PCA 1984)—at least $2^1/_2$ in. for a 10-in. wall (75

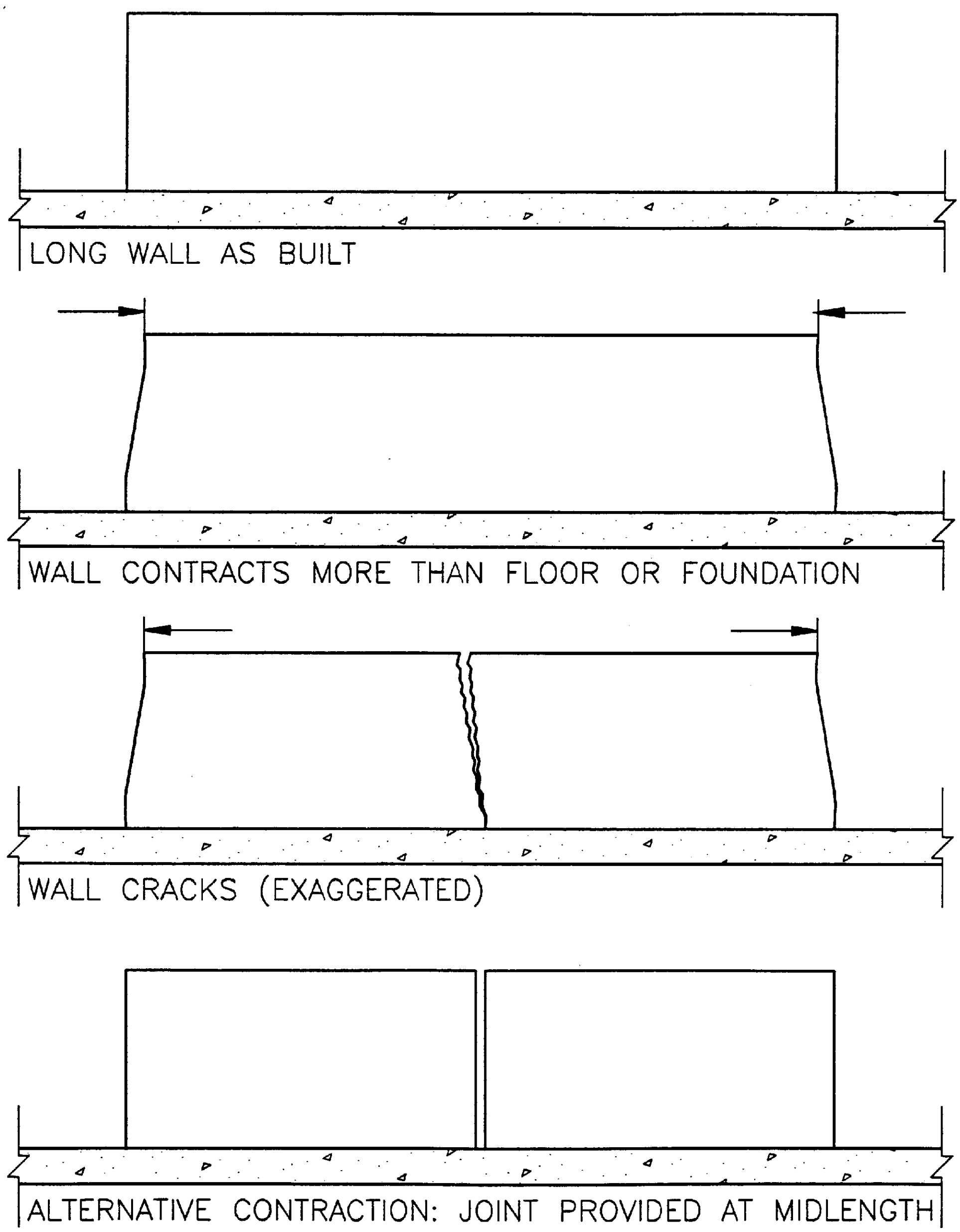
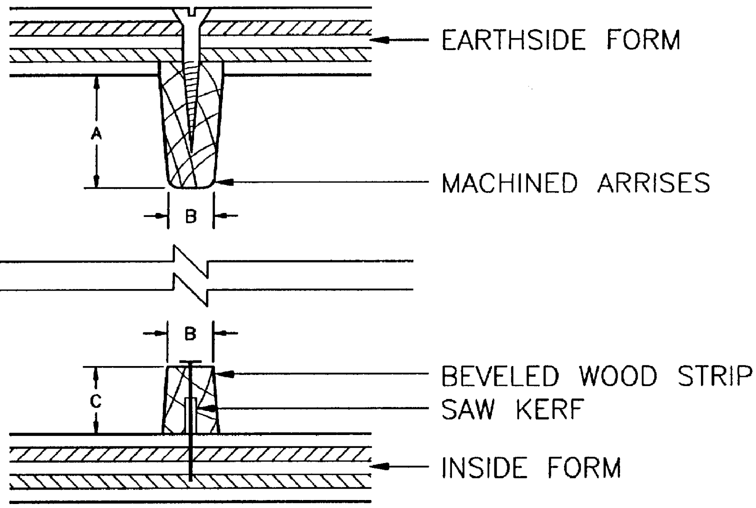


Fig. 8.1—Cracking of a long wall due to contraction

mm for a 300-mm wall). The groove or notch can be sealed to prevent excess penetration of moisture or chemicals that would promote corrosion of the wall reinforcement (see Chapter 2 and ACI 504R). Sealants such as weather-resistant polyurethane or silicone (that will remain flexible after placement) may be used. For watertightness, the exterior groove can be packed with backup material and caulked full with an elastomeric sealant (Fig. 8.3). A waterstop also can be used to prevent water from leaking through the crack that occurs in the contraction joint. PCA (1975) recommends using dowel bars that provide 0.015 times the cross-sectional area of the wall and extending 30 bar diameters each side of the joint as shown in Fig 8.4.

