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Guide to Design of Slabs-on-Ground

Reported by ACI Committee 360

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Guide to Design of Slabs-on-Ground

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Guide to Design of Slabs-on-Ground

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This guide presents information on the design of slabs-on-ground, primarily industrial floors. It addresses the planning, design, and detailing of slabs. Background information on design theories is followed by discussion of the types of slabs, soil-support systems, loadings, and jointing. Design methods are given for unreinforced concrete, reinforced concrete, shrinkage-compensating concrete, post-tensioned concrete, fiber-reinforced concrete slabs-on-ground, and slabs-on-ground in refrigerated buildings, followed by information on shrinkage and curling. Advantages and disadvantages of these slab design methods are provided, including the ability of some slab designs to minimize cracking and curling more than others. Even with the best slab designs and proper construction, it is unrealistic to expect crack-free and curl-free floors. Every owner should be advised by the designer and contractor that it is normal to expect some cracking and curling on every project. This does not necessarily reflect adversely on the adequacy of the floor's design or quality of construction. Design examples are given.

Keywords: curling; design; floors-on-ground; grade floors; industrial floors; joints; load types; post-tensioned concrete; reinforcement (steel, fibers); shrinkage; shrinkage-compensating; slabs; slabs-on-ground; soil mechanics; warping.

CONTENTS

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CHAPTER 1—INTRODUCTION

1.1—Purpose and scope

This guide presents information on the design of slabs-on-ground. Design is the decision-making process of planning, sizing, detailing, and developing specifications preceding construction of slabs-on-ground. Information on other aspects, such as materials, construction methods, placement of concrete, and finishing techniques is included only where needed in making design decisions.

In the context of this guide, slab-on-ground is defined as: a slab, supported by ground, whose main purpose is to support the applied loads by bearing on the ground. The slab is of uniform or variable thickness and it may include stiffening elements such as ribs or beams. The slab may be unreinforced or reinforced with nonprestressed reinforcement, fibers, or post-tensioned tendons. The reinforcement may be provided to limit crack widths resulting from shrinkage and temperature restraint and the applied loads. Post-tensioning tendons may be provided to minimize cracking due to shrinkage and temperature restraint, resist the applied loads, and accommodate movements due to expansive soil volume changes.

This guide covers the design of slabs-on-ground for loads from material stored directly on the slab, storage rack loads, and static and dynamic loads associated with equipment and vehicles. Other loads, such as roof loads transferred through dual-purpose rack systems, are also mentioned.

This guide discusses soil-support systems, shrinkage and temperature effects; cracking, curling or warping; and other concerns affecting slab design. Although the same general principles are applicable, this guide does not specifically address the design of roadway pavements, airport pavements, parking lots, or mat foundations.

1.2—Work of ACI Committee 360 and other relevant committees

There are several ACI committees listed below that provide relevant information concerning slabs-on-ground design and construction or similar slab types that are not addressed in this guide such as pavements, parking lots, or mat foundations. These committees provide documents where more detailed information for topics discussed in this guide can be found.

1.2.1 ACI Committee 117 develops and reports information on tolerances for concrete construction through liaison with other ACI committees.

1.2.2 ACI Committee 223 develops recommendations on the use of shrinkage-compensating concrete.

1.2.3 ACI Committee 301 develops and maintains specifications for concrete construction.

1.2.4 ACI Committee 302 develops and reports information on materials and procedures for the construction of concrete floors. ACI 302.1R provides guidelines and recommendations on materials and slab construction. ACI 302.2R provides guidelines for concrete slabs that receive moisture-sensitive flooring materials.

1.2.5 ACI Committee 318 develops and maintains building code requirements for structural concrete.

1.2.6 ACI Committee 325 develops and reports information on concrete pavements.

1.2.7 ACI Committee 330 develops and reports information on concrete parking lots and paving sites. Parking lots and paving sites have unique considerations that are covered in ACI 330R.

1.2.8 ACI Committee 332 develops and reports information on concrete in residential construction.

1.2.9 ACI Committee 336 develops and reports information on footings, mats, and drilled piers. The design procedures for combined footings and mat foundations are given in ACI 336.2R. Mat foundations are typically more rigid and more heavily reinforced than common slabs-on-ground.

1.2.10 ACI Committee 360 develops and reports information on the design of slabs-on-ground, with the exception of highways, parking lots, airport pavements, and mat foundations.

1.2.11 ACI Committee 544 develops and reports information on concrete reinforced with short, discontinuous, randomly-dispersed fibers. ACI 544.3R is a guide for specifying, proportioning, and production of fiber-reinforced concrete (FRC).

1.3—Work of non-ACI organizations

Numerous contributions of slabs-on-ground design and construction information used in this guide come from organizations and individuals outside the American Concrete Institute. The U.S. Army Corps of Engineers (USACE), the National Academy of Science, and the Department of Housing and Urban Development (HUD) have developed guidelines for floor slab design and construction. The Portland Cement Association (PCA), Wire Reinforcement Institute (WRI), Concrete Reinforcing Steel Institute (CRSI), Post-Tensioning Institute (PTI), as well as several universities and consulting engineers have studied slabs-on-ground and developed recommendations for their design and construction. In addition, periodicals such as *Concrete International* and *Concrete Construction* have continuously disseminated information about slabs-on-ground.

1.4—Design theories for slabs-on-ground

1.4.1 *Introduction*—Stresses in slabs-on-ground result from applied loads and volume changes of the soil and concrete. The magnitude of these stresses depends on factors such as the degree of slab continuity, subgrade strength and

uniformity, construction method, construction quality, and magnitude and position of the loads. In most cases, the effects of these factors are evaluated by making simplified analysis assumptions with respect to material properties and soil-structure interaction. The following sections briefly review some of the design theories of soil-supported concrete slabs.

1.4.2 *Review of classical design theories*—Design methods for slabs-on-ground are based on theories originally developed for airport and highway pavements. Westergaard developed one of the first rigorous theories of structural behavior of rigid pavement (Westergaard 1923, 1925, 1926). This theory considers a homogeneous, isotropic, and elastic slab resting on an ideal subgrade that exerts, at all points, a vertical reactive pressure proportional to slab deflection; known as a Winkler subgrade (Winkler 1867). The subgrade acts as a linear spring with a proportionality constant k with units of pressure (lb/in.^{2} [kPa]) per unit deformation (in. [m]). The units are commonly abbreviated as lb/in.^{3} (kN/m^3). This constant is defined as the modulus of subgrade reaction.

In the 1930s, the structural behaviors of concrete pavement slabs were investigated at the Arlington Virginia Experimental Farm and at the Iowa State Engineering Experiment Station. Good agreement occurred between experiential stresses and those computed by the Westergaard's theory, as long as the slab remained continuously supported by the subgrade. Corrections were required only for the Westergaard corner formula to account for the effects of slab curling and loss of contact with the subgrade. Although choosing the modulus of subgrade reaction was essential for good agreement with respect to stresses, there remained ambiguity in the methods used to determine the correction coefficient.

In the 1930s, experimental information showed that the behavior of many subgrades may be close to that of an elastic and isotropic solid. Two characteristic constants—the modulus of soil deformation and Poisson's ratio—are typically used to evaluate the deformation response of such solids.

Based on the concept of the subgrade as an elastic and isotropic solid, and assuming that the slab is of infinite extent but of finite thickness, Burmister proposed the layered-solid theory of structural behavior for rigid pavements (Burmister 1943). He suggested basing the design on a criterion of limited deformation under load. Design procedures for rigid pavements based on this theory are not sufficiently developed for use in engineering practice. The lack of analogous solutions for slabs of finite extent, for example, edge and corner cases, is a particular deficiency. Other approaches based on the assumption of a thin elastic slab of infinite extent resting on an elastic, isotropic solid have been developed. The preceding theories are limited to behavior in the linear range where deflections are proportional to applied loads. Lösberg (Lösberg 1978; Pichumani 1973) later proposed a strength theory based on the yield-line concept for ground-supported slabs, but the use of ultimate strength for slab-on-ground design is not common.

All existing design theories are grouped according to models that simulate slab and the subgrade behavior. Three models used for slab analysis are:

1. Elastic-isotropic solid;
2. Thin elastic slab; and
3. Thin elastic-plastic slab.

Two models used for subgrade are:

1. Elastic-isotropic solid; and
2. Winkler (1867).

The Winkler subgrade models the soil as linear springs so that the reaction is proportional to the slab deflection. Existing design theories are based on various combinations of these models. The methods in this guide are generally graphical, plotted from computer-generated solutions of selected models. Design theories need not be limited to these combinations. The elastic-isotropic model provides close prediction for the response of real soils, but the Winkler model is widely used for design and a number of investigators have reported good agreement between observed responses to the Winkler-based predictions.

1.4.3 *Finite-element method*—The classical differential equation of a thin elastic plate resting on an elastic subgrade is often used to represent the slab-on-ground. Solving the governing equations by conventional methods is feasible for simplified models where slab and subgrade are assumed to be continuous and homogeneous. In reality, a slab-on-ground usually contains discontinuities, such as joints and cracks, and the subgrade support may not be uniform. Thus, the use of this approach is limited.

The finite-element method can be used to analyze slabs-on-ground, particularly those with discontinuities. Various models have been proposed to represent the slab (Spears and Panarese 1983; Pichumani 1973). Typically, these models use combinations of elements, such as elastic blocks, rigid blocks, and torsion bars, to represent the slab. The subgrade is typically modeled by linear springs (Winkler subgrade) placed under the nodal joints. Whereas the finite-element method offers good potential for complex problems, graphical solutions and simplified design equations have been traditionally used for design. The evolution of modern computer software has made modeling with finite elements more feasible in the design office setting.

1.5—Construction document information

Listed below is the minimum information that should be addressed in the construction documents prepared by the designer. Refer to ACI 302.1R for information related to the installation and construction for some of these items.

- Slab-on-ground design criteria;
- Base and subbase materials, preparation requirements, and vapor retarder/barrier, when required;
- Concrete thickness;
- Concrete compressive strength, or flexural strength, or both;
- Concrete mixture proportion requirements, ultimate dry shrinkage strain, or both;
- Joint locations and details;
- Reinforcement (type, size, and location), when required;
- Surface treatment, when required;
- Surface finish;

- Tolerances (base, subbase, slab thickness, and floor flatness and levelness);
- Concrete curing;
- Joint filling material and installation;
- Special embedments;
- Testing requirements; and
- Preconstruction meeting, quality assurance, and quality control.

1.5.1 *Slab-on-ground design criteria*—It is helpful that when the slab-on-ground design criteria are well established, that it be shown on the drawings. This information is especially useful when future modifications are made to the slab or its use. Design issues, such as the slab contributing to wind or seismic resistance or building foundation uplift forces, would not be readily apparent unless noted on the drawings. Because it is not readily apparent when a slab is used as a horizontal diaphragm, it should be noted on the drawings. Removing or cutting a slab that is designed to resist uplift or horizontal forces could seriously impair the building's stability.

The design criteria should include some of the following:

- Geotechnical soil properties used for the different loading types;
- Uniform storage loading;
- Lift-truck and vehicle loadings;
- Rack loadings;
- Line loads;
- Equipment loads;
- When the slab is used to resist wind or seismic foundation uplift forces; and
- When the slab is used as a horizontal diaphragm and to resist horizontal forces or both due to tilt-walls, masonry walls, tops of retaining walls, and metal building system columns.

Refer to Appendix 7 for an example of design criteria.

1.5.2 *Floor flatness and levelness tolerances*—Tolerances for floor uses should conform to ACI 117. For additional information, including how to specify floor flatness and levelness requirements, refer to ACI 117 commentary and ACI 302.1R.

When using slabs in offices, areas of pedestrian traffic, wide aisle warehousing, and manufacturing, where the movement is intended to be random in any direction, then a random traffic tolerance system, such as F_F/F_L, should be designated. The subject areas should be shown on the construction documents and tolerances specified.

In defined traffic areas such as narrow aisle or very narrow aisle warehousing and manufacturing, where vehicle paths are restrained by rail, wire, laser, or telemetry guidance systems, a tolerance system such as F-min should be implemented (Fudala 2008), with subject areas shown on the construction documents, and tolerances specified. Table 1.1 provides typical defined traffic values for different rack heights that have been used successfully. Narrow aisle and very narrow aisle systems, however, use specialized equipment and the manufacturers should be consulted for F-min recommendations.

Table 1.1—Defined traffic values

Rack height, ft (m)	Longitudinal* F-min	Transverse† F-min
0 to 25 (0 to 7.6)	50	60
26 to 30 (7.9 to 9.1)	55	65
31 to 35 (9.4 to 10.7)	60	70
36 to 40 (11 to 12.2)	65	75
41 to 45 (12.5 to 13.7)	70	80
46 to 50 (14 to 15.2)	75	85
51 to 65 (15.5 to 19.8)	90	100
66 to 90 (20.1 to 27.4)	100	125

*Longitudinal value between the front and rear axle.
†Transverse value between loaded wheel tracks.

1.6—Further research

There are many areas that need additional research. Some of these areas are:

- Developing concrete mixture proportions that have low shrinkage characteristics and are workable, finishable, and provide a serviceable surface;
- Flexural stress in slabs with curl and applied loads and how curling stresses change over time due to creep;
- Base restraint due to shrinkage and other volume changes and how this restraint changes over time;
- Crack widths for different amounts of reinforcement for slabs-on-ground;
- Provide guidance on acceptable joint and crack widths for different slab usages;
- Provide dowel recommendations based on loadings (lift truck, rack post, and uniform storage) rather than slab thickness;
- Provide plate dowel spacing recommendations for plate dowel geometries;
- Provide design guidance for slabs with macrosynthetic fibers;
- Provide design aids for slabs with rack uplift loads due to seismic and other uplift loadings;
- Provide design aids for slabs with non-uniform rack post loads;
- Develop a standardized method for testing and specifying slab surface abrasion resistance;
- Soil properties and how they may change over time under load repetitions, wide area long-term loadings, or both; and
- Recommended joint spacing for fiber-reinforced concrete.

CHAPTER 2—DEFINITIONS

2.1—Definitions

ACI provides a comprehensive list of definitions through an online resource. "ACI Concrete Terminology," http://terminology.concrete.org. Definitions provided herein complement that resource.

curling or warping—out-of-plane deformation of the corners, edges, and surface of a pavement, slab, or wall panel from its original shape.

slab-on-ground—slab, supported by ground, whose main purpose is to support the applied loads by bearing on the ground.

CHAPTER 3—SLAB TYPES

3.1—Introduction

Chapter 3 identifies and briefly discusses the common types of slab-on-ground construction. The term "slab-on-ground" is preferred but "slab-on-grade" is often used. Slab-on-ground includes interior slabs subject to loadings as described in Chapter 5. These include industrial, commercial, residential, and related applications. Although the term might include parking lot and roadway pavements, this guide does not specifically address them.

An important responsibility of the slab designer is to discuss the requirements of the floor slab with the owner. Discussions should include the advantages and disadvantages of the different slab types and how they relate to the owner's requirements. It is important for this discussion to occur so the owner has reasonable expectations of the slab performance and required future maintenance for the slab type selected. Some of the more important expectations that should be discussed for the prospective slab type are:

- Cracking potential;
- Crack widths for slabs designed with reinforcement to limit crack widths;
- Use of doweled joints versus aggregate interlock;
- Possible future repairs including joint deterioration;
- Joint maintenance requirements and the owner's responsibility for this maintenance;
- Floor flatness and levelness requirements to meet the owner's needs;
- Changes to the flatness and levelness over time, especially in low-humidity environments;
- Advantages and disadvantages of slab placement with the watertight roofing system in place versus placing the slab in the open;
- Level of moisture vapor resistance required; and
- Advantages and disadvantages of using the building floor slab for tilt-wall construction form and temporary bracing.

3.2—Slab types

There are four basic design choices for slab-on-ground construction:

1. Unreinforced concrete slab.
2. Slabs reinforced to limit crack widths due to shrinkage and temperature restraint and applied loads. These slabs consist of:
   a. Nonprestressed steel bar, wire reinforcement, or fiber reinforcement, all with closely spaced joints; and
   b. Continuously reinforced, free-of-sawcut, contraction joints.
3. Slabs reinforced to prevent cracking due to shrinkage and temperature restraint and applied loads. These slabs consist of:
   a. Shrinkage-compensating concrete; and
   b. Post-tensioned.
4. Structural slabs designed in accordance with ACI 318:
   a. Plain concrete; and
   b. Reinforced concrete.

3.2.1 *Unreinforced concrete slab*—The thickness is determined as a concrete slab without reinforcement; however, it may have joints strengthened with steel dowels. It is designed to remain uncracked between joints when loaded and restraint to concrete volumetric changes. Unreinforced concrete slabs do not contain macrosynthetic fibers, wire reinforcement, steel fibers, plain or deformed bars, post-tensioning, or any other type of steel reinforcement. Type I or II portland cement (ASTM C150/C150M) is normally used. Drying shrinkage effects and uniform subgrade support on slab cracking are critical to the performance of unreinforced concrete slabs. Refer to Chapter 7 for unreinforced slab design methods.

3.2.2 *Slabs reinforced for crack-width control*—Thickness design can be the same as for unreinforced concrete slabs, and they are designed to remain uncracked when loaded. For slabs constructed with portland cement, shrinkage crack widths (when cracking occurs) between joints are controlled by a nominal quantity of distributed reinforcement. Slabs reinforcement can consist of bars, welded wire reinforcement sheets, steel fibers, or macrosynthetic fibers. Bar and wire reinforcement should be stiff enough to be accurately located in the upper 1/3 of the slab.

Bars or welded wire reinforcement are used to provide flexural strength at a cracked section. In this case, and for slabs of insufficient thickness to carry the applied loads as an unreinforced slab, the reinforcement required for flexural strength should be sized by reinforced concrete theory as described in ACI 318. Using the methods in ACI 318 with high steel reinforcement stresses, however, may lead to unacceptable crack widths. Building codes do not support the use of fiber reinforcement to provide flexural strength in cracked sections for vertical or lateral forces from other portions of a structure.

Other than post-tensioning or the reinforcement in a shrinkage-compensating slab, reinforcement does not prevent concrete cracking. Typically, the most economical way to increase flexural strength is to increase the slab thickness. Chapters 7, 8, and 11 contain design methods for slabs reinforced for crack-width control.

3.2.3 *Slabs reinforced to prevent cracking*—Post-tensioned slabs and shrinkage-compensating slabs are typically designed not to crack, but some incidental minor cracking may occur. For shrinkage-compensating slabs, the slab is designed unreinforced, and the reinforcement is designed to prestress the expanding slab to offset the stresses caused by the shrinkage and temperature restraint. For post-tensioned slabs, the reinforcement is typically designed to compensate for shrinkage and temperature restraint stress and applied loads.

Shrinkage-compensating concrete slabs are produced either with a separate component admixture or with ASTM C845 Type K expansive cement. This concrete does shrink, but first expands to an amount intended to be slightly greater than its drying shrinkage. To limit the initial slab expansion and to prestress the concrete, reinforcement is distributed in the upper 1/3 of the slab. Such reinforcement should be rigid and positively positioned. The slab should be isolated from fixed portions of the structure, such as columns and perimeter

Table 3.1—General comparison of slab types

Slab type	Advantages	Disadvantages
Unreinforced concrete	• Simple to construct. • Generally is less expensive to install than slabs designed by other methods.	• Requires relatively closely spaced sawcut contraction joints. • More opportunity for slab curl and joint deterioration. • Large number of joints to maintain. • Positive load transfer may be required at joints. • Flatness and levelness may decrease over time.
Reinforced with deformed bars or welded-wire reinforcement sheets for crack-width control	• Reinforcement is used to limits crack width.	• May be more expensive than an unreinforced slab. • Reinforcement can actually increase the number of random cracks, particularly at wider joint spacings. • More opportunity for slab curl and joint deterioration. • Positive load transfer may be required at joints.
Continuously reinforced with deformed bars or welded-wire reinforcement mats	• Sawcut contract joints can be eliminated where sufficient reinforcement is used. • Eliminates sawcut contraction joint maintenance. • Curling is reduced when high amounts of reinforcement are used. • Less changes in flatness and levelness with time.	• Requires relatively high amounts (at least 0.5%) of continuous reinforcement placed near the top of the slab to eliminate joints. • Typically produces numerous, closely spaced, fine cracks (approximately 3 to 6 ft [0.9 to 1.8 m]) throughout slab.
Shrinkage-compensating concrete	• Allows construction joint spacings of 40 to 150 ft (12 to 46 m). • Sawcut contraction joints are normally not required. • Reduces joint maintenance cost due to increased spacing of the joints reducing the total amount of joints. • Negligible curl at the joints. • Increases surface durability and abrasion resistance (ACI 223, Section 2.5.7—Durability).	• Requires reinforcement to develop shrinkage compensation. • Window of finishability is reduced. • Allowance should be made for concrete to expand before drying shrinkage begins. • Construction sequencing of adjacent slab panels should be considered, or joints should be detailed for expansion. • Contractor should have experience with this type of concrete.
Post-tensioned	• Construction spacings 100 to 500 ft (30 to 150 m). • Most shrinkage and flexural cracks can be avoided. • Eliminates sawcut contraction joints and their maintenance. • Negligible slab curl when tendons are draped near joint ends. • Improved long-term flatness and levelness. • Decreased slab thickness or increased flexural strength. • Resilient when overloaded. • Advantages in poor soil conditions	• More demanding installation. • Contractor should have experience with post-tensioning or employ a consultant with post-tensioning experience. • Inspection essential to ensure proper placement and stressing of tendons. • Uneconomical for small areas. • Need to detail floor penetrations and perimeter for slab movement. • Impact of cutting tendons should be evaluated for post-construction slab penetrations.
Steel fiber-reinforced concrete	• Increased resistance to impact and fatigue loadings when compared to slabs reinforced with bars or mesh. • Simple to construct.	• May require adjustments to standard concrete mixing, placement, and finishing procedures. • Fibers may be exposed on the surface of slab. • Floors subjected to wet conditions may not be suitable for steel fiber because fibers close to the surface and in water-permeable cracks will rust.
Synthetic fiber-reinforced concrete	• Helps reduce plastic shrinkage cracking. • Simple to construct. • Macrosynthetic fibers provide increased resistance to impact and fatigue loadings, similar to steel fibers. • Synthetic fibers do not corrode.	• Microsynthetic fibers do not help in controlling drying shrinkage cracks. • Joint spacing for microsynthetic fiber-reinforced slabs are the same as unreinforced slabs.
Structural slabs reinforced for building code requirements	• Slabs can carry structural loads such as mezzanines. • Reduces or eliminates sawcut contraction joints where sufficient reinforcement is used.	• Slab may have numerous fine or hairline cracks if reinforcement stresses are sufficiently low.

foundations, with a compressible material that allows the initial slab expansion.

Refer to Chapters 9 and 10 for design methods for slabs reinforced to prevent cracking.

3.2.4 *Structural slabs*—Structural plain and reinforced slabs that transmit vertical loads or lateral forces from other portions of the structure to the soil should be designed in accordance with ACI 318. Using the methods in ACI 318 with high steel reinforcement stresses, however, may lead to unacceptable crack widths.

3.3—General comparison of slab types

To assist with selecting the most appropriate slab type for the particular project, Table 3.1 provides general advantages and disadvantages for the slab types discussed in Section 3.2.

3.4—Design and construction variables

Design and construction of slabs-on-ground involves both technical and human factors. Technical factors include loadings, soil-support systems, joint types and spacings, design method, slab type, concrete mixture, development of maintenance procedures, and the construction process. Human factors include the workers' abilities, feedback to evaluate the construction process, and conformance with proper maintenance procedures for cracking, curling, and shrinkage. These and other factors should be considered when designing a slab (Westergaard 1926).

3.5—Conclusion

No single slab design method is recommended for all applications. Rather, from the number of identifiable construction concepts and design methods, a combination should be selected based on the requirements of the specific application.

CHAPTER 4—SOIL SUPPORT SYSTEMS FOR SLABS-ON-GROUND

4.1—Introduction

The design of slabs-on-ground to resist moments and shears caused by applied loads depends on the interaction between the concrete slab and the supporting materials. Properties and dimensions of the slab and the supporting materials are important in the design of a slab-on-ground. The support system should be of acceptable uniform strength and not easily susceptible to the effects of climatic changes. Slab-on-ground failures can occur because of an improper support system. Issues related to the support system of the slab-on-ground include:

- Geotechnical engineering reports providing soil properties;
- Subgrade classification;
- Modulus of subgrade reaction;
- Design of the slab support system;
- Site preparation; and
- Inspection and testing of the slab support system.

This chapter is limited to aspects of the support system necessary for proper slab-on-ground performance.

The slab support system consists of a subgrade, usually a base, and sometimes a subbase, as illustrated in Fig. 4.1. Crushed rock, gravels, or coarse sands have high strength, low compressibility, and high permeability, are commonly used as base courses. Crushed rock, gravels, sands, select soils, and stabilized soils are commonly used as subbases and may be used as base materials. Soils in the subgrade are generally the ultimate supporting materials, but bedrock, competent or weathered, may also be encountered. When the existing soil has uniform strength and other necessary properties to support the slab, the slab may be placed directly on the existing subgrade. The existing grade, however, is frequently not at the desired elevation or slope and some cut and fill is required. To improve surface drainage or to elevate the floor level, controlled fill using on-site or imported soils is required on some sites.

4.2—Geotechnical engineering reports

4.2.1 *Introduction*—Geotechnical engineering investigations supply subsurface site information primarily for design and construction of the building foundation elements and to meet building code requirements. Within the geotechnical engineering report, slab-on-ground support is frequently discussed, and subgrade drainage and preparation recommendations are given. Even when slab support is not discussed in detail, information given within these reports, such as boring or test pit logs, field and laboratory test results, and discussions of subsurface conditions, are useful in evaluating subgrade conditions relative to slab-on-ground design and construction.

4.2.2 *Boring or test pit logs*—Descriptions given on boring or test pit logs provide information on the texture of the soils and their moisture condition and relative density, when noncohesive; or consistency, when cohesive. These logs present field test results, such as the standard penetration test (ASTM D1586) in blows per 6 in. (150 mm) interval values. The log notes the location of the water table at the time of boring and depths to shallow bedrock. The Atterberg limits, and laboratory test results, such as the moisture content and dry density of cohesive soils, are often included on the boring logs or in the geotechnical report. Also, the soil is classified as discussed in Section 4.3.

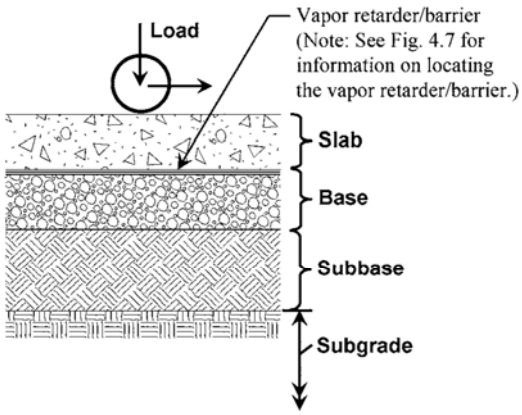


Fig. 4.1—Slab support system terminology.

4.2.3 *Report evaluations and recommendations*—Evaluations and recommendations relative to the existing subgrade material, its compaction, and supporting capability can be included in the report and should be evaluated against the design requirements. The geotechnical engineer may provide suggestions for subbase and base course materials. In some cases, local materials that are peculiar to that area, such as crushed sea shells, mine tailings, bottom ash, and other waste products, can be economically used. The local geotechnical engineer is generally knowledgeable about using these materials in the project area. The expected performance characteristics of the slab-on-ground should be made known to the geotechnical engineer before the subsurface investigation to obtain the best evaluation and recommendations. For example, some of the information that should be provided to the geotechnical engineer includes:

- Facility use and proposed floor elevation;
- Type and magnitude of anticipated loads;
- Environmental conditions of the building space;
- Floor levelness and flatness criteria; and
- Floor-covering requirements.

It may be beneficial for the geotechnical engineer to visit local buildings or other facilities of the client that have similar use. Coordination between the geotechnical engineer and the slab-on-ground designer from the beginning of the project can lead to an adequate and economical slabs-on-ground.

4.3—Subgrade classification

Soil supporting the slab-on-ground may meet the criteria for a subbase or even a base material, but should be identified and classified to estimate its suitability as a subgrade. The Unified Soil Classification System is predominantly used in the U.S. and is referred to in this guide. Table 4.1 provides information on classification groups of this system and important criteria for each soil group. Visual procedures (ASTM D2488) can be used, but laboratory test results (ASTM D2487) provide classifications that are more reliable.

Table 4.1—Unified soil classification system (Winterkorn and Fang 1975)

Field identification procedures (excluding particles larger than 3 in. [75 mm], and basing fractions on estimated weights)				Group symbol	Typical names
Coarse-grained soils (more than half of material is larger than No. 200 sieve* [75 μm])	Gravels (more than half of coarse fraction is larger than No. 4 sieve* [4.75 mm])	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well-graded gravel, gravel-sand mixtures, little or no fines
			Predominantly one size or a range of sizes with some intermediate sizes missing	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, refer to CL below)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures
			Plastic fines (for identification procedures, refer to ML below)	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures
	Sands (more than half of coarse fraction is smaller than No. 4 sieve* [4.75 mm])	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	SW	Well-grades sands, gravelly sands, little or no fines
			Predominantly one size or range of sizes with some intermediate sizes missing	SP	Poorly graded sands, gravelly sands, little or no fines
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, refer to ML below)	SM	Silty sands, poorly graded sand-silt mixtures
			Plastic fines (for identification procedures, refer to CL below)	SC	Clayey sands, poorly graded sand-clay mixtures

Identification procedures on fraction smaller than No. 40 (4.25 μm) sieve						
		Dry strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group symbol	Typical names
Fine-grained soils (more than half of material is smaller than No. 200 sieve* [75 μm])	Silts and clays (liquid limit less than 50)	None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity
		Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Slight to medium	Slow	Slight	OL	Organic silts and organic-silt clays of low plasticity
	Silts and clays (liquid limit greater than 50)	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays
		Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity
Highly organic soils		Readily identified by color, odor, spongy feel; frequently by fibrous texture			PT	Peat or other highly organic soils

*All sieve sizes herein are U.S. standard. The No. 200 sieve (75 μm) is approximately the smallest particle visible to the naked eye. For visual classifications, the 1/4 in. (6.3 mm) size may be used as equivalent for the No. 4 (4.75 mm) sieve size. Boundary classifications: soil possessing characteristics of two groups are designated by combinations of group symbols.

For example, use the plasticity chart of Table 4.2 to classify the fine-grained soils.

The following tests and test methods are useful for soil classification:

- Moisture content: ASTM D2216;
- Specific gravity: ASTM D854;
- Liquid and plastic limits: ASTM D4318; and
- Expansion Index: ASTM D4829.

The standard Proctor compaction test (ASTM D698) and modified Proctor compaction test (ASTM D1557) are not strictly classification tests. Their moisture-density relationships are useful in assessing a soil subgrade or subbase. A more detailed listing of the ASTM standards appears in Chapter 15.

4.4—Modulus of subgrade reaction

4.4.1 *Introduction*—Design methods listed in Chapter 3, including Westergaard's pioneering work on rigid pavement analysis (Westergaard 1923, 1926), employ the modulus of subgrade reaction as a single property to represent the design support strength. This modulus, also called the modulus of soil reaction or Winkler foundation, is a spring constant that assumes a linear response between load and deformation from the subgrade.

Actually, there is no single k value for a subgrade because the relationship between load and soil deformation is nonlinear and is not a fundamental soil property. Figure 4.2 depicts a typical nonlinear relationship between a normal compressive load and the resulting deformation for an area. The type of soil structure, density, moisture content, and prior loading determine the load-deformation relationship. The relationship also depends on the width and shape of the loaded area, depth of the subgrade, and position under the slab. In addition, time may be a significant factor because any deeper compressible soils may settle due to consolidation, and near-surface soils may settle due to shrinkage from alternate wetting and drying. Nevertheless, the procedures for static nonrepetitive plate load tests outlined in ASTM D1196 have been used to estimate the subgrade modulus.

Table 4.2—Laboratory classification criteria for soils (Winterkorn and Fang 1975)

Major Divisions			Group Symbols	Typical Names	Laboratory Classification Criteria
Coarse-grained soils (More than half of material is larger than No. 200 [75 μm] sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 [4.75 mm] sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 [75 μm], coarse-grained soils are classified as follows: Less than 5 per cent: GW, GP, SW, SP; More than 12 per cent: GM, GC, SM, SC; 5 to 12 per cent: *Borderline* cases requiring dual symbols†
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	
		Gravels with fines (Appreciable amount of fines)	GM* d u	Silty gravels, gravel-sand-silt mixtures	
			GC	Clayey gravels, gravel-sand-clay mixtures	
	Sands (More than half of coarse fraction is smaller than No. 4 [4.75 mm] sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines	
			SP	Poorly graded sands, gravelly sands, little or no fines	
		Sands with fines (Appreciable amount of fines)	SM* d u	Silty sands, sand-silt mixtures	
			SC	Clayey sands, sand-clay mixtures	
Fine-grained soils (More than half of material is smaller than No. 200 [75 μm] sieve)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
			OL	Organic silts and organic silty clays of low plasticity	
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
			CH	Inorganic clays of high plasticity, fat clays	
			OH	Organic clays of medium to high plasticity, organic silts	
	Highly organic soils		Pt	Peat and other highly organic soils	

$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3

Not meeting all gradation requirements for GW

Atterberg limits below "A" line or P.I. less than 4 — Above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols

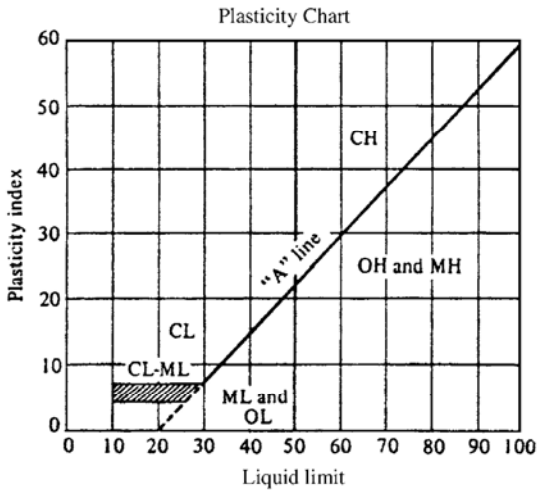
Atterberg limits below "A" line with P.I. greater than 7 — Above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols

$C_U = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3

Not meeting all gradation requirements for SW

Atterberg limits above "A" line or P.I. less than 4 — Limits plotting in hatched zone with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols

Atterberg limits above "A" line with P.I. greater than 7 — Limits plotting in hatched zone with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols



*Division of GM and SM groups into subdivisions of *d* and *u* are for roads and airfields only. Subdivision is based on Atterberg limits; suffix *d* used when L.L. is 28 or less and the P.I. is 6 or less; the suffix *u* used when L.L. is greater than 28.

†Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example, GW-GC, well-graded gravel-sand mixture with clay binder.

4.4.2 *Plate load field tests*—Determining the modulus of subgrade reaction on representative subgrade in place with a 30 in. (760 mm) diameter bearing plate, which is recommended by ASTM D1196, is time-consuming and expensive. It takes several days to plan and execute a load-testing program. Large loads may be needed to obtain significant settlement of the plates. Adjustments should be made for nonrecoverable deformation and any plate deflections. Because the load-deformation results are nonlinear, either an arbitrary load or deformation should be assumed to calculate k (Fig. 4.2).

