Guide for Construction of Concrete Pavements

Reported by ACI Committee 325

ACI 325.9R-15

American Concrete Institute
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First Printing
August 2015
ISBN: 978-1-942727-31-6

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The primary focus of this guide is pavement construction. Modern slipform paving techniques and time-proven formed construction procedures are highlighted. Quality control, quality assurance, and construction inspection, as well as the environmental, economic, and societal benefits of concrete pavement, are also presented.

This guide briefly reviews all aspects of concrete pavement construction for highways and, to some extent, local roads, streets, and airfields. Intended for field and office personnel, this guide provides a background on design issues that relate to construction and reviews material selection.

Note that the materials, processes, quality control measures, and inspections described in this guide should be tested, monitored, or performed as applicable only by individuals holding the appropriate ACI certifications or equivalent.

Keywords: concrete pavement; concrete pavement construction; concrete paving; fixed-form paving; paving materials; slipform paving; sustainability.

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ACI 325.9R-15 supersedes ACI 325.9R-91 and was adopted and published August 2015.

CHAPTER 1—INTRODUCTION AND SCOPE

1.1—Introduction

In the United States, concrete pavements have been built for over a century. The first street constructed with concrete was built in Bellefontaine, OH, in 1891; a portion of which, built in 1893, still remains in service. Concrete pavements make up an integral part of the national primary and secondary highway system, farm-to-market road system, city streets, parking lots, and airport runways. Historically, concrete pavements have exhibited a higher initial cost than asphalt pavements, but recent construction and market forces have narrowed that gap. Moreover, the longer service life and lower maintenance costs associated with concrete make it a very attractive and sustainable paving material.

1.2—Scope

This guide briefly discusses the construction of hydraulic cement concrete pavements for highways, streets, local roads, and airfields. Design issues are presented in the context of their impact on construction. Today, the slipform method of paving is preferred for roadway construction. This modern construction method is capable of producing a sustainable, high-quality, smooth pavement that can be placed quickly and economically. This guide will focus on pavement constructed using slipform methods; however, where appropriate, formed pavement constructio
are also discussed.

This guide is intended to serve as a reference for field project management, inspectors, and construction personnel by providing background information, illustrations of best practice, and information helpful in solving day-to-day jobsite problems. Designers and specification writers will also find the guide helpful in preparing contract documents and selecting construction methods that assure quality construction under normal jobsite conditions using established and proven practices. Regardless of the type of equipment used, quality construction depends, in large measure, on the skill of crews involved in the construction process and quality of materials used.

CHAPTER 2—ACRONYMS AND DEFINITIONS

2.1—Acronyms

AAR: alkali-aggregate reactivity
ABS: anti-lock braking system
ACR: alkali-carbonate reactivity
ADTT: average daily truck traffic
ASR: alkali-silica reaction
ATB: asphalt-treated base
BPN: British Pendulum Number
BPT: British Pendulum Tester
CBR: California bearing ratio
COTE: coefficient of thermal expansion
CPX: close proximity
CRCP: Continuously reinforced concrete pavement
CT meter: circular texture meter
CTB: cement-treated base
CTE: coefficient of thermal expansion
DF tester: dynamic friction tester
EAC: exposed aggregate concrete
EICM: Enhanced Integrated Climatic Model
EOT: early-opening-to-traffic
FN: friction number
FWD: falling weight deflectometer
GPR: ground-penetrating radar
HPC: high-performance concrete
HRWR: high-range water reducers
HRWRA: high-range water-reducing admixture
IFI: international friction index
IRI: international roughness index
JPCP: jointed plain concrete pavement
JRCP: jointed reinforced concrete pavement
LCA: life cycle assessment
LCB: lean concrete base
LOI: loss on ignition
LTE: load transfer efficiency
LWAS: lightweight aggregate sand
M-E: mechanistic-empirical
MIT: magnetic imaging tomography
MOR: modulus of rupture
MPD: mean profile depth
MTD: mean texture depth
NCHRP: National Cooperative Highway Research Program
ondestructive testing
NGCS: next-generation concrete surface
OBSI: On-board sound intensity
PCC: portland cement concrete
PI: plasticity index
QA: quality assurance

QC: quality control
R-value: resistance value
SE: sand equivalent
SN: skid number
SPL: sound pressure level
SSD: saturated surface-dry
VPD: vehicles per day
VPM: vibrations per minute

2.2—Definitions

ACI provides a comprehensive list of definitions though an online resource, "ACI Concrete Terminology," http://www.concrete.org/store/productdetail.aspx?ItemID=CT13. Definitions provided herein complement that resource.

dowel—mechanical devices (such as bars or plates) placed across a joint to transfer vertical load while allowing the joint to open and close.

drainage—interception and removal of water from, on, or under an area or roadway.

equivalent single-axle loads (ESAL)—number of equivalent 80 kN (18 kip) single-axle loads used to combine mixed traffic into a single design traffic parameter for thickness design according to the methodology described in the AASHTO design guide (AASHTO 1993).

falling weight deflectometer—device in which electronic sensors measure the deflection of the pavement as a result of an impact load of known magnitude; results can be used to estimate the elastic moduli of subgrade and pavement layers and the load transfer across joints and cracks.

internal curing—a method to supply water throughout a freshly placed cementitious mixture using reservoirs, via prewetted lightweight aggregates, that readily release water as needed for hydration or to replace moisture lost through evaporation or self-desiccation.

jointed plain concrete pavement—hydraulic cement concrete pavement system characterized by short joint spacing and no distributed reinforcing steel in the slab, with or without dowels.

jointed reinforced concrete pavements—hydraulic cement concrete pavement system containing dowels, characterized by long joint spacing and distributed reinforcing steel in the slab to control crack widths.

load transfer device—mechanical means designed to transfer wheel loads across a joint.

pavement structure—combination of subbase, base, rigid slab, and other layers designed to work together to provide uniform, lasting support for imposed traffic loads and distribution of loads to subgrade.

pavement surface friction—the retarding force developed at the tire-pavement interface that resists longitudinal sliding when braking forces are applied to the vehicle tires (Dahir and Gramling 1990; AASHTO 2008b).

shoulder—portion of the roadway contig
with the traveled way provided to accommodate stopped or errant vehicles for maintenance or emergency use, or to give lateral support to the subbase and some edge support to the pavement, and to aid surface drainage and moisture control of the underlying material.

soil support value—index characterizing the relative ability of a soil or aggregate mixture to support traffic loads imposed through flexible and rigid pavement structures.

stabilization—the modification of soil or aggregate layers by incorporating materials that will increase load-bearing capacity, stiffness, and resistance to weathering or displacement, and decrease swell potential.

CHAPTER 3—DESIGN ISSUES RELATING TO CONSTRUCTION

3.1—Introduction

The overall goal of pavement design is to create a structure that is reliable, economical, constructible, and maintainable throughout its design life while meeting or exceeding the needs of the traveling public, taxpayers, and owning agencies (FHWA 2012). In general, the pavement structure should be able to support the expected level of traffic and resist weathering until the next scheduled rehabilitation or reconstruction.

3.2—Design principles

3.2.1 *Introduction*—Design and construction of the roadbed is key to the long-term performance of any pavement structure. A layer of materials that provides a foundation for the riding surface characterizes a roadbed. For concrete pavements, the foundation is typically composed of a base layer on top of the subgrade soil. Proper care and attention should be paid to design and construction of the subgrade and base layers to ensure structural capacity, stability, uniformity, durability, and smoothness of any concrete pavement over its design life. Concrete pavement slabs constructed over the subgrade should have adequate strength and durability to endure exposure to traffic loadings and environmental effects (ACPA 2007).

3.2.2 *Slab characteristics*—Pavement concrete typically has a 28-day flexural strength ranging from 550 to 750 psi (3.8 to 5.2 MPa) or greater, and an elastic modulus ranging from 4 to 6 million psi (28,000 to 41,000 MPa), which helps to provide a high degree of rigidity. This rigidity enables concrete pavements to distribute loads over large areas of the supporting layers. As a result, the stresses on the layers beneath the pavement slab are low.

3.2.3 *Influence of foundation strength on pavement thickness*—The degree of support provided by the foundation for a concrete pavement structure is typically quantified in terms of the modulus of subgrade reaction, or the *k*-value. The magnitude of increase in the *k*-value from the inclusion of base layers in the design of pavements depends on the material type. Normal variations in estimated subgrade or composite *k*-values would not appreciably affect pavement thickness for a typical range of *k*-values (100 to 500 psi/in. 27 to 136 MPa/m]). It is not economical to over-design the base layers for the sole purpose of increasing the *k*-value, as adequate structural designs can be achieved by other means (for example, increasing the slab thickness or concrete strength) (ACPA 2007).

3.2.4 *Influence of foundation stiffness on stresses and strains in concrete pavement slabs*—If a concrete pavement is placed either directly on the subgrade or on any number of base layers, the properties of these foundation layers will directly influence the stresses and strains in the concrete slabs and, in turn, will have an impact on the long-term performance of the pavement structure. If a concrete slab is in complete contact with the foundation, a stiffer support will result in reduced deflections and, thus, reduced stresses under heavy loads. Stiffer support systems, however, will increase deflections and stresses under environmental effects (thermal curling and moisture warping). If a concrete pavement is constructed on a very rigid foundation, the foundation might not conform to the shape of the slab and a significant increase in curling stresses can result.

Higher curling stresses have a more damaging impact when the concrete is relatively young and has not developed the strength required to resist cracking. If the stiffness of the base becomes too great, the curling stresses in the slab will increase, and the potential for midpanel cracking will also increase. The thicker the base layer is, the greater the increase in the support stiffness. The pavement design engineer should recognize that base thickness and stiffness are important properties in the foundation design process (ACPA 2007).

3.2.5 *Drainage*—Drainage is one of the most important factors in pavement design. Water enters the pavement structure either by surface infiltration through cracks, joints, pavement surfaces, and shoulders; or as groundwater from a high water table, aquifers, and localized springs. Where water is trapped within the pavement structure due to inadequate drainage, it reduces the strength of the pavement and subgrade, and also generates high hydrodynamic pressures that might pump out fine material under the pavement, resulting in loss of support. Aggregates used for drainage purposes should satisfy filter requirements. They should be fine enough to prevent the adjacent soil from migrating into them, but coarse enough to carry water with no significant resistance (Huang 2004). In many ways, the pavements built in the United States today, particularly those on interstate highways and routes, are less vulnerable to the detrimental effects of excessive moisture than pavements built in the past because of features such as widened concrete slabs, doweled joints, stabilized base layers, and higher-quality aggregates. Still, at sites with wet climates and poorly draining soils, the need for a subsurface drainage system should be considered. This is particularly true for pavement designs likely vulnerable to moisture-related distress, such as undoweled concrete pavements on untreated aggregate base layers.

3.3—Current design procedures

3.3.1 *PCA design methodology*—The Portland Cement Association (PCA) thickness-design procedure fc highways and streets (PCA 1984) can be applied to jointed plain concrete pavement (JPCP), jointed reinforced concrete pavement (JRCP), and continually reinforced concrete pavement (CRCP). The PCA concrete pavement design procedure evaluates a candidate pavement design with respect to two potential failure modes: fatigue and erosion. The procedure was developed using the results of finite element analyses of stresses induced in concrete pavements by joint, edge, and corner loading. The analyses onsiders the degree of load transfer provided by dowels or aggregate interlock and the degree of edge support provided by a concrete shoulder. The PCA procedure, like the 1993 AASHTO procedure, employs the "composite" k concept in which the design k is a function of the subgrade soil k, base thickness, and base type (granular or cement-treated) (Huang 2004; Hall 2000).

3.3.2 *AASHTO design methodology*—The AASHTO design methodology is the most commonly used rigid pavement design method in the United States (AASHTO 1993). It is based on the empirical equations obtained from the AASHO Road Test, with modifications based on theory and experience. The empirical model for the performance of the JPCP and JRCP sections in the main loops of the AASHO Road Test predicts the log of the number of axle load applications (log W) as a function of the slab thickness, axle type (single or tandem) and weight, and terminal serviceability (Highway Research Board (HRB) 1962). This original model applies only to the designs, traffic conditions, climate, subgrade, and materials of the AASHO Road Test. It has been modified and extended to allow for the estimation of allowable axle load applications to a given terminal serviceability level for conditions of concrete strength, subgrade k-value, and concrete elastic modulus different than those of the AASHO Road Test. The AASHTO design methodology has also been extended to accommodate the conversion of mixed axle loads to equivalent single axle loads (ESALs) of 18 kip (80 kN) through the use of load equivalency factors (Huang 2004; Hall 2000).

3.3.3 *Mechanistic-empirical (M-E) design methodology*—The M-E design procedure uses mechanistic pavement responses such as stress, strain, and deflection; relates them to performance indicators such as cracking, faulting, and roughness; and calibrates them against the field data. The axle load spectra data, rather than ESALs, are used in this design procedure; climatic effects are also considered. An incremental damage concept is used in the M-E design procedure, where the damage is computed monthly and accumulated. The design life is divided into monthly increments and specific materials properties, traffic, and climatic data are used for each increment. The performance criteria considered are joint faulting and transverse cracking for JPCP, punchouts for CRCP, and the international roughness index (IRI) for both pavement types; JRCP is not included in the design methodology. Designs that meet the appropriate performance criteria at a chosen level of reliability are considered feasible from structural and functional standpoints and can be further considered for other evaluations such as life cycle cost analysis and environmental impacts P 2004; AASHTO 2008a).

3.4—Critical design inputs for construction

3.4.1 *General*—The key input parameters in any concrete pavement design procedure related to construction are outlined (3.4.1.1 through 3.4.1.3).

Table 3.4.1.1—Subgrade soil types and approximate support values (ACI 330R)

Type of soil	Support	*k*, psi/in. (kPa/mm)	California bearing ratio	*R*	Soil support value
Fine-grained soils in which silt and clay-size particles predominate	Low	75 to 120 (20 to 32)	2.5 to 3.5	10 to 22	2.3 to 3.1
Sands and sand-gravel mixtures with moderate amounts of silt and clay	Medium	130 to 170 (35 to 46)	4.5 to 7.5	29 to 41	3.5 to 4.9
Sand and sand-gravel mixtures relatively free of plastic fines	High	180 to 220 (49 to 60)	8.5 to 12	45 to 52	5.3 to 6.1

3.4.1.1 *Subgrade*—Although the *k*-value of the foundation (natural soil and embankment) can be measured by plate-bearing tests, it is usually estimated from correlations with soil type, soil strength measures such as the California bearing ratio (CBR), or by back-calculation from deflection testing on existing pavements (Table 3.4.1.1). The *k*-value is the primary subgrade design variable for concrete pavements (ACPA 2007).

3.4.1.2 *Base and subbase*—A base course provides a stable platform for construction of concrete slab, improves the smoothness achieved in the paving of the slab, could serve as a drainage layer, and protects the foundation from frost penetration. Some types of bases also significantly reduce bending stresses and deflections in the slab and improve load transfer at joints and cracks. The estimated elastic modulus of the base, its erodibility, its potential for friction and bond with the concrete slab, and its drainability are factors considered in characterizing the support to the concrete slab and the quality of subsurface drainage (ACPA 2007).

3.4.1.3 *Concrete material properties*—For the purpose of pavement thickness design, concrete is characterized by its flexural strength as well as its modulus of elasticity. Concrete flexural strength is usually characterized by the 28-day modulus of rupture (MOR) from third-point loading tests of beams, or it may be estimated from compressive strengths (Eq. (3.5.3.3)). The corresponding elastic modulus E can also be measured, but is usually estimated from strength data. In addition to its strength and stiffness, durability of concrete mixture is important to the long-term performance of the pavement (Hall 2000).

3.4.2 *Subgrade considerations*

3.4.2.1 *Load-bearing capacity*—Design methods were devised based on tests that provided an index number related to soil strength that was most commonly considered to represent the shear strength. Some of these test methods and their associated index values are discussed in this section (ACPA 2007).

3.4.2.1.1 *California-bearing ratio (CBR) test*—The CBR test measures the force required to penetrate a soil surface by a circular piston with a 3 in.^{2} (19 cm^2) piston area. The index (CBR) value is the percent of an established reference value for 0.1 and 0.2 in. (2.5 and 5.0 mm) penetration. The reference value of 100 was originally considered to represent the resistance of a well-graded crushed
of preparing specimens and conducting the test are gıven ın AASHTO T193 and ASTM D1883.

3.4.2.1.2 *Resistance value (R-value) test*—The *R*-value test is a measure of the material stiffness by way of resistance to plastic flow. This laboratory test was developed as an improved CBR test. Samples are prepared to represent the worst-case scenario during testing and are confined on all sides in the testing apparatus, resulting in a triaxial state of stress. The *R*-value is the ratio of the vertical load applied to the resultant lateral pressures. Standard *R*-value test methods are described in AASHTO T190 and ASTM D2844/D2844M.