Recommendations for joint spacing differ, depending on the type and use of wall and the service conditions. A recommended contraction joint spacing is the height of the wall for high walls and three times the height of the wall for short walls (ACI 224R). Short walls are usually considered to be less than 8 ft (2.4 m) and high walls taller than 12 ft (3.6 m). These recommendations recognize that the upper portion of the wall is likely to cool and shrink faster than the lower part of the wall, that is also more restrained. Both conditions allow additional tensile stress within the concrete that will likely cause some cracking.

Contraction joints placed in line with openings in the wall effectively control cracking at the corner of the openings. Joint spacing may be a little greater in walls without openings, but spacing should not exceed 25 ft (7.6 m) (PCA 1984). It is desirable to have a contraction joint within 10 to 15 ft (3 to 4.5 m) of a wall corner.



	6 in. (150mm)	8 in. (200mm)	10 in. (250mm)
A	1 in. (25mm)	1 1/4 in. (32mm)	1 1/2 in. (40mm)
B	1/2 in. (13mm)	1/2 in. (13mm)	1/2 in. (13mm)
C	3/4 in. (20mm)	3/4 in. (20mm)	3/4 in. (20mm)

Fig. 8.2—Forming contraction joints in walls (PCA 1984)

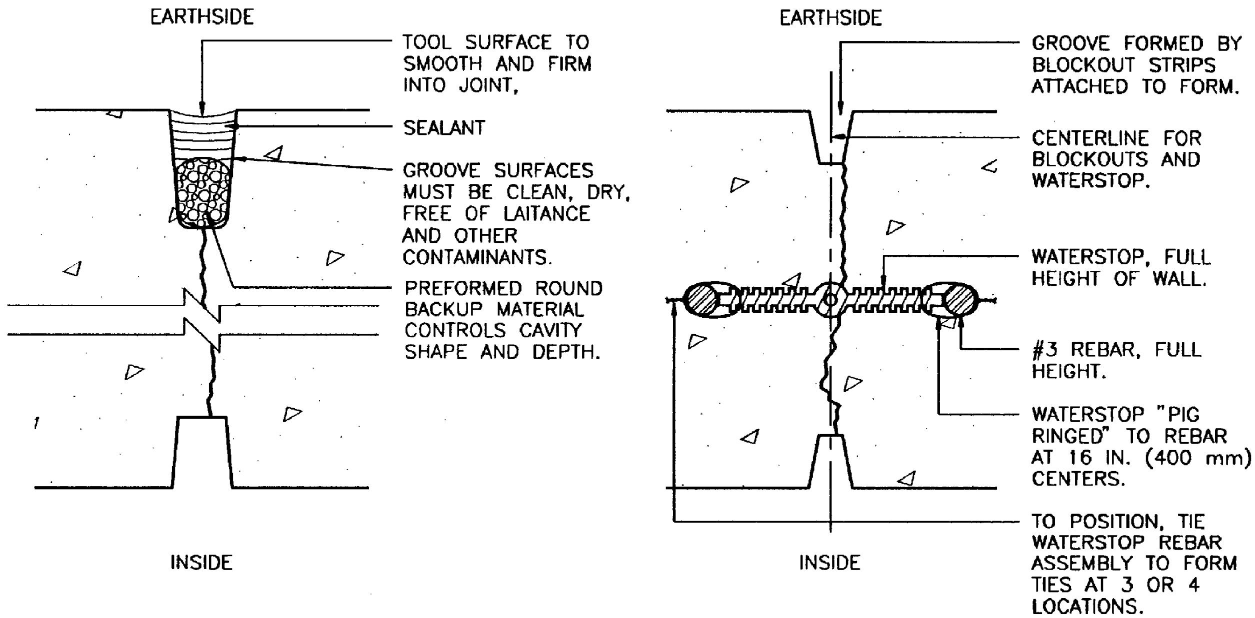


Fig. 8.3—Contraction joint for walls of water-excluding structures (PCA 1984)