Several tests over the project area are required to obtain representative k values, which generally result in a range of k values. A correction is generally necessary to account for future saturation of cohesive soil subgrades, and this requires

sampling and laboratory tests. It is usually impractical to conduct field tests on subgrade soils at their expected range of densities and moisture contents. It is also impractical to test the various possible types and thicknesses of base courses and subbases on a representative subgrade. It is difficult to test during adverse climatic conditions. Smaller plates, such as 12 in. (300 mm) diameter, have been used, but the diameter of the plate influences the results, and this is difficult to take into account when reporting a k value.

Typically, these tests are made directly on an unconfined natural or compacted subgrade or on a layer of compacted subbase or base course over a subgrade. The physical characteristics of the base course and subgrade material are necessary to properly interpret the plate bearing test results. At a minimum, data should include gradations, moisture contents, densities, and Atterberg limit of the materials in the supporting system. Before initiating a plate load field test, it is advisable to consult a geotechnical engineer familiar with site conditions to estimate price and time required and the probable results.

4.4.3 *American Association of State Highway Transportation Officials (AASHTO) approach*—For rigid pavements, AASHTO developed a design procedure using the following theoretical relationship between k values from plate bearing tests and M_R, the resilient modulus of the subgrade

$$k \text{ (lb/in.}^3\text{)} = M_R \text{ (psi)}/19.4 \text{ (in.-lb units)}$$
$$k \text{ (kN/m}^3\text{)} = M_R \text{ (kPa)} \times 2.03 \text{ (SI units)}$$

The resilient modulus is a measure of the assumed elastic property of soil considering its nonlinear characteristics. It is defined as the ratio of the repeated axial deviator stress to the recoverable axial strain and is widely recognized as a method for characterizing pavement materials. The AASHTO Test Method T 307 describes the methods for determining M_R. The value of M_R can be evaluated using a correlation with the older and more common California bearing ratio (CBR) test value (ASTM D1883) by the following empirical relationship (Heukelom and Klomp 1962)

$$M_R \text{ (psi)} = 1500 \times \text{CBR (in.-lb units)}$$
$$M_R \text{ (kPa)} = 10{,}342 \times \text{CBR (SI units)}$$

This approximate relationship has been used extensively for fine-grained soils having a soaked, saturated 96-hour CBR value of 10 or less (Heukelom and Klomp 1962). Correlations of M_R with clay content, Atterberg limits, and moisture content have also been developed.

The effective k value used for design, as recommended by AASHTO for rigid pavements, depends on several different factors besides the soil resilient modulus, including subbase types and thicknesses, loss of support due to voids, and depth to a rigid foundation. Tables and graphs in the "Guide for the Design of Pavement Structures" (AASHTO 1993) may be used to obtain an effective k for design of slabs-on-ground. The k values obtained from measured CBR and M_R data using the AASHTO relationships can yield unrealistically

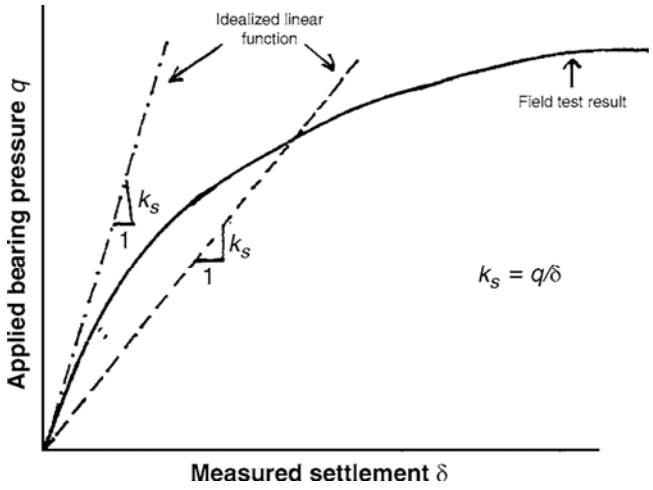


Fig. 4.2—Plate load-deformation diagram.

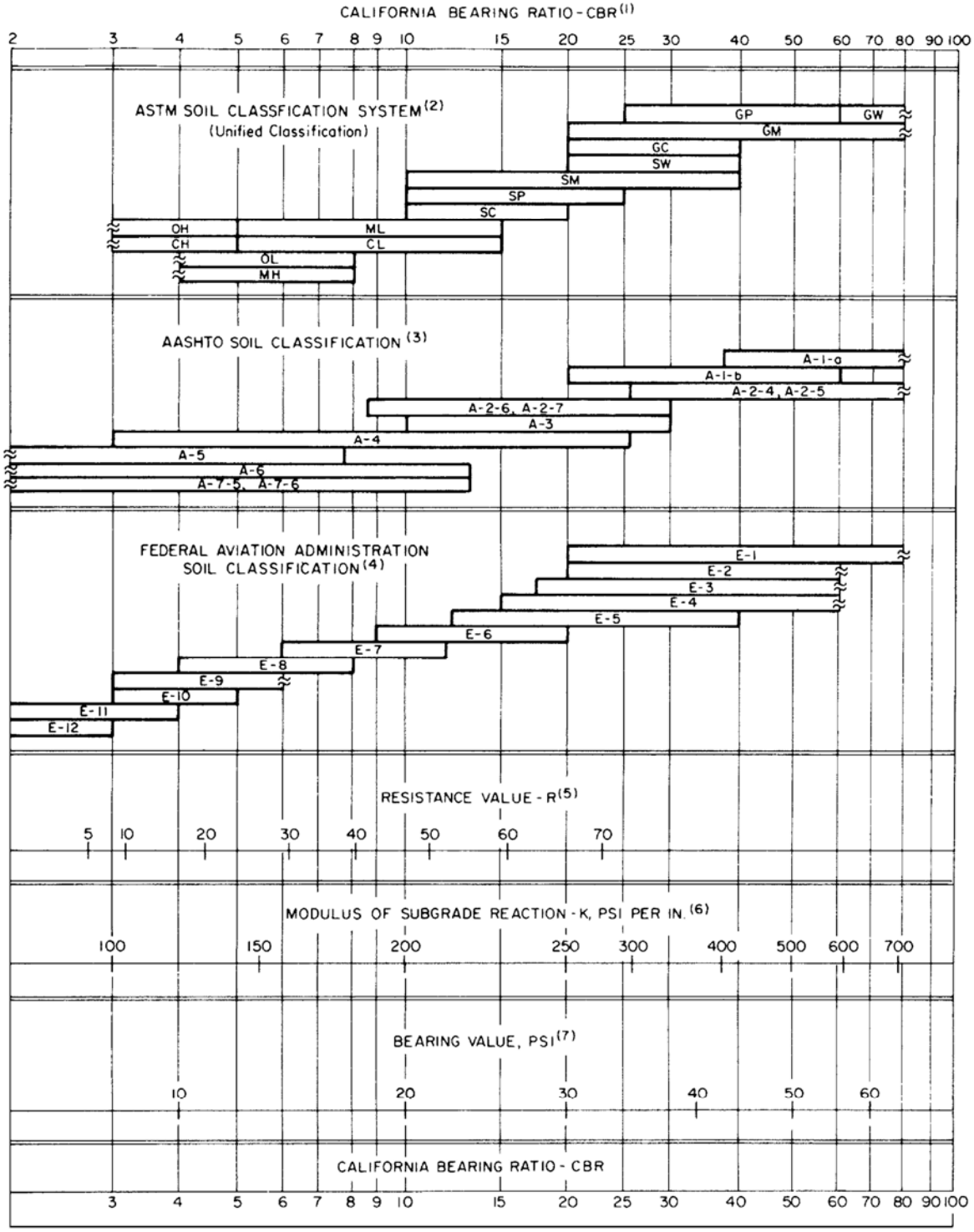
high values. It is recommended that the nomograph relationships contained in Fig. 4.3 be used to validate the results of correlated k values derived from AASHTO correlations.

4.4.4 *Other approaches*—The Corps of Engineers (COE) developed empirical relations between soil classification type, CBR, and k values, as illustrated by Fig. 4.3. These relationships are usually quite conservative. All of these test methods and procedures have been developed for pavements, not for slab-on-ground floors for buildings. Nevertheless, correlations such as these are widely used to approximate the subgrade support values for slab-on-ground design and construction.

4.4.5 *Influence of moisture content*—The moisture content of a fine-grained soil affects the modulus of subgrade reaction k at the time of testing and throughout the slab service life. Nearly all soils exhibit a decrease in k with an increase in saturation, but the amount of reduction depends chiefly on the texture of the soil, its density, and the activity of clay minerals present. In general, the higher the moisture content, the lower the supporting strength, but the relationship is unique for each type of soil. The more uniform the moisture content and dry density, the more uniform the support. Therefore, good site surface drainage and drainage of the subgrade is very important. Experience demonstrates that high water tables and broken water or drain lines cause slab-on-ground failures.

To evaluate the influence of moisture, test procedures (such as CBR), unconfined compression, and triaxial shear can be followed. Moisture and dry density ranges chosen for testing should match those anticipated in the field. Laboratory tests are more practical than field tests.

4.4.6 *Influence of soil material on modulus of subgrade reaction*—Soils found at a building site are capable of providing a range of subgrade support, as illustrated in Fig. 4.3. Clay soils, such as CL and CH materials, provide the lowest subgrade support. Well-graded, noncohesive soils, such as SW and GW material, provide the greatest support. An increase in density by compaction can improve a soil's strength, but to a limited extent. Using stabilization methods will also have a limited range of effectiveness. Drainage conditions can change the support strength of most soils, but



(1) For the basic idea, see O. J. Porter, "Foundations for Flexible Pavements," Highway Research Board *Proceedings of the Twenty-second Annual Meeting*, 1942, Vol. 22, pages 100-136.

(2) ASTM Designation D2487.

(3) "Classification of Highway Subgrade Materials," Highway Research Board *Proceedings of the Twenty-fifth Annual Meeting*, 1945, Vol. 25, pages 376-392.

(4) *Airport Paving*, U.S. Department of Commerce, Federal Aviation Agency, May 1948, pages 11-16. Estimated using values given in FAA *Design Manual for Airport Pavements*.(Formerly used FAA Classification; Unified Classification now used.)

(5) C. E. Warnes, "Correlation Between *R* Value and *k* Value," unpublished report, Portland Cement Association, Rocky Mountain-Northwest Region, October 1971 (best-fit correlation with correction for saturation).

(6) See T. A. Middlebrooks and G. E. Bertram, "Soil Tests for Design of Runway Pavements," Highway Research Board *Proceedings of the Twenty-second Annual Meeting*, 1942, Vol. 22, page 152.

(7) See item (6), page 184.

Fig. 4.3—Approximate interrelationships of soil classifications and bearing values (PCA 1988). (Note: 1 psi/in. = 0.271 kPa/mm; 1 psi = 6.90 kPa.)

this can be most significant for clays and silts. Frost action can also reduce the support strength of soils containing silt. Thus, the correlation between soil classification and supporting strength is useful for estimating the range of capability, but should be adjusted for expected site conditions.

4.4.7 *Uniformity of support*—The design charts of PCA, WRI, and COE indicate the influence that the modulus of subgrade reaction has on the required slab thickness. These design aids assume continuous slab contact with the base and a uniform subgrade modulus. Continuous intimate contact is not achieved in practice because of differences in composition, thickness, moisture content, slab curling, and subgrade density. By following the joint recommendations in Fig. 6.6, the curling stresses will be sufficiently low that the PCA, WRI, and COE methods will provide reasonable solutions. Cycles of load and climatic fluctuations of moisture may increase or decrease k, but such change is usually not uniform. Differences in subgrade support due to cuts and fills or irregular depths to shallow bedrock are common. Poor compaction control or variations in borrow material can cause fills to provide nonuniform support. Attempts to produce high subgrade moduli by compaction or stabilization may yield nonuniform support unless strict quality-control standards are implemented. Uniform high k values are difficult to achieve. After slab installation, densification of noncohesive soils, sand, and silts by vibration may yield nonuniform support. The shrinking and swelling action of cohesive soils (GC, SC, CL, and CH) causes cracks in concrete slabs, even when design and construction precautions are taken. Lack of uniform support can cause slab cracks. On some projects, a well-constructed subgrade has been compromised by utility trenches that were poorly backfilled. The importance of providing uniform support cannot be overemphasized. Inspection and testing of controlled fills should be mandatory.

4.4.8 *Influence of size of loaded area*—The k value, when derived from the plate load test, only provides information relative to the upper 30 to 60 in. (760 to 1520 mm) of the subsurface profile. This may be sufficient for the analyses of floor slabs subjected to relatively small concentrated loads, but it is not sufficient for floor slabs subjected to large, heavy loads. For example, a fully loaded warehouse bay measuring 25 x 25 ft (7.6 x 7.6 m) can load and consolidate soils 30 ft (9.1 m) or more when fills were used to develop the site. Slab settlement is not uncommon where fills were used to produce dock height floors or promote area drainage. The degree of settlement under such loading conditions typically indicates an equivalent k value of only 20 to 30% of that measured by a plate load test.

To properly consider the effect of heavy distributed loads on slab performance, a more comprehensive evaluation of subsurface conditions should be conducted. Such an evaluation may include the performance of soil test borings, laboratory tests of subgrade materials, or one of a variety of in-place testing techniques. Such information can be used to develop soil-support values that account for long-term consolidation settlements under sustained heavy distributed loads.

4.4.9 *Influence of time*—Time of load application and elapsed time are important. Short, transient loads such as lift trucks, produce smaller deformations than sustained loads; therefore, a higher k value can be used for rolling loads. With the passage of time, the subgrade and subbase is subject to load cycling. Applications of surface loads may increase the stiffness of the subgrade and subbase, and a higher k value will result. Unfortunately, this may also produce nonuniform support because the areas of load application will not usually be uniform.

Subgrade moisture change over time may also affect the soil-support system. Stability through changes of climate, such as protracted dry or wet weather conditions or cycles of freezing and thawing, should be considered.

4.5—Design of slab-support system

4.5.1 *General*—After the subgrade soils have been classified, the general range of their k values can be approximated from Fig. 4.3. Adjustments may be made on the basis of local experience, expected seasonal changes, and expected construction conditions.

With this information, a decision can be made whether to use the existing subgrade, improve it by compaction or stabilization, use a subbase and a base course, or vary the thickness of these layers. Initially, a wide range of subgrade conditions may exist across the site. The soil-support system is rarely uniform and some soil work is generally required to produce a more uniform surface to support the slab. The extent of this work, such as the degree of compaction or the addition of a base course, is generally limited by economics. Selection of crushed rock or soils in the well-graded gravel (GW) and poorly-graded gravel (GP) groups may appear costly as a base material, but the selection of these materials has distinct advantages. They improve the modulus of subgrade reaction, produce more uniform support, and provide an all-weather working surface to speed construction during inclement weather.

4.5.2 *Economics and simplified design*—Designing a slab-support system requires identification of the subgrade material and the conditions to which it will be exposed. This knowledge is essential to estimate the modulus of subgrade and the potential volume change. With knowledge of soil classification and some local experience, the engineer can select an appropriate k value and design for the specific soil conditions. The slab thickness calculation is insensitive to small changes in k, therefore the exact k value need not be known. Significant variations do not significantly change the design thickness.

For small projects, it may be advantageous to assume a relatively low k factor and add an appropriate thickness of subbase and base course material to enhance performance of the slab rather than performing an expensive plate load test. Basing design on assumed conditions increases the risk of slab failure, but there are occasions when a simplified design approach is justified. These decisions are a matter of engineering judgment and economics.

4.5.3 *Bearing support*—Calculated bearing pressures under loaded slabs-on-ground are typically significantly lower and are not critical to typical designs as compared with the allowable foundation contact pressures for building elements controlled by ACI 318. Providing uniform support conditions is extremely important for serviceable slab performance.

4.6—Site preparation

4.6.1 *Introduction*—Initially, the top layer of soil should be stripped of all organic material, debris, and frozen material. Normally, to produce a uniform support, strip and till the surface, and recompact it before placing the subbase. Hard and soft pockets of soil should be located by proof-rolling or other means. Remove them and replace with compacted soil to provide a uniform subgrade for the base, subbase, or concrete slab. Refer to ACI 302.1R for additional information.

The site should be graded to provide good surface drainage throughout the construction period and for the lifetime of the structure. Groundwater may have to be intercepted and routed around the site.

Combinations of base and subbase materials and thicknesses can be used to increase the subgrade strength. Carefully examine sinkholes, expansive soils, highly compressible materials, or other subgrade problems, as they can influence slab performance.

4.6.2 *Proof-rolling*—As discussed in ACI 302.1R, proof-rolling usually refers to driving a loaded vehicle in a grid pattern over the subgrade in an effort to locate soft and compressible areas at or near the surface. This should be a part of the quality assurance process for the soil-support system and should be documented in the project specifications. The wheel load should be sized to avoid bearing failure, but be large enough to stress at least the upper foot of subgrade. Three cycles of the wheel load over the same track are usually specified. These repeated applications may expose weak areas by rutting or pumping behavior of the surface soils. Rutting normally indicates excess moisture at the surface. Pumping of the soils under the wheels of the loaded vehicle indicates the subgrade soils are likely wet of the optimum moisture and unable to achieve and maintain compaction. Areas of poor support should be removed and replaced with compacted material to provide a more uniform subgrade. After repairs, proof-rolling can be repeated. There are no standards for proof-rolling, and quantitative assessment cannot be made from its use. Guidelines for proof-rolling are given in ACI 302.1R. When a thick layer of dry and dense material, such as a base or subbase course, exists over the surface, or the subgrade surface has become hard due to drying and construction traffic, proof-rolling may not detect any soft or compressible areas under the surface. Some projects employ proof-rolling three times after:

- Stripping (before any fill is placed);
- Installing the fill; and
- Placing the base course.

Locating suspected deeper soft areas or buried debris may require borings, test pits, resistivity, or other procedures. Proof-rolling should be scheduled so remedial work does not interfere with the construction schedule.

4.6.3 *Subgrade stabilization*—A number of methods can improve the performance of a soil subgrade. Generally, for slabs-on-ground, the soil is densified by using compaction equipment such as a sheepsfoot, rubber tire, or vibratory rollers. Chemical stabilization may also be appropriate.

Weak subgrade material can be stabilized by adding chemicals that combine with the soil, as shown in Table 4.3. Generally, portland cement, lime, or fly ash is mixed into the soil substrata with water and the mixture is recompacted. Lime and fly ash are also used to lower the plasticity index of subgrade and subbase materials. For silty soils, portland cement may be effective. A geotechnical engineer should plan, supervise, and analyze the soil conditions before chemical stabilization is used.

Depending on the situation and soil conditions, certain compactors are more effective than others. Generally, granular soils are most responsive to vibratory equipment, and cohesive soils respond best to sheepsfoot and rubber-tired rollers, but there are exceptions. The depth of compacted lifts varies with soil type and compaction equipment, but in most cases, the depth of compacted lifts should be 6 to 9 in. (150 to 230 mm). The dry density achieved after compaction is normally measured and compared with maximum dry density values obtained from laboratory compaction tests. Maximum dry density and optimum moisture content values vary with texture and plasticity. Refer to Fig. 4.4, which illustrates standard Proctor tests (ASTM D698) on eight different soils.

Because the modified Proctor test (ASTM D1557) uses a higher level of energy, the maximum dry density will be higher and the optimum moisture content will be lower than the standard Proctor test values. Furthermore, the difference will vary with the texture and plasticity of the soil (Fig. 4.5).

Specifications frequently limit only the minimum field density, such as 95% of the standard Proctor maximum density or 90% of the modified Proctor maximum dry density. To achieve a more uniform subgrade modulus, a range of density should be specified. For example, 100 ± 5% of the standard Proctor maximum density, or 95 ± 5% of the modified Proctor maximum dry density. The range specified, however, should be compatible with the soil type, soil uniformity, contractor's operation, and project needs. Specifying a lower density range for clay soils having a plasticity index of 20 or higher, for example, 92 ± 4% of the standard Proctor maximum dry density is often used to control volume changes. Frequently, moisture content within ± 3% of the optimum moisture content of the appropriate test is also specified. Higher moisture contents, from optimum moisture content to 4% above it, are frequently used to minimize volume changes.

4.6.4 *Subbase and base materials*—For many slabs-on-ground, the existing subgrade provides adequate support. Generally, the materials listed in Fig. 4.3 that yield a standard modulus of subgrade reaction above 100 lb/in.^{3} (3000 kN/m^3) can be used (Fig. 4.3). Highly compressible organic materials (OL) should be avoided, as well as high-plasticity clays (CH), as they may cause heave or swell problems. Much of the variation in support strength is the result of compaction and moisture content; for example, the k value for lean clay (CL) ranged from 70 to 250 lb/in.^{3} (2000 to 7000 kN/m^3).

The subbase material has better qualities than the subgrade, and may serve as a construction working surface and part of the floor support system. The subbase is generally omitted where the subgrades are of high quality. The use of a subbase with a base course usually represents an economical alternative for construction on a poor subgrade with an

Table 4.3—Soil stabilization with chemical admixtures

Admixture	Quantity, percent by weight of stabilized soil	Process	Applicability	Effect on soil properties
Portland cement	Varies from approximately 2-1/2 to 4% for cement treatment to 6 to 12% for soil cements.	Pulverize cohesive soil so that at least 80% will pass No. 4 (4.75 mm) sieve, mix with cement, moisten to between optimum and 2% wet, compact to at least 95% maximum density and cure for 7 or 8 days while moistening with light sprinkling or protecting by surface cover.	Forms stabilized subgrade or base course. Wearing surfaces should be added to provide abrasion resistance. Not applicable to plastic clays.	Unconfined compressive strength increased up to approximately 1000 psi (6.9 MPa). Decreases soil plasticity. Increases resistance to freezing and thawing, but remains vulnerable to frost.
Bitumen	Three to 5% bitumen in the form of cutback asphalt emulsion, or liquid tars for sandy soils. Six to 8% asphalt emulsions and light tars for fine-grained materials. For coarse-grained soils, anti-strip compounds are added to promote particle coating by bitumen.	Pulverize soil, mix with bitumen, aerate solvent, and compact mixture. Before mixing, coarse-grained soils should have moisture content as low as 2 to 4%. Water content of fine-grained soils should be several percent below optimum.	Forms wearing surface or construction stage, for emergency conditions, or for low-cost roads. Used to form working base in cohesionless sand subgrades, or for improving quality of base course. Not applicable to plastic clays.	Provides a binder to improve strength and to waterproof stabilized mixture.
Lime	Four to 8%. Fly ash, between 10 and 20%, may be added to increase pozzolanic reaction.	Spread dry lime, mix with soil by pulvimixers or discs, compact at optimum moisture to ordinary compaction densities.	Used for base course and subbase stabilization. Generally restricted to warm or moderate climates because the mixture is susceptible to breakup under freezing and thawing.	Decreases plasticity of soil, producing a grainy structure. Greatest effect in sodium clays with capacity for base exchange. Increases compressive strength up to a maximum of approximately 500 psi (3.4 MPa).

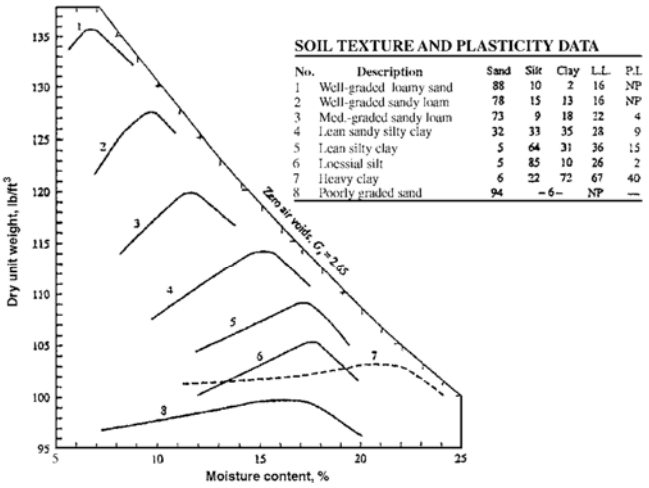


Fig. 4.4—Standard Proctor curves for various soils. (Note: 1 lb/ft^3 = 0.1571 kN/m^3.)

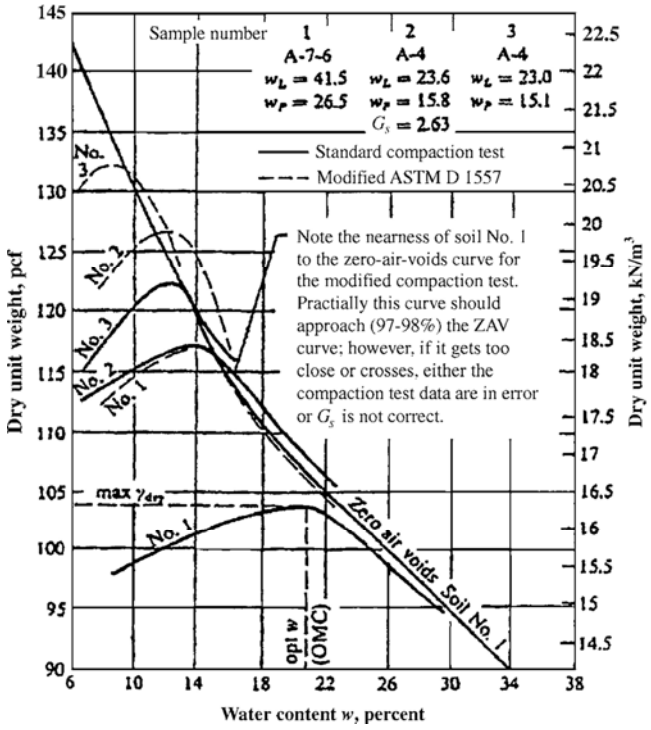


Fig. 4.5—Standard and modified Proctor curves.

expensive base course material. The subbase may be composed of:

- Stabilized subgrade soil;
- A fill of higher quality soil;
- Sand;
- Crushed rock;
- Reclaimed crushed concrete or asphalt pavement; or
- Local material with properties that satisfy project requirements.

Base material should be a clean, densely-graded, granular material with a balanced fine content. It should produce an easily constructed, low-friction surface while minimizing wicking of moisture from below. These densely-graded crushed products are commonly referred to as "crusher-run" materials. The following material sources have proven to be adequate:

1. The local Department of Transportation (DOT) approved road base material with 100% passing the 1-1/2 in. (37.5 mm) sieve, 15% to 55% passing the No. 4 (4.75 mm) sieve, and less than 12% passing the No. 200 sieve (75 μm).

2. Material satisfying the requirements of ASTM D1241, Gradation "A," "C," or "D" (with the modified allowance of less than 12% passing the No. 200 sieve [75 μm]).

Material passing the No. 200 sieve (75 μm) should be clean, granular fill with less than 3% clay or friable particles.

These materials are easily compacted and have high strengths and low compressibilities. When they have little or no fines (material passing a 200 mesh [75 μm] sieve), they are easily drained and act as a capillary break. Their effect on

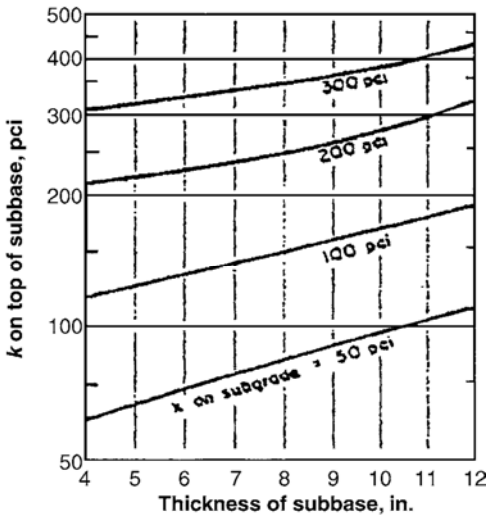


Fig. 4.6—Effect of subbase thickness on design modulus of subgrade reaction. (Note: 1 pci = 0.2714 MN/m^3; 1 in. = 25.4 mm.)

the slab support and the overall *k* value depends on the type and thickness of the base material (Fig. 4.6). Data for specific designs should be based on laboratory analysis and site-testing results. When using an open-graded, crushed rock, the surface may have to be filled in, "choked off" with sand or fine gravels, and compacted to provide a smooth, planar surface to reduce the restraint due to linear concrete shrinkage.

4.6.5 *Stabilization of base and subbase*—Base and subbase materials are often densified by mechanical compaction to improve the *k* value. Consider the relative price of alternatives such as chemical stabilization of the subgrade, use of high-quality base courses, or using a thicker slab.

Measure the mechanical compaction of clay and silt as a percent of standard Proctor density (ASTM D698) or modified Proctor density (ASTM D1557). Minimum dry densities typically specified for these materials range from 90 to 95% of the maximum dry densities of the standard and modified tests, respectively.

4.6.6 *Grading tolerance*—Usually, compliance with the initial rough and fine grading tolerance is based on a rod and level survey using a grid pattern of no more than 20 ft (6.1 m). Grading tolerances specified for a project should be consistent with the recommendations of ACI 117. When a minimum slab thickness has been specified, however, then the slab designer will need to take exception to ACI 117 slab and base tolerances requirements in the construction documents. Specifying a minimum acceptable slab thickness significantly increases the effort to minimize the base and slab tolerances. It may also increase the concrete thickness overages needed to ensure a slab minimum thickness.

4.6.7 *Vapor retarder/barrier*—All concrete is permeable to some degree. Water and water vapor can move through slabs-on-ground (Brewer 1965; Neville 1996). This can adversely affect the storage of moisture-sensitive products on the slab, humidity control within the building, and a variety of flooring materials from coatings to carpets. For storage facilities, anticipated stored goods, and methods of storage should be discussed with the owner. Emitting vapor can become trapped and condensed beneath products such as cardboard stored in direct contact with the slab. Because this moisture can damage stored products, the slab designer should consider positive moisture protection such as a vapor retarder. Products that are not stored in direct contact with the slab, but are sensitive to moisture, may require humidity control. For slabs to receive moisture-sensitive floor coverings, product manufacturers specify a maximum moisture emission rate from the slab surface, generally in the range of 3 to 5 lb/1000 ft^2 (12 to 21 N/100 m^2)/24 hours or a maximum relative humidity, generally 75 to 80% at a depth of 40% of the slab thickness. The use and the location of vapor retarders/barriers require careful consideration. Figure 4.7 provides guidance.

Excess water in the slab not taken up by chemical action evaporates through the slab top until reaching equilibrium with ambient humidity. Additionally, moisture can transpire from the subgrade and through the slab. When the base material under the slab is saturated and subjected to a hydrostatic head, as for a basement slab below a water table, liquid water may flow through cracks or joints in the concrete. When hydrostatic forces can occur, include them in the slab design considerations. The amount of flow depends on the amount of head and width, length, and spacing of the joints and cracks in the concrete. When the base material is saturated or near saturation and there is no head, moisture may transmit into the slab by capillary action of the interconnected voids in the concrete. Positive subgrade drainage is necessary where water would otherwise reach the slab base. Further, an open-graded stone is frequently used as a base course to form a break against capillary rise of moisture in the subgrade. Vapor retarders/barriers can substantially reduce vapor transmission through slabs, but some water vapor will transpire through the slab when the vapor pressure above the slab is less than that below the slab.

Climate-control systems may lower the relative humidity above the slab and result in water vapor movement through the slab. The vapor pressure is a function of temperature and relative humidity. The vapor drive is from high to low vapor pressure. The temperature of the soil base is usually lower than that of the space above the slab. The relative humidity of the subgrade is typically 100%.

Water in the subgrade under slabs-on-ground can change due to seasonal fluctuations of shallow water tables, capillary rise in the subgrade soils, poor subsurface drainage, ponding of storm water adjacent to the slab-on-ground, overwatering plants and lawns adjacent to the slab-on-ground, or from broken pipes in the subgrade. Because there are a variety of moisture sources, there is likely a nonuniform distribution of moisture beneath the slab-on-ground. Tests can be made to try to ascertain the moisture problem before a covering is placed. ASTM D4263 may detect moisture coming from the slab but does not yield a rate of moisture movement. A quantitative test method, ASTM F1869, uses a desiccant calcium chloride beneath an impermeable dome over a small slab area to calculate the moisture emission rate. These test results may be misleading when the ambient air conditions do not represent in-service conditions. ASTM F1869 requires

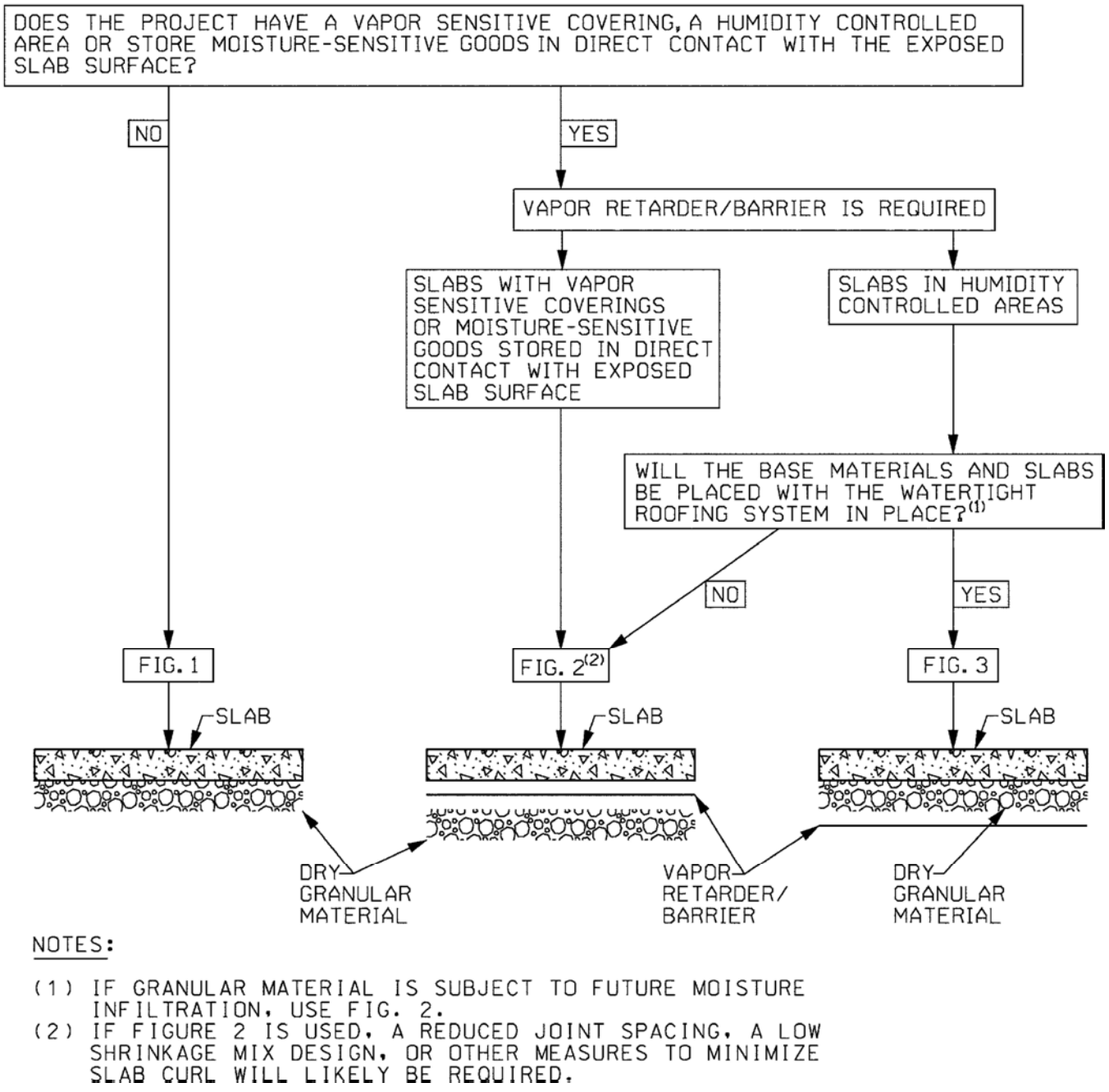


Fig. 4.7—Decision flowchart to determine when a vapor retarder/barrier is required and where it is to be placed.

an ambient air temperature of 75°F ± 10°F (24°C ± 6°C) and a relative humidity of 50% ± 10% for 48 hours before and during the test. This test measures moisture in the top 1/2 in. (13 mm) of the slab, and cannot detect moisture below 3/4 in. (19 mm). To better quantify moisture in slabs, ASTM F2170 was developed for the use of relative humidity probes.