3.4.2.1.3 *Resilient modulus of subgrade soil*—The stiffness, as an estimate of the modulus of elasticity, E, is measured by this test. The modulus of elasticity is the ratio of stress applied to the strain produced for a slowly applied load. The resilient modulus is the stress divided by the strain for a rapidly applied load. The standard resilient modulus test is given in AASHTO T307.

3.4.2.1.4 *Modulus of subgrade reaction (*k*-value)*—This bearing test, conducted in the field, provides an index to rate the support provided by a soil layer directly beneath the concrete slab. Most concrete pavement design is based on the *k*-value, as used in the Westergaard (1933) equations. The *k*-value is defined as the reaction of the subgrade per unit area of deformation and is typically given in psi/in. (kg/cm^3). Details on conducting the plate-bearing field tests are given in AASHTO T221 and T222 or in ASTM D1195/D1195M (repetitive test) and D1196/D1196M (no repetitive test). The elastic *k*-value (k_e), as determined from the repetitive plate-bearing test (ASTM D1195/D1195M) is a higher value because it considers only the elastic deformation in the *k*-value computation.

Because of slow productivity in conducting plate-bearing tests and their relatively high costs and labor intensiveness, very few agencies routinely conduct them. Instead, most agencies obtain *k*-values through correlations with other properties or through the back calculation of deflection data using the falling weight deflectometer (FWD). The use of the FWD enables collection of a large number of data points that can help evaluate the subgrade variability over a project.

3.4.2.1.5 *Cone penetrometer*—A cone penetrometer is a device used to measure the strength of in-place soil. The test results can be used to estimate the soil shear strength, CBR, and *k*-value. Because these tests are rapid and essentially nondestructive, they are ideally suited for on-site construction, and testing over large areas can evaluate uniformity. The penetrometer is driven into the ground at either a constant ate or by dropping a specific hammer over a given distance. The measured values (load needed to drive the penetrometer or blow counts per unit of depth) are then correlated to CBR, shear strength, or soil modulus value. Profiles of the changes in soil strengths across the project area can be obtained by plotting the load or blow counts versus depth. This can be used to check the depth of stabilization and to find soft or stiff layers.

3.4.2.2 *Volume stability*

3.4.2.2.1 *Introduction*—It is essential to quantify the expansion and shrinkage of soils as the overall volume change, or differential volume changes from point-to-point, can result in serious damage to a pavement structure, particularly in areas where soils remain relatively dry until wetted by an infrequent rainy period. Tests used to quantify potential volume stability issues are discussed in the following sections (ACPA 2007). ASTM D4829 gives an expansive index of soils and, based on the test results, evaluates soils from very low to very high expansion potential.

3.4.2.2.2 *California-bearing ratio and* R*-value tests*—Expansion tests are usually conducted in conjunction with the CBR and *R*-value tests. In both instances, the test specimen is compacted to a predetermined density at proper moisture content in a mold and a supply of water is made available. Surcharges, equal to the weight of the cover material that will overlay the soil in the ultimate pavement structure, are applied to the top of the specimen. The expansion that occurs during a given soaking period is measured as the change in length of the specimen. The pressure exerted by the expanding soil can be measured by means of a calibrated restraining gauge.

3.4.2.2.3 *Sand equivalent (SE) tests*—The SE test is a rapid field-testing method to detect the presence of undesirable clay-like materials in soils and aggregate materials. This method tends to magnify the volume of clay present in a sample in proportion to its detrimental effects. Details on this test method are given in AASHTO T176 and ASTM D2419. Natural sand and crushed stone have SE values of approximately 80, whereas very expansive clays have values ranging from 0 to 5.

3.4.3 *Subgrade strength and working platform*—Due to the ability of a concrete pavement to spread loads over large areas, the highest subgrade stresses will normally occur during the construction phase of a concrete pavement or the base layer. Once in place, the base layer and the concrete pavement protect the subgrade from high-stress contact by loads. Thus, the required strength of a subgrade is typically dictated by providing a stable working platform to construct successive layers. Research performed by the Wisconsin Department of Transportation has concluded that a minimum CBR value of 6 in the top 24 in. (610 mm) of subgrade provides an adequate working platform while limiting subgrade rutting under construction traffic to 0.5 in. (13 mm) or less (Crovetti and Schabelski 2002).

Compacting the subgrade to a density that provides an adequate working platform for construction equipment will provide adequate subgrade strength for the in-service concrete pavement. The AASHTO T099 field test is recommended to characterize a subgrade for accepta
departments of transportation recommend values ranging from 84 to 100 percent, but a value of 95 percent is most often specified and thus is recommended for most applications (ACPA 2005).

3.4.4 *Obtaining uniform support*—For a subgrade to provide a reasonably uniform support, the four major causes of nonuniformity should be addressed (ACPA 2007):

1) Expansive soils
2) Frost-susceptible soils
3) Wet soils
4) Pumping

3.4.4.1 *Expansive soils*—Expansive soils change volume with changes in moisture content. Expansive soils that can swell enough to cause problems are clays with a plasticity index (PI) greater than approximately 25 (ASTM D4318). Experience has indicated that volume changes of clays with a medium to low degree of expansion (PI less than 25) are not a significant concern for concrete pavements, especially if selective construction grading operations are performed, such as cross-hauling and blending of soil types to minimize or eliminate abrupt changes in soil character along the alignment. Experience has also indicated, however, that uncontrolled shrinkage and swelling of expansive soils can lead to increased stresses in concrete pavements due to nonuniform support, which accelerates pavement deterioration and negatively impacts pavement smoothness.

Construction-related factors that can further aggravate performance issues related to expansive soils include:

a) Compacting expansive soils where they are too dry, resulting in the possibility that the soil will absorb moisture and expand after the subgrade is prepared.

b) Placing a pavement on a subgrade with widely varying moisture contents, allowing differential volume change of the soil to take place along the road alignment.

c) Creating nonuniform support by ignoring abrupt changes between soil types with different capacities for volume change along the road alignment. The volume change that could occur with potentially expansive soils depends on several factors, including:

1. The moisture variation that will take place in the subgrade throughout the year or from year to year, dictated by the climate. Generally, the pavement will protect the grade to a certain degree and reduce moisture variation in an underlying subgrade, as long as the soil is not capable of drawing water from below through capillary suction.
2. The effect of weight of the soil, base layer(s), and the pavement above the expansive soil. Tests indicate that surcharge loads can reduce soil swell (Holtz and Gibbs 1956).

d) Moisture and density conditions of the expansive soil during paving.

e) Knowledge of the interrelationships between these factors leads to the selection of economical and effective control methods.

3.4.4.2 *Frost action*—Frost action is a phenomenon that
in the winter and early spring in northern climates. All surface soils undergo a certain amount of frost action, the magnitude of which depends on the prevailing local climate and precipitation level. Frost action can be divided into two phases: 1) freezing of soil water; and 2) thawing of soil water.

For pavements, frost action is a major concern where either the freezing phase is accompanied by noticeable heaving of the road surface or the thawing phase is accompanied by noticeable softening of the roadbed.

Frost heave is defined as the surface distortion caused by volume expansion within the soil or pavement structure, where water freezes and ice lenses form within the freezing zone. Ice lenses are formed when moisture, diffused within soil highly susceptible to capillary action, accumulates in a localized zone; ice initially accumulates within small collocated pores, wedging the soil apart, causing frost heave. Frost-susceptible soils are subgrade or subbase materials in which segregated ice will form, causing frost heave, under the required conditions of moisture supply and temperature.

Note that design considerations for controlling frost heave are not necessarily identical to those for controlling subgrade softening. For example, a soil with high frost-heave potential will not necessarily exhibit the maximum amount of subgrade softening. Field investigations have shown that frost damage due to frost heave, in the form of differential frost heave, has affected performance more than subgrade softening.

Subgrade softening due to frost action is not a major concern for subgrade design, as uniform support is more important than strength. Subgrade softening, however, can aggravate pumping in pavements that are constructed without adequately addressing pumping potential. Frost design for concrete pavements is concerned with providing uniform subgrade conditions. This is achieved by eliminating the moisture conditions that lead to objectionable differential frost heave, which occur where subgrade soils vary abruptly from non-frost-susceptible to highly frost-susceptible silts, at cut-fill transitions or at silt pockets, and where groundwater is close to the surface or water-bearing strata are encountered.

3.4.4.3 *Wet soils*—Wet soils can be encountered during construction for reasons ranging from a naturally high water table, seasonal rainfall, or changes in drainage conditions due to construction. Regardless of the cause, in-place wet soils should be addressed before the base layer(s) or the concrete pavement is constructed over the subgrade.

The simplest ways to mitigate the problems due to wet soils are to construct drains before construction or to let the subgrade dry out prior to constructing a base or concrete pavement on the subgrade. Construction and scheduling constraints, however, may make these solutions no longer feasible. The other procedures fall into three categories:

1. *Enhancement*—A method of removing excess moisture in wet soils by providing drainage via trenches or toe drains at the lowest point(s), compacting the subgrade using heavy equipment that forces the excess moisture out of the subgrade, or adjusting the moisture content through chemical modification.
2. *Reinforcement/separation*—A method of removing excess water by using geosynthetics. Geosynthetics are thin, pliable sheets of textile material of varying permeability. The effectiveness of geosynthetics depends on the type; the intended function, such as filtration, separation, or reinforcement; in-place soil conditions; and installation techniques.
3. *Substitution*—A method of removing excess water by removing unsuitable, unstable, or excessively wet soils and replacing it with borrow material, or by covering the wet soil with suitable material to develop necessary uniformity and stability.

3.4.4.4 *Pumping resistance*—Pumping is the forceful displacement of a mixture of soil and water through slab joints, cracks, and pavement edges. Continued, uncontrolled pumping will eventually displace enough soil to result in the loss of uniform support, leaving the slab corners and ends unsupported. This nonuniform support condition results in premature cracking at slab corners and pavement roughness in the form of faulted transverse joints. In the worst case, loads deflect concrete slabs enough to pump water and fine soil particles through joints and onto the surface of the pavement, where visible stains become evident.

For pumping to occur, these conditions should be present:

a) Significant slab deflections due to heavy loads, poor load transfer at joints, or both

b) Presence of water between the pavement and the subgrade or base layer

c) Presence of a fine-grained subgrade or erodible base material

The subgrade materials most prone to pumping are high-plasticity silts and clays. Unstabilized (granular) base materials prone to pumping are generally those with 15 percent or more fines passing through the No. 200 (75 μm) sieve.

The two most effective factors for mitigating pumping are using doweled transverse pavement joints and properly graded base courses. Using nonerodible or stabilized bases can mitigate pumping. Unstabilized bases meeting AASHTO M155 requirements will effectively prevent pumping in pavements carrying even the highest traffic volumes, assuming that other design features are appropriately selected. Using a properly graded base (stabilized or unstabilized) eliminates the fines that will go into suspension, whereas using dowels eliminates rapid differential deflection caused by frequent heavy loads.

3.4.5 *Base/subbase considerations*—The base for a concrete pavement is the untreated or treated granular layer constructed on the prepared subgrade and upon which the concrete slab is placed. A base layer provides benefits to both the construction and performance of concrete pavements. From the construction perspective, the base layer provides a stable working platform for construction equipment, which enables the contractor to provide a smoother pavement and achieve a more consistent pavement thickness than might be possible if constructing directly over the subgrade. From the performance perspective, the base layer provides uniform support to the pavement and prevents pumping of fines. econdary benefits are their help in controlling volume changes for expansive or frost-susceptible soils and reducing excessive differential frost heave. A base layer can also be used as a drainage layer; however, a careful balance between drainability and stability should be achieved. Base thickness for road and highway pavements is usually in the range of

4 to 6 in. (102 to 150 mm). In addition, granular subbase or select material up to 24 in. (600 mm) is sometimes used over weak subgrades or to provide frost protection.

The tendency in recent years has been to use the terms "subbase" and "base" interchangeably in reference to a single layer between the slab and subgrade. If another layer, such as lower-quality gravel or a filter layer separates the base from the subgrade, this second layer is referred to as the subbase.

3.4.5.1 *Types of bases*—Various types of bases have been successfully used under concrete pavements. These include unstabilized (granular) bases, and stabilized (treated) bases that include cement-stabilized bases (cement-treated or lean concrete), asphalt-treated bases, and fly ash or lime-treated bases (ACPA 2007).

Regardless of the specific base considerations, the best results are obtained by:

a) Selecting base materials that will not contribute to excessive pavement deflections under service loads and will remain stable over the design life.

b) Treating the base to ensure that it does not cause excessive friction or induce bonding to pavement slabs.

c) Specifying gradation or material controls that will ensure consistent quality.

1. *Unstabilized (granular) bases*—Unstabilized (granular) bases are the most common type of base for roadways and highways. If designed and constructed properly, unstabilized bases make an excellent support layer for concrete pavements for all types of roadways and highways. Their primary advantage is their relatively low cost compared to stabilized bases.

As a minimum, an unstabilized base should meet the requirements of AASHTO M147 (AASHTO M155 may be used if pumping is a major concern). These factors define materials that are composed of a good unstabilized base:

i. Maximum particle size of no more than one-third of the base thickness

ii. Less than 15 percent passing the No. 200 (75 µm) sieve

iii. Plasticity index of 6 or less

iv. Liquid limit of 25 or less

v. Los Angeles abrasion resistance (AASHTO T096 or ASTM C131/C131M) of 50 percent or less

vi. Target permeability of 150 to 350 ft/day (45 to 107 m/day)

The principal criterion for creating a good unstabilized base is to limit the amount of fines passing the No. 200 (75 µm) sieve. Too many fines will result in the base holding water more readily, which increases the potential for erosion and pumping.

2. *Stabilized bases*—Stabilized bases generally refer to base materials that are bound by either p

blended cement or asphalt binders. Stabilized bases fall into three categories: cement-treated, lean concrete, and asphalt-treated.

Compared with unstabilized bases, stabilized bases provide a higher degree of support to the pavement slabs (that is, a higher *k*-value). They can significantly reduce slab stresses and improve load transfer at pavement joints, especially for pavements with undoweled joints and plain concrete slabs (Colley and Humphrey 1967; Henrichs et al. 1989; Ioannides and Korovesis 1990). Their relatively high stiffness and potential bonding can, however, increase curling stresses in the slab, which can reduce the service life if not accounted for in the design process, usually by using shorter joint spacing.

3. *Cement stabilized bases*—Cement stabilized bases fall into two categories: 1) cement-treated base (CTB); and 2) lean concrete base (LCB). Fly ash or slag cement can be included in either a CTB or LCB.

Cement-treated bases have a much drier consistency, contain less cement, and are best controlled using compaction or density requirements rather than strength requirements. Because a CTB layer is best controlled using compaction or density requirements, common requirements are a level of compaction between 96 and 100 percent of the maximum density (determined by AASHTO T134 or ASTM D558). Although there is typically no strength requirement on CTB, targeting a 7-day compressive strength from 300 to 800 psi (2.1 to 5.5 MPa) assures long-term durability (Halsted et al. 2007). Cement-treated bases typically require approximately 2 to 5 percent cement by weight. The granular material typically has no more than approximately 35 percent passing the No. 200 (75 µm) sieve—a PI of 10 or less. A maximum particle size of 0.75 to 1 in. (19 to 25 mm) is preferable to permit accurate grading of the base material.

Lean concrete bases contain more cement and water than CTBs, but they contain less cement than conventional concrete. Lean concrete has the same appearance and consistency as conventional concrete, and can be placed using conventional paving equipment. For a lean concrete base, typical specifications require a 7-day compressive strength from 750 to 1200 psi (5.2 to 8.3 MPa) and an air content of 4.0 to 12.0 percent. The air content of an LCB may be used to prevent exceeding the maximum strength as well as for freezing-and-thawing resistance.

4. *Asphalt-treated bases (ATB)*—The design criteria for asphalt-treated soils and aggregate combinations focus on compaction, stability, and gradation parameters. An asphalt coating on granular material provides a membrane, which prevents the infiltration of water and thereby reduces the tendency of the material to lose strength in the presence of water. Because asphalt-treated bases rely on adhesion to bind aggregate particles together, stripping is a primary concern. Stripping

defined as the separation of the asphalt binder from the aggregate surface resulting in destabilization of the base.

The following design/material considerations are advisable for using asphalt-treated bases:

i. Asphalt-treated base may use a lower grade of asphalt binder than is required for asphalt surfaces

ii. A binder content of 4 to 4.5 percent is considered typical for ATB

iii. Durability of aggregates is an important requirement for ATB. The maximum aggregate size is typically 3/4 in. (19 mm)

iv. Aggregates meeting moderate soundness requirements and a maximum freezing-and-thawing weight loss, in water, of 10 percent and a loss in water-alcohol solution of up to 45 percent according to AASHTO T103 may be considered adequate

3.4.5.2 *When to use a base*—Base layers are appropriate if a stable and uniform construction platform will benefit construction; if the combination of subgrade soil type, water availability, and high-speed, heavy traffic are at a level conducive to cause pumping and associated distresses; or both. Therefore, a base layer is required for heavily traveled pavements, particularly those with a large amount of truck traffic.