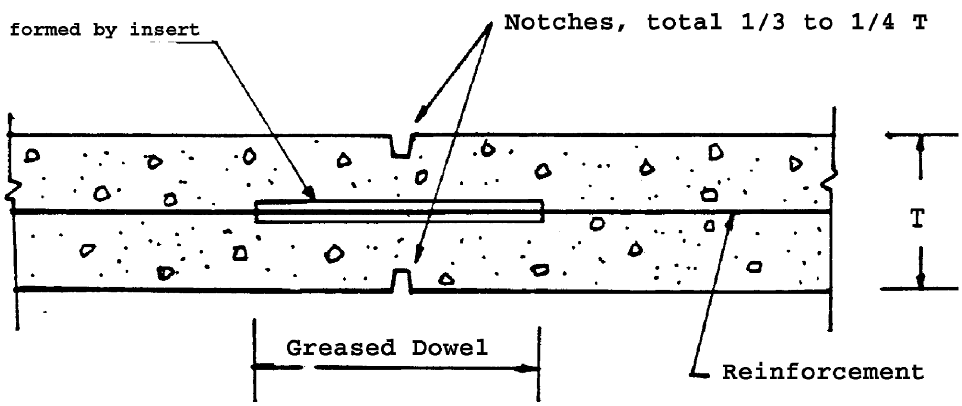


Fig. 8.4—Contraction joint with alignment dowel (Council on Tall Buildings and Urban Habitat 1978)

Recommendations for reinforcing range from stopping all reinforcing 2 to 3 in. from the joint, to allowing one-half of the reinforcing to continue through the joint (partial contraction joint). Partial contraction joints are used mostly in water-retaining or excluding structures. It is better to discontinue the reinforcing at the joint and thereby allow for full movement at the joint (Green and Perkins 1980). If alignment of joint or adjacent wall surfaces is important, dowels may be used (Fig. 8.4). Total reinforcement should continue through the joint only when the joint needs to be held either open or closed to maintain structural stability.

8.4—Isolation or expansion joints

Isolation or expansion joints are used in walls in the form of vertical joints through the concrete. They separate adjacent concrete sections and allow free movement of the adjacent parts. Independent movement of two adjacent walls prevents crushing, warping, distortion, and buckling that could result if they moved together.

These movements could be the result of compressive forces that may be developed by expansion, applied loads, or differential movement. Temperature change is an important cause of wall movement. The movement caused by a temperature change is obtained by multiplying the coefficient of linear thermal expansion by the length of the wall and the degree change in temperature. For example, a 100-ft wall section could be expected to move as much as nearly 0.5 in. at expansion joints for a temperature change of 60 F (a 30 m wall changing 35 C could move about 10 mm). Climatic conditions obviously influence the placement of expansion joints.

Expansion joints are constructed by providing a space through the full cross section between abutting wall units when the concrete is placed. The space is provided by a spacer or filler set in the forms. The material used as filler is usually compressible, elastic, and nonextruding, such as a premolded mastic or cork filler. The joint should be straight and continuous from the bottom of the wall at the foundation to the top of the wall. Reinforcement should stop 2 to 3 in. (50 to 75 mm) from the joint (PCA 1975), and dowels also may be used. Various expansion joints are shown in Fig. 8.5.

Expansion joint spacing in straight walls should range from 200 to 300 ft (60 to 100 m). An expansion joint also should be located when a direction change occurs along a wall, or when two or more walls come together from different directions. Recommended expansion joints widths range from $^{3}/_{4}$ to 1 in (20 to 25 mm).

Some buildings built without expansion joints have performed satisfactorily. Temperature expansion usually does not overcome the initial volume changes of cooling and drying shrinkage (Fintel 1974) in these cases.

8.5—Construction joints

Construction joints are planes separating the work done at different times. They accommodate the construction sequence and are designed for structural continuity. These joints may be horizontal or vertical and their location is often established before construction.

Vertical bulkheads divide the forms into sections when the concrete is placed for the full wall height. Bulkheads allow filling the section in one operation.

Surface preparation is extremely important for bonded construction joints. The surface of the concrete placed first should be cleaned of contaminants and debris such as dried loose concrete and aggregate. Sand-blasting, followed by air-blast cleaning, is often specified. Proprietary products or cement grout are sometimes used as bonding agents. For proper performance, concrete is placed before the bonding agent dries. A dry layer of bonding agent will serve as a bond breaker. Wall reinforcement is continuous through a bonded construction joint.

Wall construction joints should be spaced at 15- to 25-ft (4.5- to 7.5-m) intervals, with the first joint occurring 15 ft (4.5 m) from the corner of the structure (PCA 1984). Construction joints should also be located where abrupt changes in thickness occur in walls. Walls with frequent openings should have construction joint spacing limited to 20 ft (6 m).

Keyways are often provided in wall-to-footing construction joints, especially if no reinforcement holds the wall and footing together (PCA 1984). The keyway can be formed by pressing a slightly beveled 2-x-4 (50-x-200 mm, nominal) into the fresh footing concrete. The keyway should be oiled well before it is used so it can be removed after the concrete has hardened.

8.6—Summary

Recommendations for the location and spacing of wall isolation and contraction joints are empirical, and further study is needed to provide a more rational basis. Selection of wall joints requires study of exposure and climatic conditions, restraint imposed on the wall by surrounding structures, the likelihood of differential settlement, and the number and size of openings in the wall.

CHAPTER 9—LIQUID-RETAINING STRUCTURES

9.1—Introduction

Liquid-retaining structures are subjected to volume changes from temperature and moisture changes in the concrete as are other concrete structures. Temperature changes occur daily and seasonally.

Moisture change begins with the evaporation of excess water in the concrete during and after the curing period. Moisture will continue to decrease in the concrete until the liquid is added. The liquid can halt the contraction of the concrete and may, under hot, humid conditions, cause some expansion of the concrete.