Subgrade drainage and selecting subgrade materials have a great influence on vapor retarders/barriers performance. Protecting vapor retarders/barriers from damage during construction can significantly influence their effectiveness. Vapor retarders/barriers have been reported to affect the concrete behavior in the slab by increasing finishing time, promoting cracking, increasing slab curling, and reducing strength. These problems, however, may be less costly than performance failures related to excessive moisture transmission through the slab.

4.7—Inspection and site testing of slab support

Inspection and testing are required to control the quality of the subgrade and subbase construction and determine conformance to project specifications. Before construction begins, the subgrade soils and subbase or base-course materials should be sampled, tested in the laboratory, and the results evaluated.

In general, perform the following tests for soils and soil-aggregate mixtures:

- Particle size (ASTM D422);

- Plasticity (ASTM D4318); and
- Laboratory compaction tests (ASTM D698 or D1557).

For cohesionless and free-draining soils and aggregates gradation, perform the following:

- Maximum relative density (ASTM D4253);
- Minimum relative density (ASTM D4254); and
- Calculation of relative density.

After compaction, the in-place density can be determined in the field by:

- Drive cylinder (ASTM D2937);
- Sand cone (ASTM D1556);
- Water balloon (ASTM D2167); and
- Nuclear densometer (ASTM D2922 and D3017).

The sand cone test is the most accepted method, but the nondestructive nuclear density method is advantageous because it takes minutes to conduct, the results are available at the end of the field test, and it is widely used and accepted. This allows field density and moisture contents to be used to control construction. To check questionable results or to confirm calibrations, however, the sand cone method is generally specified. To check the nuclear gauge against the sand cone and provide an adjustment factor, conduct a series of calibration tests. Check the moisture readings against field moisture tests (ASTM D566). Testing frequency is relative to the uniformity of the materials used and the quality of compaction required. Work that does not conform to the project specifications should be corrected and retested. The subgrade should be tested in advance of installing the remainder of the slab-on-ground system. Minimum testing requirements should be established for each project. These should provide a reasonable test interval for each lift. These tests are relatively inexpensive and easy to perform.

4.8—Special slab-on-ground support problems

Because of their low shear strength and high compressibility, avoid placing slabs on topsoil. Project specifications generally require stripping topsoil from the building site.

Expansive soils are defined as fine-grained soils, as shown in Tables 4.1 and 4.2. Soils with a plasticity index of 15 or higher may have a potential for volume change that should be considered. A geotechnical engineer should examine the soil data and recommend appropriate options. Potential problems can be minimized by proper slab designs, stabilizing the soil, and preventing moisture migration through the slab. Failure to manage problems can, and often will, result in slab failure.

Frost action may be critical to silts, clays, and some fine sands. These soils can experience large volume changes and consequently heave due to the growth of ice lenses during freezing cycles and lose support due to saturation upon thawing. Three conditions are present when this problem occurs:

1. Freezing temperature in the soil;
2. Water table close to the frost level providing water for the formation of ice lenses; and
3. A soil that will transmit water from the water table into the frost zone by capillary action.

Possible remedies include lowering the water table, providing a barrier, or using a subbase or subgrade soil that is not frost-susceptible. Properly designed insulation can be beneficial. Volume changes due to frost action occur at building perimeters, under freezer areas, and under ice-skating rink floors (NCHRP 1974).

CHAPTER 5—LOADS

5.1—Introduction

Chapter 5 describes loadings, the variables that control load effects, and provides guidance for factors of safety for concrete slabs-on-ground. Concrete slabs are typically subjected to some combinations of the following loads and effects:

- Vehicle wheel loads;
- Concentrated loads;
- Distributed loads;
- Line and strip loads;
- Unusual loads;
- Construction loads; and
- Environmental effects.

Slabs should be designed for the most critical combination of these loadings, considering variables that produce the maximum stress. Figure 5.1 presents the PCA guide for selecting the most critical or controlling design considerations for various loads (Packard 1976). Because a number of factors, such as slab thickness, concrete strength, subgrade stiffness, and loading are relevant, cases where several design considerations that may control should be investigated thoroughly.

Other potential load conditions, such as loadings that change during the life of the structure and those encountered during construction (Wray 1986), should also be considered. For example, material-handling systems today make improved use of the building's volume. Stacked pallets, once considered uniform loads, may now be stored in narrow-aisle pallet racks that produce concentrated loads. The environmental exposure of the slab-on-ground is also a concern. These effects include subgrade volume changes (shrink and swell soils), buildings with equipment to reduce humidity, and temperature changes. Thermal effects may be minimized by constructing the slab after the building is enclosed, but many slabs are placed before building enclosure. Therefore, construction sequence is important in determining whether transient environmental factors should be considered in the design. Finally, thermal effects due to in-service conditions should be considered.

5.2—Vehicular loads

Most vehicular traffic on industrial floors consists of lift trucks and distribution trucks with payload capacities as high as 70,000 lb (310 kN). The payload and much of a truck's weight are generally carried by the wheels of the loaded axle for the standard (counterbalanced) lift truck. The Industrial Truck Association (ITA) (1985) has compiled representative load and geometry data for lift truck capacities up to 20,000 lb (89 kN) (Table 5.1). Vehicle variables affecting the thickness selection and design of slabs-on-ground include:

- Maximum axle load;
- Distance between loaded wheels;
- Tire contact area; and
- Load repetitions during service life.

The axle load, wheel spacing, and contact area are functions of the lift truck or vehicle specifications. When vehicle details are unknown or when the lift truck strength is expected to change in the future, the values in Table 5.1 may be used for design. The number of load repetitions, which may be used to help establish a factor of safety, is a function of the facility's usage. Knowledge of load repetitions and traffic patterns helps the designer to quantify fatigue. Consider whether these values are predictable or constant during the slab's service life. Often, the slab is designed for an unlimited number of repetitions.

The contact area between tire and slab is used in the analysis for lift truck with pneumatic or composition tires (Wray 1986). The contact area of a single tire can be approximated by dividing the tire load by the tire pressure (Packard 1976). This calculation is somewhat conservative because the effect of tension in the tire wall is not included. Assumed pressures are variable; however, pneumatic non-steel-cord tire pressures range from 85 to 100 psi (0.6 to 0.7 MPa), whereas steel-cord tire pressures range from 90 to 120 psi (0.6 to 0.8 MPa). The ITA found that the standard solid and cushion solid rubber tires have floor contact areas based on internal pressures between 180 and 250 psi (1.2 to 1.7 MPa) (Goodyear Tire and Rubber Co. 1983). Polyurethane tire pressures exceeding 1000 psi (6.9 MPa) have been measured.

Dual tires have an effective contact area greater than the actual contact area of the two individual tires. Charts are available to determine this effective contact area (Packard 1976). A conservative estimate of this effective contact area can be made by using the contact area of the two tires and the areas between the contact area. When it is not known whether the vehicle has dual wheels or what the wheel spacings are, then a single equivalent wheel load and contact area can be used conservatively.

An important consideration for the serviceability of a slab subject to vehicular loads is the design of construction and sawcut contraction joints. Joints should be stiff enough and have sufficient shear transferability to limit differential movement such that the joint filler can perform properly and resists edge spalling as a vehicle travels across the joint. Refer to Chapter 6 for more information and joint details.

5.3—Concentrated loads

Warehousing improvements in efficiency and storage densities have trended toward increased rack post loads. These changes include narrower aisles, higher pallet or material stacking, and using automated stacking equipment. Pallet storage racks may be higher than 80 ft (24 m) and may produce concentrated post loads of 40,000 lb (180 kN) or more. For the higher rack loads, racks that cover a large plan area (which affects deeper soil layers), and racks with long-term loading, consider the effect of the long-term soil settlement in the design of the slab. Cracking can also be caused by early installation of rack systems that may restrain the slab's shrinkage and thermal movement and prevent joint activation. The racks may restrain the slab with the rack system bracing or by the increase in base friction from additional storage loads.

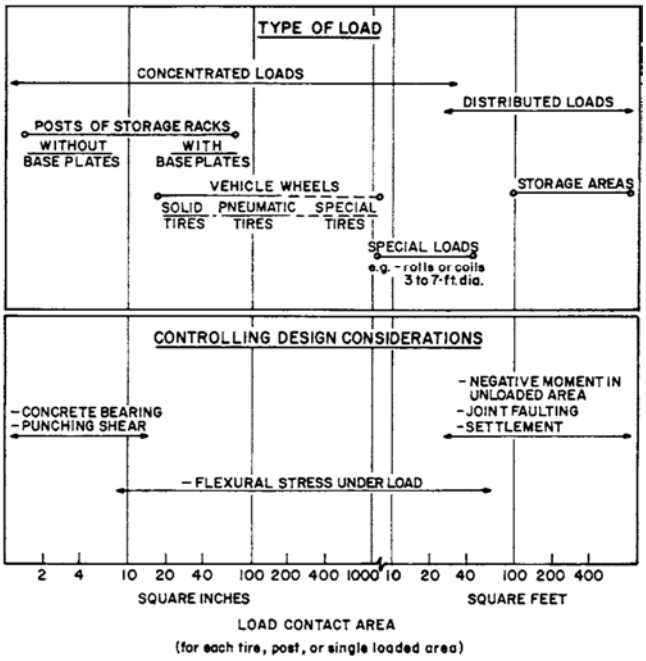


Fig. 5.1—Controlling design considerations for various types of slabs-on-ground loading (Packard 1976). (Note: 1 in.^{2} = 645.2 mm^2; 1 ft^2 = 0.09290 m^2.)

Table 5.1—Representative axle loads and wheel spacings for various lift truck capacities

Truck rated capacity, lb	Total axle load static reaction, lb	Center-to-center of opposite wheel tire, in.
2000	5600 to 7200	24 to 32
3000	7800 to 9400	26 to 34
4000	9800 to 11,600	30 to 36
5000	11,600 to 13,800	30 to 36
6000	13,600 to 15,500	30 to 36
7000	15,300 to 18,100	34 to 37
8000	16,700 to 20,400	34 to 38
10,000	20,200 to 23,800	37 to 45
12,000	23,800 to 27,500	38 to 40
15,000	30,000 to 35,300	34 to 43
20,000	39,700 to 43,700	36 to 53

Notes: Calculate concentrated reaction per tire by dividing the total axle load reaction by the number of tires on that axle. Figures given are for standard trucks. The application of attachments and extended high lifts may increase these values. In such cases, consult the manufacturer. Weights given are for trucks handling the rated loads at 24 in. from load center to face of fork with mast vertical. 1 lb = 0.004448 kN; 1 in. = 25.4 mm.

The concentrated load variables that affect slab-on-ground design include:

- Maximum or representative post load;
- Duration of load;
- Spacings between posts and aisle width;
- Location of the concentrated load relative to slab joint location and the amount of shear transfer across the slab joint; and
- Area of contact between post base plate and slab.

Material-handling systems are major parts of the building layout and should be well defined early in the project. Rack data can be obtained from the manufacturer. It is not uncommon to specify a larger base plate than is normally

supplied to reduce the flexural stress caused by the concentrated load. The base plate should be sized to distribute the load nearly uniformly over the plate area.

5.4—Distributed loads

In many warehouse and industrial buildings, materials are stored directly on the slab-on-ground. Flexural stresses in the slab are usually less than those produced by concentrated loads. The design should prevent negative moment cracks or limit crack widths for the reinforced slabs in the aisles and prevent excessive settlement. For higher load intensities, distributed loads that cover a large plan area (which affect deeper soil layers) and long-term uniform loads, consider the effect of the differential soil settlement in the slab design. The effect of a lift truck operating in aisles between uniformly loaded areas is not normally combined with the uniform load into one loading case, as the moments produced generally offset one another. The individual cases are always considered in the design.

For distributed loads, the variables affecting the design of slabs-on-ground are:

- Maximum load intensity;
- Load duration;
- Width and length of loaded area;
- Aisle width; and
- Presence of a joint located in and parallel to an aisle.

Load intensity and layout may not be constant during the service life of a slab. Therefore, the slab should be designed for the most critical case. For a given modulus of subgrade reaction and slab thickness, there is a critical aisle width that maximizes the center aisle moment (Packard 1976).

5.5—Line and strip loads

A line or strip load is a uniform load distributed over a relatively narrow area. Consider a load to be a line or strip load when its width is less than 1/3 of the radius of relative slab stiffness (Section 7.2). When the width approximates this limit, review the slab for stresses produced by line loading and uniform load. When the results are within 15% of one another, consider the load as uniform. Partition loads, bearing walls, and roll storage are examples of this load type. For higher load intensities and long-term loading, consider the effect of differential soil settlement in the design of the slab.

The variables for line and strip loads are similar to those for distributed loadings and include:

- Maximum load intensity;
- Load duration;
- Width, length of loaded area, and when the line or strip loads intersect;
- Aisle width;
- Presence of a joint in and parallel to an aisle;
- Presence of parallel joints on each side of an aisle; and
- The amount of shear transfer across the slab joint, which is especially important when the line load crosses perpendicular to a joint or is directly adjacent and parallel to a joint.

5.6—Unusual loads

Loading conditions that do not conform to the previously discussed load types may also occur. They may differ in the following manner:

- Configuration of loaded area;
- Load distributed to more than one axle; and
- More than two or four wheels per axle.

The load variables are similar to those for the load types previously discussed in Chapter 5.

5.7—Construction loads

During the building construction, various equipment types may be located on the newly placed slab-on-ground. The most common construction loads are:

- Pickup trucks;
- Scissor lifts;
- Concrete trucks;
- Dump trucks;
- Hoisting equipment and cranes used for steel erection;
- Tilt wall erection and bracing loads; and
- Setting equipment.

In addition, the slab may be subjected to loads such as scaffolding and material pallets. Because these loads can exceed design limits, anticipate the construction load case, particularly relative to early-age concrete strength. Also, consider limiting construction loads near the free edges or slab corners. The controlling load variables for construction loads are the same as for vehicle loads, concentrated loads, and uniform loads.

For construction trucks, the maximum axle load and other variables can usually be determined by referencing to local transportation laws or AASHTO standards. Off-road construction equipment may exceed these limits, but in most cases, construction equipment will not exceed the local DOT legal limits. For design, reference the values shown in Fig. 5.2.

Design codes are mostly silent on the subject of temporary construction loads during erection (Subrizi et al. 2004). In tilt-up construction, the slab-on-ground is often used as part of the temporary construction bracing. The Tilt-Up Concrete Association (TCA) has developed a temporary bracing guideline (TCA 2005) that provides some guidance for the slab's design. When the slab designer considers these tilt-up wall bracing loads during the initial design of the slab, then often a more economical bracing solution is achieved for the project (Kelly 2007). Kelly (2007) also provides some solutions when the bracing loads are considered after the slab has been constructed along with some design guidance for the slab's flexural strength.

5.8—Environmental factors

Overall slab design should consider flexural stresses produced by thermal changes, reduction in humidity, expansive soils, and moisture changes in the slab, which will affect curling due to the different shrinkage rates between the top and bottom of the slab. These effects are of particular importance for exterior slabs and for slabs constructed before the building is enclosed. Curling caused by these changes produces flexural stresses due to the slab lifting off the

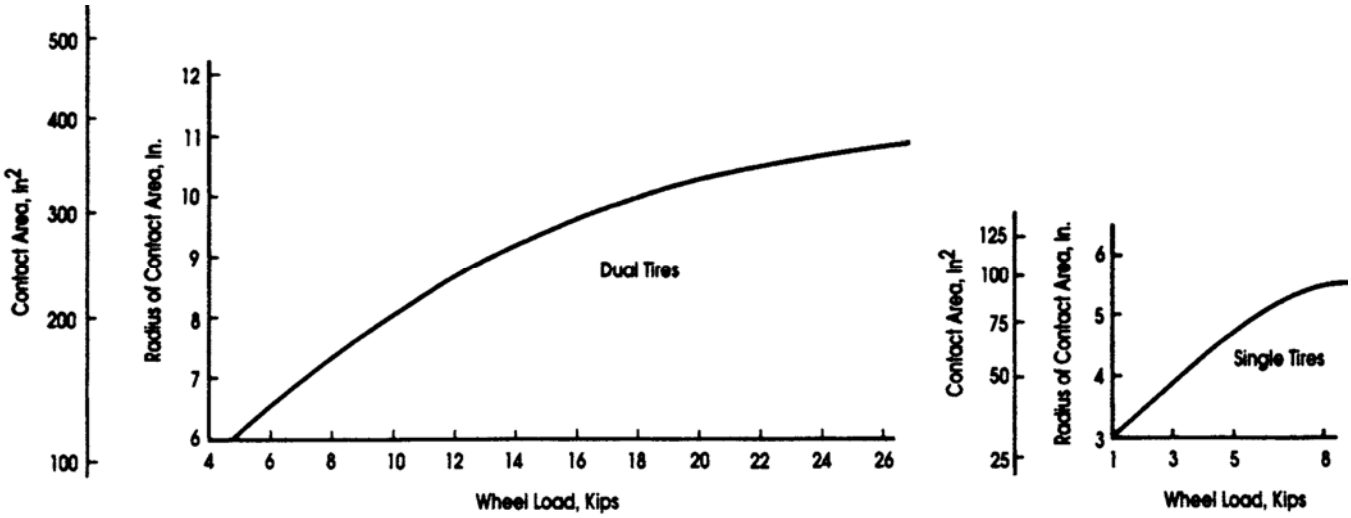


Fig. 5.2—Tire contact area for various wheel loads. (Note: 1 in. = 25.4 mm; 1 in.^{2} = 645.2 mm^2; 1 kip = 4.448 kN.)

subgrade. Generally, the restraint stresses can be ignored in short slabs because a smooth, planar subgrade does not significantly restrain the short slab movement due to uniform thermal expansion, contraction, or drying shrinkage. When the joint spacing recommendations given in Fig. 6.6 are followed, then these stresses will be sufficiently low. Built-in restraints such as foundation elements, edge walls, and pits should be avoided. Reinforcement should be provided at such restraints to limit slab crack widths. Chapter 14 discusses thermal and moisture effects. Chapter 10 discusses expansive soils.

5.9—Factors of safety

Unique serviceability requirements distinguish slabs-on-ground from other structural elements. Some of these serviceability requirements can:

- Minimize cracking and curling;
- Increase surface durability;
- Optimize joint locations and joint types for joint stability, which is the differential deflection of the adjacent slab panels edges as wheel loads cross the joint; and
- Maximize long-term flatness and levelness.

Because building codes primarily provide guidance to prevent catastrophic failures that affect public safety, the factors of safety for serviceability, inherent in building codes, are not directly addressed as are those for strength. When the slab-on-ground is part of the structural system used to transmit vertical loads or lateral forces from other portions of the structure to the soil, such as a rack-supported roof, then requirements of ACI 318 should be used for that load case.

The designer selects the factor of safety to minimize the likelihood of serviceability failure. Some items the designer should consider in selecting the factor of safety are:

- Consequences of serviceability failure, including lost productivity, lost beneficial use, and the costs for repairing areas in an active facility. For example, minimizing cracking and limiting crack widths for facilities such as pharmaceutical and food processing;
- Concrete mixture proportion and its shrinkage characteristics (test and minimize shrinkage to reduce linear drying shrinkage and curling);
- Humidity-controlled environment that increases linear drying shrinkage and slab curling;
- Subgrade smoothness and planeness to minimize restraint during linear drying shrinkage;
- Spacing and joint types;
- Geotechnical investigation to determine the shallow and deep properties of the soil;
- Number of load repetitions and traffic patterns to allow consideration of fatigue cracking;
- Impact effects;
- Storage racks installed at an early stage, which restrain linear drying shrinkage; and
- Compounding factors of safety that may produce an overly conservative design. Inclusion of cumulative factors of safety in the modulus of subgrade reaction, applied loads, compressive or flexural strength of the concrete, or number of load repetitions, may produce a very conservative and, consequently, expensive construction. The factor of safety is normally accounted for only in the allowable flexural stress in the concrete slab.

Table 5.2 shows some commonly used factors of safety for various types of slab loadings. Most range from 1.7 to 2.0, but some loading conditions use factors as low as 1.4.

A moving vehicle subjects the slab-on-ground to the effect of fatigue. Fatigue strength is expressed as the percentage of the static tensile strength that can be supported for a given number of load repetitions. As the ratio of the actual flexural stress to the modulus of rupture decreases, the slab can withstand more load repetitions before failure. For stress ratios less than 0.45, concrete can be subjected to unlimited load repetitions (PCA 2001). Table 5.3 shows various load repetitions for a range of stress ratios. The factor of safety is the inverse of the stress ratio.

Table 5.2—Factors of safety used in design of various types of loading

Load type	Commonly used factors of safety	Occasionally used factors of safety
Moving wheel loads	1.7 to 2.0	1.4 to 2.0 and greater
Concentrated (rack and post) loads	1.7 to 2.0	Higher under special circumstances
Uniform loads	1.7 to 2.0	1.4 is lower limit
Line and strip loads	1.7	2.0 is conservative upper limit*
Construction loads	1.4 to 2.0	—

*Follow appropriate building code requirements when considering a line load to be a structural load due to building function.

Table 5.3—Stress ratio versus allowable load repetitions (Portland Cement Association 1984)*

Stress ratio	Allowable load repetitions	Stress ratio	Allowable load repetitions
<0.45	Unlimited	0.73	832
0.45	62,790,761	0.74	630
0.46	14,335,236	0.75	477
0.47	5,202,474	0.76	361
0.48	2,402,754	0.77	274
0.49	1,286,914	0.78	207
0.50	762,043	0.79	157
0.51	485,184	0.80	119
0.52	326,334	0.81	90
0.53	229,127	0.82	68
0.54	166,533	0.83	52
0.55	124,523	0.84	39
0.56	94,065	0.85	30
0.57	71,229	0.86	22
0.58	53,937	0.87	17
0.59	40,842	0.88	13
0.60	30,927	0.89	10
0.61	23,419	0.90	7
0.62	17,733	0.91	6
0.63	13,428	0.92	4
0.64	10,168	0.93	3
0.65	7700	0.94	2
0.66	5830	0.95	2
0.67	4415	0.96	1
0.68	3343	0.97	1
0.69	2532	0.98	1
0.70	1917	0.99	1
0.71	1452	1.00	0
0.72	1099	>1.00	0

CHAPTER 6—JOINTS

6.1—Introduction

Joints are used in slab-on-ground construction to limit the frequency and width of random cracks caused by concrete volume changes. Generally, when limiting the number of joints or increasing the joint spacing can be accomplished without increasing the number of random cracks, floor maintenance will be reduced. The designer should provide the layout of joints and joint details. When the joint layout and joint details are not provided before project bid, the designer should provide a detailed joint layout along with the joint details before the slab preconstruction meeting or commencing construction.

Every effort should be made to avoid connecting the slab to any other element of the structure. Restraint from any source, whether internal or external, will increase the potential for random cracking.

Three types of joints are commonly used in concrete slabs-on-ground and are discussed in detail in this chapter:

1. Isolation joints.
2. Sawcut contraction joints.
3. Construction joints.

Refer to Fig. 6.1 for appropriate locations for isolation joints and sawcut contraction joints. With the designer's approval, construction joint details and sawcut contraction joint details can be interchanged. Joints in topping slabs should be located directly over joints in the base slab and, when the topping is bonded, no additional joints are required. The bonded topping slab should be designed for the shrinkage restraint due to the bond to the existing slab, and the bond should be sufficient to resist the upward tension force due to curling. For a thin, unreinforced, unbonded topping slab, consider additional joints between the existing joints in the bottom slab to help minimize the curling stress in the topping slab. The topping slab can have high curling stresses due to the bottom slab being a hard base for the topping slab. Also, any cracks in the base slab that are not stable should be repaired to ensure they will not reflect through into the topping slab.

6.1.1 *Isolation joints*—Isolation joints should be used wherever complete freedom of vertical and horizontal movement is required between the floor and adjoining building elements. Isolation joints should be used at junctions with walls (not requiring lateral restraint from the slab), columns, equipment foundations, footings, or other points of restraint such as drains, manholes, sumps, and stairways.

Isolation joints are formed by inserting preformed joint filler between the floor and the adjacent element. Where the isolation joint will restrain shrinkage, flexible closed cell foam plank should be used with a thickness that accommodates the anticipated shrinkage movement. The joint material should extend the full depth or slightly below the bottom of the slab to ensure complete separation and not protrude above it. Where the joint filler will be objectionably visible, or where there are wet conditions or hygienic or dust-control requirements, the top of the preformed filler can be removed and the joint caulked with an elastomeric sealant. Listed as follows are three methods of producing a relatively uniform joint sealant depth:

1. Use a saw to score both sides of the preformed filler at the depth to be removed. Insert the scored filler in the proper location. After the concrete hardens, use a screwdriver or similar tool to remove the top section.

2. Cut a strip of wood equal to the desired depth of the joint sealant. Nail the wood strip to the preformed filler and install the assembly in the proper location. Remove the wood strip after the concrete has hardened.

3. Use premolded joint filler with a removable top portion.

Refer to Fig. 6.2 and 6.3 for typical isolation joints around columns. Figure 6.4 shows an isolation joint at an equipment foundation. Refer to ACI 223 for guidance on isolation joints for slabs using shrinkage-compensating concrete.

6.1.2 *Construction joints*—Construction joints are placed in a slab to define the extent of the individual placements,

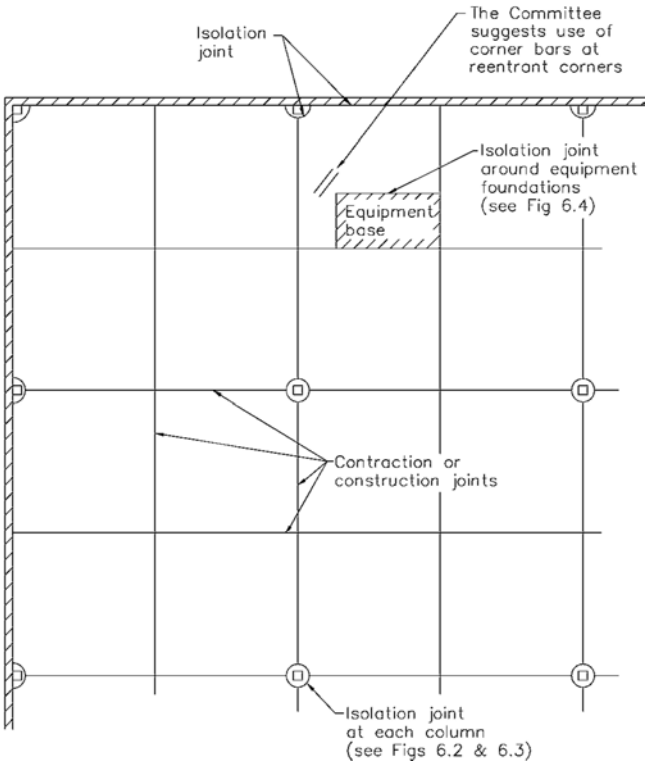


Fig. 6.1—Appropriate locations for joints.

generally in conformity with a predetermined joint layout. When concreting is interrupted long enough for the placed concrete to harden, the construction documents should provide a detail to address this unplanned event. A contingency plan for this unplanned event should also be discussed in the slab preconstruction meeting. For specialty floors such as defined traffic slabs, unplanned joints can have a significant effect on the long-term floor flatness and levelness when not detailed appropriately.

In areas not subjected to wheel traffic or when differential curling movement is not a concern, a butt joint or keyed joint may be adequate. In areas subjected to wheeled traffic, heavy loads, or both, joints with load transfer devices are recommended (Fig. 6.5). A keyed joint is not recommended for load transfer in areas subjected to wheel traffic because the male and female key components lose contact when the joint opens due to drying shrinkage. This can eventually cause a breakdown of the concrete joint edges and failure of the top side portion of the key.

Construction joints are commonly formed using bulkheads. These bulkheads should be placed to ensure specified concrete slab thickness and elevation at the slab edge. Provide the necessary support to keep the bulkheads straight, true, and rigid during the entire placing and finishing operations. When positive load transfer is required, provisions should be made along the bulkhead to ensure proper alignment of the load-transfer device during placing and finishing operations. Proper alignment can be achieved by rigidly attaching alignment devices to the bulkhead or by accurately slotting or drilling the bulkhead to accept a load-transfer device. These methods should allow the load-transfer device to be

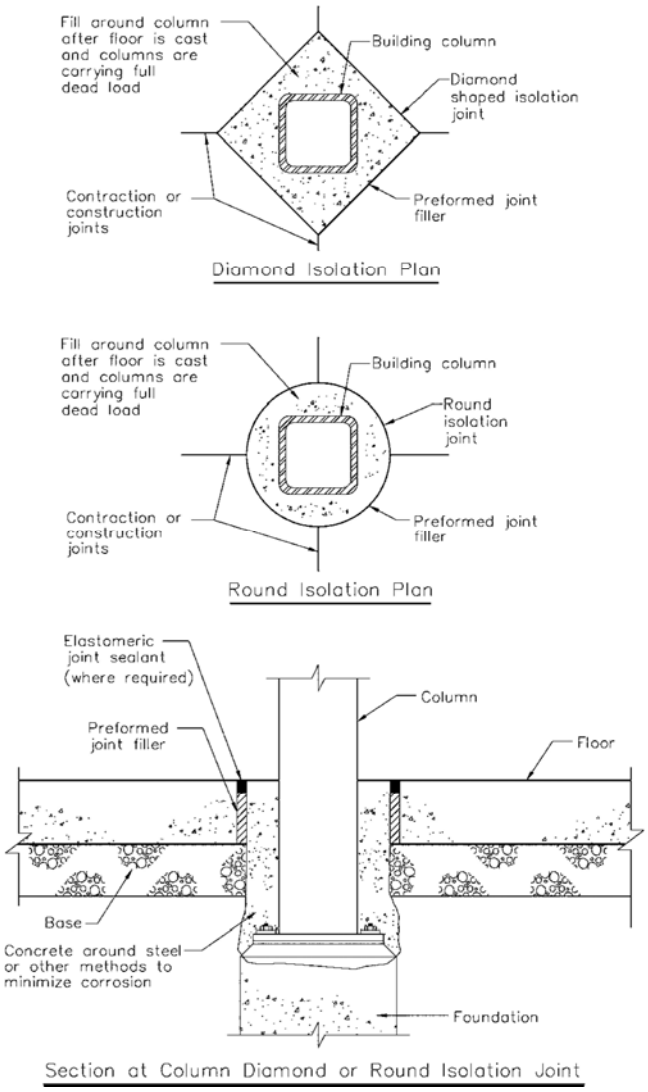


Fig. 6.2—Typical diamond and round isolation joints.

installed at the joint face while maintaining proper alignment during placing and finishing operations. The load-transfer devices should be parallel to the top surface, each other, and perpendicular to the joint face. Depending on the method used, the load-transfer device can be inserted before or after the bulkhead is removed. When incorporating alignment or installation devices into the bulkhead, which remain in the slab, the device should be manufactured with a thin rigid material that is a tight fit with the load-transfer device to minimize vertical deflection due to load. Load-transfer devices in direct contact with the concrete will be the most effective load-transfer mechanism for controlling vertical joint deflection.

All construction joints should be internally vibrated at frequent intervals to properly consolidate the concrete at the joint and around the load-transfer devices. Vibratory screeds, laser-guided screeds, and hand-rodding techniques do not provide sufficient internal vibration around the load-transfer devices.

6.1.3 *Sawcut contraction joints*—Sawcut contraction joints are used to limit random, out-of-joint floor slab cracking. Joints are usually located on column lines, with intermediate joints located at equal spaces between column lines (Fig. 6.1).

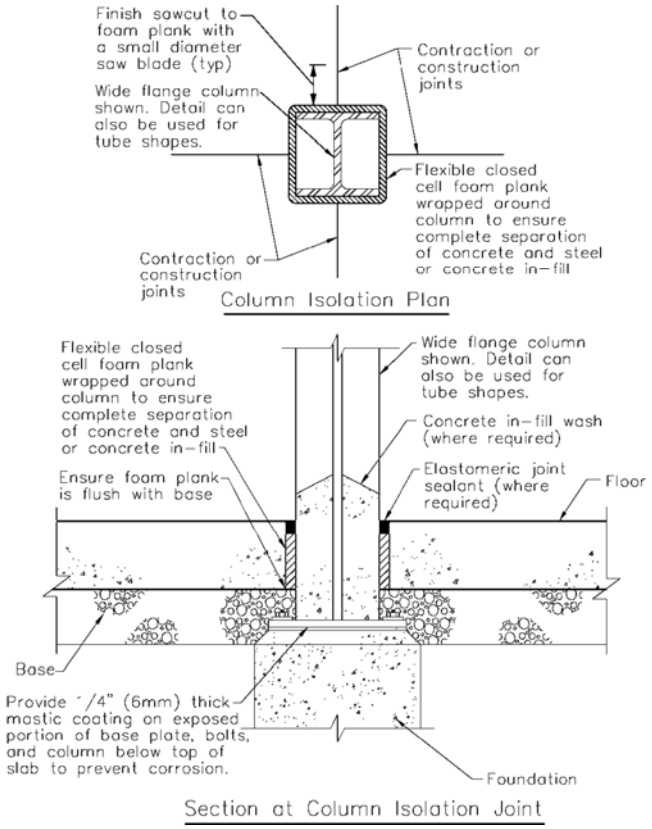


Fig. 6.3—Alternate column isolation joint.