Pavements for slow-moving trucks or light-traffic pavements, such as residential streets, secondary roads, and parking lots, are not necessarily prone to the development of pumping. Factors to consider include:

a) *Traffic*—Pavement designers often use the rule-of-thumb that a pavement expected to carry 50 trucks or fewer per day does not require a base layer to prevent pumping. The American Concrete Pavement Association recommends that pavements designed to carry less than 1 million 18 kip (80 kN) equivalent single axial loads (ESALs) over their service life do not require base layers to prevent pumping (ACPA 2007).

b) *Natural drainage*—A subgrade soil that is naturally free-draining typically will not pump as the water percolates through the subgrade and does not remain under the pavement where it can transport fines in suspension. Pavements can be built directly on natural subgrade soils with this character as long as the soil is satisfactory in other critical regards, such as frost action or expansion (ACPA 2007).

c) *Qualified subgrade soils*—Subgrade soils with less than 45 percent passing a No. 200 (75 μm) sieve and with a PI of 6 or less are adequate for moderate volumes of heavy truck traffic without the use of a base layer. In these cases, it is advisable to use doweled joints to prevent differential deflections at slab joints (ACPA 2007).

Note that increasing the thickness of concrete pavement slabs is not an acceptable measure to prevent pumping. Without proper preventive measures, pumping can occur even in the thickest of pavements (ACPA 2007).

3.5—Pavement design considerations

3.5.1 *Subgrade support*—The ability to provide uniform support conditions is the most important p
from a subgrade layer. The major causes of nonuniform support conditions, which include expansive soils, differential frost heave, pumping, and wet soils, have been discussed in detail in previous sections.

3.5.2 *Base friction*—Concrete slabs continually move throughout their life due to shrinkage, thermal effects, and moisture gradients. If the slab moves, any bonding between the slab and base develops resisting forces. These forces can lead to the development of restraint cracking. High friction between the concrete pavement and the underlying base layer is, therefore, not desirable. Also, as stiffness of the base increases, friction between the base and pavement slab increases. This in turn induces higher tensile stresses within the slab, which increases the probability of cracking. Stiffer bases by themselves do not lead to less contact with the slab unless curling or warping is occurring. The potential for bonding between concrete and the base layer can be minimized with the application of a bond breaker (ACPA 2007).

3.5.3 *Concrete materials*

3.5.3.1 *Durability*—Concrete pavements should be able to resist environmental effects such as temperature changes, freezing-and-thawing cycling, and the action of deicing chemicals in northern climates. Under these conditions, it is essential that the mixture have a low water-cementitious materials ratio (*w/cm*), adequate cement, sufficient amount of entrained air, and an adequate curing period. The amount of air entrainment needed for concrete resistant to freezing and thawing varies with maximum aggregate size and exposure conditions. The recommended amount of entrained air is given in ACI 211.1.

In addition to making the hardened concrete pavement resistant to freezing and thawing, recommended amounts of entrained air improve the properties of the concrete while it is still in the plastic state by reducing segregation, increasing workability without adding additional water, and reducing bleeding.

Aggregate selected for paving should be resistant to freezing-and-thawing deterioration (D-cracking) and not prone to alkali-silica reaction (ASR). Aggregates that meet the requirements of state highway departments for concrete paving should be acceptable in most cases. Type F fly ash, as well as other supplementary cementitious materials such as slag cement, can be used as effective mineral admixtures to help prevent deterioration due to ASR. High concentration of soil sulfates can also cause premature failure of concrete pavements. For high-sulfate soils that may be in contact with concrete pavement, recommendations of ACI 201.2R should be followed (ACI 325.12R).

3.5.3.2 *Coefficient of thermal expansion (COTE)*—The COTE is a measure of expansion of material or contraction with temperature. The COTE of concrete is approximately 3.3 to 5.5 με/°F (8 to 12 με/°C). Because aggregate comprises approximately 70 percent of the concrete, aggregate type has the greatest influence on the COTE of concrete.

Although the COTE is not featured in earlier design methods (for example, PCA [1984]; AASHTO [1993]), it s a major design input in the most recent M-E procedure available from AASHTO (2008b). The importance of the use of COTE in design is discussed in the following (NCHRP 2004):

a) The magnitude of curling stresses caused by temperature differences along the cross section of the pavement slab is very sensitive to the COTE of concrete. Under certain conditions, curling stresses comprise approximately 50 percent or more of the critical stresses experienced by a loaded pavement slab and, thus, affect slab cracking and CRCP punchouts, which is the area enclosed by two closely spaced transverse cracks, a short longitudinal crack, and the edge of the pavement or a longitudinal joint. The COTE, therefore, plays an important role in optimizing JPCP joint design, CRCP reinforcement, and in accurately computing pavement stresses and joint and crack load transfer efficiency (LTE).

b) It is an important factor in designing joint sealant reservoirs and in selecting joint sealant materials.

c) The COTE is also critical in affecting the cracking width and spacing in CRCP over the entire design life; the crack width directly affects the crack LTE, which is the key factor in punchout development.

3.5.3.3 *Strength*—Whereas loads applied to concrete pavements generate both compressive and flexural stresses in the slabs, it is the flexural stresses that are more critical, as the service loads can induce flexural stresses that may exceed the flexural strength of the slab. Because the strength of concrete is much lower in tension than in compression, the MOR is often used in concrete pavement thickness design. The MOR test is performed according to ASTM C78/C78M (third-point loading).

Concrete strength is a function of the amount and type of cementitious material, the *w/cm*, and the air content of hardened concrete. The aggregates used should be clean to ensure good aggregate-to-paste bond and should conform to the quality requirements of ASTM C33/C33M, ASTM C330/C330M, or the quality requirements established by the state departments of transportation for concrete pavement. Regardless of when the pavement is opened to traffic, the strength should be checked to verify that the design strength has been achieved.

Design methods are generally based on the results of the third-point loading test. Because the required thickness for pavement design changes approximately 0.5 in. (13 mm) for a 70 psi (0.5 MPa) change in MOR, knowledge of flexural strength is essential for economic design. MOR values for 28- or 90-day strengths are normally used in design. The use of 90-day strengths could be justified because of the limited loadings that pavements receive before this early age and could be considered as the long-term design strength. If the facility is not opened to traffic for a long period, later strengths can be used, but the designer should be aware of earlier environment and construction loadings that may cause pavement stresses that could exceed the early-age strength of concrete. Under normal conditions, concrete that has an MOR of 550 to 700 psi (3.8 to 4.8 MPa) is most economical.

While the design of concrete pavement is gener
on the tensile strength of concrete, as represented by the flexural strength, it is more common to use compressive strength testing in the field for quality-control acceptance purposes and in the laboratory for mixture design purposes. Although a useful correlation between compressive (f_c') and flexural (MOR) strengths is not readily established, an approximate relationship between them is given to facilitate these purposes by the formula

$$\text{MOR} = a_1\gamma_{conc}^{0.5}f_c'^{0.5} \quad \text{(ACI 209R)} \qquad (3.5.3.3)$$

where γ_{conc} is the concrete unit weight, and a_1 varies from 0.6 to 1.0 for units of psi (0.012 to and 0.20 for units of MPa). If desired, correlations between flexural and compressive strength can be developed for specific mixtures. The strength of concrete should not be exceeded by environmentally-induced stresses such as curling and warping, which may be critical during the first three days after placement (ACI 325.12R).

3.5.3.4 *Shrinkage*—Drying shrinkage of hardened concrete is an important factor that governs the performance of portland cement concrete (PCC) pavements. Drying shrinkage affects the crack development in CRCP as well as long-term performance of load transfer across the cracks. For JPCP, the primary effect of drying shrinkage is slab warping caused by differential shrinkage due to moisture gradient along the thickness of the pavement slab, which leads to increased cracking potential (NCHRP 2004; AASHTO 2008a).

Drying shrinkage-related inputs in AASHTO (2008b) include:

a) *Ultimate shrinkage*—Shrinkage strain that PCC will experience upon prolonged exposure to drying conditions (40 percent relative humidity). Equations to estimate ultimate shrinkage are available AASHTO (2008b). Ultimate shrinkage at the termination of the AASHTO T160 test (at 64 weeks) can also be used.

b) *Time to develop 50 percent of ultimate shrinkage*—Unless more reliable information is available, a value of 35 days (recommended by ACI 209R) is recommended to be used for the time required to develop 50 percent of ultimate shrinkage strains. If the AASHTO T160 test method is used to estimate the shrinkage in the laboratory, the time required to develop 5 percent of the shrinkage refers the number of days to reach half of the ultimate shrinkage after the specimen has been removed from a completely soaked condition.

c) *Anticipated amount of reversible shrinkage*—Refers to the reversible shrinkage strain upon rewetting of PCC. A value of 50 percent is recommended unless more reliable information is available.

d) *Mean monthly relative humidity data*—Data provided by the Enhanced Integrated Climatic Model (EICM) from the weather station data.

3.6—City streets

3.6.1 *Introduction*—Design and construction of pavements for city streets should entail both long service life and low maintenance. To ensure that these concrete pavements
rve our needs into the future, it is important that all design and construction aspects be incorporated.

Concrete pavement performs well for city streets because of its durability while being continuously subjected to traffic and, in some cases, severe weather conditions. Because of its relatively high stiffness, concrete pavements spread the imposed loads over large areas of the subgrade and are capable of resisting deformation caused by passing vehicles. Concrete pavements exhibit high wear resistance along with providing a light-reflective surface that improves driver visibility.

3.6.2 *Pavement types*—The most common type of concrete for streets and other low-volume roads is JPCP. This type of pavement does not contain reinforcement and may or may not have dowels at transverse joints. Cracking is controlled by short joint spacing. Aggregate interlock at these joints will help transfer the load across the joint and could help mitigate faulting. Pavements that contain dowels at the transverse joints are used to provide additional load transfer and prevent faulting.

Jointed reinforced concrete pavement can be used when joint spacing is greater than needed to effectively control shrinkage cracking. Distributed steel reinforcement is used to control the opening of intermediate cracks between joints. The sole function of distributed steel reinforcement is to hold together the fracture faces if cracks form.

3.6.3 *Intersections*—Concrete intersections offer long life, reduction in maintenance costs, high reflectivity at lighted intersections, high and durable skid-resistant surfaces, and the elimination of washboarding and rutting, which is a common problem associated with asphalt.

The volume of traffic at an intersection can be up to double that of either street. Where light traffic intersects heavy traffic, a thickness for heavy traffic is adequate. In the case where heavy traffic is in both directions, an additional 1 in. (25 mm) of thickness is required. The following indicated thicknesses are generally accepted for intersections: 6 in. (150 mm) for passenger cars and light trucks; 8 in. (203 mm) for moderate volumes of heavy trucks and buses; and 9 or 10 in. (225 or 250 mm) for industrial traffic and high volumes of heavy trucks. Concrete pavements should cover at least the entire functional area of the intersection. Additionally, the subgrade where trucks and buses are anticipated should contain a thin, well-compacted, granular subbase.

3.6.4 *Basis of pavement thickness design*—Pavement design methods are discussed in 3.3 of this guide. Methods used for streets and low-volume roads should have been validated and calibrated by road tests, pavement studies, and surveys of pavement performance that included thin-section, low-volume applications. The most commonly used methods are the AASHTO design guide, which was developed from performance data obtained at the AASHO road test (AASHTO 1993); and the Portland Cement Association (PCA) design procedure, which is based on pavement resistance to fatigue and deflection effects on the subgrade (PCA 1984). The PCA procedure is recommended for use in instances of overload conditions. These thickness design methods can be used for plain or reinforced pavements because the presence or lack of distributed re
no significant effect on loaded slab behavior as it pertains to thickness design.

3.6.5 *Street classification and traffic*—Comprehensive traffic studies made within city boundaries can supply data for the design of municipal pavements. A practical approach is to establish a street classification system. Streets of similar character may have similar traffic densities and axle-load intensities. The street classifications used in this guide are discussed in 3.6.5.1 through 3.6.5.6 (ACI 325.12R).

3.6.5.1 *Light residential*—These are short streets in subdivisions and may dead end with a turnaround. Light residential streets serve traffic to and from a few houses (20 to 30). Traffic volumes are low—less than 200 vehicles per day (VPD) with two to four two-axle, six-tire trucks (and heavier) in two directions (excluding two-axle, four-tire trucks). Trucks using these streets will generally have a maximum tandem axle load of 34 kip (150 kN) and an 18 kip (80 kN) maximum single-axle load. Garbage trucks and buses most frequently constitute the overloads on those types of streets.

3.6.5.2 *Residential*—These streets carry the same type of traffic as light residential streets but serve more houses (up to 300), including those on dead-end streets. Traffic generally consists of vehicles serving the homes plus an occasional heavy truck. Traffic volumes range from 200 to 1000 VPD with average daily truck traffic (ADTT) of 10 to 50. Maximum loads for these streets are 22 kip (98 kN) for single axles and 34 kip (150 kN) for tandem axles. Thicker pavement sections may be required on established bus routes in residential areas.

3.6.5.3 *Collector*—Collectors serve several subdivisions and may be several miles long. They may be bus routes and serve truck movements to and from an area even though they are not through-routes. Traffic volumes vary from 1000 to 8000 VPD with ADTT of approximately 50 to 500. Trucks using these streets generally have a 26 kip (115 kN) maximum single-axle load and a 44 kip (200 kN) maximum tandem-axle load.

3.6.5.4 *Business*—Business streets carry movements through commercial areas from expressways, arterials, or both. They carry nearly as much traffic as arterials; however, the percentage of trucks and axle weights generally tends to be less. Business streets are frequently congested and speeds are slow due to high traffic volumes, but with a low (400 to 700) ADTT; average traffic volumes vary from 11,000 to 17,000 VPD. Maximum loads are similar to collector streets.

3.6.5.5 *Arterials*—Arterials bring traffic to and from expressways and serve major movements of traffic within and through metropolitan areas not served by expressways. Truck and bus routes and state- and federal-numbered routes are usually on arterials. For design purposes, arterials are divided into minor arterial and major arterial, depending on traffic capacity and type. A minor arterial may have fewer travel lanes and carry less volume of total traffic, but the percentage of heavy trucks may be greater than that on a six-lane major arterial. Minor arterials carry 4000 to 15,000 VPD with a 300 to 600 ADTT. Major arterials carry approximately 4000 to 30,000 VPD with a 700 to 1500 ADTT. Maximum loads for minor arterials are 26 kip (115 kN) for single axles and 44 kip (200 kN) for tandem axles. Major arterials have maximum loads of 30 kip (130 kN) for single axles and 52 kip (230 kN) for tandem axles.

3.6.5.6 *Industrial*—Industrial streets provide access to industrial areas or parks. Total traffic volume may be in the lower range but the percentage of heavy axle loads is high. Typical VPD are approximately 2000 to 4000 with 300 to 800 ADTT. Truck volumes are not much different than the business class; however, the maximum axle loads are heavier—30 kip (133 kN) for single axles and 52 kip (230 kN) for tandem axles.

3.6.6 *Thickness determination for streets*—Proper selection of the slab thickness is a crucial element of a concrete pavement design. Inadequate thickness will lead to cracking and premature loss of serviceability. Suggested thickness for the design of low-volume concrete roads is found in ACI 325.12R.

Small changes in concrete thickness or an increase in concrete strength can have a significant effect on pavement fatigue life. For this reason, tolerances on pavement thickness are important.

For overloaded traffic and cases related to variable support conditions that may require the use of dowel bars at the joints, thickness designs should be developed using the PCA procedure (PCA 1984).

3.6.7 *Utilities*—Many times during the construction of new subdivisions and commercial properties, utilities will be placed in the right-of-way of the pavement area. This is mainly done to accommodate maintenance and additions to the utility system. Present and future needs should be evaluated and provisions made for them. Careful planning in the laying of these utilities can avoid the unnecessary task of removal of existing concrete pavements for work on utilities.

3.6.8 *Integral curbs and gutters*—Integral curbs are constructed with the pavement in a single operation with all concrete work being done simultaneously. They can be constructed using forms or slipform pavers.

Curbs and gutters provide many benefits to concrete streets. They confine pavement structures, outline the edges of pavements, provide easily definable borders between traveled and untraveled surfaces, and help to contain low-speed traffic within the edges of the pavements.

Pavement life can be extended through the added thickness given to edges of concrete pavements by integral curbs. These curbs increase strength and stiffness and reduce deflections induced by traffic loads.