Since the function of the structures is to retain liquid, leakage is a primary concern in design. This is especially true with leakage into potable water facilities or out of contaminated water facilities where public health is of concern.

Reinforced concrete liquid-retaining structures are designed for both strength and serviceability. As a result, concrete and steel permissible stresses and reinforcement spacing are selected to control the crack widths in the concrete.

Concrete quality is also controlled to provide a watertight structure with good chemical resistance and a shrinkage potential less than that of typical concrete. The basic means of controlling the crack size and spacing in walls is the use of a

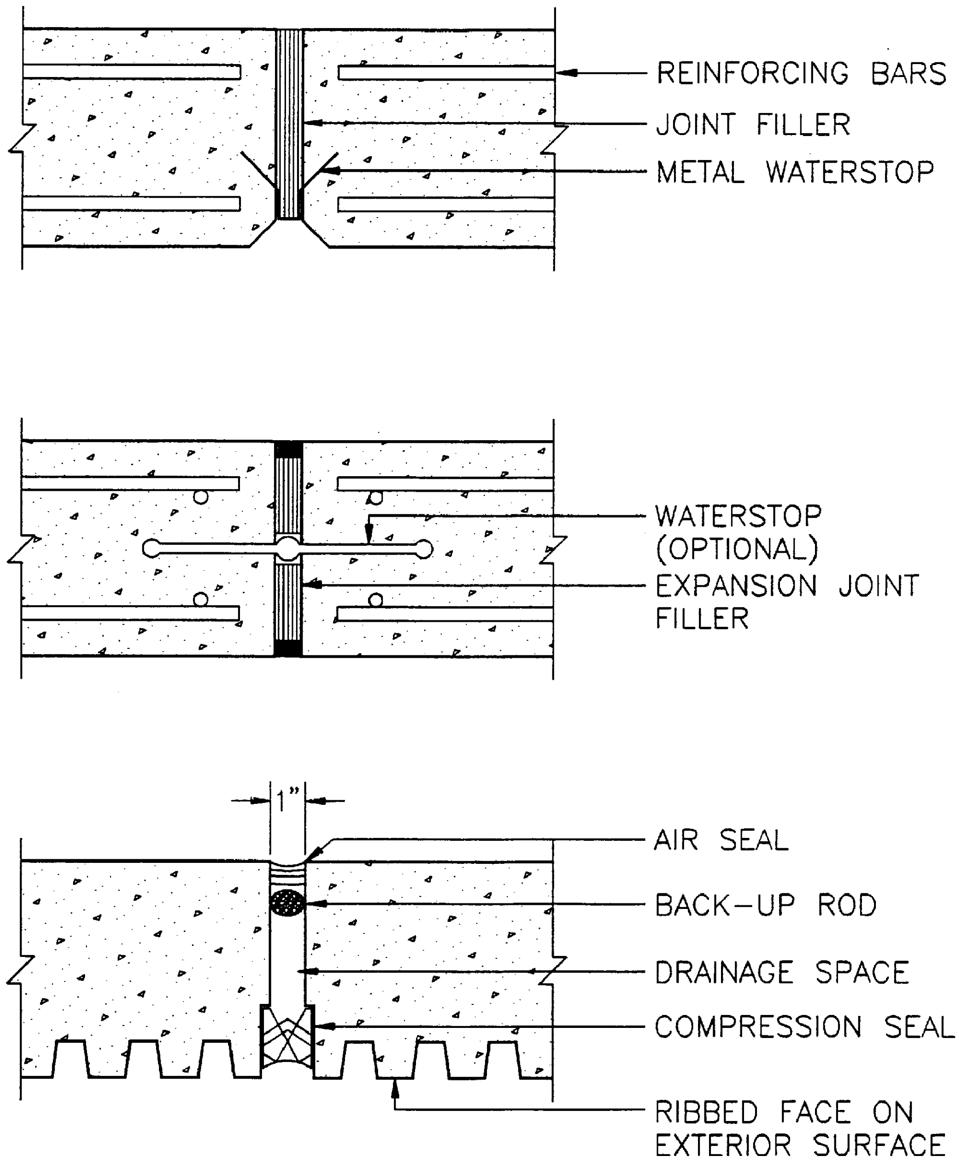


Fig. 8.5—Types of expansion joints (Perkins, 1973)

minimum of 0.28 percent of Grade 40 or Grade 60 (Grade 300 or Grade 400) reinforcement. This steel should be divided equally between the two faces and spaced not farther than 12 in. (300 mm). This minimum reinforcement has been found to be acceptable provided that movement joints are less than 30 ft (10 m) apart for concretes made with ASTM C 150 and ASTM C 595 cements. With shrinkage-compensating concrete, joint spacings up to 75 ft (25 m) have been used successfully with 0.3 percent reinforcement (ACI 350R).

Fig. 9.1 shows the ACI 350R recommendations for concretes made with ASTM C 150 and ASTM C 595 cements. Minimum temperature and shrinkage reinforcement should be No. 4 (13 mm) bars, spaced not farther than 12 in. (300 mm) on center, each face.

Movement joints are either isolation joints, expansion joints, or contraction joints. Expansion joints allow for both expansion and contraction while contraction joints exist primarily to dissipate the effects of restrained shrinkage of the concrete. Isolation joints provide complete separation between concrete walls; shear displacement as well as expansion and contraction movement is permitted.