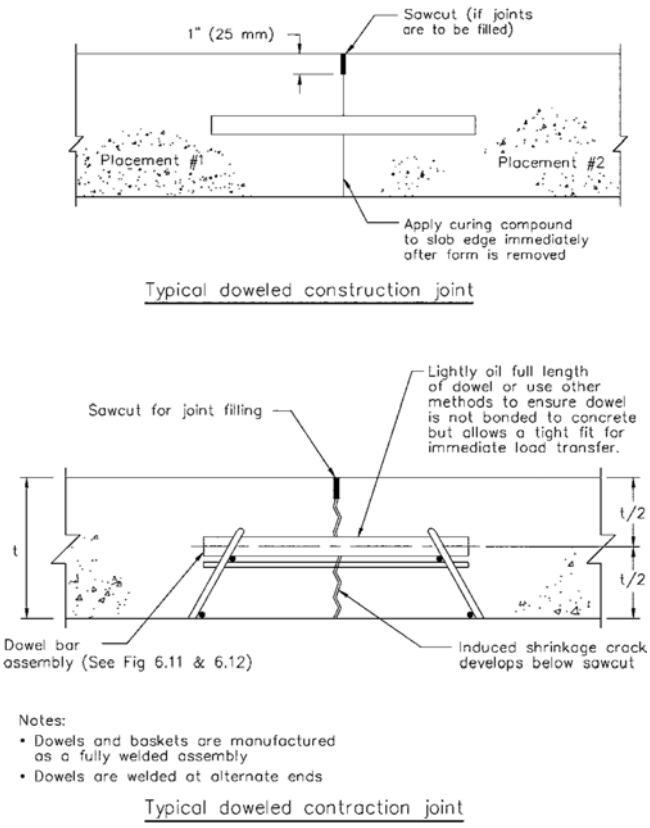


Fig. 6.5—Typical doweled joints.

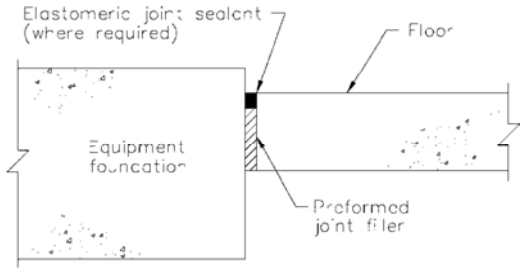
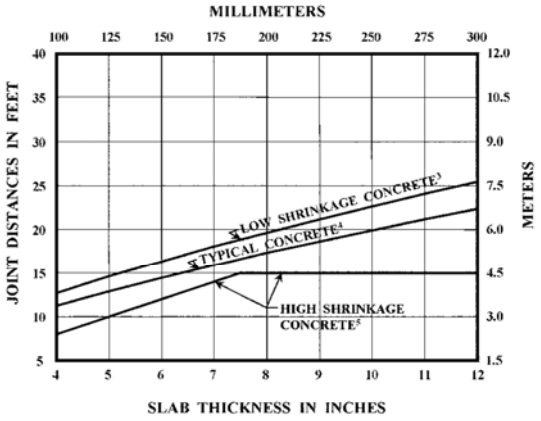


Fig. 6.4—Typical isolation joint around equipment foundation.

Consider the following when selecting spacing of sawcut contraction joints:

- Slab design method;
- Slab thickness;
- Type, amount, and location of reinforcement;
- Shrinkage potential of the concrete, including cement type and quantity; aggregate type, size, gradation, quantity, and quality; water-cementitious material ratio; type of admixtures; and concrete temperature;
- Base friction;
- Floor slab restraints;
- Layout of foundations, racks, pits, equipment pads, trenches, and similar floor discontinuities; and
- Environmental factors such as temperature, wind, and humidity.

Establishing slab joint spacing, thickness, and reinforcement requirements is the responsibility of the designer. The specified



NOTES:

1. Joint spacing recommendations based on reducing the curling stresses to minimize mid- panel cracking (Walker-Holland 2001). See discussion in Section 6.2 for joint spacing for aggregate interlock.
2. Joint spacing criteria of 36 and 24 times the slab thickness has been utilized in the past.
3. Concrete with an ultimate dry shrinkage strain of less than 520 millionths placed on a dry base material.
4. Concrete with an ultimate dry shrinkage strain of 520 to 780 millionths placed on a dry base material.
5. Concrete with an ultimate dry shrinkage strain of 780 to 1100 millionths placed on a dry base material.

Fig. 6.6—Recommended joint spacing for unreinforced slabs.

joint spacing will be a principal factor dictating both the amount and the character of random cracking to be experienced, so joint spacing should always be carefully selected. For unreinforced slabs-on-ground and for slabs reinforced only for limiting crack widths, other than continuously reinforced with more than 0.5% of steel by cross-sectional area, Fig. 6.6

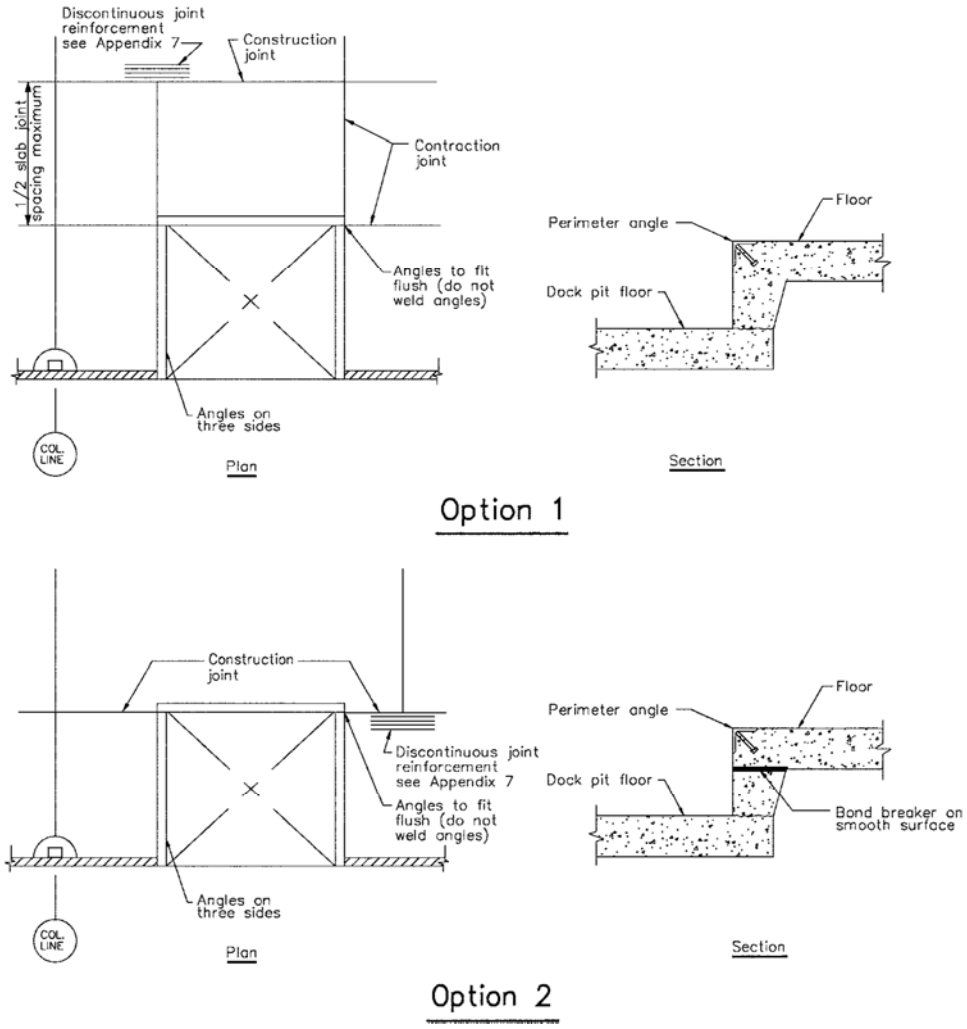


Fig. 6.7—Joint details at loading dock.

provides recommendations for joint spacing. The spacings are based on shrinkage values for specimens that have been moist-cured 7 days and then placed in air storage per ASTM C157/C157M. Use the appropriate prism size for the concrete mixture being placed. The procedures in ACI 209R can be used along with the ASTM C157/C157M test information to predict the ultimate drying shrinkage. Using only a few days of laboratory drying shrinkage data and the procedures from ACI 209R to predict the ultimate drying shrinkage may lead to unreliable values. Using 28-day laboratory shrinkage values (7 days of curing and then 21 days of air storage) and the procedures in ACI 209R to predict the ultimate drying shrinkage, however, have provided useful results.

Sawcut contraction joints should be continuous across intersecting joints, not staggered or offset. The aspect ratio of slab panels that are unreinforced, reinforced only for crack-width control, or made with shrinkage-compensating concrete should be a maximum of 1.5 to 1; however, a ratio of 1 to 1 is preferred. L- and T-shaped panels should be avoided. Floors around loading docks have a tendency to crack due to their configuration and restraints. Figure 6.7 shows two options that minimize slab cracking at reentrant corners of loading docks. In Option 1, the loading dock pit wall is placed integral with the slab and, therefore, most of the shrinkage movement is forced to the construction joint shown in the figure. To minimize the opening width of this construction joint, place the joint at 1/2 of the typical slab joint spacing. In Option 2, create a slip surface at the top of the pit wall that will help equalize the shrinkage movement on each side of the slab panel so the typical slab joint spacing can be used. By using a construction joint as shown in Option 2, there is less likelihood of cracking at the dock pit corners.

Plastic or metal inserts are not recommended for creating a contraction joint in any exposed floor surface subject to wheel traffic.

Contraction joints in industrial and commercial floors are usually formed by sawing a continuous slot in the slab to form a weakened plane so a crack will form below (Fig. 6.8).

When using load-transfer assemblies at sawcut contraction joints, the concrete around these assemblies should be internally vibrated. This properly consolidates the concrete around the load-transfer assemblies. Section 6.3 provides further details on sawcutting of joints.

6.2—Load-transfer mechanisms

Use load-transfer devices at construction and contraction joints (Fig. 6.5) when positive load transfer is required, unless a sufficient amount of post-tensioning force is provided across the construction joint to transfer the shear. Load-transfer devices force concrete on both sides of a joint to deflect approximately equally when subjected to a load. This can help prevent damage to an exposed edge when subjecting the joint to wheel traffic.

For dowels to be effective, they should be smooth, aligned, and supported so they remain parallel in both the horizontal and vertical planes during the placing and finishing operation. All dowels should have flat, square, and deburred end edges that will not restrain concrete shrinkage. Properly aligned, smooth dowels allow the joint to open freely as concrete shrinks. ACI 117 provides tolerances for the installation of slab-on-ground round dowels.

Plate dowels are now commonly used in construction and contraction joints. Manufacturers offer various plate dowel geometries and associated installation devices. Plate dowels can minimize shrinkage restraint by using a tapered shape, formed void, or by having compressible material on the vertical faces with a thin bond breaker on the top and bottom dowel surfaces (Fig. 6.9 and 6.10) (Schrader 1987, 1991; PTI 2000; Ringo and Anderson 1992; Metzger 1996; Walker and Holland 1998; American Concrete Paving Association 1992). When a formed void on the vertical sides is constructed by a stay-in-place device, then that stay-in-place device should be of a rigid material and fit tightly with the dowel surface to minimize vertical deflection at the joint. The tapered shape along with a thin bond breaker on all sides allows a void space to develop along the vertical sides of the dowel to eliminate restraint as the slab shrinks from the joint. Similarly, a formed void or the compressible material will also eliminate the restraint as the slab shrinks from the joint.

Dowel baskets (Fig. 6.11 and 6.12) should be used to maintain alignment of dowels in sawcut contraction joints, and alignment installation devices should be used in construction joints. Plate dowels in contraction joint basket assemblies with the compressible material or tapered shape allow for some horizontal misalignment with the sawcut contraction joint. In corrosive environments, the designer should consider corrosion protection for the dowels. Plate and square dowel systems that minimize horizontal restraint as shown in Fig. 6.9, 6.10, and 6.12 can be placed close to the intersection of joints, but no closer than 6 in. (150 mm). Other dowels should be placed no closer than 12 in. (300 mm) from the intersection of any joints because the maximum movement caused by horizontal dry shrinkage occurs at this point, and the corner of the slab may consequently crack.

Table 6.1 provides recommended dowel sizes and spacing for round and square shapes. Because of the various plate dowel geometries and installation devices available from the different manufacturers, the manufacturers should be consulted for their recommended plate dowel size and spacing.

A less effective load-transfer mechanism than those just discussed is aggregate interlock. Aggregate interlock depends on the irregular face of the cracked concrete at joints for load transfer. Designers that choose aggregate interlock as the load-transfer mechanism at joints are cautioned that, for unreinforced concrete slabs, the joint spacings in Fig. 6.6 are intended to minimize the potential for midpanel out-of-joint random cracking, and are independent of load transfer requirements at joints. Not all joints activate uniformly. This results in some joint opening widths that are larger than might normally be anticipated (Walker and Holland 2007b). These wide joints are often called the "dominant joints." The dominant joint behavior is made worse by the ever-increasing use of vapor retarders/barriers placed below the floor slab, which reduces base friction and makes the dominant joints more noticeable and problematic. Where aggregate interlock is anticipated as the only load-transfer mechanism in a slab-on-ground, joint spacing should be the thoughtful result of an evaluation of the anticipated, activated, joint-opening widths and the type of wheel loadings on the slab. Furthermore, when the designer cannot be sure of positive long-term shear transfer at the joints through aggregate interlock, then positive

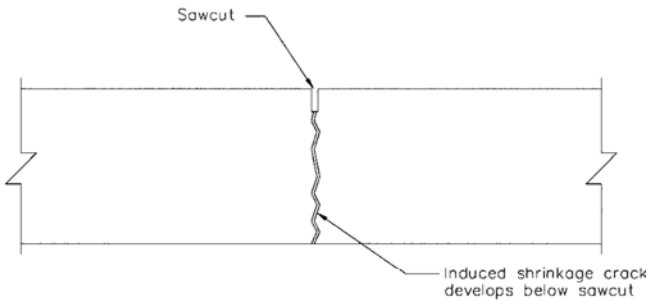


Fig. 6.8—Sawcut contraction joint.

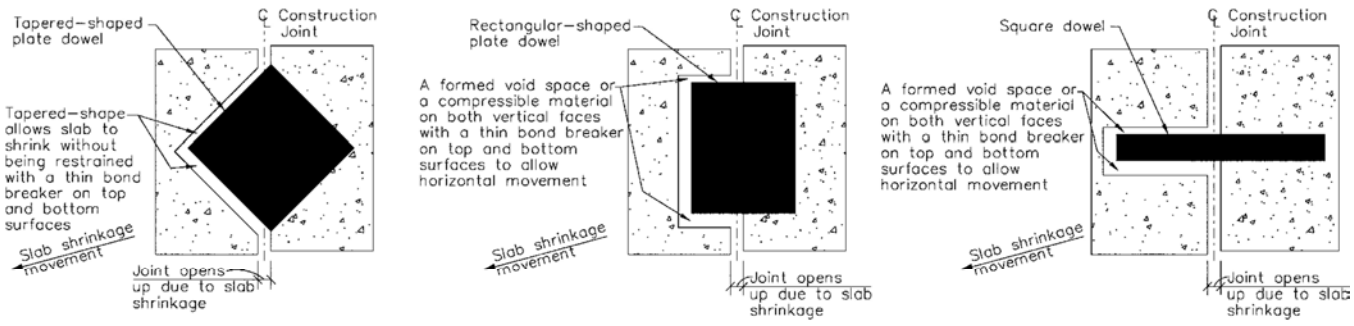


Fig. 6.9—Plan view indicating provisions for longitudinal movement at doweled construction joints.

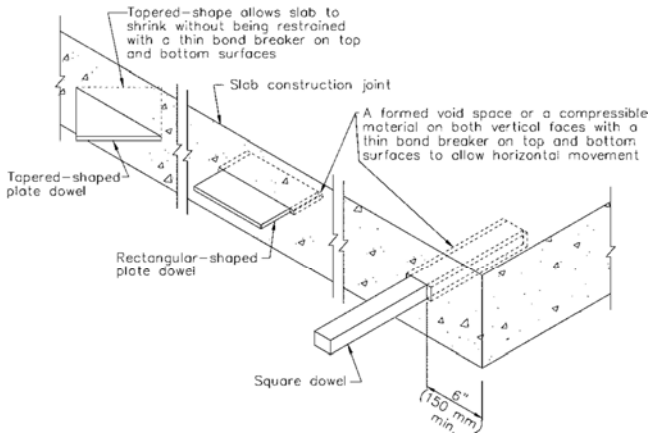


Fig. 6.10—Isometric view indicating provisions for longitudinal movement at doweled construction joints.

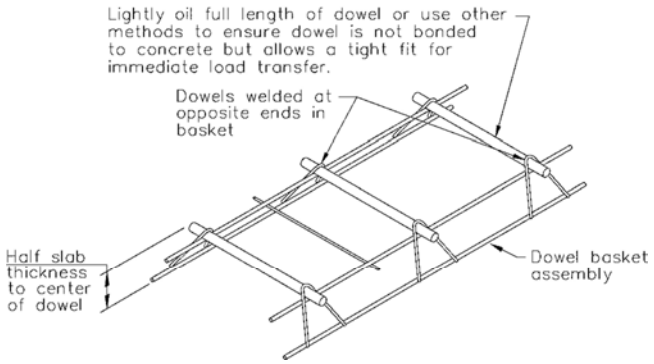


Fig. 6.11—Round dowel basket assembly.

load-transfer devices should be used at all joints subject to wheel traffic.

With respect to this issue, PCA implemented a test program to examine the effectiveness of aggregate interlock as a load-transfer mechanism (Colley and Humphrey 1967). The program tested 7 and 9 in. (180 and 230 mm) thick slabs. The test slabs were constructed using 1-1/2 in. (38 mm) maximum-size aggregate, fully supported on various base materials, and loaded using repetitive applications of 9000 lb (40 kN) on 16 in. (440 mm) diameter pads centered 9 in. (230 mm) from the joints. Among the findings were the following:

- Joint effectiveness for 7 in. (180 mm) thick slabs is reduced to 60% at an opening width of 0.025 in. (0.6 mm);
- Joint effectiveness for 9 in. (230 mm) thick slabs is reduced to 60% at an opening width of 0.035 in. (0.9 mm);
- Three values of foundation-bearing support were used. The values used were k = 89, 145, and 452 lb/in.^{3} (24,200, 39,400, and 123,000 kN/m^3). Joint effectiveness was increased with increases in foundation bearing value k; and
- Joint effectiveness increased with increased aggregate particle angularity.

Using the opening width of a joint or crack as an acceptance criterion has been used with limited success in predicting its service life. There are a number of difficulties in using the opening width. These include:

- How to measure the width, because a crack edge may not have well-defined boundaries due to spalling or other factors;
- The crack width will change with temperature;
- The crack or joint opening width will vary along the length;
- The crack is wider at the top than at the bottom, due to curling; and

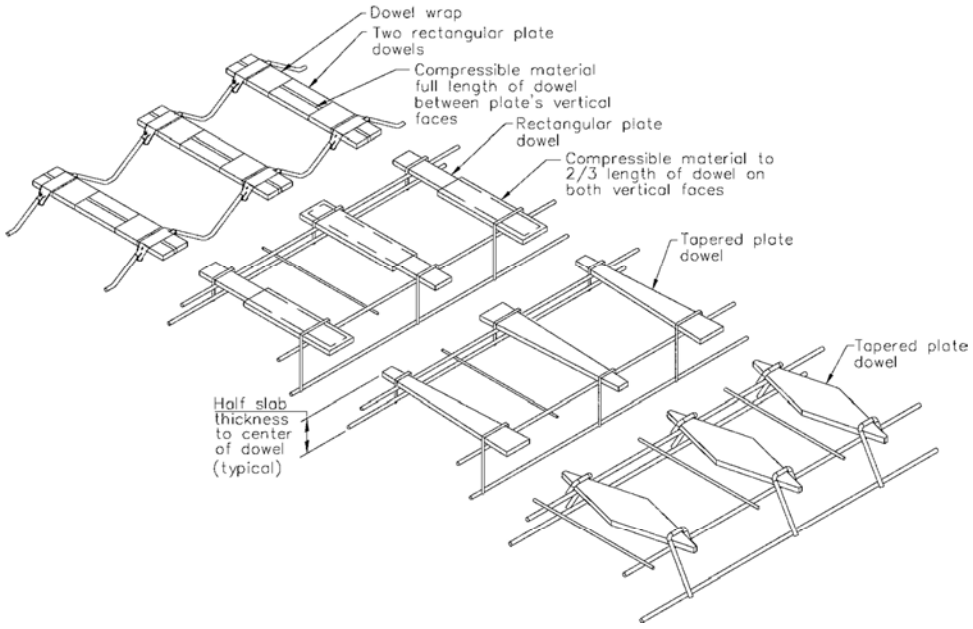


Fig. 6.12—Different plate dowel basket assemblies.

Table 6.1—Dowel size and spacing for construction and contraction joints*

Slab depth, in. (mm)	Dowel dimensions, in. (mm)					Dowel spacing center-to center,† in. (mm)		
	Construction joint		Contraction joint		Plate dowel			
	Round‡	Square§‖	Round‡	Square§‖		Round‡	Square§‖	Plate dowel
5 to 6 (130 to 150)	3/4 x 10 (19 to 250)	3/4 x 10 (19 x 250)	3/4 x 13 (19 x 330)	3/4 x 13 (19 x 330)	M/R#	12 (300)	14 (360)	18 (460)
7 to 8 (180 to 200)	1 x 13 (25 x 330)	1 x 13 (25 x 330)	1 x 16 (25 x 410)	1 x 16 (25 x 410)	M/R#	12 (300)	14 (360)	18 (460)
9 to 11 (230 to 280)	1-1/4 x 15 (32 x 380)	1-1/4 x 15 (32 x 380)	1-1/4 x 18 (32 x 460)	1-14 x 18 (32 x 460)	M/R#	12 (300)	12 (300)	18 (460)

*Table values based on a maximum joint opening of 0.20 in. (5 mm). Carefully align and support dowels during concrete operations. Misaligned dowels may lead to cracking. Spacings are based on dowels in direct contact with a thin bond breaker. Total dowel length includes allowance made for joint opening and minor errors in positioning dowels.

†Dowel spacing up to 24 in. (610 mm) for round, square, and plate dowels have been used successfully.

‡ACI Committee 325 (1956), Teller and Cashell (1958).

§Walker and Holland (1998).

‖Square dowels should have compressible material securely attached on both vertical faces.

#M/R = manufacturers' recommendations. Because of the various plate dowel geometries and installation devices available from different manufacturers, the manufacturers should be consulted for their recommended plate dowel size.

- The joint effectiveness relates to factors other than the opening width. These include load magnitude, slab thickness, soil support system stiffness, degree of curling, aggregate size and angularity.

A more reliable and definable method to evaluate the performance of a joint having a load-transfer mechanism, a joint using aggregate interlock, or a crack that has developed, is by measuring the joint or crack stability. The joint or crack stability is the differential deflection of the adjacent slab panel or crack edges when a service load crosses the joint or crack. Joint or crack stability can be easily measured. It is commonly done by using a leveled straightedge and gauge—for example, a tri-square or dial indicator—to measure the vertical distance from the upper straightedge to slab surface at 12 in. (300 mm) spacing (6 in. [150 mm] each side of the joint) to determine the amount of vertical movement under the bottom of the straightedge as it occurs (Type I Apparatus, ASTM E1155) or by using a device with an inclinometer having 12 in. (300 mm) contact point spacing located 6 in. [150 mm] on each side of the joint that gives a visual readout of the vertical movement as it occurs (Type II Apparatus, ASTM E1155). Joint or crack stability measurements below 0.010 in. (0.25 mm) for joints or cracks subjected to lift truck wheel traffic with small hard wheels will have good service life (Tarr 2004; Walker and Holland 2007a). For lift truck traffic with large, cushioned rubber wheels, a joint or crack stability measurement of 0.020 in. (0.51 mm) should have good service life (Walker and Holland 1999, 2007a). These joint stability values assume the joint is properly filled full depth with semi-rigid joint filler and that the joint filler is properly maintained. For joint or crack stability values above 0.060 in. (1.5 mm), the joint or crack would be considered unstable (Tarr 2004), have a much reduced service life, and most likely should be repaired.

Another load-transfer mechanism is enhanced aggregate interlock. Enhanced aggregate interlock depends on a combination of the effect of a small amount of deformed reinforcement continued through the joint and the irregular face of the cracked concrete at joints for load transfer. The continuation of a small percentage of deformed reinforcement (0.1% of the slab cross-sectional area) through sawcut contraction joints in combination with joint spacings (Fig. 6.6), has been used successfully by some designers to provide load-transfer capability without using dowels. A slab design that uses this small amount of deformed reinforcement to enhance aggregate interlock at the joints should conform to the following:

- Space joints as shown in Fig. 6.6;
- Place the reinforcement above mid-depth but low enough that the sawcut will not cut the reinforcement;
- Place a construction or sawcut contraction joint with a load-transfer device at a maximum of 125 ft (38 m). This forces activation at these joints when the other joints with the deformed reinforcement do not activate;
- Use an early-entry saw to cut all sawcut contraction joints; and
- The slab should be a uniform thickness.

As a general rule, the continuation of larger percentages of deformed reinforcing bars should not be used across sawcut contraction joints or construction joints because they restrain joints from opening as the slab shrinks during drying, and this increases the probability of out-of-joint random cracking. The restraint provided by the reinforcement varies with the quantity of reinforcement in the slab, expressed as a percentage of the cross-sectional area of the slab. Park and Paulay (1975) offer a method of calculating the reduction in unrestrained internal shrinkage strain that can be attributed to the presence of reinforcement. Table 6.2 provides the calculated reduction in strain that can be attributed to the presence of various percentages of reinforcement located at midheight of a slab using the following values:

E_s = modulus of elasticity of steel: 29,000,000 psi (2,000,000 MPa);

E_c = modulus of elasticity of concrete: 2,900,000 psi (20,000 MPa);

C_t = creep coefficient: 2.0; and

e_{sh} = unrestrained shrinkage strain: 0.000500.

This table suggests that the reduction in strain that could be anticipated from 0.1% reinforcement at midheight of the slab is less than 3%. This percentage is relatively minor when compared with the potential impact of variations in the

Table 6.2—Reduction in strain due to reinforcing concrete

Steel ratio, %	Concrete stress, psi (tension)	Steel stress, psi (compression)	Restrained shrinkage strain	Reduction in unrestrained shrinkage strain, %
0.1	14	14,078	0.000485	2.91
0.2	27	13,679	0.000472	5.66
0.3	40	13,303	0.000459	8.26
0.4	52	12,946	0.000446	10.71
0.5	63	12,609	0.000435	13.04
0.6	74	12,288	0.000424	15.25
0.7	84	11,983	0.000413	17.36
0.8	94	11,694	0.000403	19.35
0.9	103	11,417	0.000394	21.26
1.0	112	11,154	0.000385	23.08
3.0	229	7632	0.000263	47.37

Note: 1 psi = 0.00690 MPa.

restraint stresses due to the different coefficients of subgrade friction (Fig. 14.3) and curling stresses.

Plate, square, and round smooth dowels for slab-on-ground installation should meet the requirements of ASTM A36/A36M or A615/A615M. The designer should specify the diameter or cross-sectional area, length, shape, treatment for corrosion resistance, and specific location of dowels, as well as the method of installation, support, and debonding. Refer to Table 6.1, Fig. 6.5, and Fig. 6.9 through 6.12.

For long post-tensioned floor strips and floors using shrinkage-compensating concrete with long joint spacing, take care to accommodate significant slab movements. In most instances, post-tensioned slab joints are associated with a jacking gap. Delay the filling of jacking gaps as long as possible to accommodate shrinkage and creep. In traffic areas, armor plating of the joint edges is recommended (Fig 6.13). Figure 6.14 shows a doweled joint detail at a jacking gap in a post-tensioned slab (PTI 1996, 2004).

6.3—Sawcut contraction joints

Three types of tools are commonly used for sawcutting joints: conventional wet-cut (water-injection) saws; conventional dry-cut saws; and early-entry dry-cut saws. Timing of the sawing operations varies with manufacturer and equipment. The goal of sawcutting is to create a weakened plane as soon as the joint can be cut, preferably without creating spalling at the joint, so the floor slab will crack at the sawcut instead of randomly and create the desired visual effect.

Conventional wet-cut saws are gasoline-powered. With the appropriate blades, they are capable of cutting joints up to 12 in. (300 mm) depth or more. Dry-cut tools can use electrical or gasoline power. They provide the benefit of being generally lighter than wet-cut equipment. Most early-entry dry-cut saws cut to a maximum depth of 1-1/4 in. (32 mm), but some cut to a maximum depth of 4 in. (100 mm). Timing of the early-entry process allows joint sawcutting before significant concrete tensile stresses develop. This increases the probability of cracks forming at the joint when sufficient concrete stresses develop. Care should be taken to ensure that the early-entry saw does not ride up over hard or large coarse aggregate. The highest coarse aggregate should be

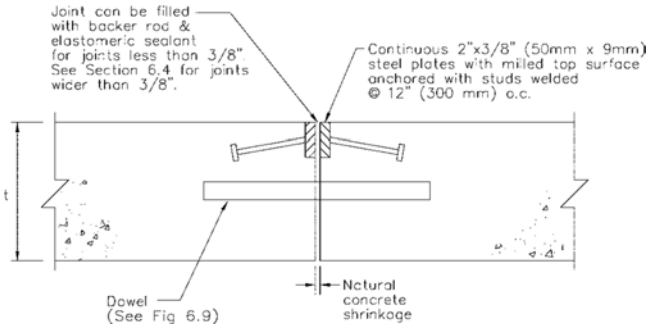


Fig. 6.13—Typical armored construction joint detail.

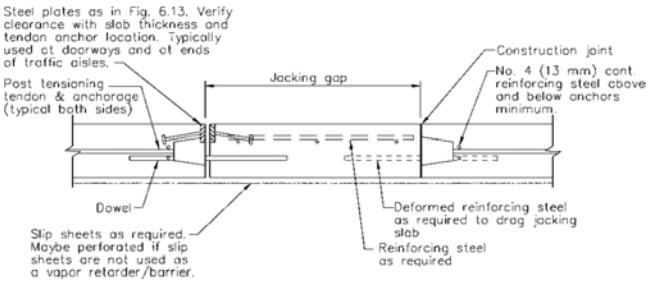


Fig. 6.14—Typical doweled joint detail for post-tensioned slab.

notched by the saw to ensure proper function of the sawcut contraction joint.

Early-entry dry-cut saws use a skid plate that helps prevent spalling. Timely changing of skid plates in accordance with manufacturer's recommendations is necessary to effectively control spalling. Typically, joints produced using conventional processes are made within 4 to 12 hours after the slab has been finished in an area—4 hours in hot weather to 12 hours in cold weather. For early-entry dry-cut saws, the waiting period will typically vary from 1 hour in hot weather to 4 hours in cold weather after completing the finishing of the slab in that joint location. Longer waiting periods may be necessary for all types of sawing for floors reinforced with steel fiber or where embedded mineral-aggregate hardeners with long-slivered particles are used. In all instances, sawing should be completed before slab concrete cooling occurs subsequent to the peak heat of hydration.

The minimum depth of sawcut using a wet conventional saw should be the greater of at least 1/4 of the slab depth or 1 in. (25 mm). The minimum depth of sawcut using an early-entry dry-cut saw should be 1 in. (25 mm) for slab depths up to 9 in. (230 mm). This recommendation assumes that the early-entry dry-cut saw is used within the time constraints noted previously. Some slab designers require cutting the slab the following day to 1/4 of the slab depth to deepen the 1 in. (25 mm) early-entry sawcut and ensure the joint activates. Restricted joint activation using a 1 in. (25 mm) sawcut is a particular concern with doweled joints because dowels may restrain slab movement. For this situation, plate or square dowels cushioned on the vertical sides by compressible material or tapered plate dowels are available in dowel basket assemblies and can reduce this restraint (Fig. 6.12).

For slabs containing steel fibers, the sawcut using a wet conventional saw should be 1/3 of the slab depth. When timely cutting is done with an early-entry saw, the depth can be the same as for unreinforced (plain) concrete for lower fiber concentrations and preferably 1-1/4 in. (32 mm) minimum for higher fiber concentrations up to a 9 in. (230 mm) thick slab. The sawing should be done shortly after the final set, but timing of the sawing is critical so as not to pull up the steel fiber. New, clean saw blades are recommended for crisp sawcuts. Attempt a 5 ft (1.5 m) long cut and evaluate it for spalling or raveling before cutting the entire slab section. When fibers are pulled up, delay the sawing and repeat the procedure until sawing reveals no fibers. Regardless of the process chosen, sawcutting should be performed before concrete starts to cool, as soon as the concrete surface is firm enough not to permit dislodging or spalling of steel fibers close to the floor surface to be torn or damaged by the blade, and before random drying-shrinkage cracks can form in the concrete slab. Shrinkage stresses build in the concrete as it sets and cools. When sawing is unduly delayed, the concrete can crack randomly before it is sawed. Additionally, delays can generate cracks that run off from the saw blade toward the edge of the slab at an obtuse or skewed angle to the sawcut.

6.4—Joint protection

Joints should be protected to ensure long-term performance. Regardless of the materials chosen for protection, the joint needs to have adequate load transfer and the surfaces of adjacent slabs should remain in the same plane.

For wheel traffic, there are two ways to protect a joint: fill the joint with a material to restore surface continuity or armor the edges with steel plates (Fig. 6.13). Certain types of semirigid epoxy or polyurea are the only materials known to the committee that can fill joints and provide sufficient shoulder support to the edges of the concrete and prevent joint breakdown. Such joint materials should be 100% solids and have a minimum Shore A hardness of 80 when measured in accordance with ASTM D2240. Refer to Section 6.5 for more details on joint filling and sealing.

For large slab placements where sawcut contraction joints are not used and the joint width at the construction joints may open significantly, such as post-tensioned slabs, steel-fiber joint-free slabs, or slabs cast with shrinkage-compensating concrete, it is recommended that the joints be protected with back-to-back steel bars (Fig. 6.13 and 6.14). It is critical that the top surfaces of the bars used to armor the joint are horizontally level with the top of the slab. Milling may be required to produce a flat surface when conventional bar stock material is used. Angles have been used to armor joints with limited success. Milling the top surface of the angle to correct camber, sweep, and out-of-square is not practical. Installing and maintaining the top angle leg horizontal during concrete placing and finishing operations is difficult while consolidating the concrete under the angle leg bearing area. Steel-armored joints less than 3/8 in. (9 mm) wide can be sealed with an elastomeric sealant as described in ACI 504R.

Armored joints equal or greater than 3/8 in. (9 mm) wide should be filled full depth with semi-rigid epoxy or polyurea joint filler or joint filler that contains an integral sand extender. This should provide a smooth transition for wheel traffic and minimize damage to tire tread.

Unstable construction and sawcut contraction joints will not retain any type of joint filler. Joints are unstable when there is horizontal movement due to continued shrinkage or temperature changes, or vertical movement due to inadequate load transfer. Regardless of the integrity of initial construction, the continued movement of a filled, curled, undoweled joint under traffic may prematurely fatigue the filler and concrete interface and cause failure. Joint edge protection provided by supportive filler increases when load-transfer provisions are incorporated in the joint design.

6.5—Joint filling and sealing

Where there are wet conditions, hygienic and dust control requirements, and the slab is not exposed to wheel traffic, contraction and construction joints can be filled with joint filler or an elastomeric joint sealant. Joints exposed to wheel traffic should be treated as discussed in Section 6.4.

Isolation or other joints are sometimes sealed with an elastomeric sealant to minimize moisture, dirt, or debris accumulation. Elastomeric sealants should not be used in interior joints subjected to vehicular traffic unless protected with steel armored edges. Refer to ACI 504R for more information on elastomeric sealants.