3.6.9 *Street widths*—Street widths vary according to the traffic that the street is designed to carry. In normal cases, the minimum recommended width is 25 ft (7.6 m) with a maximum transverse slope of 2 percent. Traffic lanes are customarily 10 to 12 ft (3 to 3.6 m) wide. Due to driver safety, lane widths over 12 ft (3.6 m) are not recommended because experience has shown that some drivers will attempt to pass on wider, single lanes.

Typically, lane widths of 12 ft (3.6 m) are used to accommodate buses. Locations for bus lanes may be ei
curb or in the median. With curb-side bus lanes, bicyclists and right turners are usually permitted. Median lanes are normally located on wide boulevards and are less likely to be congested by other traffic. Parking lanes along the curb are usually 7 to 8 ft (2 to 2.5 m) wide. In areas where passenger vehicles are the norm, a lane width of 7 ft (2 m) is recommended and those lanes, which should accommodate trucks, a width of 8 ft (2.5 m) is recommended. On major streets, parking lanes are 10 to 12 ft (3 to 3.6 m) wide and they can also be used as travel or turning lanes. For those streets where parking is prohibited, an extra 2 ft (0.6 m) of width is generally provided along the curb as nontraveled space.

3.7—Drainage issues

3.7.1 *Cross slopes*—Because a concrete surface maintains its shape, flatter grades (but typically at least 2 percent for most high-volume facilities) can be used to provide adequate surface drainage. This minimizes the amount of earthwork during construction and may result in a greater spacing of inlets.

Cross slopes (or crowns) built into concrete pavements provide for draining the water from the surface rapidly. Generally, a minimum slope of 1 percent is recommended. Slopes greater than 1 percent can be used provided they are consistent with road safety and drivability of the vehicles using them. For paved sections constructed between and abutting buildings, the type of center drain or inverted crown used in alleys may be best. The design should be for uniform slopes toward the center of not less than 1 percent.

3.7.2 *Integral curbs and gutters*—Curbs and gutters collect water from crowned pavements and convey it to points of collection. This also allows for a reduction of water, which would otherwise penetrate underneath the pavement. It is important that the aprons have adequate hydraulic capacity to carry runoff from most rainstorms. Making aprons wider reduces the opportunity for rainwater to move down through joints between curbs and pavements.

In areas where there are storm sewers, the flow in gutters is diverted through inlets built into the curbs and gutters. In areas where the rains are infrequent, the inlets may only be gaps in the curb to allow the runoff to exist. Inlets, which are more or less anchored in place, should be isolated from curbs and gutters.

CHAPTER 4—MATERIAL SELECTION

4.1—Introduction

All materials used in concrete pavements should be furnished from supply sources approved before shipments are started and used only so long as the materials continue to meet the requirements of the contract documents. The basis of approval of such sources should be the ability to consistently produce materials of the quality and in the quantity required. Unless local conditions such as quality, cost, and general availability indicate a need for modification, materials should meet the standard specifications of the governing agency or those listed in 4.2.

4.2—Foundation materials

4.2.1 *Subgrade soil*—Subgrade is the underlying surface of soil over which the pavement will be constructed. The required pavement thickness and performance of the pavement will depend in large part on the strength and unifor-

Table 4.2.1—Improvements in modulus of subgrade reaction *k (ACI 330R)**

Subgrade *k*-value, psi/in. (kPa/mm)	Subbase thickness			
	4 in. (100 mm)	6 in. (150 mm)	9 in. (225 mm)	12 in. (300 mm)
Granular aggregate subbase				
50 (14)	65 in. (1625 mm)	75 in. (1875 mm)	85 in. (2125 mm)	110 in. (2750 mm)
100 (27)	130 in. (3250 mm)	140 in. (3500 mm)	160 in. (4000 mm)	190 in. (4750 mm)
200 (54)	220 in. (5500 mm)	230 in. (5750 mm)	270 in. (6750 mm)	320 in. (8000 mm)
300 (81)	320 in. (8000 mm)	330 in. (8250 mm)	370 in. (9250 mm)	430 in. (10,750 mm)
Cement-treated subbase				
50 (14)	170 in. (4250 mm)	230 in. (5750 mm)	310 in. (7750 mm)	390 in. (9750 mm)
100 (27)	280 in. (7000 mm)	400 in. (10,000 mm)	520 in. (13,000 mm)	640 in. (16,000 mm)
200 (54)	470 in. (11,750 mm)	640 in. (16,000 mm)	830 in. (20,750 mm)	—
Other treated subbase				
50 (14)	85 in. (2125 mm)	115 in. (2875 mm)	170 in. (4250 mm)	215 in. (5375 mm)
100 (27)	175 in. (4375 mm)	210 in. (5250 mm)	270 in. (6750 mm)	325 in. (8125 mm)
200 (54)	280 in. (7000 mm)	315 in. (7875 mm)	360 in. (9000 mm)	400 in. (10,000 mm)
300 (81)	350 in. (8750 mm)	385 in. (9625 mm)	420 in. (10,500 mm)	490 in. (12,250 mm)

mity of the subgrade. Preliminary information on the engineering properties of the soil at a particular project site can be obtained from the U.S. Department of Agriculture soil survey maps or geotechnical investigations conducted for adjacent roads or buildings; however, for final design of the pavement structure, soil conditions and subgrade properties should be determined by appropriate soil testing. Refer to 3.4.2 for a discussion of properties and tests used to characterize subgrade soils.

Uniform subgrade support is the goal of proper site preparation. The designer can require grading operations to blend soil types to improve uniformity. Properties of the subgrade soil can be improved by compaction, stabilization, and moisture control.

Although the typical values of *k* for various subgrade soil types and moisture conditions are given in Table 3.4.1.1, they should be considered as a guide only. Low-strength subgrades can be stabilized to upgrade the support rating listed in Table 3.4.1.1. Where granular subbase materials are used, there may be a moderate increase in the modulus of subgrade reactions, or *k*-values that can be incorporated in the thickness design. The suggested increase in *k*-values for design purposes is shown in Table 4.2.1. Usually, it is not economical to use a granular subbase for the sole purpose of increasing *k*-values or reducing the concrete pavement thickness.

4.2.2 *Subbase material*—As discussed in 3.4.3, a subbase is a layer of select material placed under a concrete slab primarily for bearing uniformity, pumping control, and erosion resistance. The select material may be unbound or stabilized. It is more important, however, that the subbase be well drained to prevent excess pore pressure to resist pumping-induced erosion than to achieve a greater stiffness in the overall pavement.

The subbase serves many important purposes, and in some cases is used to provide a stable surface for cc diency. This could be applicable in wet-freeze climates where the use of a stabilized subbase is recommended; water can easily collect under a slab due to freezing-and-thawing action.

A contractor may find it advantageous to use a subbase or a stabilized subgrade to provide a more stable working platform during construction. Although subbases are not generally used for local streets and roads, they can be effective in controlling erosion of the subgrade materials where traffic conditions warrant such measures. Subbase materials include untreated, treated, and drainable material.

4.2.3 *Base*—Unlike asphalt pavement systems, it is not always necessary that concrete highway pavements have both a base and subbase layer. Airfield pavements handling heavy aircraft loads often have both a base and a subbase (Kohn et al. 2003). Base materials include untreated, treated, and drainable materials.

4.3—Pavement concrete materials

4.3.1 *Introduction*—Concrete mixtures for paving should be proportioned following accepted proportioning procedures. The concrete should be designed to produce the specified strength, provide adequate durability, rideability, and skid resistance. The mixture should provide adequate workability so that it can be efficiently placed, finished, and textured with the equipment the contractor will use. Paving mixtures should use the largest nominal maximum size aggregate consistent with the requirements for placing the concrete. This will minimize shrinkage cracking, enhance load transfer, and produce the most economical concrete. Mixtures with excessive natural fine aggregates should be avoided, as these mixtures tend to increase water demand and the potential for uncontrolled shrinkage cracking. Construction of a test section, using the production equipment and paving train to be used on the actual job is also recommended.

4.3.2 *Strength and durability*—Because concrete strength is a function of the type and amount of cementitious material (hydraulic cement plus supplementary cementitious materials), the completeness of the hydration, and the water-cementitious materials ratio (*w/cm*) selected for the mixture, water-reducing admixtures, or internal curing also can be used to increase strength while maintaining sufficient workability of the fresh mixture. Internal curing can improve strength at 3, 7, 28, and 90 days, and improve durability and the interfacial transition zone, as well as reduce cracking, permeability, and warping. Detailed information on hydraulic cements and supplementary cementitious materials are found in ACI 225R, 232.1R, 233R, and 234R. Aggregates should be clean to provide good aggregate-to-paste bond and should conform to the quality requirements of ASTM C33/C33M. Cubical-shaped coarse aggregates have been shown to increase flexural strength. Air entrainment is a critical item in concrete durability. Curing also ensures proper strength gain and durable concrete pavement. Mixtures designed for high early strength can be provided if the pavement should be used by construction equipment or opened to traffic earlier than normal (4 to 24 hours). Regardless of when the pavement is opened to traffic, the concrete strength should be checked to verify that the design strength has been achieved.

For concrete pavement applications, flexural strength is used by most design procedures as the most direct indicator of load capacity. Flexural strength values indicate the tensile strength at the bottom of the slab where wheel loads induce tensile stresses (ACI 325.11R). Flexural strength tests from ASTM C78/C78M are very sensitive to the beam fabricating and testing procedures. Many agencies realize this shortcoming and use compressive strength tests (ASTM C39/C39M) to evaluate concrete for acceptance and opening (ACI 325.11R). While design of concrete pavement is generally based on the tensile strength of the concrete, as represented by the flexural strength, it may be useful to use compressive-strength testing in the field for quality-control acceptance purposes and in the laboratory for mixture design purposes. Refer to 3.5.3.3 for a useful correlation between compressive strength and flexural strength.

If desired, however, a specific flexural-to-compressive strength correlation can be developed for specific mixtures. The strength of the concrete should not be exceeded by environmentally-induced stresses (curling and warping), which may be critical during the first 72 hours after placement.

4.3.3 *Concrete materials*—Concrete can be described as a paste (cement and water) binding together aggregate particles. Paving concrete in common use today makes use of supplementary cementitious materials (SCMs), chemical admixtures, and air entrainment to improve the properties of the paste and hardened concrete. Future use may include high-performance concrete (HPC) using materials for internal curing to improve durability and viscosity-modifying admixtures to improve extrusion properties for slipform paving applications. Aggregates make a major contribution to the properties and performance of paving concrete. Aggregates are usually local materials and include a wide variety of rock types as well as recycled materials.

During construction, other materials such as dowel bars, tie bars, and reinforcement may be incorporate pavement system. Other materials such as joint sealants, evaporation retarders, and curing materials are also used in the construction process. All these materials affect the construction process and the long-term performance of a concrete pavement. The following sections of this chapter describe these materials and the influences they have on the construction process as well as the performance of the pavement.

4.3.3.1 *Cementitious materials*—Hydraulic cement is the most important component of the concrete paste. Cement combines with water (thus the term “hydraulic cement”) to form the paste, which gives the concrete initial workability, and then reacts to harden and bind aggregate particles together.

There are several types of hydraulic cement, but portland cement, which is cement meeting the requirements of ASTM C150/C150M or AASHTO M085, is the most widely used hydraulic cement. Intergrinding produces blended hydraulic cements or blended cement with supplementary cementitious materials. ASTM C595/C595M or AASHTO M240 is used to specify blended cements. ASTM C1157/C1157M is a performance specification for hydraulic cement. Portland cements meeting the ASTM C150/C150M Type I and Type II requirements are the most common cements used in paving applications (Table 4.3.3.1). Type III cement is commonly used for patching and may be used in mixtures for fast-track paving. Type IV is a low-heat-of-hydration cement, but it has been largely replaced in modern mixtures by incorporating supplementary cementitious materials. Type V cements are used in areas where soils have high sulfate concentrations. The manufacture of hydraulic cement clinker releases carbon dioxide into the atmosphere, so environmental responsibility dictates that procedures and materials should be used in the concrete mixture that minimizes the clinker content of concrete. The use of blended cements and ASTM C1157/C1157M general-purpose and moderate-sulfate-resistant cements is growing in some areas of the country.

4.3.3.1.1 *Portland cement*—Portland cement is hydraulic cement that meets the chemical composition and physical property requirements of ASTM C150/C150M or AASHTO M085. The fundamental difference between portland cement types is the chemical composition and physical properties. These specifications cover five major types of portland cement; however, not all types may be available in any given geographic area. While the standard specifications contain requirements for air-entraining portland cements and Type IV low-heat cements, other technologies have replaced these cement products and they are generally not available in the United States. ASTM C150/C150M cautions that “in advance of specifying the use of cement other than Type I,” the specifier should “determine whether the proposed type of cement is, or can be made, available.”

The cement type or types to be used should conform to the requirements of the contract documents and those of the specified ASTM or AASHTO standards. Even though cements from different sources meet the requirements of e specification, there may be subtle differences that might impact concrete performance or compatibility with SCMs and admixtures used in the concrete mixture. For this reason, many agencies specify that all cement used in a given approved mixture should be from the same source. In the event that its use from more than one source is antici-

Table 4.3.3.1—Hydraulic cement types

	Types						
Cement specification	General purpose	Moderate heat of hydration	High early strength	Low heat of hydration	Moderate sulfate resistance	High sulfate resistance	Resistance to alkali-silica reactivity (ASR)
Portland cements ASTM C150/C150M AASHTO M85	Type I	Type II(MH)	Type III	Type IV	Type II	Type V	Low alkali option
Blended hydraulic cements ASTM C595/C595M AASHTO M240	IS(x) IP(x) IT(Ax)(By)	IS(<70)(MH) IP(x)(MH)		IP(x)(LH)	IS(<70)(MS) IP(x)(MS)	IS(<70)(HS) IP(HS)	Low reactivity option
Performance hydraulic cements ASTM C1157/C1157M	GU	MH	HE	LH	MS	HS	Option R

Note: IS = Type IS (portland blast-furnace slag cement); IP = Type IP (portland-pozzolan cement); IT = Type IT (ternary blend); GU = Type GU (hydraulic cement for general construction); HE = Type HE (high early strength); MS = Type MS (moderate sulfate resistance); HS = Type HS (high sulfate resistance); MH = Type MH (moderate heat of hydration); and LH = Type LH (low heat of hydration).

pated, mixture designs for each source to be used should be developed and submitted for approval.

The term "Type I/II portland cement" is a frequently used and misunderstood term. Type I/II is not an actual ASTM designation, but instead denotes that the cement being represented has a C_3A content of 8 percent or less and meets all of the requirements of both ASTM C150/C150M Type I and Type II. This is particularly helpful to the concrete producer who has limited silo storage capacity, and for whom the ability to inventory a single cement that meets both ASTM C150/C150M Type I and Type II specifications in one silo is a convenience. "Type II modified" is another term frequently misunderstood. The word "modified" can mean modified by such characteristics as lower alkali content, coarser fineness, or significantly lower C_3A content. If the term "Type II modified" is used, the purchaser should request that the manufacturer define the modification employed to ensure that the product is appropriate for the intended application.

The speed with which a concrete mixture sets and develops strength is a result of the hydration characteristics of a particular combination of cement, supplementary cementitious materials, and admixtures. Cements play a major role in setting, strength gain, and heat generation. These properties depend on the interaction of the individual chemical compounds in the cement, SCMs, and admixtures used (ACI 225R).

Finely ground cements, such as Type III cement, have increased surface areas, which allows for more cement contact with mixing water. This additional contact results in cement that hydrates faster and generates more heat from the hydration process. Due to the higher fineness of Type III cement (which provides a much greater surface area for the hydration reaction), early strength normally develops faster than other types of portland cement. Type III has been successfully used in cold weather and selective fast-track paving operations and in pavement patching.

Due to the faster hydration reaction associated with Type III cement, early setting of the concrete mixture should be expected. The materials engineer and contractor should be aware of this. Tests should be conducted using the same cement that the contractor will use in construction.

With proper proportioning, concretes using Type I or Type II portland cement also can produce adequate characteristics for accelerated-concrete paving. To develop adequate early strength, concrete made from these cements will usually require either accelerating admixtures or low water-cement ratios (*w/c*), dense graded aggregate proportions, and the use of curing as an adjunct to standard external curing.

4.3.3.1.2 *Blended cement*—Blended hydraulic cements are usually made by grinding portland-cement clinker with calcium sulfate (gypsum) and a quantity of a suitable reactive material such as slag cement, fly ash, silica fume, or raw or calcined natural pozzolan. They may also be made by blending the finely ground ingredients. For specification purposes, portland and blended hydraulic cements are designated by type depending on their chemical composition and properties. The availability of a given type of cement can vary widely among geographical regions. The use of blended cements, though presently small, is growing in response to needs for use in concrete requiring special properties, conservation of energy, and raw materials.

The use of silica fume in blended cements has also attracted interest. Typically, the properties of cements containing silica fume as a blending material may be expected to be the same as if the silica fume were added separately. As with any blended cement, there will be a loss in flexibility in mixture proportioning with respect to the exact amount of silica fume in a given concrete mixture.