A wall restrained at its base by being placed atop previous concrete construction will tend to have full height cracks spaced at $1^1/_2$ to 2 times the wall height. Larger crack spacings are found with less base restraint. Contraction joints may be used to locate the full-height wall cracks.

There are no exact rules for locating contraction joints. Each structure should be examined individually to determine where the contraction joints should be placed. The following guidelines are suggested:

• For walls 9- to 12 ft- (3- to 4-m) high with openings, contraction joint spacing should be 15 to 20 ft (5 to 6.5 m). Walls without openings or taller walls with openings may have joints up to 25 ft (8.3 mm) apart. For shorter walls the spacing of contraction joints should be reduced.

For walls 9- to 12-ft- (-3 to 4-m-) high:

• Locate joints within 10 to 15 ft (3 to 5 m) from wall cor-

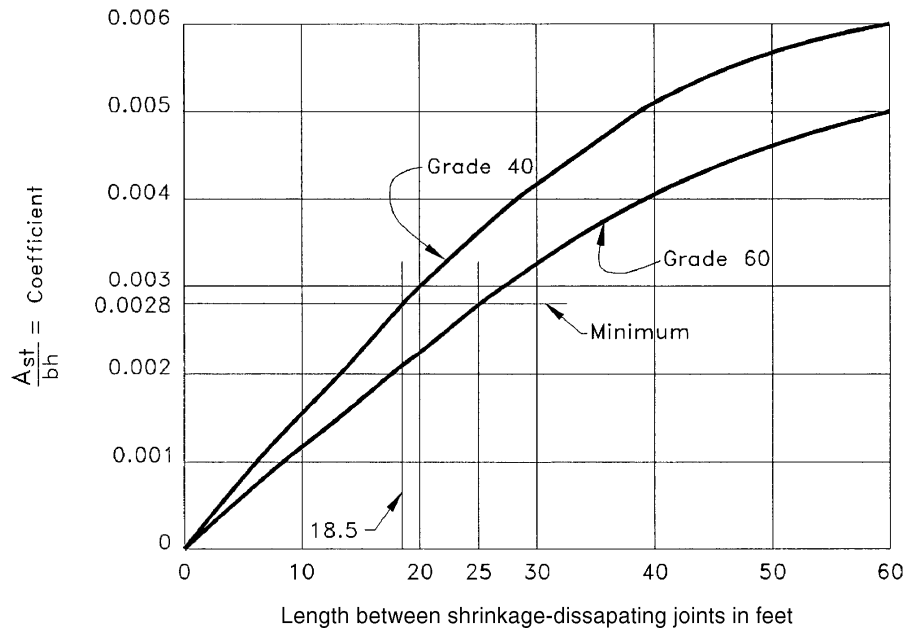


Fig. 9.1—Shrinkage and temperature reinforcement for environmental engineering concrete structures (ACI 350R)

ners if possible. Placing joints closer than 10 ft (3 m) may result in excessive deformation in the joint.

• Locate joints at the edge of openings, changes in wall thickness, or any other obvious locations for a potential vertical crack.

9.2—Contraction joints

Contraction joints are designed to create a plane of weakness where cracks are likely to form. The cross section may be reduced or a bond breaker placed through the wall thickness. Notches that reduce the concrete cross section should be the larger of 20 percent of the wall thickness, or 2 in. (50-mm) deep on each face of the concrete. Dowels or shear keys can be designed into the joint to transfer shear forces.

Contraction joints can be either "full" or "partial." A "full" contraction joint has no reinforcement crossing the joint. A "partial" contraction joint has 50 percent or less of the wall reinforcement crossing the joint. If more than 50 percent of the reinforcement crosses the joint, the joint should be classified as a construction joint.

Contraction joint spacing should not exceed 30 ft (9.1 m) (see ACI 350R). The reinforcement through the joint is usually placed at both faces to permit flexural and tensile load transfer. If the reinforcement at the crack is protected by sealant, it seems reasonable to permit a stress at this location that exceeds the service load limits recommended in ACI 350R. This may be done to achieve the required strength while still providing a movement joint. The reinforcement passing through the joint should extend at least 1.7 times the development length on both sides of the joint. This will reduce the tendency for cracking at the end of the bar lap.

For liquid-retaining structures, the joint should be caulked with an elastomeric sealant suitable for submerged surface on the moist or liquid side to provide a watertight joint. A waterstop can be placed across the joint. It is preferable to use sealant and a waterstop when corrosion protection is desired for the reinforcement crossing the joint.

9.3—Isolation or expansion joints

Expansion and isolation joints include some type of preformed filler, a joint sealant at the liquid face, and a suitable waterstop made of rubber or plastic. Each is selected to allow for the anticipated movement. Refer to Chapter 2 or ACI 504R for additional information on joint materials.

Rubber waterstops permit more joint movement than polyvinylchloride (PVC) waterstops. Either type should be at least 9-in. (225-mm) wide for adequate concrete embedment and should be $^3/_8$- to $^1/_2$-in. (9- to 12-mm) thick.

A preformed joint filler should ideally be able to compress to half of its original width and be able to re-expand to fill the joint as the joint enlarges. Cork, rubber, foam, and other materials conforming to ASTM D 994, ASTM D 1751, and ASTM D 1752 are satisfactory.