6.5.1 *Time of filling and sealing*—Concrete slabs-on-ground continue to shrink for years; most shrinkage takes place within the first year. It is advisable to defer joint filling and sealing as long as possible to minimize the effects of shrinkage-related joint opening on the filler or sealant. Ideally, when the building is equipped with an HVAC system, it should be run for two weeks before joint filling. This is especially important where using joint filler in traffic-bearing joints because such materials have minimal extensibility. When the joint is filled before most of the shrinkage has occurred, expect separation between the joint edge and the joint filler or within the joint filler itself. These slight openings can subsequently be filled with a low-viscosity compatible material.

When construction schedule dictates that joints be filled early or when it is decided to fill the joints early to minimize the damage to the joints due to construction traffic, then the construction documents should have provisions to require that the contractor return at a pre-established date, typically between 6 months and 1 year, to repair the joint filler separations using the same manufacturer's product. Early filling results in greater separation and requires more substantial correction. This separation does not indicate a failure of the filler or installation. The construction documents should identify the parties responsible for this repair and address payment requirements.

For cold-storage and freezer-room floors, joint fillers specifically developed for cold temperature applications should be installed only after the room has been held at its planned operating temperature for at least 48 hours. For

freezer rooms with operating temperatures below 0°F (–18°C), the operating temperature should be maintained for 14 days before starting filling joints.

There should be an understanding among all parties as to when the joints will be filled and whether provisions should be made for refilling the joints at a later time when additional concrete shrinkage has taken place.

6.5.2 *Installation*—Elastomeric sealants should be installed over a preformed joint filler, backer rod, or other bond breaker as described in ACI 504R. Semirigid epoxy and polyurea joint fillers should be installed full sawcut depth to bottom of sawcut so that the sawcut ledge provides support for the filler material. Joints should be suitably cleaned to provide optimum contact between the filler or sealant and bare concrete. Remove dirt, debris, saw cuttings, curing compounds, and sealers. Vacuuming is recommended rather than blowing the joint out with compressed air. Cured epoxy and polyurea fillers should be flush with the floor surface to protect the joint edges and recreate an interruption-free floor surface. Installing the joint filler flush with the top of the slab can best be achieved by overfilling the joint and shaving the top of the filler level with the slab surface after the material has hardened.

CHAPTER 7—DESIGN OF UNREINFORCED CONCRETE SLABS

7.1—Introduction

The thickness of unreinforced concrete slabs is determined using an allowable concrete flexural tensile stress. Although the effects of any welded wire reinforcement, plain or deformed bars, post tensioning, fibers, or any other type of reinforcement are not considered, joints may be reinforced for load transfer across the joint. Slabs are normally designed to remain uncracked due to applied loads with a factor of safety of 1.4 to 2.0 relative to the modulus of rupture.

It is important to note that, as set forth in ACI 318, slabs-on-ground are not considered structural members unless they are used to transmit vertical or horizontal loads from other elements of the building's structure (Chapter 12). Consequently, cracking, joint instability, and surface character problems are considered slab serviceability issues and not relevant to the general integrity of the building structure.

Concrete floor slabs employing portland cement, regardless of slump, will begin to experience a reduction in volume as soon as they are placed. This continues as long as water, heat, or both, are released into the surroundings. Because the drying and cooling rates at the top and bottom of the slab are dissimilar, the shrinkage will vary with the depth. This distorts the as-cast shape and reduces volume. Resistance to this distortion introduces internal concrete stresses that, when unrelieved, may cause cracks.

Controlling the effects of drying shrinkage is critical to the performance of unreinforced concrete slabs. Two principal objectives of unreinforced slab-on-ground design are to avoid the formation of random, out-of-joint cracks and to maintain adequate joint stability. The slab's anticipated live loading governs its thickness and cross-joint shear transfer requirements, whereas shrinkage considerations dictate the maximum joint spacing.

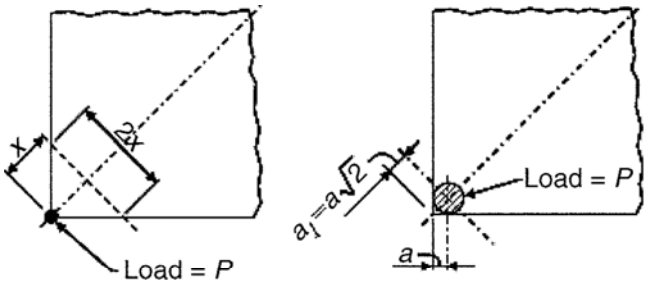


Fig. 7.1—Corner load on slab-on-ground.

Current design and construction procedures are based upon limiting cracking and curling, due to restrained shrinkage, to acceptable levels, but not eliminating them. ACI 302.1R suggests that cracking in up to 3% of the slab panels in a normally jointed floor is a realistic expectation. Refer to ACI 224R for further discussion of cracking in reinforced and unreinforced concrete slabs.

In jointed, unreinforced slabs-on-ground, the design intends to cause shrinkage cracks to occur beneath sawcut contraction joints. In industrial construction, this can result in a floor slab that is susceptible to relative movement of the joint edges and joint maintenance problems when exposed to wheel traffic. When the designer cannot be sure of positive long-term shear transfer at the joints through aggregate interlock, then positive load-transfer devices should be used at all joints subject to wheel traffic. Refer to Section 6.2 for additional information.

7.2—Thickness design methods

When the slab is loaded uniformly over its entire area and supported by uniform subgrade, stresses will be due solely to restrained volumetric changes; however, most slabs are subjected to nonuniform loading. In warehouses, the necessity for maintaining clear aisles for access to stored materials results in alternating loaded and unloaded areas. Rack post and lift truck wheel loads present a more complex loading pattern.

As noted in Chapter 1, the analysis of slabs supporting concentrated loads is based largely on the work of Westergaard (1923, 1925, 1926). Three separate cases, differentiated on the basis of the location of the load with respect to the edge of the slab, might be considered (Winter et al. 1964). These cases are provided to illustrate the effect of load location, particularly at free corners or edges. Most of the generally used structural design methods discussed do not provide for loading at free edges and corners. The designer should carefully consider such loading.

Case 1: *Wheel load close to corner of large slab*—With a load applied at the corner of a slab, the critical stress in the concrete is tension at the top surface of the slab. An approximate solution assumes a point load acting at the corner of the slab (Fig. 7.1). At short distances from the corner, the upward reaction of the soil has little effect, and the slab is considered to act as a cantilever. At a distance x from the corner, the bending moment is Px; it is assumed to be

uniformly distributed across the slab section width at right angles to the bisector of the corner angle. For a 90-degree corner, the section width is $2x$, and the bending moment per unit width of slab is

$$\frac{Px}{2x} = \frac{P}{2}$$

When h is the thickness of the slab, the tensile stress at the top surface is

$$f_t = \frac{M}{S} = \frac{P/2}{h^2/6} = \frac{3P}{h^2} \quad (7\text{-}1)$$

This equation provides reasonably close results only in the immediate vicinity of the slab corner, and only when the load is applied over a small contact area.

In an analysis that considers the reaction of the subgrade, and that considers the load to be applied over a contact area of radius a (Fig. 7.1), Westergaard derives the expression for critical tension at the top of the slab, occurring at a distance $2\sqrt{a_1 L}$ from the corner of the slab

$$f_t = \frac{3P}{h^2}\left[1 - \left(\frac{a\sqrt{2}}{L}\right)^{0.6}\right] \quad (7\text{-}2)$$

where f_t is concrete tensile stress, psi (Pa); a is the radius of load contact area, in. (m); P is the load on the slab-on-ground, lb (N); h is the slab thickness, in. (m); and in which L is the radius of relative stiffness [in. (m)], equal to

$$L = \sqrt[4]{\frac{Eh^3}{12(1-\nu^2)k}} \quad (7\text{-}3)$$

where E is elastic modulus of concrete, psi (Pa); ν is Poisson's ratio for concrete—approximately 0.15; and k is modulus of subgrade reaction, lb/in.^{3} (N/m^3).

The value of L reflects the relative stiffness of the slab and the subgrade. It will be large for a stiff slab on a soft base, and small for a flexible slab on a stiff base.

Case 2: *Wheel load considerable distance from edges of slab*—When the load is applied some distance from the edges of the slab at approximately four times the relative stiffness ($4L$), the critical stress in the concrete will be in tension at the bottom surface. This tension is greatest directly under the center of the loaded area, and is given by the expression

$$f_b = 0.316\frac{P}{h^2}[\log(h^3) - 4\log(\sqrt{1.6a^2 + h^2} - 0.675h) - \log(k) + 6.48] \quad (7\text{-}4)$$

Case 3: *Wheel load at edge of slab, but removed considerable distance from corner*—When the load is applied at a point along an edge of the slab, the critical tensile stress is at the bottom of the concrete, directly under the load, and is equal to

$$f_b = 0.572\frac{P}{h^2}[\log(h^3) - 4\log(\sqrt{1.6a^2 + h^2} - 0.675h) - \log(k) + 5.77] \quad (7\text{-}5)$$

For Eq. (7-4) and (7-5), use P in pounds (lb), h in inches (in.), and k in pounds per cubic inch (lb/in.^{3}), then f_b will be in pounds per square inch (lb/in.^{2}). The logarithms are base 10.

If the flexural tensile stress, as given by the previous equations, exceeds the allowable concrete flexural tensile stress, it is necessary to increase slab thickness, increase concrete flexural strength, or provide reinforcement. Such reinforcement is usually designed to provide for all the tension indicated by the analysis of the assumed homogeneous, elastic slab.

Case 4: *Loads distributed over partial areas*—In addition to concentrated loads, uniform loads distributed over partial areas of slabs may produce the critical design condition. Again, in warehouses, heavy loads alternate with clear aisles. With such a loading pattern, cracking is likely to occur along the centerline of the aisles.

In an analysis based on such loading, Rice (1957) derived an expression for the critical negative moment in the slab M_c that occurs at the center of the aisle

$$M_c = \frac{w}{2\lambda^2}e^{-\lambda a}[\sin(\lambda a)] \quad (7\text{-}6)$$

where

M_c = slab moment at the center of the aisle, in.-lb/in. (m-N/m);
λ = $\sqrt[4]{k/4EI}$, in.$^{-1}$ (m^{-1});
E = elastic modulus of concrete, psi (Pa);
I = moment of inertia, in.^{4} (m^4);
a = half-aisle width, in. (m);
k = modulus of subgrade reaction, lb/in.^{3} (N/m^3);
w = uniform load, psi (N/m^2); and
e = base of natural logarithms.

Recognizing that the aisle width cannot always be predicted exactly, Rice suggests that a "critical aisle width" be used. This width is such as to maximize the above for bending moment (Westergaard 1926).

Generally accepted thickness design methods for unreinforced slabs-on-ground are the:

- PCA method (Section 7.2.1);
- WRI method (Section 7.2.2); and
- COE method (Section 7.2.3).

Each of these methods, described in Chapter 1, seek to avoid live load-induced cracks through the provision of adequate slab cross section by using an adequate factor of safety against rupture. The PCA and WRI methods only address live loads imposed on the slab's interior, whereas the COE method only considers live loads imposed on the slab's edges or joints. All three methods assume that the slab remains in full contact with the ground at all locations. Curl-induced stresses are not considered. Design examples in Appendixes 1, 2, and 3 show how to use all three methods.

7.2.1 *Portland Cement Association design method*—The PCA method is based on Pickett's analysis (Ringo 1986). The variables used are flexural strength, working stress,

wheel contact area, spacing, and the subgrade modulus. Assumed values are Poisson's ratio (0.15) and the concrete modulus of elasticity (4,000,000 psi [28,000 MPa]). The PCA method is for interior loadings only; that is, loadings are not adjacent to free edges.

7.2.1.1 *Wheel loads*—Slabs-on-ground are subjected to various types, sizes, and magnitudes of wheel loads. Lift truck loading is a common example of wheel loads. Small wheels have tire inflation or contact pressures in the range of 85 to 100 psi (0.6 to 0.7 MPa) for pneumatic tires, 90 to 120 psi (0.6 to 0.8 MPa) for steel-cord tires, and 180 to 250 psi (1.2 to 1.7 MPa) for solid or cushion tires (Goodyear Tire and Rubber Co. 1983). Some polyurethane tire pressures exceeding 1000 psi (6.9 MPa) have been measured. Large wheels have tire pressures ranging from 50 to 90 psi (0.3 to 0.6 MPa). Appendix l shows use of the PCA design charts for wheel loadings.

7.2.1.2 *Concentrated loads*—Concentrated loads can be more severe than wheel loads. Design for concentrated loads is the same as for wheel loads. Consider also the proximity of rack posts to joints. Generally, flexure controls the concrete slab thickness. Bearing stresses and shear stresses at the bearing plates should also be checked in accordance with ACI 318. Section A1.3 shows the PCA design charts used for concentrated loads as found in conventionally spaced rack and post storage.

7.2.1.3 *Uniform loads*—Uniform loads do not stress the concrete slab as highly as concentrated loads. The two main design objectives are to prevent top cracks in the unloaded aisles and to avoid excessive settlement due to consolidation of the subgrade. Top cracks are caused by tension in the top of the slab and depend largely on slab thickness, load placement, and short- and long-term subgrade deflections. The PCA tables for uniform loads (Appendix l) are based on the work of Hetenyi (1946), considering the flexural strength of the concrete and the subgrade modulus as the main variables. Values other than the flexural strength and subgrade modulus are assumed in the tables.

7.2.1.4 *Construction loads*—The PCA method does not directly address construction loading. If, however, such loading can be determined as equivalent wheel loads, concentrated loads, or uniform loads, the same charts and tables can be used.

7.2.2 *Wire Reinforcement Institute design method*

7.2.2.1 *Introduction*—The WRI design charts, for interior loadings only, are based on a discrete element computer model. The slab is represented by rigid bars, torsion bars for plate twisting, and elastic joints for plate bending. Variables are slab stiffness factors, modulus of elasticity, subgrade modulus, and trial slab thickness; diameter of equivalent loaded area; distance between wheels; flexural strength; and working stress.

7.2.2.2 *Wheel loads*—Slabs-on-ground subject to wheel loadings are discussed in Section 7.2.1.1. The WRI thickness selection method starts with an assumption of slab thickness so that the stiffness of slab relative to the subgrade is determined. The moment in the slab caused by the wheel loads and the slab's required thickness are then determined. Appendix 2 shows the use of the WRI design charts for wheel loadings.

7.2.2.3 *Concentrated loads*—The WRI charts do not address concentrated loads directly. Because it is possible, however, to determine a wheel load that represents an equivalent concentrated loading, the charts can be used.

7.2.2.4 *Uniform loads*—The WRI provides other charts (Appendix 2) for design of slab thickness where the loading is uniformly distributed on either side of an aisle. In addition to the variables listed in Section 7.2.2.1, the aisle width and the uniform load are variables in this method.

7.2.2.5 *Construction loads*—Construction loads such as equipment, cranes, concrete trucks, and pickup trucks may affect slab thickness design. As with the PCA design method, these are not directly addressed by WRI. Thickness design, however, may be based on an equivalent load expressed in terms of wheel loads or uniform loads.

7.2.3 *The COE design method*—The COE design charts are intended for wheel and axle loadings applied at an edge or joint only. The variables inherent in the axle configuration are built into the design index category. Concentrated loads, uniform loads, construction loads, and line and strip loads are not addressed.

The COE method is based on Westergaard's formula for edge stresses in a concrete slab-on-ground. The edge effect is reduced by a joint transfer coefficient of 0.75 to account for load transfer across the joint. Variables are concrete flexural strength, subgrade modulus, and the design index category.

The design index is used to simplify and standardize design for the lighter-weight lift trucks, generally having less than a 25,000 lb (110 kN) axle load. The traffic volumes and daily operations of various sizes of lift truck for each design index are considered representative of normal warehouse activity and are built into the design method. Assumed values are an impact factor of 25%, concrete modulus of elasticity of 4,000,000 psi (28,000 MPa), Poisson's ratio of 0.20, the wheel contact area, and the wheel spacings. The latter two values are fixed internally for each index category.

Appendix 3 illustrates the use of the design index category and the COE charts. Additional design charts for pavements with protected and unprotected corners have been developed by the COE for pavements, although they may be applied to slabs-on-ground in general.

7.3—Shear transfer at joints

A principal concern governing the spacing of sawcut contraction joints is edge curl (Walker and Holland 1999). Effective shear transfer at both construction and intermediate sawcut contraction joints is required to avoid a loaded free edge. Also, curl and shrinkage can reduce joint stability by disengaging aggregate interlock or keyed joints, allowing the free edges to deflect independently under wheel traffic. Excessive curling and shrinking can also reduce the joint stability of doweled joints. Positive load-transfer devices, such as dowels, should be used for joints subjected to wheel traffic where the joint is expected to open more than 0.025 to 0.035 in. (0.6 to 0.9 mm). Chapter 6 contains an expanded discussion of jointing of slabs-on-ground and protecting the

joints. The PCA (2001) provides considerations for the effectiveness of shear transfer at joints.

7.3.1 *Steel dowels*—Steel dowels are the most effective means to provide effective load transfer and to ensure adjacent curled joint edges deflect together. Refer to Chapter 6 for a discussion of different doweling approaches.

When dowels are installed across a joint, the slab edges abutting the joint may still curl and deflect when loaded, but they do so in unison. When the wheel reaches the joint, no significant relative vertical displacement between the panels is encountered, and the impact loads imposed on the edges are greatly reduced.

7.4—Maximum joint spacing

Assuming the subgrade is relatively free from abrupt changes in elevation, such as that caused by uncorrected wheel rutting, the tensile stresses created in the shrinking panel by subgrade frictional restraint are relatively minor in comparison to curling-induced stresses. These higher curling stresses are likely the principal cause of shrinkage cracking in most unreinforced concrete floor slabs (Walker and Holland 1999).

In general, joint spacing should not exceed the spacing in Fig. 6.6 and as discussed in Chapter 6. Refer to Chapter 14 for discussion on how joint spacing affects curling-induced stress.

CHAPTER 8—DESIGN OF SLABS REINFORCED FOR CRACK-WIDTH CONTROL

8.1—Introduction

The thickness of slabs-on-ground should be selected to prevent cracking due to external loadings as discussed in Chapter 7. Slab thickness calculations should be based on the assumption of an uncracked and unreinforced slab. Reinforcement may be used in slabs-on-ground to improve performance of the slab under certain conditions. These reinforcement benefits include:

- Limiting shrinkage crack width;
- Use of longer joint spacings than unreinforced slabs; and
- Providing flexural strength and stability at cracked sections.

Reinforcement will not prevent cracking, but will actually increase crack frequency while reducing crack widths. Properly proportioned and positioned, reinforcement limits crack widths such that the cracks should not affect slab serviceability. The appearance of cracks for this slab type should, however, be discussed with the owner so that the owner has the expectation that cracks will possibly occur.

8.2—Thickness design methods

The inclusion of reinforcement, even in large quantities, has very little effect on the uncracked strength of the slab. The PCA, WRI, and COE thickness design methods described in Chapter 7 may all be applied to the design of reinforced slabs-on-ground by ignoring the presence of the reinforcement.

8.3—Reinforcement for crack-width control only

Reinforcement required for crack-width control is a function of joint spacing and slab thickness. To eliminate sawcut contraction joints, a continuous amount of reinforcement with a minimum steel ratio of 0.5% (PCA 2001) of the slab cross-sectional area in the direction where the contraction joints are eliminated is recommended. For slabs that will not be exposed to view, or where appearance is not important, the reinforcement should be located as close to the slab top surface as possible while maintaining minimum concrete cover over the reinforcement. For slabs that will be exposed to view and the surface appearance is important, consideration should be given to specifying sufficient cover to minimize possible bar shadowing and subsidence cracking longitudinally over the reinforcement (Babaei and Fouladgar 1997; Dakhil and Cady 1975). A common practice is to specify that the steel have 1.5 to 2 in. (38 to 51 mm) cover from the top surface of the concrete to the bar to minimize the bar shadowing and subsidence cracking.

CHAPTER 9—DESIGN OF SHRINKAGE-COMPENSATING CONCRETE SLABS

9.1—Introduction

This chapter deals with shrinkage-compensating concrete slabs made with cement conforming to ASTM C845. The design procedure differs significantly from that for conventional concrete with ASTM C150/C150M portland cement and blends conforming to ASTM C595.

When concrete dries, it contracts or shrinks and when it is wetted again, it expands. These volume changes with changes in moisture content are an inherent characteristic of hydraulic-cement concrete. ACI 224R discusses this phenomenon in detail. Volume changes also occur with temperature changes.

Shrinkage-compensating concrete is expansive cement concrete that, when restrained by the proper amount of reinforcement or other means, will expand an amount equal to or slightly greater than the anticipated drying shrinkage. Subsequent drying shrinkage reduces the expansion strains, but ideally, a residual compressive stress remains in the concrete, thereby minimizing shrinkage cracking and curling. Sections 9.1.1 and 9.1.2 explain how shrinkage-compensating concrete differs from conventional concrete with respect to volume changes.

9.1.1 *Portland-cement and blended-cement concrete*—The shortening of cement and blended-cement concrete due to shrinkage is restrained by reinforcement and friction between the ground and the slab. This shortening occurs at an early age, and the friction can cause concrete tension restraint stress in excess of its early tensile strength, thereby cracking the slab.

As drying shrinkage continues, cracks open wider. This may present maintenance problems, and when the crack width exceeds 0.025 in. (0.6 mm), aggregate interlock (load transfer) becomes ineffective. Refer to Section 6.2 for additional information on aggregate interlock. Cracking due to shrinkage restraint can be minimized by closer joint spacing or post-tensioning, or crack widths can be minimized with additional distributed reinforcement.

9.1.2 *Shrinkage-compensating concrete compared with conventional concrete*—Shrinkage-compensating concrete

is used to limit cracking and curling. Shrinkage-compensating concrete is made with cement conforming to ASTM C845 rather than ASTM C150/C150M or C595/C595M. Therefore, the volume change characteristics are different. Shrinkage-compensating concrete undergoes an initial volume increase during the first few days of curing, and then undergoes drying shrinkage. The drying-shrinkage characteristics of shrinkage-compensating concrete are similar to those of portland-cement concrete. The drying shrinkage of shrinkage-compensating concrete is affected by the same factors as portland-cement concrete. These include water content of the concrete mixture, type of aggregate used, aggregate gradation, and cement content. The water content influences the expansion during curing and subsequent shortening due to drying shrinkage. Figure 9.1 illustrates the typical length-change characteristics of shrinkage-compensating and portland-cement concrete prism specimens tested in accordance with ASTM C878/C878M (ACI Committee 223 1970).

In shrinkage-compensating concrete, the expansion is restrained by the bonded reinforcement, which causes tension in the reinforcement. As a result of this expansive strain causing tension in the reinforcement, compression develops in the concrete to oppose this tension. These stresses are relieved over time by drying shrinkage and creep. It is intended that the restrained expansion be greater than the resultant long-term shrinkage, as shown in Fig. 9.2, so the concrete remains in compression. The minimum recommended amount of concrete expansion for slabs-on-ground, measured in accordance with ASTM C878/C878M, is 0.03%.

9.2—Thickness determination

For a shrinkage-compensating concrete slab-on-ground, the determination of the slab thickness is similar to that used for other slab-on-ground design methods. The PCA, WRI, and COE methods are all appropriate. Refer to Chapter 7 and Appendixes 1, 2, and 3. Appendix 5 illustrates other design considerations specific to shrinkage-compensating concrete.

9.3—Reinforcement

9.3.1 *Restraint*—An elastic type of restraint, such as that provided by internal reinforcement, should be provided to develop shrinkage compensation. Other types of restraint such as adjacent structural elements, subgrade friction, and integral abutments, are largely indeterminate, and may provide either too much or too little restraint. Subgrade frictional coefficients in the range of 1 to 2 have been used with acceptable results. High restraint, however, induces a high compressive stress in the concrete but provides little shrinkage compensation. To reduce subgrade frictional restraint, which allows easier expansion, two sheets of polyethylene have been used successfully. Subgrade frictional coefficients as low as 0.20 have been measured (Timms 1964) for two sheets of polyethylene in the laboratory. Due to the construction variations in the base, however; a more realistic subgrade friction value of 0.30 for two sheets of polyethylene is likely and recommended for projects with

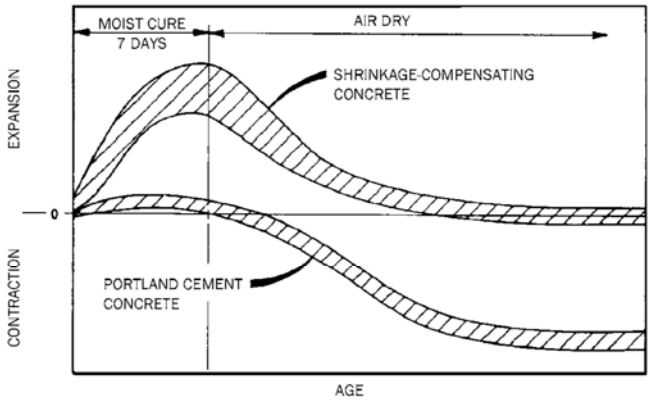


Fig. 9.1—Typical length change characteristics of shrinkage-compensating and portland-cement concretes (ACI Committee 223 1970).

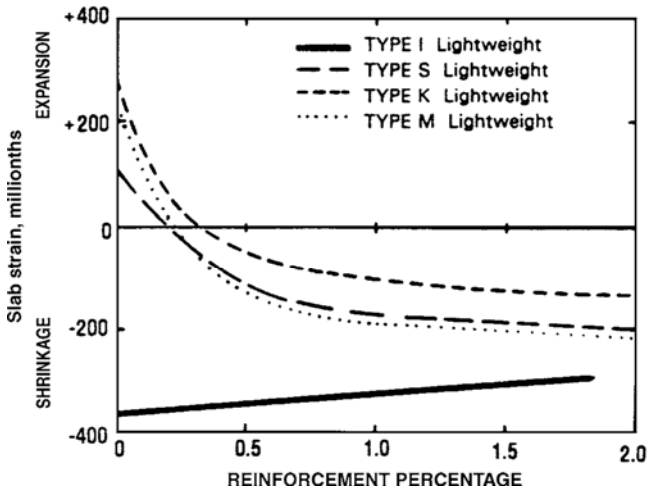


Fig. 9.2—Effect of reinforcement on shrinkage and expansion at 250 days (Russell 1980).

smooth and level bases. Wherever possible, the designer should specify the reinforcement recommended in ACI 223.

9.3.2 *Minimum reinforcement*—A minimum ratio of reinforcement area to gross concrete area of 0.0015 should be used in each direction that shrinkage compensation is desired. This minimum ratio does not depend on the reinforcement yield strength. When procedures outlined in ACI 223 are followed, however, a reinforcement ratio of less than 0.0015 may be used.

9.3.3 *Effect of reinforcement location*—The position of the steel is critical to both slab behavior and internal concrete stress. ACI 223 recommends positioning reinforcement 1/3 of the depth from the top. The top reinforcement balances the restraint provided by the subgrade and provides elastic restraint against expansion. Exercise caution when using smaller percentages of reinforcement because small-gauge bars and wires may be more difficult to position and maintain in the top portion of the slab. Use lower reinforcement percentages with stiffer, more widely spaced reinforcement such as ASTM A497/A497M, deformed wire reinforcement ASTM A615/A615M, ASTM A996/A996M, and ASTM A706/A706M deformed bars.

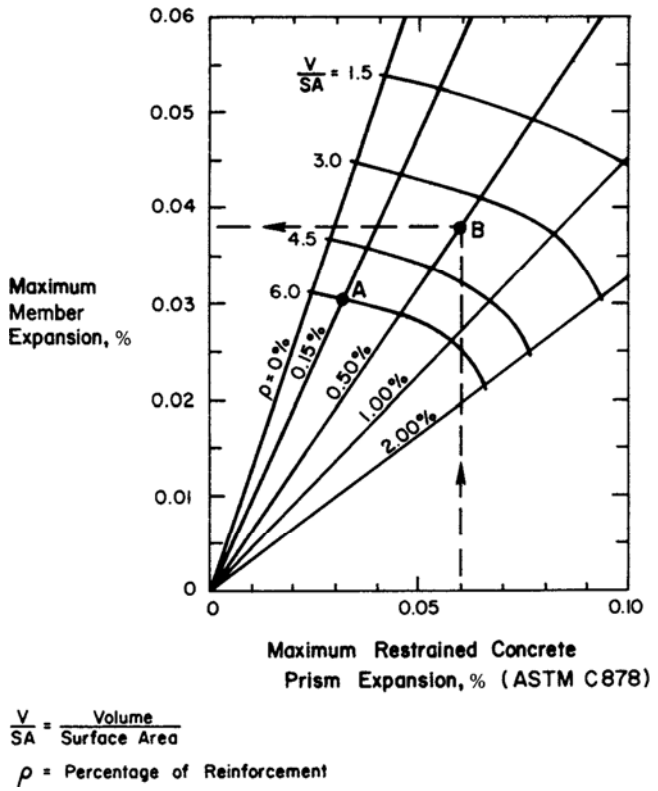


Fig. 9.3—Slab expansion versus prism expansion for different volume-surface ratios and reinforcement percentages (from ACI 223).

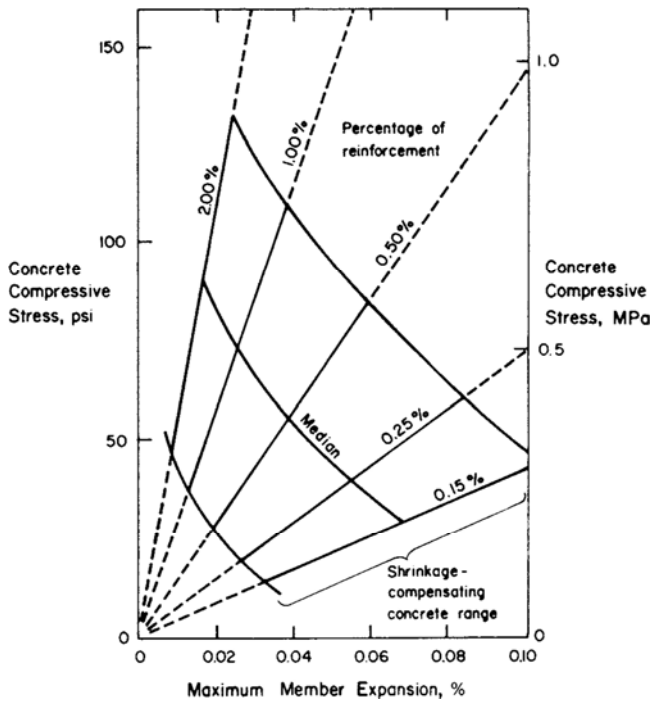


Fig. 9.4—Calculated compressive stresses induced by expansion (ACI 223).

9.3.4 *Maximum reinforcement*—The objective of full shrinkage compensation is to attain restrained member expansive strains equal to or greater than the restrained shrinkage strains. Kesler et al. (1973) cautioned that the maximum level of reinforcement should be approximately 0.6% because, at that point, restrained expansion strains equaled restrained shrinkage strains. This maximum ratio does not depend on the reinforcement yield strength. To prevent concrete from shrinking more than the restrained expansion, use lighter percentages of steel. Should high steel ratios be required for structural design conditions, higher expansion levels in the concrete, as measured by ASTM C878/C878M prisms, would be required.

The required level of ASTM C878/C878M prism expansion strains can be determined by using Fig. 9.3. The figure shows the relationship between prism expansions, reinforcement percentages, volume-surface relationship, and resulting concrete slab expansions. Use the volume-surface ratio for different slabs and different reinforcement percentages to estimate the anticipated member shrinkage strains. When the resulting slab expansions are greater than the resulting shrinkage strains for a given volume-surface relationship, then full shrinkage compensation occurs. This prism value is the minimum value that should be specified or verified in the lab with trial mixtures; the minimum recommended amount of concrete expansion for slabs-on-ground measured in accordance with ASTM C878/C878M is 0.03% (Russell 1973).

9.3.5 *Alternative minimum restraint levels*—Russell concluded that restrained expansion should be equal to or greater than restrained shrinkage (Keeton 1979). The concrete shrinkage depends on aggregate type and gradation, unit water content, volume-surface ratios, environment, and other conditions. Volume-surface ratio mathematically expresses the drying surface or surfaces in comparison to the volume of a concrete member. Slabs-on-ground have single-surface (top) drying, whereas walls and elevated structural slabs have two faces for drying. Thus, 6:1 is the volume-surface ratio for a 6 in. (150 mm) slab drying on the top surface. The expansion strain depends largely on the expansion capability of the concrete mixture, which in turn depends on cement factor, curing, admixture, and the level of internal and external restraint.

The minimum reinforcement required to properly control expansion for shrinkage compensation depends on the potential shrinkage of the slab and the restrained prism expansion of the concrete mixture measured according to ASTM C878/C878M. For a given volume-surface ratio and a minimum standard prism expansion level, which has been verified with trial batch data, the internal restraint levels provided by less than 0.15% steel in a typical 6 in. (150 mm) slab can be used (ACI SP-64 (ACI Committee 223 1980)). When the slab expansion is greater than the shrinkage strain for a surface-volume ratio of 6:1, using Russell's (1980) data, full compensation can be achieved. Figure 9.4 shows circumferential curves depicting shrinkage strains for volume-surface ratios for other slab thicknesses.

Exercise care when using low reinforcement ratios. When light reinforcement is used, it may accidentally be depressed into the bottom 1/3 of the slab, which can lead to subsequent warping and cracking. Light, but stiff, reinforcement can be obtained by using larger bars or wire at a wider spacing. The maximum spacing of reinforcing bars should not exceed

three times the slab thickness. For smooth wire reinforcement, the spacing should not be more than 14 in. (360 mm), even though a wider spacing is easier for workers to step through. Deformed welded wire reinforcement can be spaced in the same manner as reinforcing bars. When tests and design calculations are not performed, the minimum 0.15% reinforcement is often specified.

9.4—Other considerations

9.4.1 *Curvature benefits*—Keeton (1979) investigated portland-cement concrete and shrinkage-compensating concrete slabs that were allowed to dry only from the top surface for 1 year after both types were given similar wet curing. The expansion and shrinkage profiles of both slabs were monitored. Expansive strains of the shrinkage-compensating concrete were greater at the top fibers than at the lower fibers of a slab-on-ground, setting up a convex profile that was the opposite of the concave profile of portland-cement concrete slabs. This occurred despite having reinforcement located in the top 1/4 of the slab. Both reinforced and plain slabs, as well as fiber-reinforced slabs, displayed this behavior.

9.4.2 *Prism and slab expansion strains and stresses*—Because the reinforcement percentage varies, use the ASTM C878/C878M restrained concrete prism test to verify the expansive potential of a given mixture. Use Fig. 9.3 to determine the amount of slab expansion (strain) using the known prism expansion value and the percent of slab reinforcement.

The amount of internal compressive force acting on the concrete can be estimated by using Fig. 9.4 knowing the maximum member (slab) expansion and the percent of internal slab reinforcement.

9.4.3 *Expansion/isolation joints*—Because a slab may be restrained externally on one side by a previously cast slab, the opposite side should accommodate the expansive strains. When a slab is adjacent to a stiff wall, pit wall, or other slab, external restraint on two opposite sides is present. When external restraints are stiff, compressive stresses as high as 45 to 172 psi (0.31 to 1.19 MPa) may prevent the concrete from expanding and elongating the steel (Russell 1973).