ASTM C595/C595M applies to blended cements that are intended for use in general concrete construction. AASHTO M240 is the equivalent AASHTO specification. Blended cements covered by these specifications are Type IS (portland blast furnace slag cement) and Type IP (portland-pozzolan cement). The percentage of slag or pozzolan in the product is expressed by a whole number based on the mass of the final blended product. The percentage is designated by adding the suffix (x) to the type designation. Additionally, these blended cements may have special properties such as air-entraining cement (A), moderate sulfate resistance (MS), moderate heat of hydration (MH), and low heat of hydration (LH). These special properties will be specified by adding the suffix (A), (MS), (MH), or (LH) to the blended type designation. Type IT designates a ternary blend with (Ax) and (By) indicating the percentage of the A and B SCM components of the blend.

4.3.3.1.3 *Performance cement*—Performance-based cements provide an option to the standard prescriptive-based cements such as those of ASTM C150/C150M and ASTM C595/C595M. These cements can provide equal performance to many of the standard cements used on previous work. Many of the performance-based cements produced today are more environmentally friendly and require less energy to produce.

ASTM C1157/C1157M classifies cements based on specific requirements for general use, high early strength, resistance to attack by sulfates, and heat of hydration. Optional requirements are provided for the property of low reactivity with alkali-silica-reactive aggregates (ASTM C1157/C1157M). This specification covers six basic types of cements and is: Type GU (hydraulic cement for general construction), Type HE (high early strength), Type MS (moderate sulfate resistance), Type HS (high sulfate resistance), Type MH (moderate heat of hydration), and Type LH (low heat of hydration). There is no equivalent AASHTO specification for performance-based cement.

4.3.3.2 *Special-purpose cements*—In addition to portland and blended cements, other cements may be available for use in pavements.

4.3.3.2.1 *Rapid-hardening hydraulic cements*—Rapid-hardening hydraulic cements are similar in composition to other blended cements, except that they are specially formulated with functional additions to provide design strengths in approximately 3 to 12 hours. Regular blended cements normally provide design strengths in 7 to 28 days. Very-early-strength blended cements can be used in the same application as portland and blended cement. They are usually used in applications where early-strength development is highly beneficial, such as in repair applications or accelerated paving applications. ASTM C1600/C1600M provides performance requirements for four categories of rapid hardening hydraulic cements:

a) *Type URH*—Ultra rapid hardening, for use where ultra-high early strength is desired

b) *Type VRH*—Very rapid hardening, for use where very high early strength is desired

c) *Type MRH*—Medium rapid hardening, for use where midrange rapid hardening, high early strength is desired

d) *Type GRH*—General rapid hardening, for use if the higher strength property of Type VRH or Type MRH cement is not required

4.3.3.2.2 *Magnesium phosphate cements*—Magnesium phosphate cements are rapid hardening, no
cements that are primarily used in highway and airport pavement repairs. They may be two-part cements consisting of a dry powder and a phosphoric acid liquid with which the powder should be mixed, or they may be one-component products to which only water is added.

4.3.3.2.3 *Expansive cement*—Paving applications for expansive or shrinkage-compensating cements covered by ASTM C845 are very limited. Some agencies have used them in bridge decks and pavement patching applications. These cements are designed to expand a small amount during the first few days of hydration to offset the effects of later drying shrinkage. They are used to reduce cracking resulting from drying shrinkage, and to cause stressing of reinforcing steel. Shrinkage-compensating cements manufactured in the United States depend on the formation of a higher-than-usual amount of ettringite during hydration of the cement to cause the expansion. The expansive ingredient—an anhydrous calcium sulfoaluminate—can be purchased separately. Magnesium oxide or calcium oxide, which are used in Europe and Japan, can also be used as expansive agents.

4.3.4 *Supplementary cementitious materials*—Supplementary cementitious materials (SCMs) are materials that, if used in conjunction with portland or blended cement, contribute to the properties of the hardened concrete through hydraulic or pozzolanic activity, or both. In the past, the term "mineral admixture" was used to refer to these materials, but because of the large quantities of these materials used in modern concrete, this term has fallen out of favor.

Several types of SCMs can be used in pavement concrete. These materials have gained popularity over the years due to the desirable concrete properties that result from their use, and sustainability issues. In particular, SCMs have shown to improve the durability of concrete through a variety of mechanisms. Some SCMs are incorporated in blended cements discussed in 3.2.2, but they are also introduced as a separately batched material at the concrete mixing plants. Supplementary cementitious materials are classified into four primary groups:

1) Fly ash (ASTM C618)
2) Slag cement (ASTM C989/C989M)
3) Silica fume (ASTM C1240)
4) Natural pozzolan—volcanic ash and calcined clays (ASTM C618)

4.3.4.1 *Fly ash*—Fly ash—a by-product of coal combustion from electric power plants—became available in large quantities in the 1930s. Initially, fly ash was used as a partial mass or volume replacement of portland cement, which is the expensive cementing component in concrete. However, as the use of fly ash increased, the potential for improved properties of concrete containing fly ash was recognized. Fly ash is now used in concrete for many reasons, including improvements in workability of fresh concrete, reduction in temperature rise during initial hydration, improved resistance to sulfate attack, reduced expansion due to alkali-silica reaction (ASR), reduced cost, and other contributions to the durability and strength of hardened concrete.

commonly used specification for fly ash, ASTM C618, establishes two classifications: Class C and Class F, both of which are used in concrete pavements. Used in paving concrete, fly ash will lower water demand, improve workability, and increase long-term concrete strength. All fly ashes display pozzolanic properties, but some Type C ashes possess varying degrees of cementitious value without the addition of calcium hydroxide or portland cement because they contain some lime. Type F has the added benefit of minimizing the effects of ASR, but has less cementitious value to contribute to the hydraulic reaction. Fly ash containing higher levels of calcium (typically classified as Type C) usually comes from sub-bituminous and lignite coals, whereas fly ash containing lower levels of calcium (typically classified as Type F) comes from bituminous or anthracite coals.

Fly ash in concrete makes efficient use of the products of hydration of portland cement: 1) solutions of calcium and alkali hydroxide, which are released into the pore structure of the paste, then combine with the silica in the fly ash, and form a cementing medium similar to that formed by the portland cement; and 2) the heat generated by hydration of portland cement is an important factor in initiating the reaction of the fly ash. In properly cured concrete containing fly ash, fly-ash reaction products fill in the spaces between hydrating cement particles, thus lowering the concrete permeability to water and aggressive chemicals. The slower hydration reaction rate of many fly ashes compared to portland cement limits the amount of early heat generation and the detrimental early temperature rise in massive structures. Properly proportioned fly ash mixtures impart properties to concrete that may not be achievable through the use of portland cement alone.

4.3.4.1.1 *Effects of fly ash on properties of fresh concrete*—Using fly ash in a concrete mixture can impact the workability, bleeding characteristics, set time, finishing characteristics, heat generation characteristics, and air entrained in the fresh concrete. These effects can be helpful or detrimental to pavement construction depending on project requirements.

4.3.4.1.1.1 *Workability*—The absolute volume of cement plus fly ash normally exceeds that of cement in similar concrete mixtures not containing fly ash. While it depends on the proportions used, this increase in paste volume produces a concrete with improved plasticity and better cohesiveness (Lane 1983). In addition, the increase in the volume of fines from fly ash can compensate for deficient aggregate fines. The generally spherical shape of fly ash particles normally permits the water in the concrete to be reduced for a given workability (Brown 1980). Ravina (1984) reported on a Class F fly ash that reduced the rate of slump loss compared to non-fly ash concrete in hot-weather conditions. Fly ash particles vary in size from less than 1 µm to more than 100 µm, with the typical particle size measuring under 20 µm (Mehta 1984), the smaller size favorably influencing workability and density.

4.3.4.1.1.2 *Bleeding*—Using fly ash in air-entrained and non-air-entrained concrete mixtures usually reduces bleeding by providing greater surface area of solid particles and a lower water content for a given worka

4.3.4.1.1.3 *Time of setting*—The use of fly ash may extend the time of setting of concrete if the portland cement content is reduced. Jawed and Skalny (1981) found that Class F fly ashes retarded early C_3S hydration. Grutzeck et al. (1985) also found retardation with Class C fly ash. The setting characteristics of concrete are influenced by ambient and concrete temperature, cement type, source, content, fineness, water content of the paste, water-soluble alkalis, use and dosages of other admixtures, the amount of fly ash, and the fineness and chemical composition of the fly ash (Plowman and Cabrera 1984). If these factors are given proper consideration in the concrete mixture proportioning, an acceptable time of setting can usually be obtained. The actual effect of a given fly ash on time of setting may be determined by testing if a precise determination is needed or by observation if a less precise determination is acceptable.

4.3.4.1.1.4 *Finishability*—Where fly ash concrete has a longer time of setting than concrete without fly ash, such mixtures should be finished at a later time than mixtures without fly ash. Failure to do so could lead to premature finishing, which can seal the bleed water under the top surface, creating a plane of weakness. Longer times of setting may increase the probability of plastic shrinkage cracking or surface crusting under conditions of high evaporation rates. Using very wet mixtures containing fly ash with significant amounts of very light unburned coal particles or cenospheres can cause these particles to migrate upward and collect at the surface, which may lead to an unacceptable appearance. Some situations are encountered where the addition of fly ash results in stickiness and consequent difficulties in finishing. In such cases, the concrete may have too much fine material or too high an air content.

4.3.4.1.1.5 *Temperature rise*—The heat generated during cement hydration has an important bearing on the rate of strength development and on early stress development due to differential volume change in concrete. The rate of hydration and heat generation depends on the quantity, fineness, and type of cement; mass of the structure; method of placement; temperature of the concrete at the time of placement; and curing temperature. Using fly ash as a portion of the cementitious material in concrete can reduce the temperature rise. As the amount of portland cement is reduced, the heat of hydration of the concrete is generally reduced (Mather 1974). However, some Class C fly ashes do contribute to early temperature rise in concrete (Dunstan 1984). When heat of hydration is of critical concern, the proposed concrete mixture should be tested for temperature rise.

4.3.4.1.1.6 *Air entrainment*—The use of fly ash in air-entrained concrete will generally require a change in the dosage rate of the air-entraining admixture. Some fly ashes with loss on ignition (LOI) values of less than 3 percent require no appreciable increase in air-entraining admixture dosage. Some Class C fly ashes may reduce the amount of air-entraining admixture required, particularly for those with significant water-soluble alkalis in the fly ash (Pistilli 1983). To maintain constant air content, admixture dosages should usually be increased, depending on the carbon content as ndicated by LOI, fineness, and amount of organic material in the fly ash. When using a fly ash with a high LOI, more frequent testing of air content at the point of placement is desirable to maintain proper control of air content in the concrete. Required air-entraining admixture dosages may increase with an increase in the coarse fractions of a fly ash.

Adjustments should be made as necessary in the admixture dosage to provide concrete with the desired air content at the point of placement.

Meininger (1981) and Gebler and Klieger (1983) have shown that there appears to be a relationship between the required dosage of air-entraining admixture to obtain the specified air content and the loss of air in fly ash concrete with prolonged mixing or agitation prior to placement. Those fly ashes that require a higher admixture dosage tend to suffer more air loss in fresh concrete. When this problem is suspected, air tests should be made as the concrete is placed to measure the magnitude of the loss in air and to provide information necessary to properly adjust the dosage level for adequate air content at the time of placement.

The loss of air depends on a number of factors, such as properties and proportions of fly ash, cement, and fine aggregate; duration of mixing; and type of air-entraining admixture used (Gaynor 1980; Meininger 1981). Neutralized vinsol resin air-entraining admixtures did not perform well with fly ashes having high LOI values. For a given fly ash, the most stable air content was achieved with the cement-fine-aggregate combinations that had the highest air-entraining admixture requirement. However, a change in fly ash that requires a higher admixture dosage to obtain the specified air content is more likely to cause loss of air if the mixture is agitated or manipulated for a period of time.

High LOI of fly ash is often, but not always, an indicator of the likelihood of air-loss problems; so far, the problem seems to be confined to the lower-CaO, Class F fly ashes. The foam-index test (ACI 233R) is a rapid test that can be used to check successive shipments of fly ash to detect a change in the required dosage of air-entraining admixture in concrete.

4.3.4.1.2 *Effects of fly ash on properties of hardened concrete*—The hardened properties of concrete are also affected by the use of fly ash in the mixture. These effects may improve or have a detrimental impact on the long-term performance of the pavement.

4.3.4.1.2.1 *Compressive strength and rate of strength gain*—Strength at any given age and rate of strength gain of concrete are affected by the characteristics of the particular fly ash, the cement with which it is used, and their proportions in the concrete. Compared with concrete without fly ash, concrete containing a typical Class F fly ash may develop lower strength at 7 days of age or before if tested at room temperature (Abdun-Nur 1961). If equivalent 3- or 7-day strength is desired, it may be possible to provide the desired strength by using accelerators or water reducers, or by changing the mixture proportions (Swamy et al. 1983). After the rate of strength contribution of portland cement slows, the continued pozzolanic activity of fly ash contributes to increased strength gain at later ages, provided moisture is available to continue the hydration react

to 4.6 and 5.3 for further information on curing) being a reliable method; therefore, concrete containing fly ash with equivalent or lower strength at early ages may have equivalent or higher strength at later ages than concrete without fly ash. Other tests, comparing concrete with and without fly ash showed significantly higher performance for the concrete containing fly ash at ages up to 10 years (Mather 1965).

Class C fly ashes often exhibit a higher rate of reaction at early ages than Class F fly ashes, and typically give very good strength results at 28 days as well. In fact, Cook (1981) reported that some Class C fly ashes were as effective as portland cement on an equivalent-mass basis. However, certain Class C fly ashes may not show the later-age strength gain typical of Class F fly ashes. Changes in cement source may change concrete strengths with Class F fly ash as much as 20 percent. Cements with alkali contents of 0.60 percent Na_2O equivalent or more typically perform better with fly ash for strength measured beyond 28 days.

4.3.4.1.2.2 *Modulus of elasticity*—Lane and Best (1982) report that the modulus of elasticity of Class F fly ash concrete is somewhat lower at early ages and slightly higher at later ages than similar concretes without fly ash. Cain (1979) concluded that cement and aggregate characteristics will have a greater effect on modulus of elasticity than the use of fly ash.

4.3.4.1.2.3 *Abrasion resistance*—Compressive strength, curing, finishing, and aggregate properties are the major factors controlling the abrasion resistance of concrete (ACI 201.2R; 210R). At equal compressive strengths, properly finished and cured concretes with and without fly ash will exhibit essentially equal resistance to abrasion.

4.3.4.1.2.4 *Resistance to freezing and thawing*—The resistance to damage from freezing and thawing of concrete made with or without fly ash depends on the adequacy of the air-void system, the soundness of the aggregates, age, maturity of the cement paste, and moisture condition of the concrete (Larson 1964). Use care proportioning mixtures to ensure that the concrete has adequate strength when first exposed to cyclic freezing and thawing—that is, approximately 3500 psi (24 MPa) or more. When compared on this basis in properly air-entrained concrete, investigators found no significant difference in the resistance to freezing and thawing of concretes with and without fly ash (Lane and Best [1982] for Class F fly ash; Majko and Pistilli [1984] for Class C fly ash).

4.3.4.1.2.5 *Permeability and corrosion protection*—Through its pozzolanic properties, fly ash chemically combines with calcium hydroxide and water to produce desirable hydration products (C-S-H), thus reducing the risk of leaching calcium hydroxide. Additionally, the long-term reaction of fly ash improves the pore structure of concrete to reduce the ingress of chloride ions. As a result of the improved pore structure, permeability is reduced (Manmohan and Mehta 1981). Despite concerns that the pozzolanic action of fly ash could reduce the pH of concrete, researchers have found that an alkaline environment, very similar to that in concrete without fly ash, remains to preserve the passivity of

inforcement (Ho and Lewis 1983).

4.3.4.1.2.6 *Reduction of expansion caused by ASR*—The reaction between the siliceous glass in fly ash and the alkali hydroxides in the portland-cement paste consumes alkalis, which reduces their availability for expansive reactions with reactive aggregates. The use of adequate amounts of some fly ashes can reduce the amount of aggregate reaction and reduce or eliminate harmful expansion of the concrete. Often, the amount of fly ash necessary to prevent damage due to alkali-aggregate reaction will be more than the optimum amount necessary for improvement in strength and workability properties of concrete. Particular replacement levels of some high-alkali fly ashes increase the problem of ASR and higher replacement levels of the same fly ash reduce the problem of ASR. The pessimum level of a particular fly ash is an important consideration for selecting mixture proportions using potentially reactive aggregates.