Polyurethane rubber elastomers, as described in ACI 504R, may be suitable as sealants if the joint movement is not more than 25 percent of the joint width. Polysulfide sealants may not be suitable in sewage treatment plants, since they have a low resistance to the chemical and biological reactions in sewage treatment. Sealants used in liquid-retaining structures should be suitable for submerged service.

9.4—Construction joints

Construction joints are joints formed by the interruption of concrete placements. Construction joints should be placed at locations planned for expansion or contraction joints. If not placed at planned movement joint locations, the construction joint should be sealed as if it were a contraction joint, unless the plane of the joint is compressed by prestressing.

The joint should be constructed with grooves to permit a place for the sealant, unless a waterstop provides a watertight joint. The face of the construction joint should be made intentionally rough to maintain shear integrity and water tightness. Expanded metal or fine mesh can be used at the vertical stop, or sandblasting of the surface can provide the clean, rough surface desired.

CHAPTER 10—MASS CONCRETE

10.1—Introduction

The primary concern in massive concrete construction is cracking caused by differential volume changes. These result from heat generated by the hydration of cement and cooling of the exterior concrete. Measures commonly used to prevent thermal cracking in mass concrete include:

- Use of moderate heat cement and a suitable pozzolan
- Use of the minimum cement content consistent with structural and durability requirements
- Careful selection of aggregates and mixture proportions to produce concrete having the best resistance to cracking or the greatest tensile strain capacity
- Limiting the rate of placement when no cooling is used
- Precooling concrete ingredients
- Postcooling after the concrete is placed
- Insulating the exposed surfaces during cold weather
- Controlling the time of year when placement is permitted. This is especially true where large seasonal differences exist and the mass can be placed in a short period of time, say two months.

ACI 224R discusses these measures in detail.

Besides these preventive measures, joints should be provided at proper intervals and locations in mass concrete to control random cracking, to accommodate volumetric changes, and to facilitate construction. The two principal types of joints used in mass concrete are contraction and construction joints.

10.2—Contraction joints

The massive concrete structure is constructed in blocks separated by a contraction joint to control the formation of cracks as the mass concrete undergoes a volumetric change due to drying or cooling. In some dams, contraction joints also provide a space for grouting when thermal contraction is complete. Solid grouting at maximum opening prevents any intermediate crack that appears on the concrete face from propagating further into, and possibly through, the dam.

The location and spacing of contraction joints are generally governed by the physical features of the structure, the results of temperature studies, placement methods, the probable concrete mixing plant capacity, and the type of concrete being used. A spacing of 40 to 60 ft (12 to 18 m) has proven satisfactory.

Massive concrete structures, such as concrete gravity dams, are generally designed so that each individual monolith (concrete block) is capable of carrying its load to the foundation without transfer of loads from, or to, adjacent monoliths. Therefore, contraction joints are generally constructed so that no bond exists between the concrete blocks separated by the joint, and no reinforcement is extended across the joint. However, if the load transfer between adjacent concrete elements is required, appropriate reinforcement should be extended across the contraction joint as specified in the design.

10.3—Construction joints

Horizontal or nearly horizontal construction joints are placed in massive concrete structures to divide the structure into convenient working units, or to permit installation of embedded items.

The spacing of construction joints in massive concrete structures is controlled by the type and size of the structure. Other considerations are concrete plant capacity, prevailing climate during construction, construction schedule required, and other temperature control requirements. The vertical spacing of construction joints is generally 5 to 7 $^1/_2$ ft (1.5 to 2.25 m) for gravity dams and 10 ft (3 m) or more in thin arch dams, piers, and abutments.

Proper preparation of construction joints before placing fresh concrete upon the construction joint surfaces is important in assuring the integrity of a concrete structure. There are several methods for preparing construction joints, including green cutting, sandblasting, and high-pressure water jet. Detailed methods of preparation of horizontal construction joints in massive concrete structures are given in ACI 207.1R.

CHAPTER 11—REFERENCES

11.1—Recommended references

AASHTO

American Association of State Highway and Transportation Officials *Standard Specifications for Highway Bridges*, 14th Edition, 1989.

ACI

207.1R Mass Concrete
209R Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures
223 Standard Practice for the Use of Shrinkage-Compensating Concrete
224R Control of Cracking in Concrete Structures
302.1R Guide for Concrete Floor and Slab Construction
311.1R Manual of Concrete Inspection
318 Building Code Requirements for Reinforced Concrete
325.7R Recommendations for Designing Prestressed Concrete Pavements
330R Guide for Design and Construction of Concrete Parking Lots
350R Environmental Engineering Concrete Structures
360R Guide for Construction of Slabs-on-Grade

504R Guide to Joint Sealants in Concrete Structures
504.1R State-of-the-Art Report on Sealing Joints in Tunnels