Normal asphaltic premolded fiber isolation joints are far too stiff to provide adequate isolation and accommodate expansion as their minimum strength requirements are in the 150 psi (1.0 MPa) range at a compression of 50% of the original joint thickness. A material with a maximum compressive strength of 25 psi (0.17 MPa) at 50% deformation according to ASTM D1621 or D3575 should be used.

For a slab allowed to expand only at one end during initial expansion, the width of the isolation joint should be equal to two times the anticipated slab expansion using Fig. 9.3 and multiplied by the length of the longest dimension of the slab. For a 100 x 120 ft (30 x 37 m) slab with expansion strain of 0.00035, the required joint width is

$$\text{Joint width} = 2 \times 120 \times 12 \times 0.00035 = 1.008 \text{ in.} \quad \text{(in.-lb)}$$
$$(2 \times 36.6 \times 1000 \times 0.00035 = 25.6 \text{ mm} \quad \text{[SI])}$$

Use 1 in. (25 mm) thick joint material when the slab is to expand only at one end, and use 1/2 in. (13 mm) thick joint material if allowed to expand at both ends.

9.4.4 *Construction joints*—When using shrinkage-compensating concrete, slabs may be placed in areas as large as 16,000 ft^2 (1500 m^2) without joints (ACI 223). Placements of this size should only be considered in ideal conditions. Placements of 10,000 ft^2 (930 m^2) or less are more common with joint spacing of 100 ft (30 m).

Slab sections should be as square as possible, and provisions should be made to accommodate differential movement between adjacent slabs in the direction parallel to the joint between the two slabs. ACI 223 provides explanation and details.

9.4.5 *Placing sequence*—The placement sequence should allow the slab's expansive strains to occur against a free and unrestrained edge. The opposite end of a slab, when cast against a rigid element, should be free to move. A formed edge should have the brace stakes or pins loosened after the final set of the concrete to accommodate the expansive action.

The placing sequence should be organized so that the edges of slabs are free to move for the maximum time possible before placing adjacent slabs. At least 70% of the maximum measured laboratory expansion per ASTM C878/C878M should occur before placing adjacent slabs when a slab is not free to expand on two opposite ends. Refer to ACI 223 for placement pattern examples. Avoid checker-boarded placements unless placing a compressible joint material between the slabs before concrete placement as per Section 9.4.3.

Before establishing the placement sequence, consider the results of expansion testing per ASTM C878/C878M. A minimum level of prism expansion of 0.03% is recommended for slabs-on-ground. Higher expansion results accommodate larger slab placements or slabs that have higher amounts of reinforcements. Trial batches for the tested mixture proportion should use materials identical to those that will be used during construction and tested at the proposed slump that will be used in the field.

9.4.6 *Concrete overlays*—Overlays are used at times to increase the thickness of a slab during initial construction or as a remedial measure. Improved wear performance or new finished floor elevation are the common reasons for using overlays. The two types of overlays—bonded and unbonded—are covered in ACI 302.1R as Class 7 and Class 8 floors.

Bonded overlays are generally a minimum of 3/4 in. (19 mm) thick, but thicknesses of 3 in. (76 mm) or more are not uncommon. Bonded overlays are used to improve surface abrasion resistance with the use of a wear-resistant aggregate. To improve the abrasion resistance and impact resistance of floor surfaces, employ a more ductile material, such as graded iron.

Joints in a deferred topping slab should accommodate shrinkage strains by matching the base slab joints. The base slab joints should be carefully coordinated with the topping slab joints and continued through the topping, or a crack will develop. Base slabs that contain cracks that move due to slab motion will often reflect cracks into the topping. Therefore, these cracks should be repaired. When the base slab contains

shrinkage-compensating concrete, the portland-cement concrete bonded topping should be applied at least 10 days after placing the base slab. This allows the base slab to display volume change characteristics similar to portland-cement concrete, as both the topping and the base slab shorten simultaneously. For bonded toppings, joints in addition to those matching joints in the base slab do not serve a purpose.

A bonded topping of shrinkage-compensating concrete should not be attempted as an overlay on a portland-cement concrete base slab. The base slab restraint negates the expansion action of the topping and leads to cracking or possibly delamination.

CHAPTER 10—DESIGN OF POST-TENSIONED SLABS-ON-GROUND

10.1—Introduction

Post-tensioning of ground-supported slabs began in the early 1960s. In 1967, the first three ground-supported slabs using a system of post-tensioned reinforcement approved by the Federal Housing Administration were installed in Houston. In January 1968, tests on a 20 x 40 ft (6 x 12 m) post-tensioned residential ground-supported slab were reported (Thompson and Anderson 1968). In June 1968, these tests and previous experience with completed construction led the U.S. Department of Housing and Urban Development to approve the use of post-tensioned ground-supported slabs throughout the U.S. The only requirement placed on using this reinforcement method was that the slab be designed by a licensed professional engineer. Since June 1968, millions of square feet of ground-supported slabs for residential, commercial, and industrial applications have been constructed using post-tensioned concrete.

Slabs-on-ground may be post-tensioned using bonded or unbonded tendons; however, bonded tendons are rarely used. Tendons are post-tensioned and anchored after the concrete obtains sufficient strength to withstand the force at the anchorage. The primary advantages of a post-tensioned slab-on-ground are:

- Increased joint spacing—only construction joints are necessary—no sawcut contraction joints;
- Minimizes shrinkage and flexural cracks due to active prestress;
- Lower life-cycle cost, fewer joints to maintain, and higher durability due to precompression;
- Enhanced serviceability and minimum facility down-time for joint repair or maintenance;
- Better preservation of floor flatness and levelness by minimizing the number of joints and joint curling;
- Decreased slab thickness;
- Increased load strength; and
- Resilience and recovery capability from overloading.

It is not likely that a ground-supported slab will deflect sufficiently to exceed the yield strength of the post-tensioning steel, which means that cracks due to overload are likely to close up after load removal. Post-tensioned slabs require competent construction supervision and coordination. Tendons for all applications should be properly placed, stressed, and anchored. Also, the concrete properties should meet the design criteria to be able to receive the forces introduced through the post-tensioning anchorages. The post-tensioning system (PTI 2006) used for industrial floor applications should meet PTI "Specifications for Unbonded Tendons" (PTI 2000).

10.2—Applicable design procedures

Several design procedures are used to design post-tensioned slabs-on-ground. A careful consideration of these procedures along with the expected performance of the slab is used to determine which procedure is appropriate.

10.2.1 *Thickness design*—The required thickness of post-tensioned slabs may be determined by increasing the permissible tensile stress of the concrete by the net precompression from the prestressing force. Refer to the PCA, WRI, and COE methods described in Chapter 7 and illustrated in Appendixes 1, 2, and 3.

10.2.2 *Crack-control design*—This design is normally used for slabs with light loading, usually with no rack post loading. Post-tensioning is mainly used to increase the joint spacing and to provide preservation of floor flatness and levelness. Calculate the minimum post-tensioning force to provide some residual compression over the tension resulting from subgrade drag due to concrete shrinkage and temperature effects.

10.2.3 *Industrial floor design*—This design is normally used for slabs with higher loads, especially concentrated loading. The design of a typical post-tensioned industrial floor can be accomplished by following these steps:

1. Determine slab geometry, placement sizes, estimate slab thickness (normally 5 to 10 in. [125 to 250 mm]).
2. Calculate the subgrade drag and the friction losses in the post-tensioning tendons.
3. Estimate long-term losses to arrive at the final effective prestress force. For floors subjected to large temperature changes, consider the effects of temperature on the concrete in determining the final effective post-tensioning force.
4. Analyze the loading effects using the Westergaard equations or similar analysis yielding the stresses under the concentrated, uniform lift-truck or line loads. Different formulas are available for loads in the middle of a slab and at the edge.
5. Verify that the total superimposed stresses and deflections do not exceed the allowable values. Depending on the results, a modification of the slab thickness or slab placement layout may be necessary.

10.2.4 *Post-Tensioning Institute method*—In 2004, the PTI published a third edition of *Design and Construction of Post-Tensioned Slabs-on-Ground* (PTI 2004), which contains recommendations for establishing strength and serviceability requirements for residential post-tensioned concrete slabs on either stable, expansive, or compressible soils. These strength and deflection requirements are based on the assumption of an uncracked section.

The PTI design procedure uses post-tensioning as the primary reinforcement for a ribbed and uniform thickness slab. A ribbed slab is reinforced to provide sufficient strength and deflection control in swelling and compressible soil conditions. A post-tensioned slab is able to remain

uncracked for higher loads and therefore control slab movements due to its greater stiffness.

Design and Construction of Post-Tensioned Slabs-on-Ground presents equations for the determination of the moment, deflection, and shear for slabs cast on expansive or compressible soils. These equations were developed by a log-linear regression analysis based on the results of 768 separate analyses that represented full consideration of both center lift and edge lift conditions using a finite element plate-on-elastic-half-space foundation (Wray 1978). The results of each analysis were screened for the maximum values of moment, shear, and differential deflection in both the long and short direction. These values were used in the regression analysis that developed the design equations. The engineer should consider site-specific circumstances such as climate, trees, slopes with cut and fill, and surface water drainage conditions, as these can change the results considerably. Moisture-sensitive soils should be stabilized by minimizing exchanges in moisture content. The PTI (2004) document also provides slab design recommendations and design examples.

10.3—Slabs post-tensioned for crack control

10.3.1 *Design methods*—For lightly loaded slabs, typically with no rack post loads, the crack control design is used. The practical minimum slab thickness is approximately 4 in. (100 mm) to provide necessary concrete cover to the prestressed reinforcement. This minimum slab thickness with residual compression from post-tensioning will considerably increase the load strength of the slab.

10.3.2 *Post-tensioning force required*—Calculate the post-tensioning force P_r (lb/ft [N/m]) required to overcome the subgrade friction by

$$P_r = W_{slab}\frac{L}{2}\mu \qquad (10\text{-}1)$$

where W_{slab} is the self-weight of the foundation slab, lb/ft^2 (Pa) (unit weight in lb/ft^3 [kg/m^3] adjusted to the slab thickness); L is the slab length in the direction being considered, ft (m); and μ is the coefficient of friction between the slab and subgrade.

The following coefficients of friction μ are recommended for slabs constructed on polyethylene sheeting (Timms 1964):

- Slabs using two sheets of polyethylene (refer to Section 9.3.1 for discussion of coefficient of friction value): 0.30. For post-tensioned slabs over 100 ft [30 m], 0.50 might be used to account for variations in base elevation over longer distances;
- Slabs on one layer of polyethylene sheeting: 0.50 to 0.75; and
- Slabs constructed on a sand base: 0.75 to 1.00.

For longer-ribbed slabs, consider the restraint due to the rib geometry.

The slab designer determines the residual compressive force after all prestress losses and the subgrade friction losses based on the slab geometry, loading, and usage. Recommended residual prestress levels are:

Residential foundations:	50 to 75 psi (0.3 to 0.5 MPa)
Industrial floors up to 100 ft (30 m) long:	75 to 100 psi (0.5 to 0.7 MPa)
Industrial floors up to 200 ft (60 m) long:	100 to 150 psi (0.7 to 1.0 MPa)
Industrial floors up to 300 ft (90 m) long:	150 to 200 psi (1.0 to 1.4 MPa)
Industrial floors up to 400 ft (120 m) long:	200 to 250 psi (1.4 to 1.7 MPa)

Calculate the friction losses, elastic shortening, and the long-term losses in the tendons according to Zia et al. (1979).

The tendon spacing S_{ten} (ft [m]) required to overcome slab subgrade friction and maintain a residual compression at the center of a solid slab is given by the equation

$$S_{ten} = \frac{P_e}{f_p WH + P_r} \qquad (10\text{-}2)$$

where P_e is the effective prestress force per tendon, lb (N); f_p is the minimum average residual prestress (required compressive stress), psi (MPa); W is the slab unit strip width (12 in./ft [1000 mm/m]); H is slab thickness, in. (mm); and P_r is from Eq. (10-1), lb/ft (N/m).

10.3.3 *Floating slab*—A floating slab is a slab-on-ground isolated from all restraining elements that would resist contractions and expansions due to drying shrinkage, elastic shortening due to post-tensioning, or temperature effects. Crack-control reinforcement is only effective when the slab is allowed to shorten due to drying shrinkage and the elastic shortening due to the post-tensioning force. Any slab restraints such as columns, walls, footings, and loading docks should be isolated so as not to prevent slab movement. The concrete placement layout should be designed in such a way so that the adjacent placements do not restrain the relative slab movements. Dowels or other joint reinforcement going through a joint should have a compressible material on the side of relative movement.

10.3.4 *Tendon stressing*—The stressing sequence should be adjusted to the project requirements. One-time stressing may be used for most residential slabs after the concrete reaches sufficient strength to transfer the force from the anchorages. Industrial floors may need two stressing stages to prevent early shrinkage cracks from appearing. Typically, the initial stage (partial) stressing should be completed within 24 hours after concrete placement. The partial tendon stress should be based on compression strength cylinders stored in the same environmental conditions as the slab. The cylinders should be broken immediately before partial stressing the tendons.

10.3.5 *Tendon layout*—Depending on the slab usage (flexible rack layout, random traffic, heavy racks with fixed layout) and the placement layout and sequence (long narrow strips, rectangular sections), post-tensioning can be provided in one or both directions. One-way post-tensioning is common for narrow strip placements. Two-way post-tensioning is used for random traffic areas and rectangular placement areas. Typically, post-tensioning tendons are placed in the direction of vehicle traffic. Sometimes the post-tensioning tendons cross the traffic joint to maintain compression across the joint and keeping it from opening. This enhances the durability of the traffic joint and eliminates a need for more severe measures

such as dowels or armored joint. Coordinate slab penetrations and drilling anchors placed after construction with tendon locations to avoid severing tendons. Use metal detectors or similar devices to locate tendons.

10.4—Industrial slabs with post-tensioned reinforcement for structural support

10.4.1 *Design methods*—The thickness design methods listed in Section 10.2.1 can be used. These industrial floor design methods allow for an accurate analysis of the effects from the common load cases for warehouse slabs. Conventional structural concrete design methods should be used when the assumption of an uncracked section is not valid. Apply the provisions of Section 10.3 to control cracking and to help determine the placement and tendon layouts.

10.4.2 *Factors of safety*—Avoid the use of multiple factors of safety. Post-tensioning provides reserve capacities, and factors of safety greater than those for nonprestressed slabs should not be used. Cracking under the concentrated load is permissible for post-tensioned slabs, and it can be taken into account by using structural design requirements of ACI 318.

10.4.3 *Subgrade friction reduction*—Refer to Section 10.3.2 for the recommended range of friction coefficients. To use the advantages of post-tensioned slabs, the strip lengths or placement sizes should be as large as practical. For this reason, subgrade friction reduction is desirable. For slabs, one or two layers of polyethylene sheeting directly beneath the slab are typically used. Curling can be reduced by perforated sheeting. However, the sheeting should not be perforated when these sheets are also used for a vapor retarder/barrier. A thin layer of compactible, easy-to-trim, granular material can be used under the slab to help reduce subgrade friction and provide other benefits noted in Section 4.6.7.

10.4.4 *Joint requirements*—There are no joints in post-tensioned slabs besides the construction joints surrounding the section being placed. There is no need for sawcut contraction joints. Because lengths between the joints are long and the shortening of the slabs should not be restricted, note the following considerations.

10.4.4.1 *Strip placements*—Normally, every other strip is placed then the adjacent slabs are placed in a second phase. This allows the initial slabs to shorten in their long (and short) direction before placing the adjacent slab. Typically, no dowels or other reinforcement are necessary across this joint unless a load transfer is required, as in areas of joint-crossing traffic. To significantly improve the durability of such a joint, post-tensioning perpendicular to the joint can be provided in this area. The short direction joint on the end of a long strip may open more than is desirable. To offset this effect, a placement strip can remain open as long as practical to allow most of the shortening to occur before the placement strip is closed. Also, stage stressing reduces long-direction shortening as the young concrete is only partially prestressed. The elastic shortening decreases with the age of concrete at load transfer.

10.4.4.2 *Placement of rectangular sections*—All of the criteria from the section for strip placements apply. Typically, post-tensioning is provided in both directions. Provisions should be made for the tendons crossing the joints so as not to restrain the shortening of the adjacent slabs by using compressible sleeves around the tendons.

10.4.5 *Special considerations*—The post-tensioning tendons should be located near the center of the slab. Occasionally, the engineer may determine that the tendons need to be in another position based on other considerations. A higher position of the tendons in the slab reduces the risk of surface cracking. A lower position reduces the potential of cracking under concentrated loads.

The tendons can be supported on support bars, especially when using one-way post-tensioning. Support bars can also serve as crack-width control reinforcement. Use special slab-on-ground chairs of the required height to ensure that the location of the tendons remains unchanged during concrete placement.

CHAPTER 11—FIBER-REINFORCED CONCRETE SLABS-ON-GROUND

11.1—Introduction

Synthetic and steel fibers have been used in concrete slabs-on-ground for over 30 years to improve concrete's plastic (early-age) and hardened properties. Synthetic fibers of nylon and polypropylene have been used to provide control of random plastic-shrinkage cracking. Steel fibers and some macrosynthetic fibers have been used to provide random crack control in concrete after it reaches a hardened state. Combining these products in concrete provides plastic and hardened state benefits. This chapter presents synthetic and steel fiber-reinforced concrete (FRC) material properties and design methods for FRC slabs-on-ground. The performance of FRC slabs-on-ground depends on the mixture proportions and all mixture constituents, including fiber type and quantity. A design example in Appendix 6 shows how to design with FRC.

For more information on steel fibers, refer to publications from ACI Committee 544, Fiber Reinforced Concrete, and industry literature. Information on properties of synthetic fibers, including elastic modulus, tensile strength, and specific gravities, are available in ACI 544.1R and 544.3R.

11.2—Synthetic fiber reinforcement

Synthetic fibers are used to reinforce concrete against plastic shrinkage and drying shrinkage stresses. Fine micro monofilament (diameters less than 0.012 in. [0.30 mm]) or fibrillated synthetic fibers are typically added at low volume addition (LVA) rates of 0.1% or less of concrete volume for plastic shrinkage crack control. Macrosynthetic fibers (diameters equal or greater than 0.012 in. [0.30 mm]) are typically added at high volume addition (HVA) rates of 0.25 to 1% by volume for drying shrinkage crack control.

The length of fibers used for slab-on-ground applications can range between 0.5 to 2.5 in. (13 to 64 mm).

11.2.1 *Properties of synthetic fibers*—Adding microsynthetic fibers to concrete for plastic shrinkage crack control provides a mechanism that increases concrete's tensile strength in the plastic state (Banthia and Yan 2000). This is achieved by the reduction in bleeding and particle settlement while the concrete is in its plastic state. Microsynthetic fibers

provide support for the coarse aggregate and enhance the mixture uniformity. As the modulus of elasticity of concrete increases with hardening, micro fibers provide insufficient restraint to inhibit cracking. Microsynthetic fibers are normally used in the range of 0.05 volume percent to 0.20 volume percent. These volume percentages equate to 0.75 to 3.0 lb/yd^3 (0.44 to 1.8 kg/m^3). Some microsynthetic fibers can increase the fracture toughness of concrete slabs-on-ground in the hardened state. Use macrosynthetic fibers in the range of 0.20 to 1.0% by volume, which equates to 3.0 to 15 lb/yd^3 (1.8 to 8.9 kg/m^3). Macrosynthetic fibers provide increased post-cracking residual strength to slabs-on-ground.

Adding microsynthetic fibers at quantity rates of 1% by volume or less does not significantly alter the ultimate compressive, flexural, and tensile strength of concrete. Adding macrosynthetic fibers at quantities between 0.20 to 1.0% by volume significantly increases the flexural toughness of concrete. ASTM C1399, C1550, and C1609/1609M provide quantitative measures that are useful in evaluating performance of synthetic FRC in the hardened state. The results of these test methods can be used to optimize the proportions of FRC, to determine compliance with construction specifications, and to evaluate existing FRC.

11.2.2 *Design principles*—The design principles for microsynthetic FRC are the same as those used for unreinforced concrete, including using the joint spacing recommendations shown in Fig. 6.6.

Macrosynthetic fibers provide increased post-cracking residual strength to concrete slabs-on-ground. The same design principles for steel FRC in Section 11.3.3 can be used for macrosynthetic FRC.

11.2.3 *Joint details*—Construction and sawcut contraction joint details and spacing for microsynthetic FRC are the same as those used for unreinforced concrete. Macrosynthetic fibers at quantities between 0.2 to 1% by volume increase the post-cracking residual strength of the concrete. This material behavior permits longer sawcut contraction joint spacing; however, load transfer stability may be reduced at sawn contraction joints and should be considered carefully at the longer joint spacing.

11.3—Steel fiber reinforcement

Steel fibers are used to reinforce concrete slabs-on-ground to provide increased strain strength, impact resistance, flexural toughness, fatigue endurance, crack-width control, and tensile strength (ACI 544.4R). Steel fibers are smooth or deformed. Deformations provide mechanical anchorage in the concrete. The matrix bond and anchorage allows steel fibers to bridge cracks that develop in the hardened state and redistribute the accumulated stress caused by applied loads and shrinkage stresses. The length of steel fibers used for slab-on-ground applications can range from 0.75 to 2.50 in. (19 to 64 mm).

11.3.1 *Properties of steel fibers*—Steel fibers for concrete reinforcement are short, discrete lengths of steel having an aspect ratio, or ratio of length-to-diameter, from about 20 to 100, with several types of cross sections. They are sufficiently small to be randomly dispersed in an unhardened concrete mixture using common mixing procedures. ASTM A820/A820M provides classification for five general types of steel fibers, based primarily on the product or process used in their manufacture:

- Type I: Cold-drawn wire;
- Type II: Cut sheet;
- Type III: Melt-extracted;
- Type IV: Mill cut; and
- Type V: Modified cold-drawn wire.

ASTM A820/A820M establishes tolerances for aspect ratio, length, and diameter (or equivalent diameter), minimum tensile strength, and 90-degree bending requirements for steel fibers.

Steel fibers are made of low-carbon, high-carbon, or stainless steel. Carbon steel fibers are either uncoated or galvanized. High-carbon fibers are typically used with concrete mixtures of 8000 psi (55 MPa) compressive strength and higher. Stainless steel fibers may be used when the concrete will be exposed to extremely high temperatures.

Steel fiber bond to the matrix is enhanced by mechanical anchorage, surface area, alloying, surface roughness, or a combination of these. Long-term loading does not adversely influence the mechanical properties of steel FRC (ACI 544.1R).

11.3.2 *Properties of steel FRC*—The properties of FRC in the freshly mixed and hardened state are a consequence of its composite nature. The performance of hardened FRC is related to the fiber aspect ratio, fiber spacing, fiber tensile strength, anchorage characteristics, and volume percentage (Johnston and Skarendahl 1992; Trottier et al. 1997; Balaguru et al. 1992; Clements 1996). Procedures for mixing the steel fibers into the concrete will affect the design parameters. Consult ACI 544.3R to ensure that proper mixing, placing, and finishing guidelines are met.

11.3.2.1 *Random crack control*—Steel fibers are commonly used for random crack control. As in the case with conventional reinforcement, the fibers do not prevent cracking, but serve to hold cracks tight such that the slab performs as intended during its service life. The degree of random crack control by the fibers is directly related to the fiber type and quantity.

11.3.2.2 *Crack width opening*—As with conventional reinforcement, steel fibers at volumes of 0.25 to 0.5% (33 to 66 lb/yd^3 [20 to 39 kg/m^3]) can increase the number of cracks and thus, reduce crack widths. Using steel FRC with conventional, deformed, or smooth continuous reinforcement has synergistic effects, and can be designed to share the applied tensile forces with the continuous reinforcement, thereby adding to crack-width control. The degree of crack-width control is directly related to the fiber type and quantity.

11.3.2.3 *Flexural toughness*—Flexural toughness of steel FRC is determined by testing beams or panels in a laboratory using ASTM C1399, C1550, and C1609/1609M or JSCE SF4. It is generally accepted that the presence of steel fibers in quantities less than 0.5% by volume, as expected in most slabs-on-ground, does not affect the concrete's modulus of rupture. Toughness is a measure of the post-cracking energy-absorbing strength of steel FRC, and is defined as the area under the test beam load-deflection curve.

Use residual strength factors $R_{e,3}$ and average residual strength (ARS), determined using ASTM C1399 and JSCE SF4, in slab-on-ground design. These factors represent an average value of load-carrying strength of the test beam over a deflection interval. The ARS is reported in psi (MPa) and represents a portion of the modulus of rupture. $R_{e,3}$ is reported as a percentage of the modulus of rupture. Refer to ACI 544.2R, ASTM STP 169D, and ACI SP-155 (Stevens et al. [1995]) for more information. This guide uses the residual strength factor $R_{e,3}$ to represent the post-crack characteristics of steel FRC. The degree of flexural toughness is directly related to the mixture proportion and all mixture constituents, including fiber type and quantity.

11.3.2.4 *Impact resistance*—The impact resistance of steel FRC is three to 10 times greater than plain concrete when subjected to explosive charges, dropped weights, and dynamic flexural, tensile, and compressive loads (Williamson 1965; Robins and Calderwood 1978; Suaris and Shah 1981). The degree of impact resistance is directly related to the mixture proportion and all mixture constituents, including fiber type and quantity.

11.3.2.5 *Flexural fatigue resistance*—The flexural fatigue resistance at two million cycles for plain concrete is approximately 50% of the static rupture modulus. This is the basis for the well-known factor of safety of 2.0 shown in the PCA design document (Spears and Panarese 1983). Steel FRC mixtures have shown fatigue strengths of 65 to 90% of the static rupture modulus at two million cycles when nonreversed loading is used (Ramakrishnan and Josifek 1987; Ramakrishnan et al. 1987). The fatigue strength is slightly less when full reversal of loads is used (Batson et al. 1972). The degree of fatigue resistance relates directly to the mixture proportions and all mixture constituents, including fiber type and quantity.

11.3.2.6 *Shear resistance*—Steel FRC can provide higher punching shear strength and anchor bolt pullout strength compared with plain concrete (Concrete Society 2003). Shear strength is directly related to the mixture proportion and all mixture constituents, including fiber type and quantity.

11.3.2.7 *Freezing-and-thawing resistance*—Steel fibers do not inherently increase freezing-and-thawing resistance of concrete. The same mixture proportion principles as those discussed in ACI 201.1R should be followed for steel FRC exposed to freezing and thawing.

11.3.2.8 *Durability in corrosive environments*—Plain carbon steel fibers are protected from corrosion by the alkaline environment of the cementitious matrix and their electrical discontinuity. Laboratory and field testing of intact steel FRC shows that in the long term, the corrosion of steel fiber exposed at the surface is limited to a concrete depth of 0.1 in. (2.5 mm). Laboratory and field testing of cracked steel FRC in an environment containing chlorides indicates that fibers passing across the crack can corrode similarly to conventional reinforcement but without causing spalling (Hoff 1987). Previous studies show that crack widths less than 0.004 in. (0.1 mm) do not allow corrosion of steel fibers passing the crack (Morse and Williamson 1977). Studies that are more recent show that crack widths up to 0.02 in. (0.5 mm) have no adverse effect on corrosion of steel fibers (Lambrechts et al. 2003). When cracks wider than 0.02 in (0.5 mm) are limited in depth, the consequences of this localized corrosion may not be structurally significant.

11.3.3 *Thickness design methods*—Five methods available for determining the thickness of steel FRC slabs-on-ground are described in this section:

1. The PCA, WRI, and COE thickness design methods;
2. Elastic method;
3. Yield line method;
4. Nonlinear finite modeling; and
5. Combined steel FRC and bar reinforcement.

These design methods depend on steel FRC attaining a minimum level of residual strength. In addition, Table 11.1 provides suggested performance levels for various floor loading conditions. These values represent a compilation of performance values obtained from trade literature.

11.3.3.1 *The PCA/WRI/COE method*—The PCA/WRI/COE methods described in Chapter 7 may all be applied to the design of steel FRC slabs-on-ground. With this approach, use steel fiber reinforcement for serviceability design issues such as temperature and shrinkage crack control, enhanced joint stability, and impact and fatigue resistance. Consult fiber manufacturers for specific designs or steel fiber quantities.

11.3.3.2 *Elastic method*—The thickness of slabs-on-ground with steel fibers should be selected to prevent cracking due to external loading as discussed in Chapter 7 with the following modifications: Account for steel fibers by setting the allowable stress equal to the equivalent flexural strength of the composite steel FRC

$$f_b = R_{e,3}/100 \times f_r$$

where f_b is allowable flexural tensile stress, psi (MPa); f_r is the modulus of rupture of concrete, psi (MPa); and $R_{e,3}$ is the residual strength factor determined using JSCE SF4, %.

For example, using a $R_{e,3}$ = 55 and modulus of rupture = 570 psi, the allowable bending stress is

$$F_b = 55/100 \times \text{modulus of rupture} = 0.55 \times 570 \text{ psi} = 314 \text{ psi}$$

Compare this with an unreinforced slab that has an allowable flexural strength of 0.50 × 570 psi = 285 psi.

11.3.3.3 *Yield line method*—Yield line analysis accounts for the redistribution of moments and formation of plastic hinges in the slab. These plastic hinge regions develop at points of maximum moment and cause a shift in the elastic moment diagram. Using plastic hinges permits the use of the full moment strength of the slab and an accurate determination of its ultimate load strength. Because the formation of plastic hinges depends on toughness, the minimum $R_{e,3}$ residual strength should be greater than 30%. The results of tests (Beckett 1995) led to the adoption of yield-line design methods based on the work of Meyerhof (1962) and Lösberg (1961).

The work of Meyerhof (1962) presents three separate cases, differentiated on the basis of load location with respect to the edges of the slab, which may be considered.

Case 1: *Central load on large slab*

$$P_o = 6\left[1 + \frac{2a}{L}\right]M_o$$

For this case, express the value of M_o as

$$M_o = M_n + M_p = \left[1 + \frac{R_{e,3}}{100}\right] \times \frac{f_r \times b \times h^2}{6}$$

Case 2: *Edge load*

$$P_o = 3.5\left[1 + \frac{3a}{L}\right]M_o$$

For this case, express the value of M_o as

$$M_o = M_n + M_p = \left[1 + \frac{R_{e,3}}{100}\right] \times \frac{f_r \times b \times h^2}{6}$$

Case 3: *Corner load*

$$P_o = 2\left[1 + \frac{4a}{L}\right]M_o$$

For this case, express the value of M_o as

$$M_o = M_n = \frac{f_r \times b \times h^2}{6}$$

In the previous formulas:

a = radius of circle with area equal to that of the post base plate, in. (mm);
b = unit width (1 in. [1 mm]);
f_r = concrete modulus of rupture, psi (MPa);
h = slab thickness, in. (mm);
L = radius of relative stiffness, in. (mm); Section 7.2;
M_n = negative moment strength of the slab, tension at top slab surface, in.-lb (N-mm);
M_p = positive moment strength of the slab, tension at bottom slab surface, in.-lb (N-mm);
P_o = ultimate load strength of the slab, lb (N); and
$R_{e,3}$ = residual strength factor determined by JSCE SF4, %.

The term $f_r[1 + R_{e,3}/100]$ is an enhancement factor that accounts for the toughness of steel FRC slabs-on-ground. Apply the same factor of safety as those in Chapter 5.

11.3.3.4 *Nonlinear finite element computer modeling*—Proprietary finite-element modeling techniques can be used to model nonlinear material behavior. Such designs may include linear shrinkage, curling, and applied loads and are typically iterative. Once final stresses are determined, calculate a residual strength factor $R_{e,3}$ to determine the steel fiber quantity.

Table 11.1—Steel fiber concentrations and residual strength factors for slabs-on-ground

Fiber concentration, lb/yd^3 (kg/m^3)	Application (typical residual strength factors)	Anticipated type of traffic
Over 33 (over 20)	Random crack-width control (20 to 40%)	Commercial and light industrial with foot traffic or infrequent lift trucks with pneumatic tires
33 to 50 (20 to 30)	Light dynamic loading (30 to 50%)	Industrial vehicular traffic with pneumatic wheels or moderately soft solid wheels
40 to 60 (24 to 36)	Medium dynamic loading (40 to 60%)	Heavy-duty industrial traffic with hard wheels or heavy wheel loads
60 to 125 (36 to 74)	Severe dynamic loading, extending joint spacing design (60% or higher)	Industrial and heavy-duty industrial traffic

11.3.3.5 *Steel fibers combined with bar reinforcement*—Serviceability requirements often control over strength considerations in fluid-tight slabs-on-ground. ACI 544.4R quantifies the effect of steel fibers in conjunction with bar reinforcement on serviceability. Equations presented estimate the reduction in reinforcing bar stress due to the presence of steel fibers. Such reductions are useful in meeting serviceability requirements presented in ACI 318 and 350.

11.3.4 *Joint details*—Three joint types commonly used are isolation joints, sawcut contraction joints, and construction joints. Isolation and construction joints for SFRC floors should be designed as discussed in Chapter 6.

Steel fibers may offer additional shear load transfer through fiber-enhanced aggregate interlock compared with unreinforced concrete where the joint opening width remains small enough to not impair the bond between concrete and fiber. The performance of fibers at sawn contraction joints depends on slab thickness, contraction joint spacing, joint opening width, sawcut depth, and mixture constituents, including fiber type and quantity.

As mentioned in Chapter 6, sawcut contraction joints are usually located on column lines, and locate intermediate joints at predetermined spacings. In addition to the amount of steel fibers added to the mixture, consider the other factors listed in Section 6.1.3 when selecting sawcut contraction joint spacing. Sawcut contraction joint spacings for steel FRC slabs-on-ground with quantities less than 0.25% by volume (33 lb/yd^3 [20 kg/m^3]) should follow the same guidelines as those for plain concrete or slabs with minimum conventional reinforcement. Joint spacings greater than those shown in Fig. 6.6 require higher quantities of steel fiber reinforcement to ensure proper crack control and shear-load transfer across the sawn joints. When increased sawcut contraction joint spacings are required, consider: blended aggregate gradation optimization (as recommended in Table 3.1 of ACI 544.1R), water-reducing admixtures, adequate curing, a choker run base material, and a slip membrane. For case studies, refer to Shashanni et al. (2000) and Destree (2000).

To sawcut joints in steel FRC slabs, the same tools discussed in Chapter 6 can be used. Chapter 6 also provides the depth of sawcut using a conventional wet saw and early-entry saw.

CHAPTER 12—STRUCTURAL SLABS-ON-GROUND SUPPORTING BUILDING CODE LOADS

12.1—Introduction

There are cases where the slab-on-ground transmits vertical loads or lateral forces from other portions of the structure to the soil. Examples of this are:

- Storage rack columns that support the building roof structure;
- Mezzanine posts supported by the slab-on-ground;
- Load-bearing walls supported by the slab-on-ground;
- Perimeter building walls tied to the slab-on-ground to resist lateral and vertical loads; and
- Pre-engineered metal building columns vertically supported by the slab-on-ground or that use the slab to resist lateral loads.

These structural slabs should be designed in accordance with ACI 318. Also, where the soil is used to form the slab and the slab should resist the loads by spanning to piles, piers, or other foundation elements, these slabs should be designed in accordance with ACI 318.