4.3.5 *Slag cement*

4.3.5.1 *Introduction*—Blast-furnace slag is a nonmetallic product, consisting essentially of silicates and aluminosilicates of calcium and of other bases that is developed in a molten condition simultaneously with iron in a blast furnace. Slag cement is the glassy granular material formed where molten blast-furnace slag is rapidly chilled, such as by immersion in water. The composition of blast-furnace slag is determined by that of the ores, fluxing stone, and impurities in the coke charged into the blast furnace. Typically, silicon, calcium, aluminum, magnesium, and oxygen constitute 95 percent or more of the blast-furnace slag.

A discussion of the basic principles of slag cement hydration makes it possible to identify the primary factors that, in practice, will influence the effectiveness of the uses of slag cement in concrete and mortar. The factors determining cementitious properties are:

a) Chemical composition of the slag cement and portland cement

b) Alkali ion concentration in the reacting system

c) Glass content of the slag cement

d) Fineness of the slag cement and the portland cement

e) Temperature during the early phases of the hydration process

4.3.5.2 *Proportioning with slag cement*—The proportion of slag cement in a concrete mixture will depend on the purposes for which the concrete is to be used, curing temperature, grade (activity) of the slag cement, and portland cement or other activator. In most cases, slag cements have been used in proportions of 25 to 70 percent by mass of the total cementitious material.

Other considerations that determine the proportion of slag cement could include the requirements for permeability, temperature rise control, time of setting and finishing, sulfate resistance, and the control of expansion due to ASR.

4.3.5.3 *Environmental considerations*—Use of slag cement in concrete and mortar is an environmentally sound and efficient use of existing resources. The use of slag cement has several benefits, including reduced energy, reduced greenhouse gas emissions, and reduced virgin raw materials. Recognizing the positive environmental impacts of using slag cement, the Environmental Pr
(EPA) actively encourages the expanded use of slag cement.

4.3.5.4 *Effects on properties of fresh concrete*

4.3.5.4.1 *Workability*—Fulton (1974) investigated workability in great detail and suggested that a cementitious matrix containing slag cements exhibited greater workability due to the increased paste content and increased cohesiveness of the paste. Wood (1981) reported that the workability and placeability of concrete containing slag cement was improved where compared with concrete containing no slag cement.

4.3.5.4.2 *Time of setting*—Delays in setting time can be expected if more than 25 percent slag cement is used as a replacement for portland cement in concrete mixtures. The degree to which the setting time is affected depends on the temperature of the concrete, amount of slag cement used, *w/cm*, and characteristics of the portland cement (Fulton 1974). The amount of portland cement is also important. Hogan and Meusel (1981) found that for 50 percent slag cement, the initial setting time is increased 0.5 to 1 hour at 23°C (73°F); little if any change was found above 29°C (85°F).

Although significant retardation has been observed at low temperatures, the addition of conventional accelerators, such as calcium chloride or nonchloride accelerating admixtures, can reduce or eliminate this effect. Because the amount of portland cement in a mixture usually determines setting characteristics, reducing the slag cement-portland cement ratio may be considered in cold weather.

4.3.5.4.3 *Bleeding*—Bleeding capacity and bleeding rate of concrete are influenced by a number of factors, including the ratio of the surface area of solids to the unit volume of water, air content, subgrade conditions, and concrete thickness. When slag cements are used, bleeding characteristics can be estimated depending on the fineness of the slag cement compared with that of the portland cement, and the combined effect of the two cementitious materials. For slag cement that is finer than portland cement and is substituted on an equal-mass basis, bleeding may be reduced; conversely, for slag cement that is coarser, the rate and amount of bleeding may increase.

4.3.5.4.4 *Slump*—Meusel and Rose (1983) indicated that concrete containing slag cement at 50 percent substitution yielded slump loss equal to that of concrete without slag cement. Experiences in the United Kingdom indicated reduced slump loss, particularly where the portland cement used in the blend, exhibited rapid slump loss such as that caused by false-set characteristics of the cement (Lea 1971).

4.3.5.5 *Effects on properties of hardened concrete and mortar*

4.3.5.5.1 *Strength*—Compressive and flexural strength gain characteristics of concrete containing slag cement can vary over a wide range. The extent to which slag cement affects strength depends on the slag activity index of the particular slag cement and the fraction in which it is used in the mixture. The activity index is the ratio of the compressive strength of a mortar cube made with a 50 percent slag cement blend to that of a mortar cube made with a reference cement. Compared with portland-cement concrete, the se of Grade 120 slag cement typically results in reduced strength at early ages (1 to 3 days) and increased strength at later ages (7 days and beyond) (Hogan and Meusel 1981). The use of Grade 100 slag cement results in lower strengths at early ages (1 to 21 days) but equal or greater strength at later ages. Grade 80 slag cement typically gives reduced strength at early ages, although, by the 28th day, the strength may be equivalent to or slightly higher than a 100 percent portland cement mixture.

4.3.5.5.2 *Modulus of rupture (MOR)*—Of particular interest is the effect of slag cement for concrete that is tested for flexural strength MOR. If comparisons are made between concrete with and without slag cement, where the slag cement is used at proportions designed for greatest strength, the blends generally yield higher MOR at ages beyond 7 days (Fulton 1974; Malhotra 1980; Hogan and Meusel 1981).

This is believed to be a result of the increased density of the paste and improved bond at the aggregate-paste interface.

4.3.5.5.3 *Creep and shrinkage*—Published data on creep and shrinkage of concrete containing slag cement indicate somewhat conflicting results for comparison with concrete containing only portland cement. These differences are likely to be affected by differences in maturity and characteristics of the portland cement from which the concrete specimens were made. Overall, the published information suggests that drying shrinkage is similar in portland-cement concrete and concrete containing slag cement.

4.3.5.5.4 *Color*—Slag cement is considerably lighter in color than most portland cement and will produce a lighter color in concrete after curing. In certain operations, up to 30 percent slag cement has been used to replace white portland cement without a noticeable color difference in the cured product.

4.3.5.5.5 *Permeability*—The use of slag cement in hydraulic structures is well documented. The permeability of mature concrete containing slag cement is much lower than that of concrete not containing slag cement (Hooton and Emery 1990; Roy 1989; Rose 1987). As the slag cement content is increased, permeability of the concrete decreases.

4.3.5.5.6 *Resistance to sulfate attack*—Partial replacement of portland cement with slag cement improves the sulfate resistance of concrete. High resistance to sulfate attack has been demonstrated where the slag cement proportion exceeds 50 percent of the total cementitious material where Type II cements was used (Hogan and Meusel 1981).

4.3.5.5.7 *Reduction of expansion due to alkali-silica reaction*—The use of slag cement as a partial replacement for portland cement is known to reduce the potential expansion of concrete due to ASR (Bakker 1980; Hogan and Meusel 1981).

Results of tests using slag cement as a partial replacement for high-alkali cement with aggregate known to exhibit alkali-silica and alkali-carbonate reactions were reported by Soles et al. (1989). After 2 years of observation, the slag cement blends were found to be effective in reducing expansion, but the reduction was less than that found with the low-alkali cement. Used in combination with high-alkali cement, blends of 50 percent slag cement appear to be e
reducing the potential of ASR.

4.3.5.5.8 *Resistance to freezing and thawing*—Many studies related to freezing-and-thawing resistance have been made using concrete containing slag cement. Results of these studies generally indicate that where concrete made with slag cement (ASTM C989/C989M) was tested in comparison with Type I and Type II cements, they are the same (Fulton 1974; Klieger and Isberner 1967; Mather 1957).

4.3.5.5.9 *Resistance to deicing chemicals*—Although some laboratory tests with Type IS cement indicate less resistance to deicing salts, many researchers have found, in field exposure, little difference compared with concrete not containing slag cement (Klieger and Isberner 1967). Similar results were reported using blends of 50 percent slag cement and 50 percent portland cement (Hogan and Meusel 1981), or slag cement percentages in excess of 35 percent (Afrani and Rogers 1994).

4.3.5.6 *Silica fume*

4.3.5.6.1 *Introduction*—Silica fume—a by-product of the ferrosilicon industry—is a highly pozzolanic material that is used to enhance mechanical and durability properties of concrete. Silica fume is a very fine noncrystalline silica (more than 95 percent of the particles are less than 1 μm) produced in electric arc furnaces as a by-product of the production of elemental silicon or alloys containing silicon. Silica fume can be added directly to concrete as an individual ingredient or in a blend of portland cement and silica fume. In the United States, silica fume is used predominantly to produce concrete with greater resistance to chloride penetration for applications such as parking structures, bridges, and bridge decks. Silica fume is normally not used in paving applications because of cost and availability of other supplementary cementitious materials. It has been used in curb and gutter applications to improve durability and by the U.S. military for roads and airfields in the Middle East since the mid-1990s.

Silica fume was initially viewed as a cement replacement material, but currently the most important reason for its use is the production of high-performance concrete, where adding silica fume provides enhancements in concrete properties. In this role, silica fume has been used to produce concrete with enhanced compressive strength and very high levels of durability.

Because of the fineness of the material, adding silica fume to concrete mixtures usually increases water demand. To produce high-performance, durable concrete, it is necessary to maintain (or decrease) the *w*/*cm*. Consequently, high-range water-reducing admixtures (HRWRAs), sometimes combined with water-reducing admixtures (WRAs), are used to obtain the required performance and workability. Silica-fume concrete will be more cohesive than ordinary concrete; consequently, a somewhat higher slump will normally be required to maintain the same apparent degree of workability.

4.3.5.6.2 *Effects of silica fume on properties of fresh concrete*

.6.2.1 *Color*—Most silica fumes range from light to dark gray. Because SiO_2 is colorless, the color is determined by the nonsilica components, which typically include carbon and iron oxide.

In general, the higher the carbon content, the darker the silica fume. Silica fume is typically used in amounts between

5 and 10 percent by mass of the total cementitious material (Kosmatka et al. 2002).

4.3.5.6.2.2 *Water demand*—The water demand of concrete containing silica fume increases with increasing amounts of silica fume (Carette and Malhotra 1983; Scali et al. 1987). Primarily, the high surface area of the silica fume causes this increase. To achieve a maximum improvement in strength and durability, silica-fume concrete should contain a WRA, HRWRA, or both. The dosage of the HRWRA will depend on the amount of silica fume and the type of HRWRA used (Jahren 1983).

4.3.5.6.2.3 *Workability*—Fresh concrete containing silica fume is more cohesive and less prone to segregation than concrete without silica fume. As the silica fume content is increased, the concrete becomes sticky.

4.3.5.6.2.4 *Slump loss*—The presence of silica fume by itself will not significantly change the rate of slump loss of a given concrete mixture. The slump loss of silica-fume concrete will be determined by the presence and characteristics of a WRA or HRWRA, or by any of the other factors that affect slump loss of any concrete, such as high cement temperature, high concrete temperature, or failure to account for aggregate moisture correctly. Trial batches conducted at conditions simulating the concrete placing environment using project materials are recommended to establish slump-loss characteristics for a particular situation.

4.3.5.6.2.5 *Time of setting*—Experience indicates that the time of setting is not significantly affected by the use of silica fume by itself. The chemical admixtures typically used in silica-fume concrete may affect the time of setting of the concrete. Practical control of the time of setting may be achieved by using appropriate chemical admixtures.

4.3.5.6.2.6 *Segregation*—Concrete containing silica fume normally does not segregate appreciably because of the fineness of the silica fume and the use of HRWRA. Segregation can occur in many types of concrete, including those (with or without silica fume) with excessive slump, improper proportioning, improper handling, or prolonged vibration. The use of silica fume will not overcome poor handling or consolidation practices.

4.3.5.6.2.7 *Bleeding and plastic shrinkage*—Concrete containing silica fume shows significantly reduced bleeding. As silica fume dosage is increased, bleeding will be reduced. This effect is caused primarily by the high surface area of the silica fume to be wetted; there is very little free water left in the mixture for bleeding (Grutzeck et al. 1982). Plastic-shrinkage cracks occur if the rate of water evaporation from the concrete surface exceeds the rate at which water appears at the surface due to bleeding, or if water is lost into the subgrade. Because silica-fume concrete exhibits significantly reduced bleeding, the potential for plastic-shrinkage cracking is increased.

4.3.5.6.2.8 *Air entrainment*—The dosage admixture to produce a required volume of air in concrete usually increases with increasing amounts of silica fume. Typically, the increase in air-entraining admixture will be approximately 125 to 150 percent of that used in similar concrete without silica fume. This increase is attributed to the very high surface area of silica fume and possibly to the effect of carbon where the latter is present (Carette and Malhotra 1983).

4.3.5.6.3 *Effects of silica fume on mechanical properties of concrete*—The effects of silica fume on the properties of hardened concrete can be directly related to the physical and chemical mechanisms by which silica fume functions. The primary changes in the concrete are in pore structure, cement paste-aggregate transition zone, and chemical composition, particularly the content of calcium hydroxide and alkalinity of the pore solution.

Many of the improvements in mechanical properties appear to be related to improvements in bond strength between the paste and aggregate. Therefore, the influence of the aggregate properties on the mechanical properties of concrete becomes more important in silica-fume concrete. The size, durability, and engineering properties (strength, modulus of elasticity, Poisson's ratio) become important factors to consider in selecting the appropriate aggregate for the concrete.

4.3.5.6.4 *Effects of silica fume on durability of concrete*—Durability is a complex subject, and a number of mechanisms can be involved in the degradation of concrete, concerning both the transport of substances into and out of concrete and the effect of these substances on the concrete. The most important reason for using silica fume is its contribution to improved durability in concrete. Overall, silica fume can improve the durability of conventional concrete by:

a) Reducing rates of transport of aggressive fluids through the pore structure

b) Reducing the rate of chloride-ion ingress

c) Providing equal or improved resistance to freezing and thawing, as well as to deicer scaling

d) Improving chemical attack resistance

e) Improving erosion and abrasion resistance

f) Providing similar fire resistance

g) Improving resistance to deleterious expansion due to ASR

h) Improving sulfate resistance

i) Increasing electrical resistivity

4.3.5.6.5 *Silica-fume concrete and cracking*—Silica-fume concrete is usually high strength and has a low *w/cm*. Such concrete can be susceptible to two basic types of cracking. First, there is early-age cracking relating to the material itself and to construction practices. This form of cracking is reasonably well understood, and appears to be controlled by adopting appropriate construction practices. Second, there is later-age cracking that is apparently related to performance under load. This form of cracking is not well understood.

4.3.5.7 *Natural pozzolans*—Volcanic ash and calcined clays exhibit pozzolanic properties and have been used in construction for centuries. These materials, termed "natural ozzolan," are specified using ASTM C618. In North America, natural pozzolan has been used in public works construction since the early part of the twentieth century. Volcanic ash is available in the western part of the United States, but the most common natural pozzolans are calcined clay, calcined shale, and metakaolin. To make metakaolin, high-purity kaolin clay is calcined at a low temperature. The product is ground to a particle size of 1 to 2 micrometers. Metakaolin is used in special applications where concrete having a very low permeability or very high strength is needed. Typical addition rates are approximately 10 percent of the cement mass.

4.3.6 *Water*—Water quality is a concern because chemicals in it, even in very small amounts, sometimes change the setting time of the mixture on the long-term performance of the concrete. Almost any drinkable water can be used to make concrete. Treated city water (tap water) from almost all major cities in the United States and Canada are suitable for making concrete. Some water that is not drinkable—most notably, recycled water from concrete production and other industrial processes—is used for mixing water. ASTM C1602/C1602M provides detailed requirements for mixing water and provides procedures for testing water to ensure that it is suitable for use. Compressive strength of concrete made with the nonpotable source proposed for the job must not be less than 90 percent of the strength of the same concrete made with potable or distilled water and the time of set must be no more than 1 hour earlier to 1 hour and 30 minutes later than the comparison concrete. If water from concrete production is recycled, the density of the recycled water must be monitored daily. ASTM C1602/C1602M also contains maximum concentration requirements for chlorides, sulfate, alkalis, and total solids.

4.3.7 *Chemical admixtures*

4.3.7.1 *Introduction*—Admixtures are typically used to modify the properties of concrete so that it will be more suitable for a particular purpose. Their use to obtain desirable characteristics should be based on appropriate evaluation of their effects on specific combinations of materials and on economic considerations. Air-entraining and water-reducing admixtures are commonly used in concrete for paving.

Consult ACI 212.3R if considering admixture use in concrete. Experience records on use of specific admixture with concreting materials commonly used in the area should also be considered. If admixtures are required by the general specifications, or permitted by the engineer, they should conform to the appropriate specifications:

a) ASTM C260/C260M; AASHTO M154 (air-entraining admixtures)

b) ASTM C494/C494M; AASHTO M194 (water-reducing/set-controlling admixtures)

c) ASTM D98; AASHTO M144 (calcium chloride)

NCHRP Report 578 (Nagi et al. 2007) contains helpful information on the use and selection of air-entraining admixtures.