ASTM
C 14 Specification for Concrete Sewer, Storm Drain and Culvert Pipe
C 76 Specification for Reinforced Concrete Culvert, Storm Drain and Sewer Pipe
C 150 Specification for Portland Cement
C 361 Specification for Reinforced Concrete Low-Head Pressure Pipe
C 443 Specification for Joints for Circular Concrete Sewer and Culvert Pipe, Using Rubber Gaskets
C 595 Specification for Blended Hydraulic Cements
C 877 Specification for External Sealing Bands for Noncircular Concrete Sewer Storm Drain and Culvert Pipe
D 994 Specification for Preformed Expansion Joint Filler for Concrete (Bituminous Type)
D 1751 Specification for Preformed Expansion Joint Fillers for Concrete Paving and Similar Construction (Non-Extruding and Resilient Bituminous Types)
D 1752 Specification for Preformed Sponge Rubber and Cork Expansion Joint Fillers for Concrete Paving and Structural Construction
D 2240 Test Method for Rubber Hardness Property-Durometer Hardness

AWWA
C 300 Reinforced Concrete Pipe, Steel-Cylinder Type, for Water and Other Liquids
C 301 Prestressed Concrete Pipe, Steel-Cylinder Type, for Water and Other Liquids
C 302 Reinforced Concrete Pressure Pipe, Noncylinder Type, for Water and Other Liquids
C 303 Reinforced Concrete Pressure Pipe, Steel Cylinder Type, Pretensioned, for Water and Other Liquids

The above publications are available from the following organizations:

American Association of State Highway and Transportation Officials
444 N. Capitol Street, N.W., Suite 225
Washington, D.C. 20001

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

ASTM
100 Barr Harbor Dr.
West Conshohocken, Pa. 19428

American Water Works Association
6666 West Quincy Avenue
Denver, CO 80235

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APPENDIX A

Temperatures used for calculation of T (after National Academy of Sciences 1974)

The method proposed by the National Academy of Sciences (1974) for expansion joint spacing in single and multistory construction requires location-specific information. Values of T_m, T_w, and T_c for selected locations throughout the United States are given below. This information is taken from the SCSE report (National Academy of Sciences 1974) and was derived from data now available in ASHRAE (1981).

T_m = temperature during the normal construction season in the locality of the building, assumed to be the continuous period in a year during which the minimum daily temperature equals or exceeds 32 F (0 C)

T_w = temperature exceeded, on average, only 1 percent of the time during the summer months of June through September

T_c = temperature equaled or exceeded, on average, 99 percent of the time during the winter months of December, January, and February

Location	T_w	T_m	T_c
Alabama			
Birmingham	97	63	19
Huntsville	97	61	13
Mobile	96	68	2
Montgomery	98	66	22
Alaska			
Anchorage	73	51	-25
Barrow	58	38	-45
Fairbanks	82	50	-53
Juneau	75	48	-7
Nome	66	45	-32
Arizona			
Flagstaff	84	58	0
Phoenix	108	70	31
Prescott	96	64	15
Tucson	105	67	29
Winslow	97	67	9
Yuma	111	72	37
Arkansas			
Ft. Smith	101	65	15
Little Rock	99	65	19
Texarkana	99	65	22
California			
Bakersfield	103	65	31
Burbank	97	64	36
Eureka/Arcata	67	52	32
Fresno	101	63	28
Long Beach	87	63	41
Los Angeles	94	62	41
Oakland	85	57	35
Sacramento	100	60	30
San Diego	86	62	42
San Francisco	83	56	35
Santa Maria	85	57	3
Colorado			
Alamosa	84	60	-17
Colorado Springs	90	61	-1
Denver	92	62	-2
Grand Junction	96	64	-5
Pueblo	96	64	-5
Connecticut			
Bridgeport	90	60	4
Hartford	90	61	1
New Haven	88	59	5
Delaware			
Wilmington	93	62	12
Florida			
Daytona Beach	94	70	3
Ft. Myers	94	74	38
Jacksonville	96	68	29
Key West	90	77	55
Lakeland	95	72	35
Miami	92	75	44
Miami Beach	91	75	45
Orlando	96	72	33
Pensacola	92	68	29
Tallahassee	96	68	25
Tampa	92	72	36
West Palm Beach	92	75	40
Georgia			
Athens	96	61	17
Atlanta	95	62	18