12.2—Design considerations

Strength and serviceability are the main slab-on-ground design considerations. The strength requirements of ACI 318 should be met; however, the serviceability requirements of ACI 318 may not be sufficient for many types of slab-on-ground installations. Consider the serviceability requirements for the different slab types as discussed in the other chapters.

CHAPTER 13—DESIGN OF SLABS FOR REFRIGERATED FACILITIES

13.1—Introduction

Chapter 13 provides design guidance for concrete slabs in refrigerated buildings. The typical floor in a refrigerated building is a slab on a slip sheet on insulation on a vapor retarder/barrier on either a soil base or a subslab (Fig. 13.1).

The floor slab is considered a slab-on-ground. The slip sheet, typically a polyethylene film (6 mil [0.15 mm] minimum thickness), is a bond break between the slab and the insulation. The insulation may be in single or multiple layers, depending on the thermal requirements. For a room at a temperature above 32°F (0°C), insulation is typically not required. Insulation is typically extruded polystyrene board, rigid polyurethane board, or cellular glass board insulation. The vapor retarder/barrier is under the insulation and is usually a polyethylene film (10 mil [0.25 mm] minimum thickness), 45 mil (1.14 mm) EPDM membrane (EPDM is an acronym for Ethylene-Propylene-Diene-Monomer, which is a type of synthetic rubber), or bituminous materials in the form of liquid-applied coatings or composite sheets. For refrigerated buildings, install vapor retarders/barriers on the warm side of the insulation. Under the vapor retarder/barrier, there is a soil base or a subslab. Many times, a subslab is installed for ease of insulated floor-system construction, or to encase a grid of heating pipes or conduits. Refrigerated buildings with operating temperatures below freezing require an under-floor heating system to prevent the ground from freezing and heaving. The insulated floor system can be installed over a structural slab supported by deep foundations such as piles.

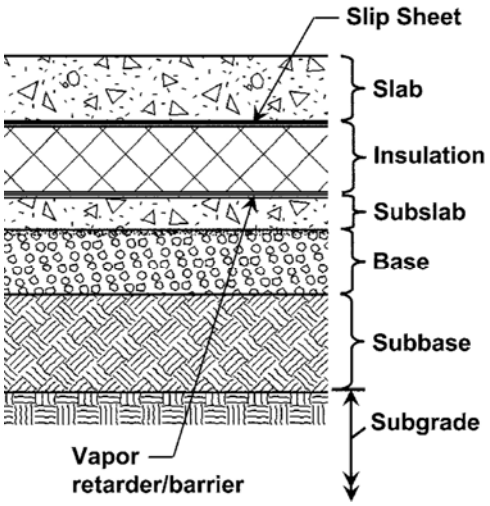


Fig. 13.1—Typical slab-on-ground construction for refrigerated building.

13.2—Design and specification considerations

Design a floor slab installed over insulation as a slab-on-ground. The slab type and method of design can be any type described in other chapters. Slab thickness and reinforcement design should follow the methods and guidelines as described in this guide. The following paragraphs describe the differences and special considerations for slabs in refrigerated buildings.

13.2.1 *Insulation modulus*—For slab-on-ground design, the strength of the soil support system directly below the slab is considered. In the case of a floor in a refrigerated building, consider the strength of the insulation in a similar manner. Some design methods use the modulus of subgrade reaction to account for soil properties. Insulation has a similar modulus.

The insulation's modulus of subgrade reaction should be determined using the plate-bearing test described in Chapter 4 (ASTM D1196). The value of k is normally defined as the pressure to cause a 30 in. (760 mm) diameter plate to deflect 0.05 in. (1.3 mm). The COE, however, determines k for the deformation obtained under a 10 psi (0.07 MPa) pressure. Do not use data provided by the manufacturer using ASTM D1621 with the design methods in Chapter 13.

13.2.2 *Compressive creep*—Compressive loading on insulation causes deformation in the insulation. Insulation deformation increases when the load continues to be applied. In addition to the instantaneous deformation described by the insulation modulus, there is a gradual, permanent deformation of the insulation known as compressive creep.

Long-term creep should be limited to 2% of the thickness over a 20-year period by limiting live loads to 1/5 the

compressive strength and dead loads to 1/3 the compressive strength of the insulation. Manufacturer guidelines may vary.

13.2.3 *Reinforcement*—Slabs in refrigerated facilities do not require special considerations for reinforcement because of room temperature. The design of reinforcement should follow methods described elsewhere in this guide. When reinforcement is to be used, such as deformed bars, post-tensioning cables or welded wire, reinforcing supports with runners or plates should be used so as to not penetrate the insulation or vapor retarder/barrier.

13.2.4 *Joints*—Locations of slab joints in refrigerated buildings follow the same guidelines as slabs in nonrefrigerated buildings. Load-transfer devices, such as dowel bars, should be used in joints. Keyed joints and sawcut joints using aggregate interlock for load transfer are inadequate in a refrigerated building. The temperature shrinkage in the slabs causes the joints to open wider and makes those joints ineffective in load transfer. Joints should be filled after refrigerated rooms reach operating temperature. This allows the slab to contract and stabilize due to temperature reduction. The colder the room or the greater the temperature reduction, the more the slab will contract. The slab takes longer to stabilize at the operating temperature than the room air; consequently, it is best to wait as long as possible to fill the joints. Armoring the construction joints, for example, with embedding steel bars in the joint edges is a viable option to reduce joint maintenance. This is particularly applicable in rooms operating at freezing temperatures, where maintenance occurs less often because of cold temperatures and the limited availability of products that work at these temperatures. Refer to Chapter 6 for more information on floor joints.

13.2.5 *Curing*—Proper curing is very important for slabs in refrigerated areas. Because a vapor retarder/barrier is directly beneath the slab, and therefore all water leaving the slab passes through the upper surface, there can be a higher incidence of curling.

13.2.6 *Underslab tolerance*—Elevation tolerance for the soil base or for the subslab, when used, is important because the rigid board floor insulation mirrors the surface upon which it bears. The surface of the insulation typically cannot be adjusted. When using a subslab, the surface should have a smooth flat finish or a light steel-trowel finish. Avoid irregularities in the soil base or subslab because the insulation may bear on high points and extend higher than the adjacent insulation board. A high point may create a rocking situation, which means that the insulation is not fully supported. The elevation of the base or subslab should conform to a tolerance of +0/–1/2 in. (+0/–13 mm). This is more stringent than ACI 117, and should be specified in the project documents.

13.2.7 *Forming*—Typically, slab-on-ground forms are staked into the soil below. For a refrigerated building, this is not acceptable because of the floor insulation and the vapor retarder/barrier. Construct forms for this floor type with a form mounted vertically on a horizontal base, such as plywood. Place this L-shaped form on the insulation and use sandbags to hold it in place.

13.3—Temperature drawdown

Gradual temperature reduction for a refrigerated room controls cracking caused by differential thermal contraction and allows drying to remove excess moisture from the slab after curing. A typical drawdown schedule:

	Temperature	Time
1. Ambient to 35°F (2°C)	10°F (6°C)	Per day (24 hours)
2. Hold at 35°C (2°C)	—	2 to 5 days
3. 35°C (2°C) to final	10°F (6°C)	Per day

CHAPTER 14—REDUCING EFFECTS OF SLAB SHRINKAGE AND CURLING

14.1—Introduction

Chapter 14 provides design methods to reduce the effect of drying shrinkage and curling or warping in slabs-on-ground. The material is largely based on Ytterberg's three articles (Ytterberg 1987). For further analysis and discussion, refer to Walker and Holland (1999). For information on concrete shrinkage, refer to ACI 209R and Ytterberg (1987).

To be workable enough for placement, virtually all concrete is produced with approximately twice as much water as needed to hydrate the cement. Because water primarily leaves the concrete from the upper surface of slabs-on-ground, a moisture gradient is created between the top and bottom of the slab. Moist subgrades and low humidity at the top surface magnify such moisture gradients. Shrinkage occurs in all three dimensions, but moisture evaporation from the top surface causes the upper half of the slab to shrink more than the lower half. Curling is caused primarily by the difference in drying shrinkage between the top and bottom surfaces of the slab. The effects of shrinkage and curling due to loss of moisture from the slab surface are often overlooked by designers, although curling stresses can be quite high. Analysis by Walker and Holland (1999) indicates curling stresses can easily range from 200 to 450 psi (1.4 to 3.1 MPa). Slab design should consider the two most important items that affect curling: moisture content of the subgrade, and shrinkage potential of concrete. Often, compressive strength and slump testing of the slab concrete are used to evaluate the shrinkage potential and neither of the two tests are good indicators of future drying shrinkage and curling. Higher compressive strength, however, generally correlates with greater shrinkage and curl.

Significant curling of slabs-on-ground has become more prevalent in the past 30 years. This is partly due to the emergence of more finely ground cements, smaller maximum-size coarse aggregates, and gap-graded aggregates, all of which increase the water demand in concrete. The problem might also be compounded by increases in the specified compressive strength, resulting in a higher modulus of elasticity. Such strength increases are usually achieved by increasing the total volume of water and cement per cubic yard, even though the *w*/*cm* should be reduced. This results in a higher modulus of elasticity, increased brittleness, and decreased curl relaxation due to creep. For slabs-on-ground, the commonly specified 28-day compressive strength of 3000 psi (21 MPa) in years past has been increased to as much as 5000 psi (34 MPa) to permit reduc-

tion of calculated slab thickness. Under certain conditions, however, higher compressive strength can actually decrease load-carrying strength due to increased curling stress (Walker and Holland 1999). Higher strengths can improve durability, but designers should consider alternatives to high 28-day compressive strength when attempting to reduce slab thickness.

Shrinkage and curling problems have become more common because slabs are being constructed on less desirable, higher-moisture-content subgrades as the availability of cost-effective industrial land has decreased. Slab thickness has not increased, nor have well-designed vapor retarder/barrier and aggregate blotter systems been specified to offset this increase in subgrade moisture. Furthermore, the modulus of subgrade reaction of subgrades and subbases is seldom determined by the plate test, as suggested in Chapter 4. Excess subgrade moisture adds to the moisture gradient already present in slabs-on-ground and increases slab curling.

Appropriate design and specification provisions can reduce shrinkage cracking and curling. Such provisions should include:

- Relative shrinkage of various concrete mixtures;
- Type and location of reinforcement;
- Subgrade friction;
- Concrete planarity;
- Permeability;
- Slab thickness;
- Shrinkage restraints;
- Location of sawcut contraction joints; and
- Properly designed vapor retarder/barrier and aggregate blotter systems.

14.2—Drying and thermal shrinkage

Typical portland-cement concrete, along with shrinkage-compensating concrete, shrinks approximately 0.04 to 0.08% due to drying (PCA 2002). For slabs-on-ground, the shrinkage restraint from the subgrade varies with the coefficient of friction and planarity of the surface of the subbase. Thermal movement is caused by a change in slab temperature from the time of initial placement. Consider this for any floor when casting concrete at a significantly different temperature than the normal operating temperature. Thermal contraction can be calculated by using the concrete's coefficient of thermal expansion of 5.5×10^{-6} per °F (9.9×10^{-6} per °C). For example, lowering the temperature of a floor slab from 70 to 0°F (21 to –18°C) can shorten a 100 ft (30 m) slab by 0.46 in. (12 mm), assuming no subgrade restraint.

14.3—Curling and warping

"Curling" and "warping" are used interchangeably in this guide, in conformance with "ACI Concrete Terminology" (American Concrete Institute 2009).

Curling occurs at slab edges because of differential shrinkage. The upper part of the slab-on-ground almost always has the greater shrinkage because the top surface is commonly free to dry faster, and the upper portion has higher unit water content at the time of final set. Higher relative humidity in the ambient air at the upper surface reduces the severity of curling even though the concrete may contain high shrinkage materials. Curvature occurs over the entire slab panel, but according to Walker and Holland (1999), the edges can actually lift off the subgrade for a distance of 2 to 7 ft (0.6 to 2.1 m) from all slab edges, from cracks wider than hairline, and from joints with or without positive load-transfer devices. Refer to Fig. 14.1 and 14.2 for the curling effect in exaggerated fashion.

Curling of concrete slabs at joints and cracks is directly related to drying shrinkage. Reducing drying shrinkage reduces curling. There can also be curling due to the temperature differential between the top and bottom slab's surfaces. For interior slabs, temperature differential is a small amount.

Table 14.1—Cumulative effect on adverse factors on concrete shrinkage (Tremper and Spellman 1963)

Effect of departing from use of best materials and workmanship	Equivalent increase in shrinkage, %	Cumulative effect
Temperatures of concrete at discharge allowed to reach 80°F (27°C), whereas with reasonable precautions, temperatures of 60°F (16°C) could have been maintained	8	1.00 × 1.08 = 1.08
Used 6 to 7 in. (150 to 180 mm) slump where 3 to 4 in. (76 to 100 mm) could have been used	10	1.08 × 1.10 = 1.19
Excessive haul in transit mixture, too long a waiting period at job site, or too many revolutions at mixing speed	10	1.19 × 1.10 = 1.31
Use of 3/4 in. (19 mm) maximum-size aggregate under conditions where 1-1/2 in. (38 mm) could have been used	25	1.31 × 1.25 = 1.64
Use of cement having relatively high shrinkage characteristics	25	1.64 × 1.25 = 2.05
Excessive "dirt" in aggregate due to insufficient washing or contamination during handling	25	2.05 × 1.25 = 2.56
Use of aggregates of poor inherent quality with respect to shrinkage	50	2.56 × 1.50 = 3.84
Use of an admixture that produces high shrinkage	30	3.84 × 1.30 = 5.00
Total increase	Summation 183%	Cumulative 400%

14.4—Factors that affect shrinkage and curling

Drying shrinkage and curling can be reduced by reducing the total water content (not necessarily the *w*/*c*) in the concrete. Tremper and Spellman (1963) found that drying shrinkage is the product, not merely the summation, of eight individual factors that control the water requirements of concrete. Table 14.1 shows the cumulative effect of these eight factors, resulting in about a fourfold increase in drying shrinkage rather than a twofold increase when arithmetically added. The influence of four of these factors on the water demand of concrete is discussed.

14.4.1 *Effect of maximum-size coarse aggregate*—Table 14.1 shows that using 3/4 in. (19 mm) maximum-size aggregate under conditions where 1-1/2 in. (38 mm) maximum-size aggregate could have been used will increase concrete shrinkage approximately 25% because of the greater water demand of the 3/4 in. (19 mm) maximum-size aggregate. In addition to the water demand effect, aggregate generally acts

to control (reduce) shrinkage by restraining cement paste shrinkage. To minimize shrinkage of the cement paste, the concrete should contain the maximum practical amount of incompressible, clean, well-graded aggregate.

In actual practice, the dry-rodded volume of coarse aggregate is approximately 50 to 66% of the concrete volume when 1/2 in. (13 mm) maximum-size aggregate is used, but can be as high as 75% when 1-1/2 in. (38 mm) maximum-size aggregate is used (ACI 211.1). Using large-size coarse aggregates may be more expensive than smaller-size aggregates, but it can save on cement content. Designers should specify the nominal top-size coarse aggregate if a larger size is desired.

14.4.2 *Influence of cement*—Table 14.1 shows the possibility of a 25% increase in concrete shrinkage if a cement with relatively high shrinkage characteristics is used. Twenty-eight-day design strengths are usually most economically achieved using Type I or Type III cement because these cements usually give higher early strength than Type II cement, with all else being equal. Type I and III cements can cause higher concrete shrinkage than Type II cement because of their physical and chemical differences. Specifying minimum concrete compressive strength without regard to cement type or relative cement mortar shrinkage can contribute to slab shrinkage and curling. Designers should therefore specify the type of cement used for slabs-on-ground. Because the quality of cement may vary from each brand and within a brand, comparative cement mortar shrinkage tests (ASTM C157/C157M) conducted before starting the project are desirable.

14.4.3 *Influence of slump*—A 6 to 7 in. (150 to 180 mm) slump concrete will have only 10% more shrinkage than a 3 to 4 in. (76 to 100 mm) slump concrete (Table 14.1). This increase in shrinkage potential would be anticipated when the slump increase was due to additional water or admixtures that increase the shrinkage. When shrinkage is to be kept to a minimum, then slump control is only a small factor in the equation. Slump by itself is not an adequate indicator of expected shrinkage. The specification and control of many factors are necessary for satisfactory slab shrinkage in the hardened state.

14.4.4 *Influence of water-reducing admixtures*—Water reductions of approximately 7% may be achieved with ASTM C494/C494M Type A water-reducing admixtures, but their effect on shrinkage and curling is minimal. Chloride-based admixtures of this type definitely increase shrinkage of the concrete (Tremper and Spellman 1963).

Some water-reducing admixtures increase concrete shrinkage, even at reduced mixing water contents, as shown by numerous investigators (Ytterberg 1987). A reduction in mixing water content, permitted by the use of water-reducing admixtures, will not always decrease shrinkage proportionally. Introducing a water-reducing or high-range water-reducing admixture (Types A and F, ASTM C494/C494M) or decreasing slump from 5 to 3 in. (130 to 76 mm) does not significantly change shrinkage in many cases. Note that ASTM C494/C494M allows concrete manufactured with admixtures to have up to 35% greater shrinkage than the same concrete without the admixture.

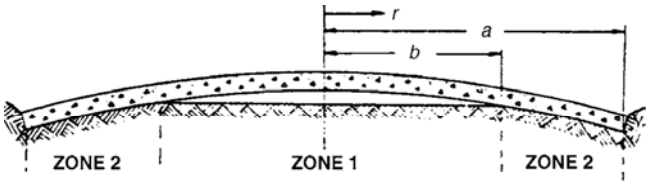


Fig. 14.1—Highway slab edges curl downward at edges during the day when the sun warms the top of the slab.

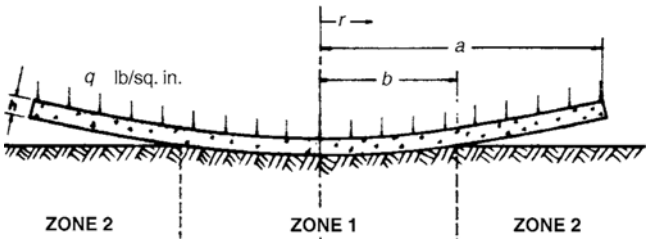


Fig. 14.2—Slabs indoors curl upward because of the moisture differential between the top and bottom of slabs.

14.4.5 *Shrinkage-reducing admixtures*—Since their introduction in Japan in 1983 (Sato et al. 1983; Tomita et al. 1986) and in the U.S. in the mid 1990s, the use of shrinkage-reducing admixtures (SRAs) has grown. Hundreds of structures incorporate the technology. According to the Capillary Tension Theory (Tomita 1992), the main causes of drying shrinkage is the surface tension developed in the small pores of the cement paste of concrete. When these pores lose moisture through evaporation, a meniscus forms at the air/water interface. Surface tension in the meniscus pulls the pore walls inward; the concrete responds to these internal forces by shrinkage. This shrinkage mechanism occurs only in pores within a fixed range of sizes. The amount of cement-paste shrinkage caused by surface tension depends primarily on the *w*/*c*. Cement type and fineness, and other ingredients such as admixtures and supplementary cementitious materials that affect pore size distribution in the hardened paste, also affect cement-paste shrinkage. The SRA reduces shrinkage by reducing the surface tension of water in the pores between 2.5 and 50 nanometers in diameter. The SRA disperses in the concrete during mixing, after the concrete hardens the admixture remains in the pore system, where it continues to reduce the surface tension effects that contribute to drying shrinkage (Ai and Young 1997). Shrinkage-reducing admixtures have also been shown to reduce curling and the time required to reach specified moisture vapor emission rates (MVER) (Berke and Li 2004). Projects have been completed with extended joint spacing exceeding 60 ft (18 m) with minimal or no cracking (Bae 2004).

14.5—Compressive strength and shrinkage

In the competitive concrete supply market, increases of 1-day, 3-day, and 28-day compressive strengths are often obtained at the expense of increased shrinkage. More cement and more water per cubic yard (cubic meter) (not necessarily a higher *w*/*c*), a higher shrinkage cement, or a water-

reducing admixture that increases shrinkage, are the typical means used for increasing compressive strength.

The main reason for controlling compressive strength and, therefore, modulus of rupture, is to ensure that the unreinforced slab thickness is of sufficient strength to transmit loads to the subgrade. Consider a 60-day, 90-day, or longer strength test, rather than a 28-day strength test, for designing slab thickness. This assumes that the design loads would not be applied during the first 60 or 90 days.

Instead of using a higher design strength to minimize slab thickness, consider adding conventional reinforcement or post-tensioning. For another approach to minimize slab thickness, quadrupling the slab contact area of equivalently stiff base plates beneath post loads (8 x 8 in. [200 x 200 mm] plates instead of 4 x 4 in. [100 x 100 mm] plates) could decrease the required slab thickness by more than 1 in. (25 mm).

14.6—Compressive strength and abrasion resistance

Abrasion resistance is a function of the *w*/*cm* (and compressive strength) at the top surface of the concrete. The cylinders or cubes tested to measure compressive strength are not a measure of surface abrasion resistance.

The slab's upper part has a higher water content than the lower portion because of the gravity effect on concrete material before set takes place. Compressive strengths are always higher in the lower half of floors, and shrinkage is always higher in the upper half (Pawlowski et al. 1975).

The finishing process, primarily the type and quality of the troweling operation, significantly affects the abrasion resistance at the top surface. When concrete cannot resist the expected abrasive action, special metallic or mineral aggregate shake-on hardeners may be used to improve surface abrasion of floors placed in a single lift. A separate floor topping with low *w*/*cm* can also improve abrasion resistance.

14.7—Removing restraints to shrinkage

It is important to isolate the slab from anything that could restrain contraction or expansion. Frequently, designers use the floor slab as an anchor by detailing reinforcing bars from foundation walls, exterior walls, and pit walls to the floor slab. When there is no other way to anchor these walls except by tying them into the floor, then unreinforced slabs should be jointed no more than 10 to 15 ft (3.0 to 4.5 m) from the wall so that the remainder of the floor is free to shrink and move. Also, the first joint from the anchored wall tends to open much wider because all of the shrinkage from the restrained panel occurs at this joint.

In most industrial slabs-on-ground, it is desirable to reduce joints to a minimum because joints are a maintenance problem when exposed to high-frequency lift truck traffic. Therefore, it may be better to anchor walls to a separate slab under the finished floor slab with at least 6 in. (150 mm) of base material between the two slabs to minimize joints in the finished floor slab. This is not often done, but is recommended where reduction of cracks and joints is important.

Besides isolating the slab-on-ground from walls, columns, and column footings, the slab should be isolated from guard posts (bollards) that penetrate the floor and are anchored into the ground below. The slab should be isolated from any other slab shrinkage restraints, such as drains. A compressible material should be specified full slab depth around all restraints to allow the slab to shrink and move relative to the fixed items. Electrical conduit and storm drain lines should be buried in the subgrade so they do not reduce the slab thickness or restrain drying shrinkage.

Restraint parallel to joints due to conventional round dowels can be eliminated by using square, tapered-plate, or rectangular-plate dowel systems with formed voids or compressible isolation material on the bar or plate sides. This allows transverse and longitudinal movement while transferring vertical load. Refer to Chapter 6 for more information.

14.8—Base and vapor retarders/barriers

A permeable base, with a smooth, low-friction surface reduces shrinkage cracking because it allows the slab to shrink with minimal restraint. A relatively dry base also allows some water from the bottom of the slab to leave by acting as a blotter before the concrete sets. A vapor retarder/barrier should be used where required to control moisture transmission through the floor system. A vapor retarder/barrier in direct contact with the slab may increase slab curling. A vapor retarder/barrier, aggregate blotter system design, or both, should be evaluated as set forth in Chapter 4. An option discussed in Chapter 4 is to cover the retarder/barrier with at least 4 in. (100 mm) of reasonably dry, trimmable, compactible granular material to provide a permeable, absorptive base directly under the slab. Using 4 in. (150 mm) or more of this material over the retarder/barrier will improve constructibility and minimize damage.

A slab cast on an impervious base can experience serious shrinkage cracking and curling (Nicholson 1981). When the base is kept moist by groundwater or when the slab is placed on a wet base, this increases the moisture gradient in the slab and increases curl. When the aggregate material over the vapor retarder/barrier is not sufficiently dry at concrete placement, it will not act as a blotter and can exacerbate curling and moisture problems. In spite of the inherent problems with placing concrete directly on the vapor retarder/barrier, it is better to do so when there is a chance that the aggregate blotter will not be relatively dry. Chapter 4 provides more information. When using crushed stone as a base material, the upper surface of the crushed stone should be choked off with fine aggregate material to provide a smooth surface that allows the slab-on-ground to shrink with minimum restraint.

When polyethylene is required only to serve as a slip sheet to reduce friction between slab and base, and the base is to remain dry, then the polyethylene can be installed without a stone cover. Holes should be drilled in the sheet, while the sheet is still folded or on a roll, at approximately 12 in. (300 mm) centers to allow water to leave the slab bottom before the concrete sets.

Figure 14.3 shows the variation in values for base friction. In post-tensioned slabs over two sheets of polyethylene, the friction factor may be taken as 0.3 (Timms 1964). For post-

tensioned slabs over 100 ft (30 m), 0.5 might be used to account for variations in base elevation over longer distances.

14.9—Distributed reinforcement to reduce curling and number of joints

The upper half of a floor slab has the greater shrinkage. Reinforcement should be in the upper half of the slab so that the steel restrains concrete shrinkage. A reinforcement ratio of 1.0% (Abdul-Wahab and Jaffar 1983) could be justified in the direction perpendicular to the slab edge to minimize slab curling deflection. One and one-half to 2 in. (38 to 51 mm) of concrete cover is preferred. Reinforcement in the lower part of the slab may actually increase upward slab curling for slabs under roof and not subject to surface heating by the sun. To avoid being pushed down by the feet of construction workers, reinforcing wire or bars should preferably be spaced a minimum of 14 in. (360 mm) in each direction. The bar reinforcement should be located and supported based on the most stringent recommendations of CRSI 10MSP and CRSI 10PLACE. The wire reinforcement should be supported based on the most stringent recommendations of WRI-TF-702 (WRI 2008a) and WRI-WWR-500 (WRI 2008b). Neuber (2006) provides additional information for supporting light welded wire reinforcement.

For unreinforced slabs, the joint spacing recommendations of Fig. 6.6 have generally produced acceptable results. A closer joint spacing is more likely to accommodate the higher shrinkage concrete mixtures often encountered (Fig. 6.6). When greater joint spacings are desirable to reduce maintenance, consider a continuously reinforced, a post-tensioned, or a shrinkage-compensating concrete slab as a means to reduce the number of joints in slabs-on-ground. Joint locations should be detailed on the slab construction drawings.

14.10—Thickened edges to reduce curling

Curling is greatest at corners of slabs and corner curling reduces as slab thickness increases (Child and Kapernick 1958). Corner curling vertical deflections of 0.05 and 0.11 in. (1.3 and 2.8 mm) were measured for 8 and 6 in. (200 and 150 mm) thick slabs, respectively, after 15 days of surface drying.

Thickening free edges reduces the slab stress due to edge loads. Edge curling may also be reduced by thickening slab edges. The thickened edge adds weight and reduces the surface area exposed to drying relative to the volume of concrete, both of which reduce upward curling. Typically, thickened free slab edges should only be used in situations such as between the slab and equipment foundations or other free edges at isolation joints where positive load-transfer devices, such as dowels, should not be used. The free slab edges at these locations should be thickened 50% with a gradual 1-in-10 slope. Provided that the subgrade is smooth with a low coefficient of friction, as described in Section 14.8, then thickened edges should not be a significant linear shrinkage restraint; however, curling stresses are increased somewhat.

14.11—Relation between curing and curling

Because curling and drying shrinkage are both a function of potentially free water in the concrete at the time of concrete set, curing methods that retain water in the concrete delay shrinkage and curling of enclosed slabs-on-ground.

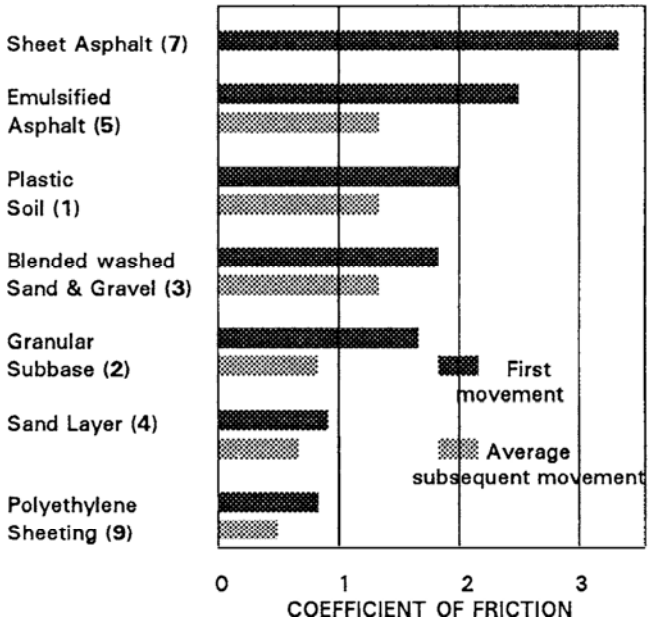


Fig. 14.3—Variation in values of coefficient of friction for 5 in. (125 mm) slabs on different bases (based on Design and Construction of Post-Tensioned Slabs-on-Ground *[PTI 2004]).*

Curing did not decrease curling in a study of concrete pavements where test slabs cured for 7 days under wet burlap, then ponded until load tests for the flat, uncurled, slabs were completed (Child and Kapernick 1958). After completing load tests on the flat slabs, usually within 5 to 6 weeks, the water was removed and the slabs were permitted to dry from the top. The load tests were repeated on the curled slabs. Adding water to the surface reduced the curl, especially hot water, but after removing the water, the slabs curled again to the same vertical deflection as before the application of the water.

Water curing can saturate the base and subgrade, creating a reservoir of water that may eventually transmit through the slab (Chapter 4).

Because all curing methods are used for a limited time, curing methods will not be as effective as when the surface is exposed to long-term, high ambient relative humidity for improving surface durability. Extended curing only delays curling; it does not reduce curling.

14.12—Warping stresses in relation to joint spacing

Warping stress increases as the slab length increases only up to a certain slab length (Kelley 1939; Leonards and Harr 1959; Walker and Holland 1999). The slab lengths at which these warping stresses reach a maximum are referred to as critical slab lengths, and are measured diagonally, corner to corner. Critical lengths, in feet (meters), are shown below for slabs 4 to 10 in. (100 to 250 mm) thick and temperature gradients T of 20, 30, and 40°F (11, 17, and 22°C). A modulus of subgrade reaction k of 100 lb/in.^{3} (27 kPa/mm)

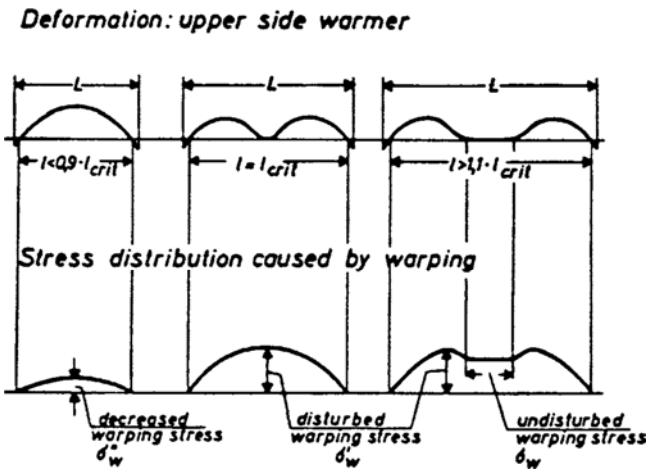


Fig. 14.4—Effect of slab length on warping and warping stress in an exposed highway slab (Eisenmann 1971).

and a modulus of elasticity E of 3×10^6 psi (21,000 MPa) were used in determining these values:

Slab thickness	T = 20°F (11°C)	T = 30°F (17°C)	T = 40°F (22°C)
4 in. (100 mm)	21 ft (6.4 m)	—	—
6 in. (150 mm)	26 ft (7.9 m)	27 ft (8.2 m)	—
8 in. (200 mm)	—	34 ft (10.4 m)	35 ft (10.7 m)
10 in. (250 mm)	—	38 ft (11.6 m)	40 ft (12.2 m)

Computer studies indicate that these lengths increase mostly with slab thickness and temperature gradient, and increase only slightly with changes in modulus of elasticity and modulus of subgrade reaction. Figure 14.4 shows deformation and warping stress curves for three highway slabs with lengths less than, equal to, and greater than the critical slab length. Warping stress does not increase as slab length increases beyond the critical length because vertical deformation does not increase.

There will be a marked loss of effectiveness of aggregate interlock at sawcut contraction joints when the joints are too far apart (Spears and Panarese 1983). Positive load transfer using dowels or plates should be provided where joints are expected to open more than 0.025 to 0.035 in. (0.6 to 0.9 mm) for slabs subjected to wheel traffic. Refer to Section 6.2 for additional information. Slabs may be more economical when sawcut contraction joint spacing is increased beyond lengths noted previously by using distributed reinforcement designed for crack-width control, but not less than 0.50% of the cross-sectional area. Floor and lift truck maintenance cost may be lower with the least number and length of joints, as long as curling is not sufficient to cause cracking or joint spalling. Increased joint spacings larger than the critical slab length will not increase warping stresses.

14.13—Warping stresses and deformation

Using the concept of a subgrade reaction modulus, Westergaard (1927) provided equations for warping stress and edge deflections caused by temperature gradients in slabs-on-ground. Although his research does not refer to moisture gradients, it is equally applicable to either temperature or moisture gradients across the thickness of a slab-on-ground. The only shortcoming is the incorrect assumption that slabs-on-ground are fully supported by the subgrade when they are warped from temperature gradients.

When slabs-on-ground warp from temperature or moisture gradients, they are not fully supported by the subgrade, and unsupported edges have higher stresses than when they are supported. Consider these factors as described in Walker and Holland (1999).

Bradbury (1938) extended Westergaard's work with a working stress formula called the Westergaard-Bradbury formula. In 1939, Kelley (1939) used the Westergaard-Bradbury formula to calculate the warping stresses shown in Fig. 14.5 for 6 and 9 in. (150 and 230 mm) slabs-on-ground. Note that Kelley calculated a maximum stress of approximately 390 psi (2.7 MPa) for a 9 in. (230 mm) slab with a length of 24 ft (7.3 m).

In 1959, Leonards and Harr (1959) calculated the warping stresses shown in Fig. 14.6, presented herein for general understanding. The upper center set of curves in Fig. 14.6 shows a maximum warping stress of approximately 560 psi (3.9 MPa) for almost the same assumptions made by Kelley when he computed a stress of 390 psi (2.7 MPa). The only significant difference is that Kelley used a 27°F (15°C) change in temperature across the slab whereas Leonards and Harr used a 30°F (17°C) temperature difference across the slab thickness. Adjusting for this gradient difference by multiplying by the ratio, 30/27, Kelley's stress would be 433 psi (3.0 MPa) instead of 390 psi (2.7 MPa). Leonards and Harr's 560 psi (3.9 MPa) stress, however, is still 29% greater than the stress Kelley calculated due to their better assumptions. Walker and Holland (1999) had similar results to Leonards and Harr (1959).