Air-entraining admixtures should be used to improve durability and workability. Water-reducing admixtures may reduce total water content and *w/cm*, thus increasing compressive strength, flexural strength, and dura

decreasing permeability, shrinkage, and creep. Some admixtures accelerate the time of setting of concrete, permitting earlier finishing, removal of forms, and opening of lanes to traffic, as well as reduce the time of protection from freezing during cold weather. Others can retard the time of setting of concrete where rapid setting is undesirable. Many retarding admixtures accelerate strength gain once initial set is attained.

Admixtures are tested for one or more reasons:

1) Determination of their compliance with specifications

2) Evaluation of the effect of admixture on the properties of concrete to be produced with job materials under the anticipated ambient conditions and construction procedures

3) Determination of the uniformity of product

Although ASTM tests afford a valuable screening procedure for selection of admixtures, continuing use of admixtures in production of concrete should be preceded by testing that allows observation and measurement of the performance of the chemical admixture under concrete plant operating conditions in combination with concrete-making materials then in use. Uniformity of results is as important as the average result with respect to each significant property of the admixture or the concrete.

Although specifications deal primarily with the influence of admixtures on standard properties of fresh and hardened concrete, the concrete supplier, contractor, and owner of the construction project are interested in other features of concrete construction. Of primary concern may be workability, placing and finishing qualities, and early strength development. These additional features are often of great importance when determining the selection and dosage rate of an admixture.

4.3.7.2 *Cost effectiveness*—Economic evaluation of any given admixture should be based on the results obtained with the particular concrete in question under conditions simulating those expected on the job. This is highly desirable because the results obtained are influenced by the characteristics of the cement, supplementary cementitious materials and aggregate, as well as their relative proportions, temperature, humidity, and curing conditions.

Water-reducing and set-retarding admixtures permit placement of large volumes of concrete over extended periods, thereby minimizing the need for forming, placing, and joining separate units. Accelerating admixtures reduce finishing and forming costs.

4.3.7.3 *Other considerations*—Careful attention should be given to the instructions provided by the manufacturer of the admixture. The effects of an admixture should be evaluated as possible by use with the particular materials and conditions of use intended. Such an evaluation is particularly important if: 1) the admixture has not been used previously with the particular combination of materials; 2) special types of cement are specified; 3) more than one admixture is to be used; and 4) mixing and placing is done at temperatures well outside generally recommended concreting temperature ranges.

Furthermore, note that: 1) a change in type or source of or amount of cement, or a modification of aggregate grading or mixture proportions, may be desirable; 2) many admixtures affect more than one property of concrete, sometimes adversely affecting desirable properties; 3) the effects of some admixtures are significantly modified by such factors as water content and cement content of the mixture, by aggregate type and grading, and by type and length of mixing.

Admixtures that modify the properties of fresh concrete can cause problems through early stiffening or undesirable retardation that prolong the time of setting. The cause of abnormal setting behavior should be determined through studies of how such admixtures affect the cement to be used; early stiffening often is caused by changes in the rate of reaction between C_3A and sulfate. Retardation can be caused by an overdose of admixture or by a lowering of ambient temperature, both of which delay the hydration of the calcium silicates.

4.3.8 *Aggregates*—Aggregates for pavement concrete should conform to the quality requirements of ASTM C33/C33M, or applicable state department of transportation specifications. If lightweight aggregate sand (LWAS) is specified for internal curing, aggregates for pavement concrete should conform to the quality requirements of ASTM C33/C33M and C330/C330M, considering that the minus 100 and 200 sizes of ASTM C330/C330M material are pozzolanic in action. For evaluating potential reactivity of an aggregate, methods are provided in the appendix of ASTM C33/C33M. Following the recommendations in ACI 201.2R can reduce the danger of aggregate-alkali reactivity distress.

A number of recycled and industrial by-product materials (RIBMs) are becoming attractive for use in pavement construction from a sustainability perspective. Some of these materials include recycled concrete aggregate, recycled asphalt pavement (RAP), air-cooled blast-furnace slag (ACBFS), steel furnace slag, and foundry sand. However, these materials possess properties that are different from natural aggregates, so they should be adequately characterized and carefully evaluated before incorporating into a construction project. In general, recycled materials to be used as aggregate for pavement concrete should meet the same requirements as virgin aggregate materials, unless the lower-quality recycled material is used in the lower layer of a two-course pavement.

Laboratory tests on aggregates depend on the potential modes of deterioration, but should include absorption, specific gravity, abrasion resistance, and soundness; some typical values are given in Table 4.3.8. More detailed information can be found in ACI 221.1R.

4.3.8.1 *Gradation*—The desired gradation limits for the project should be stipulated, along with permissible day-to-day variations within the specification limits. To avoid segregation, coarse aggregates should be furnished in at least two separate sizes, with the separation at the 0.75 in. (19 mm) sieve if combined material graded from No. 4 to 1-1/2 in. (4.75 to 37.5 mm) nominal maximum size (or 2 in. [50 mm] maximum size) is specified, and at the 1 in. (25 mm) sieve if combined material graded from No. 4 to 2 in. (4.75 to 50 mm) nominal maximum size (
mm] maximum size) is specified. If the nominal maximum size of coarse aggregate is 1 in. (25 mm) or less, such separation is not necessary.

Consideration of grading and aggregate particle shape may optimize early and long-term concrete strength. Typical procedures consider the proportions of coarse and fine aggregates without specifying the combined or total grading. Consequently, concrete producers draw aggregate from two stockpiles at the plant site: one for coarse aggregate and one for fine aggregate. To improve aggregate grading, additional intermediate sizes of material (blend sizes) at the plant site during project construction may be required.

Table 4.3.8—Typical natural aggregate properties

Property	Natural aggregate
Particle shape and texture	Well rounded, smooth (gravels) to angular and rough (crushed stone)
Absorption capacity	0.8 to 3.7 percent
Specific gravity	2.4 to 2.9
L.A. abrasion test mass loss	15 to 30 percent
Sodium sulfate soundness mass loss	7 to 21 percent
Magnesium sulfate soundness mass loss	4 to 7 percent
Chloride content	0 to 2 lb/yd^3 (0 to 1.2 kg/m^3)

Grading data indicate the relative composition of aggregate by particle size. Sieve analyses of source stockpiles are necessary to characterize the materials. The best use of such data is to calculate the individual proportions of each aggregate stockpile in the mixture to obtain the designed combined-aggregate grading. Well-graded mixtures generally have a uniform distribution of aggregates on each sieve. An optimum combined-aggregate grading efficiently uses locally available materials to fill the major voids in the concrete to reduce the need for mortar. One approach to evaluate the combined-aggregate grading is to assess the percentage of aggregates retained on each sieve (Shilstone and Shilstone 2002). A grading that approaches the shape of a bell curve on a standard grading chart indicates an optimal distribution, as shown in Fig. 4.3.8.1(a). Blends that leave a deficiency of aggregates retained in the No. 8 through No. 30 (2.36 through 0.6 mm) sieves are gap-graded mixtures, shown in Fig. 4.3.8.1(b).

There is a definite relationship between aggregate grading and concrete strength, workability, and long-term durability. Intermediate-size aggregates fill voids typically occupied by less-dense cement paste and thereby optimize concrete density. Increasing concrete density in this manner results in:

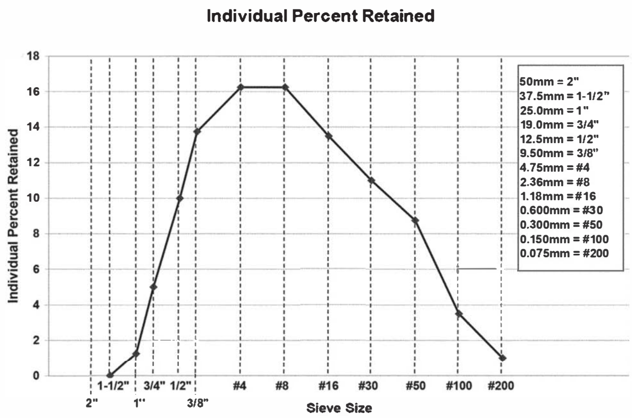
a) Reduced mixing water demand and improved strength because less mortar is necessary to fill space between aggregates

b) Increased durability through reduced avenues for water penetration in the hardened concrete

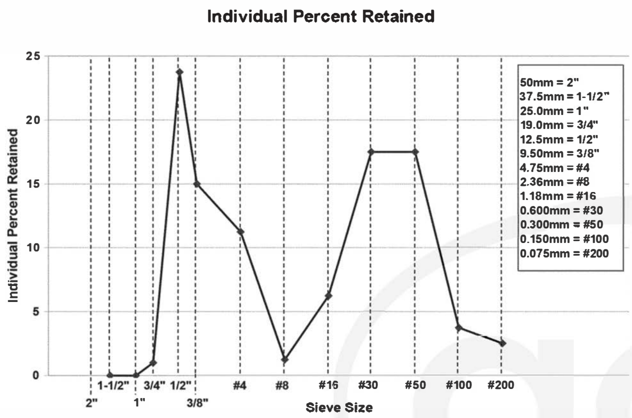
c) Better workability and mobility because large aggregate particles do not bind in contact with other large particles under the dynamics of finishing and vibration

d) Less edge slump because of increased particle-to-particle contact

Well-graded aggregates also influence workability and ease the placing, consolidating, and finishing of concrete. While engineers traditionally look at the slump test as a measure of workability, it does not necessarily reflect that characteristic of concrete. Slump evaluates only the fluidity of a single concrete batch and provides a relative measure



(a) well graded



(b) gap graded

Fig. 4.3.8.1—Well graded versus gap graded (Missouri Department of Transportation (MoDOT) 2005).

of fluidity between separate concrete batches of the same mixture proportions.

Concrete with a well-graded aggregate often will be much more workable at a low slump than a gap-graded mixture at a higher slump. A well-graded aggregate may change concrete slump by 3-1/2 in. (90 mm) over a similar gap-graded mixture. This is because approximately 540 to 650 lb/yd^3 (320 to 385 kg/m^3) less water is necessary to maintain mixture consistency than is necessary with gap grading (Shilstone and Shilstone 2002).

Although ASTM C33/C33M and C330/C330M are acceptable specifications, Table 5.4.1 in ACI 302.1R-96 recommends preferred grading specifications for the toppings for Class 7 floors. These gradations limit the amount of material passing the No. 50 and No. 100 (300 and 150 μm) sieves to reduce the amount of cement paste needed. However, ACI 302.1R-96 cautions that where fine aggregates contain minimum percentages of material passing the No. 50 and No. 100 (300 and 150 μm) sieves, the likelihood
sive bleeding is increased and limitations on water content of the mixture become increasingly important. Natural sand is preferred to manufactured sand (which can increase the harshness of the mixture and make it very difficult to work); the gradation indicated in Table 5.4.1 in ACI 302.1R-96 will minimize water demand. For lightweight fine aggregates (governed by ASTM C330/C330M), the material passing the No. 100 and No. 200 (150 and 300 μm) are usually pozzolanic in nature and improve the characteristics of the concrete.

The use of large aggregate is generally desired for lower water demand and shrinkage reduction. However, it is important to recognize the overall gradation of all the aggregate. According to ACI 302.1R-96, gradations requiring between 8 and 18 percent for large top-size aggregates (such as 1-1/2 in. [37.5 mm]) or 8 and 22 percent for smaller top-size aggregates (such as 1 in. or 0.75 in. [25 or 19 mm]) retained on each sieve below the top size and above the No. 100 (150 μm) sieve have proven to be satisfactory in reducing water demand while providing good workability. It is recommended that the ideal range for No. 30 and No. 50 (600 and 300 μm) sieves be 8 to 15 percent retained on each, and often, a third aggregate is required to achieve this gradation.

Typically, 0 to 4 percent retained on the top size sieve and 1.5 to 5.0 percent on the No. 100 (150 μm) sieve will be a well-graded mixture. This particle size distribution is appropriate for round or cubically shaped particles in the No. 4 through No. 16 (4.75 mm through 1.19 mm) sieve sizes. If the available aggregates for these sizes are slivered, sharp, or elongated, 4 to 8 percent retained on any single sieve is a reasonable compromise. Mixture proportions should be adjusted where individual aggregate grading varies during the course of the work.

4.3.8.2 *Particle shape and texture*—The shape and texture of aggregate particles impact concrete properties. Sharp and rough particles generally produce less-workable mixtures than rounded and smooth particles with the same *w/cm*. Particle shape and texture are important to the response of the concrete to vibration, especially in the intermediate sizes.

The bond strength between aggregate and cement mortar improves as aggregate texture becomes rougher. The improved bond will improve concrete flexural strength, and is the result of increased mechanical interlock. Concrete mixtures containing natural coarse aggregates and natural sands are easily consolidated, and cube-shaped crushed aggregates are also more easily consolidated than flat or elongated aggregate. The good mobility allows concrete to flow easily around the baskets, chairs, and reinforcing bars, and is ideal for pavements.

Flat or elongated intermediate and large aggregates can cause mixture problems. These shapes generally require more mixing water or fine aggregate for workability and, consequently, result in a lower concrete flexural strength, unless more cementitious materials are added. Allowing no more than 15 percent flat or elongated aggregate by weight of the total aggregate is advisable. Use ASTM D4791 to deter-
e maximum quantity of flat or elongated particles.

4.3.8.3 *Durability*

4.3.8.3.1 *Alkali-aggregate reactivity*—In many parts of the world, precautions should be taken to avoid excessive expansion due to alkali-aggregate reactivity (AAR) in many types of concrete construction. Alkali-aggregate reactivity may involve siliceous aggregates (such as ASR) or carbonate aggregates (alkali-carbonate reactivity [ACR]); failure to take precautions may result in progressive deterioration, requiring costly repair and rehabilitation of concrete structures to maintain their intended function. Extensive knowledge is available regarding the mechanisms of the reactions, the aggregate constituents that may react deleteriously, and precautions that can be taken to avoid resulting distress. As a result of extensive research, concrete structures can now be designed and built with a high degree of assurance that excessive expansion due to AAR will not occur and cause progressive degradation of the concrete.

4.3.8.3.2 *Alkali-silica reactivity*—Alkali-silica reactivity was first recognized in a California concrete pavement by Stanton (1940) of the California State Division of Highways. Stanton's early laboratory work demonstrated that expansion and cracking resulted if certain combinations of high-alkali cement and aggregate were combined in mortar bars stored in containers at very high relative humidity. Two important conclusions were drawn from this work:

1) Expansions resulting from ASR in damp mortar bars were negligible where alkali levels in cement were less than 0.60 percent, expressed as equivalent sodium oxide (percent Na_2O_e = percent Na_2O + 0.658 percent K_2O).

2) The partial replacement of high-alkali cement with a suitable pozzolanic material prevented excessive expansions. Replacement of a portion or all of a normalweight, nonreactive sand with lightweight (expanded shale) sand can reduce the ASR in concretes containing a known, reactive, normalweight coarse aggregate (Boyd et al. 2000).

Thus, foundations for the engineering control of the reaction were developed. Test methods currently in use to determine potential for expansive reactivity, particularly in the United States, were derived primarily from work carried out in the 1940s. However, recent research efforts in several countries indicate a promise of newer, more reliable tests to identify potentially deleteriously reactive cement-aggregate combinations. Common aggregates that could be susceptible to ASR include chert, shale, and rhyolite.

4.3.8.3.3 *Alkali-carbonate reactivity*—Alkali-carbonate reactivity was identified as causing a type of progressive deterioration of concrete by Swenson (1957) of the National Research Council of Canada. He found that an alkali-sensitive reaction had developed in concrete containing argillaceous calcite dolomite aggregate that appeared to be different than the ASR. Because rock susceptible to this type of reaction is relatively rare and often unacceptable for use as concrete aggregate for other reasons, reported occurrences of deleterious ACR in actual structures are relatively few. The only area where it appears to have developed to any great extent is in southern Ontario, Canada, in the vicinities of Kingston and Cornwall. Isolated occurrences in concrete structures have been found in the United S
Kentucky, Tennessee, and Virginia. So-called alkali-dolomite reactions involving dolomitic limestone and dolostones have also been recognized in China (Tang et al. 1996). Additional information on AAR and ways to mitigate it can be found in ACI 211.1.

4.3.8.4 *D-cracking*—D-cracks are a series of cracks in concrete near and roughly parallel to joints, edges, and structural cracks. D-cracking is commonly associated with distress due to freezing and thawing of critically saturated aggregate particles in concrete pavements. Three conditions are necessary for D-cracking to occur:

a) The aggregate should be susceptible to D-cracking; such aggregates that have weakened planes and deleterious pore size

b) Pavement joints are poorly drained, making moisture available

c) The pavement should be subject to freezing and thawing

D-cracking can be prevented by using nonporous aggregate or reducing the top size of the aggregate (Schwartz 1987).