Location	T_w	T_m	T_c
Augusta	98	64	20
Columbus	98	65	23
Macon	98	65	23
Rome	97	62	16
Savannah/Travis	96	67	24
Hawaii			
Hilo	85	73	59
Honolulu	87	76	60
Idaho			
Boise	96	61	4
Idaho Falls	91	61	-12
Lewiston	98	60	6
Pocatello	94	60	-8
Illinois			
Chicago	95	60	-3
Moline	94	63	-7
Peoria	94	61	-2
Rockford	92	62	-7
Springfield	95	62	-1
Indiana			
Evansville	96	65	6
Fort Wayne	93	62	0
Indianapolis	93	63	0
South Bend	92	61	-2
Iowa			
Burlington	95	64	-4
Des Moines	95	64	-7
Dubuque	62	63	-11
Sioux City	96	64	-10
Waterloo	91	63	-12
Kansas			
Dodge City	99	64	3
Goodland	99	65	-2
Topeka	99	69	3
Wichita	102	68	5
Kentucky			
Covington	93	63	3
Lexington	94	63	6
Louisville	96	64	8
Louisiana			
Baton Rouge	96	68	25
Lake Charles	95	68	29
New Orleans	93	69	32
Shreveport	99	66	22
Maine			
Caribou	85	56	-18
Portland	88	58	-5
Maryland			
Baltimore	94	63	12
Frederick	94	63	7
Massachusetts			
Boston	91	58	6
Pittsfield	86	58	-5
Worcester	89	58	-3
Michigan			
Alpena	87	57	-5
Detroit-Metro	92	58	4
Escanaba	82	55	-7
Flint	89	60	-1
Grand Rapids	91	62	2
Lansing	89	59	2
Marquette	88	55	-8
Muskegon	87	59	4
Sault Ste. Marie	83	55	-12
Minnesota			
Duluth	85	55	-19
International Falls	86	57	-29
Minneapolis/St. Paul	92	62	-14
Rochester	90	60	-17
St. Cloud	90	60	-20
Mississippi			
Jackson	98	66	21
Meridian	97	65	20
Vicksburg	97	66	23
Missouri			
Columbia	97	65	2
Kansas City	100	65	4
St. Joseph	97	66	-1
St. Louis	98	65	4
Springfield	97	64	5
Montana			
Billings	94	60	-10
Glasgow	96	60	-25
Great Falls	91	58	-20
Havre	91	58	-22
Helena	90	58	-17
Kalispell	88	56	-7
Miles City	97	62	-19
Missoula	92	58	-7
Nebraska			
Grand Island	98	65	-6
Lincoln	100	64	-4
Norfolk	97	64	-11
North Platte	97	64	-6
Omaha	97	64	-5
Scottsbluff	96	62	-8

Location	T_w	T_m	T_c
Nevada			
Elko	94	61	-13
Ely	90	59	-6
Las Vegas	108	66	23
Reno	95	62	2
Winnemucca	97	63	1
New Hampshire			
Concord	91	60	-11
New Jersey			
Atlantic City	91	61	14
Newark	94	62	11
Trenton	92	61	12
New Mexico			
Albuquerque	96	64	14
Raton	92	64	-2
Roswell	101	70	16
New York			
Albany	91	61	-5
Binghampton	91	67	-2
Buffalo	88	59	3
New York	94	59	11
Rochester	91	59	2
Syracuse	90	59	-2
North Carolina			
Asheville	91	60	13
Charlotte	96	60	18
Greensboro	94	64	14
Raleigh/Durham	95	62	16
Wilmington	93	63	23
Winston/Salem	94	63	14
North Dakota			
Bismarck	95	60	-24
Devils Lake	93	58	-23
Fargo	92	59	-22
Minot	91	—	-24
Williston	94	59	-21
Ohio			
Akron/Canton	89	60	1
Cincinnati	94	62	8
Cleveland	91	61	2
Columbus	92	61	2
Dayton	92	61	0
Mansfield	91	61	1
Sandusky	92	60	4
Toledo	92	61	1
Youngstown	89	59	1
Oklahoma			
Oklahoma City	100	64	11

Location	T_w	T_m	T_c
Tulsa	102	65	12
Oregon			
Astoria	79	50	27
Eugene	91	52	22
Medford	98	56	21
Pendleton	97	58	3
Portland	91	52	21
Roseburg	93	54	25
Salem	92	52	21
Pennsylvania			
Allentown	92	61	3
Erie	88	59	7
Harrisburg	92	61	9
Philadelphia	93	63	11
Pittsburgh	90	63	5
Reading/Scranton Wilkes-Barre	89	61	2
Williamsport	91	61	1
Rhode Island			
Providence	89	60	6
South Carolina			
Charleston	95	66	23
Columbia	98	64	20
Florence	96	64	21
Greenville	95	61	19
Spartanburg	95	60	18
South Dakota			
Huron	97	62	-16
Rapid City	96	61	-9
Sioux Falls	95	62	-14
Tennessee			
Bristol/Tri City	92	63	11
Chattanooga	97	60	15
Knoxville	95	60	13
Memphis	98	62	17
Nashville	97	62	12
Texas			
Abilene	101	65	17
Amarillo	98	66	8
Austin	101	68	25
Brownsville	94	74	36
Corpus Christi	95	71	32
Dallas	101	66	19
El Paso	100	65	21
Fort Worth	102	66	20
Galveston	91	70	32
Houston	96	68	28
Loredo AFB	103	74	32
Lubbock	99	67	11

Location	T_w	T_m	T_c
Midland	100	66	19
Port Arthur	94	69	29
San Angelo	101	65	20
San Antonio	99	69	25
Victoria	98	71	28
Waco	101	67	21
Wichita Falls	103	66	15
Utah			
Salt Lake City	97	63	5
Vermont			
Burlington	88	57	-12
Virginia			
Lynchburg	94	62	15
Norfolk	94	60	20
Richmond	94	64	14
Roanoke	94	63	15

Location	T_w	T_m	T_c
Washington, D.C.			
National Airport	94	63	16
Washington			
Charleston	92	63	9
Huntington	95	63	10
Parkersberg	93	62	8
Wisconsin			
Green Bay	88	59	-12
La Crosse	90	62	-12
Madison	92	61	=9
Milwaukee	90	60	-6
Wyoming			
Casper	92	59	-11
Cheyenne	89	58	-6
Lander	92	58	-16
Sheridan	95	59	-12

ACI 224.3R-95 was submitted to letter ballot of the committee and approved in accordance with ACI balloting procedures.