4.3.8.5 *Recycled aggregates*—Disposal of exiting concrete pavements is often a problem faced on many pavement reconstruction projects. Recycling concrete, as an aggregate product, is encouraged by the Federal Highway Administration (FHWA 2002) Policy Memorandum and is common practice by many public and private organizations. Removed concrete pavement can be processed into aggregate product for use as granular or stabilized base, subbase, or shoulder materials. It can also be processed into aggregate material used for bedding, backfill, granular embankment, and aggregate for asphalt or hydraulic-cement concrete. Specific guidelines for use of recycled concrete pavement as aggregate in hydraulic-cement concrete pavement can be found in FHWA Technical Advisory T 5040.37 (FHWA 2007a).

In general, the recycled materials used for concrete paving projects should meet the same quality requirements normally used for virgin aggregates. Although recycling demonstrates good environmental stewardship and can reduce the cost of a paving project, economic and environmental costs are different for each project.

4.3.9 *Rapid-setting concrete mixtures*

4.3.9.1 *Introduction*—For the most part, early-opening-to-traffic (EOT) concrete is composed of the same constituents as normal paving concrete. Coarse and fine aggregates are blended with hydraulic cement, water, and admixtures to produce a stiff but moldable mass that hardens by hydration. In the resulting stone-like mass, the aggregates have been bound together by the hydration products formed through chemical reactions between the water and cement. Air is also entrapped, entrained, or both, typically making up 5 to 7 percent of the total mixture volume (NCHRP 2005). The emerging technology is to use HPC, with a *w/cm* of 0.33 to 0.43, usually at ±0.38. This is an opportunity for the contractor using internal curing to place the concrete in service sooner.

4.3.9.2 *Cement*—ASTM C150/C150M Types I, II, or III portland cement can produce successful accelerated paving mixtures. Certain ASTM C595/C595M blended cements, STM C1157/C1157M cements, rapid-setting hydraulic cements meeting ASTM C1600/C1600M, and several proprietary cements that develop high early strengths can also be useful for accelerated paving applications. Not every portland cement will gain strength rapidly, however, and testing is necessary to confirm the applicability of each cement. The materials engineer and contractor should be aware of these phenomena in testing mixtures and trial batches. Tests should be conducted using the same cement that the contractor will use in construction.

4.3.9.3 *Cement factor*—The cement factor (or cement content) of EOT concrete is typically much higher than that used in conventional paving concrete. These high cement factors contribute to increased paste porosity, as reflected in an increase in percent of permeable voids, absorption, and sorptivity. Further, the increase in paste volume increases the amount of shrinkage, potentially producing more cracking and reducing durability of the mixture. This suggests that increasing cement content will not necessarily improve the early or long-term strength of the EOT concrete. Instead, other methods of increasing early strength, such as lowering the *w/c* and internal curing, are likely to be more effective. Therefore, mixtures with lower cement contents, as well as those with corresponding higher aggregate volumes, should be investigated for use in EOT concrete (NCHRP 2005).

4.3.9.4 w/cm—Decreasing the *w/cm* of the mixture (over the range of 0.43 to 0.33) will increase the various measures of strength at all ages of testing, decrease absorption, and improve paste homogeneity with no observed disadvantages other than increasing autogenous shrinkage, as long as workability is maintained. It is therefore advantageous both from the perspective of strength gain and durability to use a *w/cm* at or below 0.40 for 6- to 8-hour EOT concrete mixtures, although a slightly higher *w/cm* appears to be acceptable for 20- to 24-hour EOT concrete mixtures.

4.3.9.5 *Accelerating admixtures*—Accelerating admixtures, which are also called accelerators (Type C or E) in AASHTO M194 are common in EOT concrete, profoundly affecting strength gain and, potentially, durability. Although calcium chloride is the most common accelerator used in concrete, it promotes corrosion of embedded steel and could have other adverse impacts on concrete durability. Calcium nitrite is the most common nonchloride accelerator used in concrete (NCHRP 2005).

4.3.9.6 *Water reducer*—Water reducers (AASHTO M194 Type A, Type E, and Type F) are often used in 6- to 8-hour EOT concrete mixtures and occasionally 20- to 24-hour EOT concrete mixtures to assist in producing workable concrete at low *w/cm*. The use of the Type F HRWRs may negatively impact the air-void system parameters, creating a network of rather large bubbles with insufficient spacing factors, thus compromising the freezing-and-thawing performance of the concrete (Whiting and Nagi 1998). Various water-reducing admixtures are available for use in EOT concrete; it is impossible to categorize their interaction with other concrete constituents. However, the final selection of the water-reducing admixture should be done only after testing the job mixture, including evaluation of the impact on both strength and durability characteristics. This testing is of importance if HRWRs are being considered, as difficulties have been reported in obtaining satisfactory air void systems in mixtures containing Type F HRWRs (NCHRP 2005).

4.3.9.7 *Coarse aggregate*—The type of coarse aggregate used affects concrete density and coefficient of thermal expansion (CTE). The coarse aggregate can also impact some strength properties of the mixtures and scaling resistance. Thus, care should be exercised in selecting coarse aggregate that will provide both the desired strength and the durability properties (NCHRP 2005).

4.4—Reinforcement, dowels, and tie bars

The desired types of reinforcing steel and accessories should be specified in accordance with the following applicable specifications.

4.4.1 *Steel wire fabric reinforcement*—Steel wire fabric reinforcement should conform to ASTM A1064 or ASTM A884/A884M.

4.4.2 *Bar mats (ASTM A184/A184M)*—Member size and spacing should be shown on the plans. All intersections of longitudinal and transverse bars should be securely wired, clipped, or welded together in the plant of the steel supplier.

4.4.3 *Reinforcing bars*—Reinforcing bars should conform to the requirements of one of the standard specifications:

a) ASTM A615/A615M, Grade 40, or Grade 60

b) ASTM A996/A996M

c) ASTM A775/A775M and ASTM A934/A934M specify materials, surface preparation procedures, and coating requirements for protective epoxy coatings

d) ASTM A955/A955M specifies requirements for stainless steel materials.

Guidance for the use of fiber-reinforced concrete can be found in ACI 544.1R.

4.4.4 *Surface condition*—Reinforcing steel should be free from dirt, oil, paint, grease, or other organic materials that may adversely affect or reduce bond with the concrete. Rust, mill scale, or a combination of both should be considered acceptable provided the minimum dimensions, weight, and physical properties of a hand-wire-brushed test specimen are not less than the applicable ASTM specification requirements (refer to ASTM A615/A615M, for example).

4.4.5 *Tie bars*—Tie bars should be deformed steel bars conforming to the requirements of the governing specifications for reinforcing bars except that only grades of steel bars should be used that can be bent and restraightened without damage if this procedure is indicated. Tie bars may be inserted in the fresh concrete, placed on chairs ahead of the paver, or drilled into the slab after paving (the latter case being when separate pavings are made, such as a tied shoulder that is added later).

4.4.6 *Dowels*---(ASTM A615/A615M) Dowels should be plain bars conforming to the requirements of the specifications for plain round bars. Dowels should not be burred, roughened, or deformed out of round in such a manner as to hinder slippage in the concrete. If expansion caps are used for expansion joints, they should cover the ends of the dowels for not less than 2 in. (50 mm) or more than 3 in. (75 mm). Caps be closed at one end, and should provide for adequate expansion and not interfere with proper load transfer. It should be of such rigid design that the closed end will not collapse during construction. Epoxy coatings are commonly applied for corrosion protection, although plastic coatings are occasionally used. In addition, several alternative dowel

Table 4.5—Joint sealants and filler materials

Material type	Properties	Material	Common specification
Field molded	Hot applied	Four types, performance specification Elastomeric Jet-fuel resistant	ASTM D6690 Fed Spec SS-S-1614A
Field molded	Cold applied Jet-blast resistant	Single component silicone	ASTM D5893/D5893M Fed Spec SS-S-200E
Preformed	Compression seal	Preformed polychloroprene elastomeric seal Lubricant for installing compression seals	ASTM D2628 ASTM D2835
Filler	Preformed expansion joint Preformed expansion joint Preformed joint filler	Bituminous Nonextruded and resilient bituminous Sponge, rubber, cork, and recycled PVC	ASTM D994/D994M ASTM D1751 ASTM D1752

bar materials that are either noncorrodible (for example, fiber reinforced polymer) or have a high resistance to corrosion (for example, stainless steel) are being used by a few agencies in severe exposure conditions.

4.4.6.1 *New developments in load transfer*—In addition to round steel bars, there are a number of new materials and dowel configurations available that may offer some advantages over the traditional round dowel. Elliptical dowels, plate dowel bars, and tapered plate dowel bars provide increased bearing surface with a smaller steel cross section and improved joint opening characteristics. Fiberglass-reinforced polymer dowels can offer adequate load transfer and rider comfort in joints while reducing issues with corrosion. These dowels are beneficial in areas where metal dowels would interfere with pavement performance, such as toll booth sensors. Alternative dowels should be designed and selected to provide equivalent performance to the standard round dowel size and spacing for a typical pavement thickness or for the specific performance criteria of the designer.

4.4.7 *Chairs*—Chairs, which are used to support reinforcing steel, dowels, or tie bars on subbases, should be of adequate strength and designed to resist displacement or deformation before and during concrete placing. On loose or sandy bases, chairs should have base plates such that they do not sink into the subgrade under steel or concrete load.

4.4.8 *Stakes*—Stakes used to support expansion joint fillers should be metal. Their length and stiffness should be adequate to keep the fillers in proper position during concrete placement.

4.5—Joint sealants and fillers

Joint sealants are installed in concrete pavements to prevent the entry of water and solid foreign materials into joints. Joint fillers help to exclude water and debris from the joint and provide support for sealants applied to the exposed surface; however, sealants should be capable of minimizing the amount of water that enters the pavement structure and keeping incompressible material out of the joint (FHWA 1990). Many types of sealants are available, but the materials fall into one of four general categories; 1) mastic; 2) field-molded thermoplastics; 3) field-mo
ting materials; and 4) preformed compression seals. Table 4.5 provides an overview of materials and commonly used specifications. The recommendations found in ACPA (1991, 1995) should be consulted for more details on the selection and use of sealants and fillers.

4.6—Curing materials

The specifications should stipulate the type or types of curing material to be used and require conformance to the appropriate specification. The general requirements of curing practice as recommended by ACI 308R should be followed.

4.6.1 *Burlap*—Burlap should be made from jute or keaf and at the time of use, be in good condition, and be free from holes, dirt, clay, or any substance that interferes with its absorptive quality. It should not contain any substance that would have a deleterious effect on the concrete. Additional details can be found in AASHTO M182. Burlap that will not absorb water readily if dipped or sprayed and that weighs less than 7 oz/yd^2 (240 g/m^2) clean and dry should not be used. Burlap made into mats should be handled with care to avoid marring the finished surface of the concrete.

4.6.2 *Waterproof paper and impermeable sheets*—Waterproof paper and impermeable sheets should conform to the water retention requirements of ASTM C171.

4.6.3 *Liquid membrane-forming compounds*—Liquid membrane-forming compounds should conform to the requirements of ASTM C309. Type 2, white-pigmented curing compound, is generally preferred for concrete pavements. Type 1, clear or translucent, and Type 3, light gray pigmented, are also used.

4.6.4 *Fogging*—Where there is little or no wind or adequate protection with screens, fogging provides excellent protection against surface drying if applied properly and frequently and where the air temperature is well above freezing. Its primary purpose is to increase the humidity of the air and reduce the rate of evaporation (ACI 308R).

4.6.5 *Internal curing*—Internal curing is not a replacement of burlap, fogging, waterproof paper, or liquid membrane, but as an adjunct to them, because they do not adequately supply water into the interior of the concrete and do not supply water for hydration in low-*w*/*c* mixtures. Because it is widely distributed throughout the concrete, lightweight aggregate sand can supply water to the cement particles that are not sufficiently hydrated by the mixing water or the external curing water.

CHAPTER 5—CONSTRUCTION

5.1—Foundation preparation

5.1.1 *Introduction*—Performance of a concrete pavement system is highly dependent on the construction of the subgrade, subbase, and base layers. Delatte (2008) points out that there is nothing more expensive than a cheap foundation. The construction team should recognize that the goal in building the foundation is to insure that the pavement substructure will provide the characteristics the designer assumed would support the pavement surface. If drainage, uniform strength, or other properties are not achieved, the long-term performance of the pavement will be compromised.

5.1.2 *Grade preparation*—Construction of the subgrade begins with clearing and grubbing, which is the removal of all trees, shrubs, and stumps, along the route. All vegetation should be removed and hauled to a disposal site. Topsoil removed from the construction area is often stockpiled for reuse in vegetated areas constructed along the route. Organic material and poor soil should not be buried in deep fills where settlement could cause pavement failure. Variations from subgrade materials expected at the site shown in the construction documents should be reported to the design agency. Subgrade soils should be compacted at moisture contents and to densities that will ensure stable and uniform support for the pavement. In areas where there is an abrupt change in soil type, cross hauling and mixing of soils is often used to achieve uniform support conditions. In cut-and-fill areas, the better soil types should be placed in the top of the subgrade. Extremely poor soils can be treated with cement, lime, or replaced with better soil.

The agency and contractor should have contingency plans in the event that unsuitable soils are encountered during subgrade preparation. Options include removing unsatisfactory material and replacing it with suitable material from nearby areas, soil stabilization, removing undesirable soil and replacing it with crushed stone, and placing a geogrid and 10 in. (250 mm) lift of crushed stone over soft areas (Delatte 2008).

Embankments are placed in uniform layers and compacted to the density specified in the contract documents. Optimum moisture content and maximum dry density for compaction can be specified using either the standard proctor test (ASTM D698) or the modified proctor test (ASTM D1557). Additional information on subgrade compaction for specific soil types can be found in Delatte (2008). Special techniques and precautions to control expansive soils and frost heave can be found in ACPA (2007).

5.1.3 *Subbase and base*—Terminology relating to subbase and base used in other types of pavements, airport construction, and concrete highway construction can be confusing. Concrete highways usually have a single layer of subbase under the concrete pavement. Subbases and bases can be constructed using untreated, cement-treated, lean-treated, asphalt-treated, and permeable bases. Detailed information on the construction of each subbase or base type is found in ACPA (2007). Whatever type is used, the final l
the concrete surface should be uniform in strength, smooth, and provide the characteristics assumed by the designer and contained in the job specifications.

5.2—Production, placing, consolidation, and finishing concrete pavement

5.2.1 *Introduction*—Quality concrete pavement can be constructed using a variety of methods. Today, large paving projects are built using slipform paving machines and concrete produced at a dedicated site production facility. Smaller jobs may use concrete from commercial concrete plants and use simple formed construction techniques. The following section on production applies equally to site and commercial production roadways, as shown in Fig. 5.2.1. Construction discussion will focus on slipform methods, and information on formed paving construction will be presented.

5.2.2 *Concrete production*

5.2.2.1 *Handling and storing materials*—Aggregates should be handled and stored in a manner that minimizes segregation, degradation, contamination, or mixing different kinds and sizes. A preferred method of stockpiling coarse aggregates to minimize segregation is construction of the stockpile in successive horizontal layers not more than 6 ft (2 m) thick, with each layer completed over the entire stockpile area before the next is started. Successive lifts should not be allowed to cascade over lower lifts. Radial stacking conveyors can be a great source of segregation. The drop from the conveyor should be minimized. Stacker discharge should be used to build a windrow, which is then leveled before the next stockpile lift is added. If operation of hauling equipment on a stockpile is necessary, boards should cover all ramps and runways on the stockpile, or rubber-tired vehicles should be used to minimize degradation. Rejected material can be reprocessed and returned to the stockpile, provided the reprocessed materials meet the applicable specifications. Use care in removing the aggregates from the stockpiles to prevent segregation. Information about stockpiling in specific situations is found in ACI 304R and 221.1R.

Frozen aggregates or aggregates containing frozen lumps should be thawed before use. Aggregate moisture content should be reasonably uniform when delivered to the mixer. Wetting of dry aggregates prior to batching will affect cooling by evaporation and can, if carefully done, minimize moisture variations and reduce excessive absorption of mixing water. Fine aggregates, including those produced or manipulated by hydraulic methods, should be allowed to drain for at least 12 hours before use. Stockpiles, or cars and barges equipped with seep holes, are considered to offer suitable opportunity for drainage.

If lightweight aggregate sand is used, it should be saturated surface-dry (SSD) when batched into the concrete mixer. It is appropriate to have it delivered to the batch plant in such condition and to use sprinklers to maintain it at SSD. The batching sequence should be to batch the lightweight
ate sand (LWAS) along with a sizable part of the water before other ingredients are batched.

5.2.2.2 *Storage of cementitious materials*—Cementitious materials should be stored in closed, watertight facilities. If cement and SCMs are stored in adjacent silos, the common wall should be a double wall with a void that is inspectable