Leonards and Harr (1959) calculated warping stress with a computer model that permitted the slab to lift off the subgrade when the uplift force was greater than the gravity force. Figure 14.7 shows their vertical deflection curves for the same six cases of slabs shown in Fig. 14.6. The upward slab edge lift and downward slab center deflection shown in Fig. 14.7 is the usual case for slabs inside buildings. Temperature gradient is small for slabs inside a building, but the moisture gradient can be equivalent to about a 5°F per in. (2.8°C per 25 mm) of slab thickness temperature gradient for such slabs under roof. Leonards and Harr assumed a 30°F (17°C) gradient across all the slabs, shown in Fig. 14.6 and 14.7, no matter what the thickness. They also assumed a cold top and a hot slab bottom, which is not the usual temperature gradient, but it is a usual equivalent moisture gradient for slabs inside buildings with a very moist bottom and very dry top.

Ytterberg's 1987 paper documents the conflict between the Westergaard assumption of a fully supported slab-on-ground and the reality of either unsupported slab edges or supported slab centers. Because the three commonly used slab thickness design methods (PCA, WRI, and COE) all are based on Westergaard's work and the assumption that slabs are always fully supported by the subgrade, they give erroneous results for slab thickness where the slab is not in contact with the subgrade. This is called the cantilever effect. The thickness

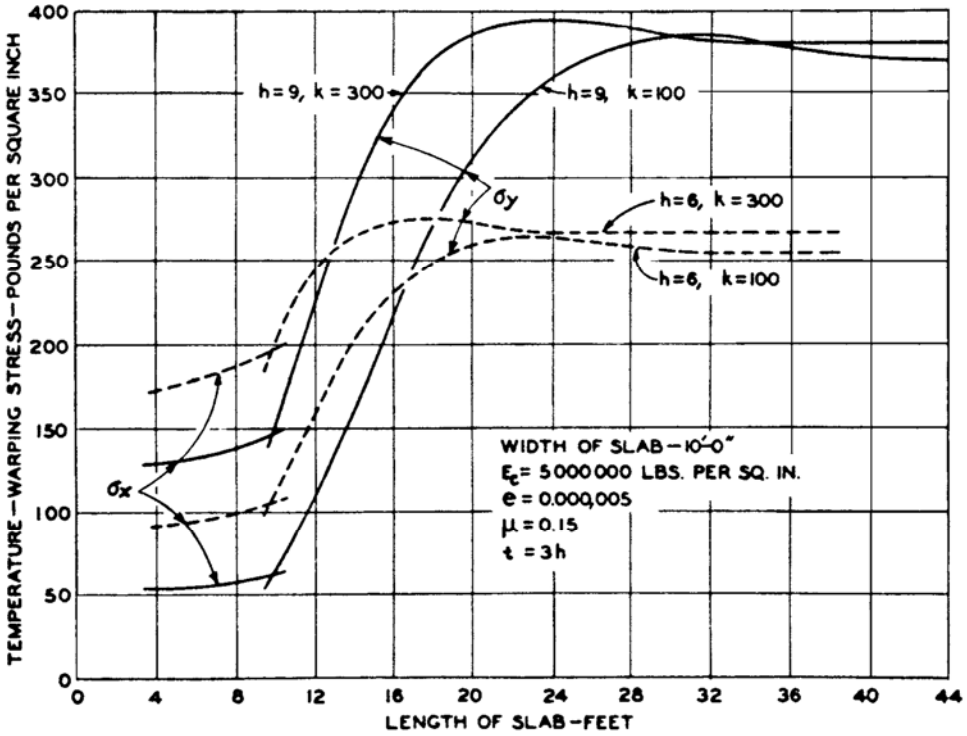


Fig. 14.5—Increases in slab length, beyond a certain amount, do not increase warping stress in the slab interior (Kelley 1939). (Note: 1 psi = 0.0069 MPa; 1 ft = 0.3048 ft.)

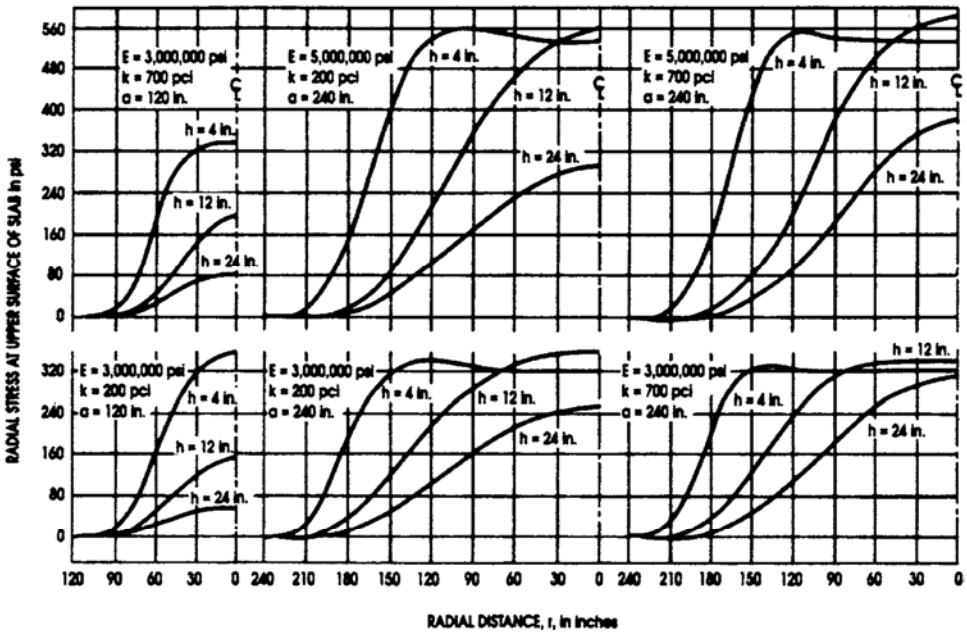


Fig. 14.6—Representative radial stresses for an effective temperature difference of 30°F between top and bottom (Leonards and Harr 1959). (Note: 1 psi = 0.0069 MPa; 1 in. = 25.4 mm; 1°F = 0.56°C.)

design of the outer 3 to 5 ft (0.9 to 1.5 m) of slab panels on ground might be based on a cantilever design when warping is anticipated.

Another anomaly is that the three slab thickness design methods permit thinner slabs as the modulus of subgrade reaction increases. A higher subgrade reaction modulus actually increases the length of unsupported curled slab edges because the center of the slab is less able to sink into the subgrade. Thus, curling stress increases as the subgrade stiffens, and resultant load-carrying strength decreases for edge loadings; however, higher k with loadings away from the edges allows thinner slabs. Designers should consider these factors.

Highway slabs-on-ground should be designed for a 3°F (1.7°C) per in. (25 mm) daytime positive gradient (down-

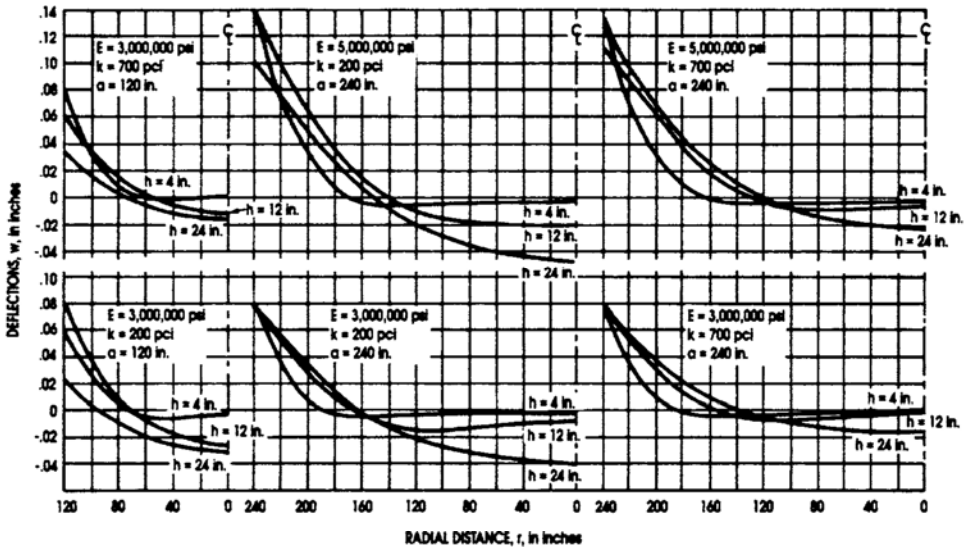


Fig. 14.7—Representative curling deflection curves for 20 and 40 ft slabs with an effective temperature difference of 30°F between top and bottom (Leonards and Harr 1959). (Note: 1 psi = 0.0069 MPa; 1 in. = 25.4 mm; 1°F[ΔT] = 0.56°C; 1 lb/ft^3 = 0.2714 MN/m^3.)

ward curl) and a 1°F (0.6°C) per in. (25 mm) nighttime negative gradient (upward curl) (ACI Committee 325 1956). Enclosed slabs-on-ground should be designed for a negative gradient (upward curl) of 3 to 6°F per in. (1.7 to 3.4°C per 25 mm) (Leonards and Harr 1959).

The Westergaard-Bradbury formula (Yoder and Witczak 1975) concluded that warping stress in slabs is proportional to the concrete's modulus of elasticity, and partially proportional to the modulus of elasticity of aggregates. To reduce slab warping, low-modulus aggregates, such as limestone or sandstone, are preferable to higher-modulus aggregates, such as granite and especially traprock; however, when no hardener or topping is used, many low-modulus aggregates will not be as durable for some applications.

14.14—Effect of eliminating sawcut contraction joints with post-tensioning or shrinkage-compensating concrete

Post-tensioned and shrinkage-compensated slabs do not have intermediate sawcut contraction joints. The total amount of drying shrinkage movement is forced to occur at the placement perimeter when concrete is placed in large areas without intermediate sawcut contraction joints. The construction joints surrounding 10,000 to 12,000 ft^2 (930 to 1100 m^2) of post-tensioned or shrinkage-compensated concrete slabs will commonly open much more than construction joints for the same areas of conventional jointed portland-cement concrete slabs. This is because the intermediate sawcut contraction joints within the latter slabs take up most of the shrinkage.

Where vehicle traffic crosses construction joints in post-tensioned or shrinkage-compensated slabs-on-ground, the joints should be doweled or provide a sufficient amount of post-tensioning force across the joint to transfer the shear. The top edges of the construction joints should be protected with back-to-back steel bars, epoxy-armored edges, or by other equally durable material.

14.15—Summary and conclusions

Designers of enclosed slabs-on-ground can reduce shrinkage cracking and shrinkage curling by considering the features that affect these phenomena and addressing them:

14.15.1 *Subgrade conditions*

- Before and during slab installation, check for smoothness, dryness, and permeability of the base and subgrade. Measure and record the base and subgrade moisture content immediately before concrete placement; and
- Do not use a vapor retarder/barrier unless required to control moisture transmission through the slab. When used, consider using an aggregate blotter over the vapor retarder/barrier.

14.15.2 *Design details*

- Where random cracking is acceptable, specify distributed reinforcement in the upper half of the slab or use the appropriate fiber concentrations to minimize or eliminate sawcut contraction joints. Shrinkage reinforcement is not needed in the bottom half of slabs-on-ground;
- When reinforcement is used, select practical spacings and diameters of wires and bars, considering at least 14 in. (350 mm) spacing and 3/8 in. (9 mm) diameter;
- Consider square, tapered-plate or rectangular-plate dowel systems that eliminate restraint longitudinally and transversely while transferring vertical load;
- Eliminate as many slab restraints as possible, isolating those that remain;
- Specify largest practical size of base plate for rack posts. Include the base plate size in the slab thickness design process, with a base plate thickness adequate enough to distribute post load over area of plate; and
- Consider shrinkage-compensating concrete or post-tensioning as design options.

14.15.3 *Control of concrete mixture*

- Specify workable concrete with the largest practical maximum size of coarse aggregate and with aggregate gap-grading minimized;
- Consider specifying 60- or 90-day concrete design strength to permit using concrete with lower shrinkage than is obtained with the same compressive strength at 28 days. Use lowest compressive strength and corresponding minimum cement content feasible and mineral or metallic hardener or topping when surface durability is a concern;
- Before slab installation, consider shrinkage testing of various cements (mortars), aggregate gradations, and concrete mixtures;
- Specify the cement type and brand;
- Consider a daily check of aggregate gradation to ensure uniform water demand and shrinkage of concrete; and
- Consider plant inspection to perform the aforementioned testing and monitor batching for uniformity, referring to ACI 311.5 for guidance.

CHAPTER 15—REFERENCES

15.1—Referenced standards and reports

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

American Association of State Highway & Transportation Officials

GDPS-4-M	Guide for the Design of Pavement Structures
T 307	Determining the Resilient Modulus of Soils and Aggregate Materials

American Concrete Institute

117	Specifications for Tolerances for Concrete Construction and Materials and Commentary
201.1R	Guide for Conducting a Visual Inspection of Concrete in Service
209R	Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures
211.1	Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
223	Standard Practice for the Use of Shrinkage-Compensating Concrete
224R	Control of Cracking in Concrete Structures
302.1R	Guide for Concrete Floor and Slab Construction
302.2R	Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials
311.5	Guide for Concrete Plant Inspection and Testing of Ready-Mixed Concrete
318	Building Code Requirements for Structural Concrete and Commentary
330R	Guide for Design and Construction of Concrete Parking Lots
336.2R	Suggested Analysis and Design Procedures for Combined Footings and Mats
350	Code Requirements for Environmental Engineering Concrete Structures and Commentary
504R	Guide to Sealing Joints in Concrete Structures (withdrawn)
544.1R	Report on Fiber Reinforced Concrete
544.2R	Measurement of Properties of Fiber Reinforced Concrete
544.3R	Guide for Specifying, Proportioning, and Production of Fiber-Reinforced Concrete
544.4R	Design Considerations for Steel Fiber Reinforced Concrete

ASTM International

A36/A36M	Standard Specification for Carbon Structural Steel
A497/A497M	Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete Reinforcement
A615/A615M	Standard Specification for Deformed and Plain Carbon Steel Bars for Concrete Reinforcement
A706/A706M	Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A820/A820M	Standard Specification for Steel Fibers for Fiber-Reinforced Concrete
A996/A996M	Standard Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement
C150/C150M	Standard Specification for Portland Cement
C157/C157M	Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete
C494/C494M	Standard Specification for Chemical Admixtures for Concrete
C595/C595M	Standard Specification for Blended Hydraulic Cements
C845	Standard Specification for Expansive Hydraulic Cement
C878/C878M	Standard Test Method for Restrained Expansion of Shrinkage-Compensating Concrete
C1399	Standard Test Method for Obtaining Average Residual-Strength of Fiber-Reinforced Concrete
C1550	Standard Test Method for Flexural Toughness of Fiber Reinforced Concrete (Using Centrally Loaded Round Panel)
C1609/C1609M	Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading)
D422	Standard Test Method for Particle-Size Analysis of Soils
D566	Standard Test Method for Dropping Point of Lubricating Grease
D698	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using

	Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))
D854	Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer
D1196	Standard Test Method for Nonrepetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements
D1241	Standard Specification for Materials for Soil-Aggregate Subbase, Base, and Surface Courses
D1556	Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method
D1557	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3))
D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
D1621	Standard Test Method for Compressive Properties of Rigid Cellular Plastics
D1883	Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils
D2167	Standard Test Method for Density and Unit Weight of Soil in Place by the Rubber Balloon Method
D2216	Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
D2240	Standard Test Method for Rubber Property—Durometer Hardness
D2487	Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
D2488	Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
D2922	Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)
D2937	Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method
D3017	Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)
D3575	Standard Test Methods for Flexible Cellular Materials Made from Olefin Polymers
D4253	Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table
D4254	Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density
D4263	Standard Test Method for Indicating Moisture in Concrete by the Plastic Sheet Method
D4318	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
D4829	Standard Test Method for Expansion Index of Soils
E1155	Standard Test Method for Determining F_F Floor Flatness and F_L Floor Levelness Numbers
F1869	Standard Test Method for Measuring Moisture Vapor Emission Rate of Concrete Subfloor Using Anhydrous Calcium Chloride
F2170	Standard Test Method for Determining Relative Humidity in Concrete Floor Slabs Using in situ Probes
STP-169D	Significance of Tests and Properties of Concrete and Concrete-Making Materials

Concrete Reinforcing Steel Institute (CRSI)

10MSP	Manual of Standard Practice
10PLACE	Placing Reinforcing Bars

Japan Society of Civil Engineers (JSCE)

JSCE-SF4	Standard for Flexural Strength and Flexural Toughness

The above publications may be obtained from the following organizations:

American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
www.concrete.org

ASTM International
100 Barr Harbor Drive
West Conshohocken, PA 19428
www.astm.org

Concrete Reinforcing Steel Institute
933 North Plum Grove Road
Schaumburg, IL 60173
www.crsi.org

Japan Society of Civil Engineers
Yotsuya 1-chome, Shinjuku-ku
Tokyo, Japan 160-0004
www.jsce-int.org

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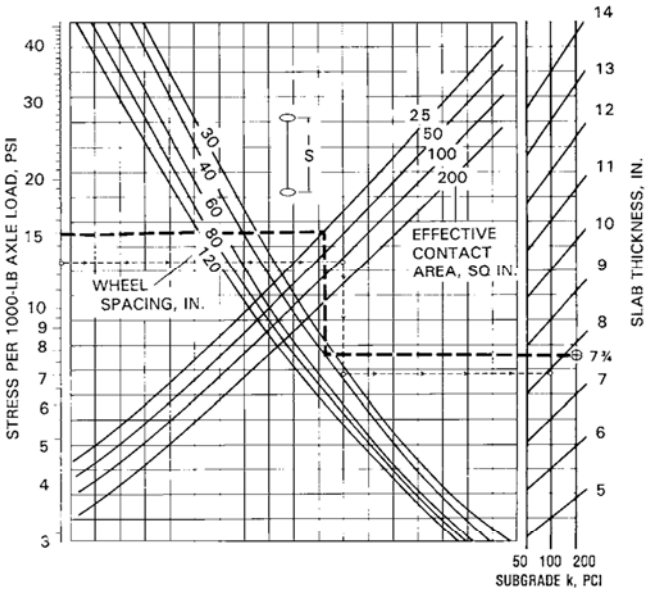


Fig. A1.1—The PCA design chart for axles with single wheels.

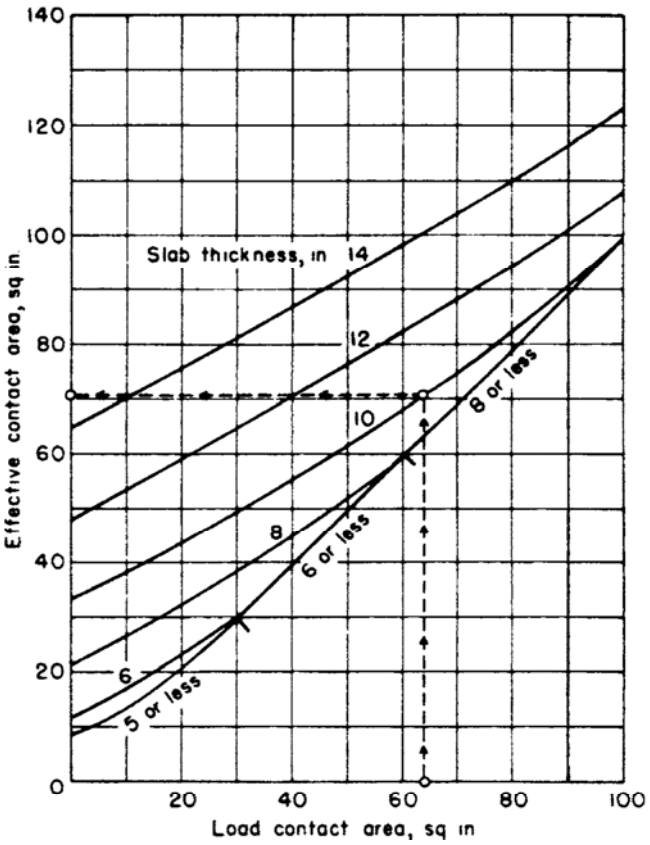


Fig. A1.2—Relationship between load contact area and effective load contact area.

APPENDIX 1—DESIGN EXAMPLES USING PORTLAND CEMENT ASSOCIATION METHOD

A1.1—Introduction

The following two examples show the determination of thickness for a slab-on-ground using design charts published by PCA in *Concrete Floors on Ground* (2001). Both examples select the thickness based on limiting the tension on the bottom of the slab. The following examples presented are in inch-pound units. A table for converting the examples to SI units, along with an example of the process, is provided at the end of the Appendixes.

A1.2—The PCA thickness design for single-axle load

This procedure selects the thickness of a concrete slab for a single-axle loading with single wheels at each end. Use of the design chart (Fig. A1.1) is illustrated by assuming the following:

Loading: axle load = 22.4 kips
Effective contact area of one wheel = 25 in.^{2}
Wheel spacing = 40 in.
Subgrade modulus k = 200 lb/in.^{3}

Material: concrete
Compressive strength = 4000 psi
Modulus of rupture = 570 psi

Design: Selected safety factor = 1.7
Allowable stress = 335 psi
Stress/1000 lb of axle load = 335/22.4 = 14.96 = 15
Solution: thickness = 7-3/4 in., as determined from Fig. A1.1.

Figures A1.2 and A1.3 are also included for determining the effective load contact area and for the equivalent load factor.

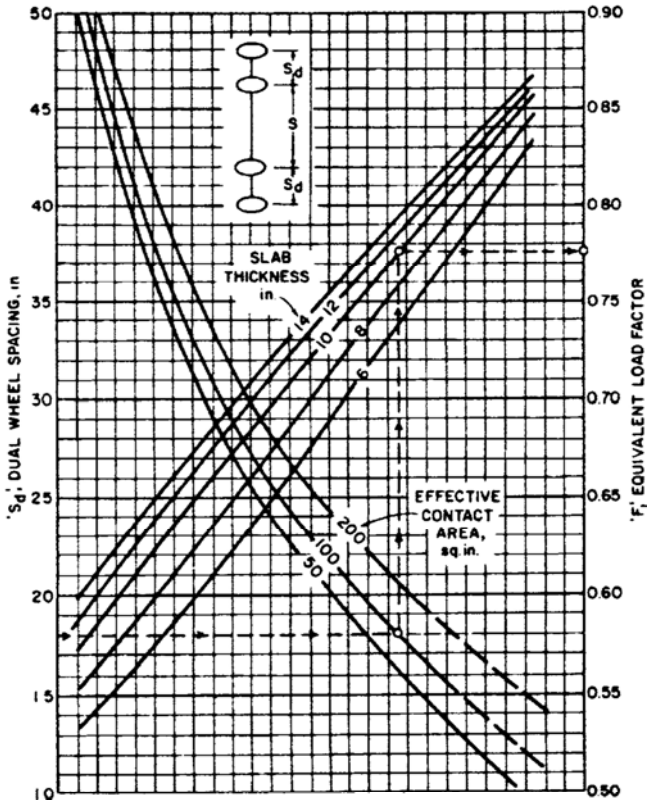


Fig. A1.3—The PCA design chart for axles with dual wheels.

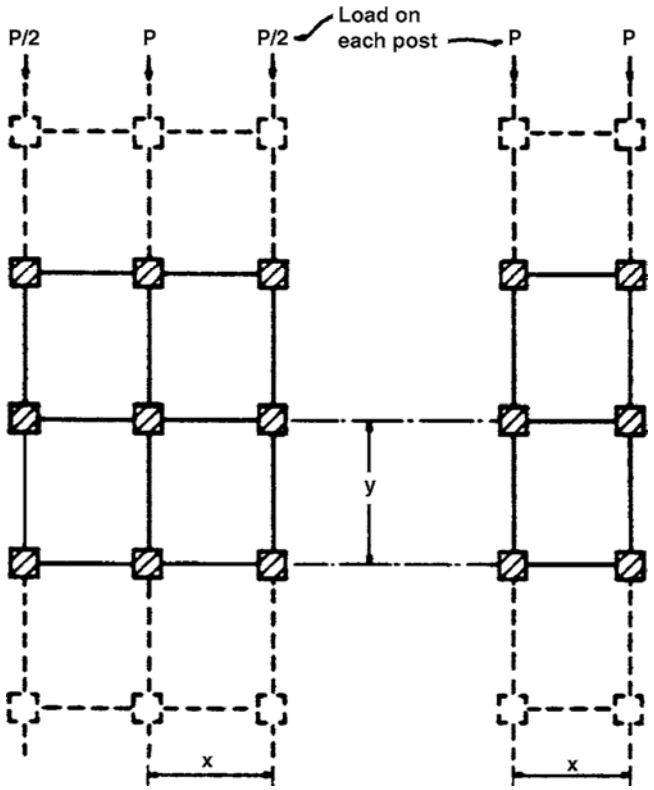


Fig. A1.4—Post configurations and loads.

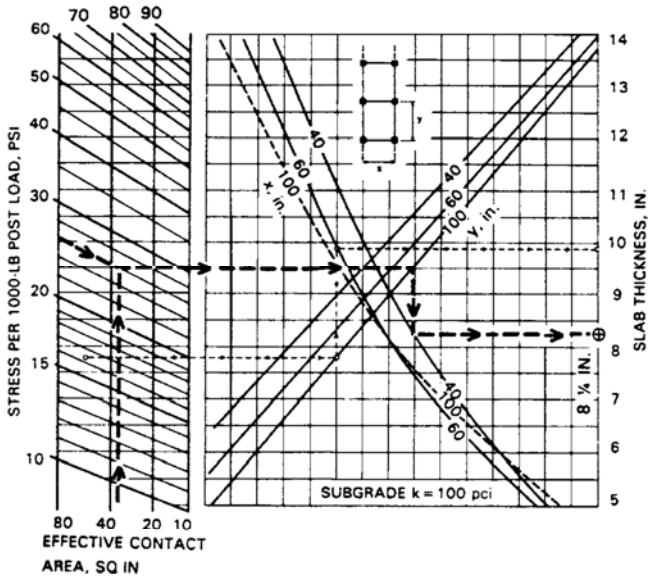


Fig. A1.5—The PCA design chart for post loads where subgrade modulus is 100 pci.

A1.3—The PCA thickness design for slab with post loading

This procedure selects the slab thickness due to loading by a grid of posts shown in Fig. A1.4, such as from rack storage supports. The use of the design chart (Fig. A.1.5) is illustrated assuming the following:

Loading: post load = 15.5 kips
Plate contact area for each post = 36 in.^{2}
Long spacing y = 100 in.

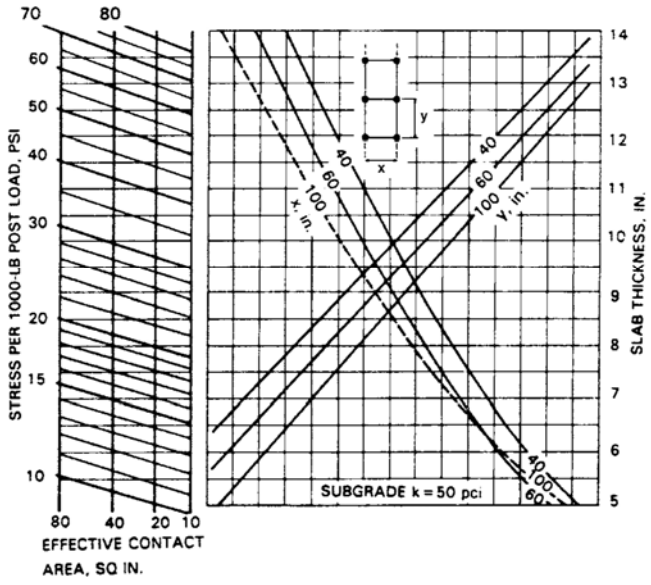


Fig. A1.6—The PCA design chart for post loads where subgrade modulus is 50 pci.

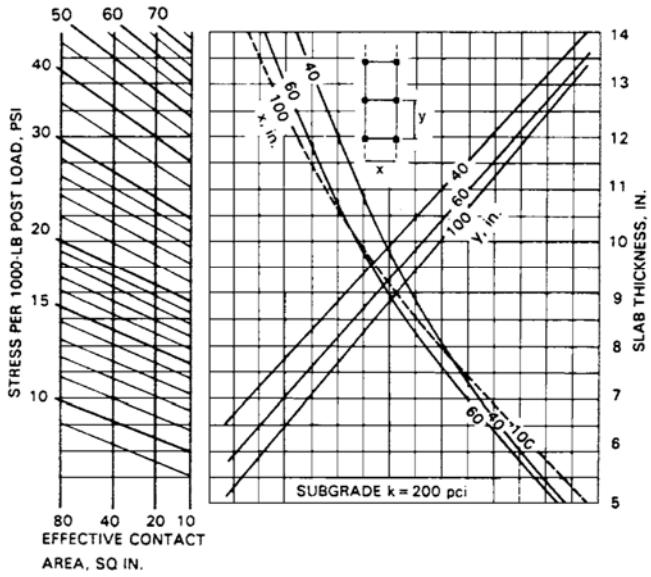


Fig. A1.7—The PCA design chart for post loads where subgrade modulus is 200 pci.

Short spacing x = 40 in.

Material: concrete
Compressive strength = 4000 psi
Modulus of rupture = 570 psi
k = 100 lb/in.^{3}

Design: selected safety factor = 1.4
Allowable stress = 407 psi
Stress per 1000 lb of post load = 407/15.5 = 26.3
—Use 26

Solution: Thickness = 8-1/4 in., as determined from Fig. A1.5.

Figures A1.6 and A1.7 are also included for rack and post loads with subgrade modulus values of k = 50 and 200 lb/in.^{3}, respectively.

Table A1.1—Allowable distributed loads for unjointed aisle with nonuniform loading and variable layout (Packard 1976)

Slab thickness, in.	Subgrade k,* lb/in.^{3}	Allowable load, lb/ft^2†			
		Concrete flexural strength, psi			
		550	600	650	700
5	50	535	585	635	685
	100	760	830	900	965
	200	1075	1175	1270	1370
6	50	585	640	695	750
	100	830	905	980	1055
	200	1175	1280	1390	1495
8	50	680	740	800	865
	100	960	1045	1135	1220
	200	1355	1480	1603	1725
10	50	760	830	895	965
	100	1070	1170	1265	1365
	200	1515	1655	1790	1930
12	50	830	905	980	1055
	100	1175	1280	1390	1495
	200	1660	1810	1965	2115
14	50	895	980	1060	1140
	100	1270	1385	1500	1615
	200	1795	1960	2120	2285

*k of subgrade; disregard increase in k due to subbase.
†For allowable stress equal to 1/2 flexural strength.
Note: Based on aisle and load widths giving maximum stress.

A1.4—Other PCA design information

Tables A1.1 and A1.2 are also included for uniform load applications. Refer to examples of their uses in PCA (2001) and Ringo (1985).

APPENDIX 2—SLAB THICKNESS DESIGN BY THE WIRE REINFORCEMENT INSTITUTE (WRI) METHOD

A2.1—Introduction

The following two examples show the determination of thickness for a slab-on-ground based on an unreinforced slab. Place a nominal quantity of distributed reinforcement in the upper 1/3 of the slab. The primary purpose of this reinforcement is to limit the width of any cracks (when they occur) that may form between the joints. The following examples presented are in inch-pound units. A table for converting the examples to SI units, along with an example of the process, is provided at the end of the Appendixes.

The design charts are for a single axle loading with two single wheels and for the controlling moment in an aisle with uniform loading on either side. Tension on the bottom of the slab controls the first situation. Tension on the top of the slab controls the second situation. Both procedures start with use of a relative stiffness term D/k, and require the initial assumption of the concrete modulus of elasticity E and slab thickness H, as well as selected the allowable tensile unit stress and the appropriate subgrade modulus k.

A2.2—The WRI thickness selection for single-axle wheel load

This procedure selects the concrete slab thickness for a single axle with wheels at each end of the axle, using Fig. A2.1,

Table A1.2—Allowable distribution loads, unjointed aisles, uniform loading, and variable layout; PCA method

Slab thickness, in.	Working stress, psi	Critical aisle width*, in.	Allowable load, lb/ft^2					
			At critical aisle width	At other aisle widths				
				6 ft aisle	8 ft aisle	10 ft aisle	12 ft aisle	14 ft aisle
Subgrade k = 50 lb/in.^{3}†								
5	300	5.6	610	615	670	815	1050	1215
	350	5.6	710	715	785	950	1225	1420
	400	5.6	815	820	895	1085	1400	1620
6	300	6.4	670	675	695	780	945	1175
	350	6.4	785	785	810	910	1100	1370
	400	6.4	895	895	925	1040	1260	1570
8	300	8.0	770	800	770	800	880	1010
	350	8.0	900	935	900	935	1025	1180
	400	8.0	1025	1070	1025	1065	1175	1350
10	300	9.4	845	930	855	850	885	960
	350	9.4	985	1085	1000	990	1035	1120
	400	9.4	1130	1240	1145	1135	1185	1285
12	300	10.8	915	1065	955	915	925	965
	350	10.8	1065	1240	1115	1070	1080	1125
	400	10.8	1220	1420	1270	1220	1230	1290
14	300	12.1	980	1225	1070	1000	980	995
	350	12.1	1145	1430	1245	1170	1145	1160
	400	12.1	1310	1630	1425	1335	1310	1330
Subgrade k = 100 lb/in.^{3}†								
5	300	4.7	865	900	1090	1470	1745	1810
	350	4.7	1010	1050	1270	1715	2035	2115
	400	4.7	1155	1200	1455	1955	2325	2415
6	300	5.4	950	955	1065	1320	1700	1925
	350	5.4	1105	1115	1245	1540	1985	2245
	400	5.4	1265	1275	1420	1760	2270	2565
8	300	6.7	1095	1105	1120	1240	1465	1815
	350	6.7	1280	1285	1305	1445	1705	2120
	400	6.7	1460	1470	1495	1650	1950	2420
10	300	7.9	1215	1265	1215	1270	1395	1610
	350	7.9	1420	1475	1420	1480	1630	1880
	400	7.9	1625	1645	1625	1690	1860	2150
12	300	9.1	1320	1425	1325	1330	1400	1535
	350	9.1	1540	1665	1545	1550	1635	1795
	400	9.1	1755	1900	1770	1770	1865	2050
14	300	10.2	1405	1590	1445	1405	1435	1525
	350	10.2	1640	1855	1685	1640	1675	1775
	400	10.2	1875	2120	1925	1875	1915	2030
Subgrade k = 200 lb/in.^{3}†								
5	300	4.0	1225	1400	1930	2450	2565	2520
	350	4.0	1425	1630	2255	2860	2990	2940
	400	4.0	1630	1865	2575	3270	3420	3360
6	300	4.5	1340	1415	1755	2395	2740	2810
	350	4.5	1565	1650	2050	2800	3200	3275
	400	4.5	1785	1890	2345	3190	3655	3745
8	300	5.6	1550	1550	1695	2045	2635	3070
	350	5.6	1810	1810	1980	2385	3075	3580
	400	5.6	2065	2070	2615	2730	3515	4095
10	300	6.6	1730	1745	1775	1965	2330	2895
	350	6.6	2020	2035	2070	2290	2715	3300
	400	6.6	2310	2325	2365	2620	3105	3860
12	300	7.6	1890	1945	1895	1995	2230	2610
	350	7.6	2205	2270	2210	2330	2600	3045
	400	7.6	2520	2595	2525	2660	2972	3480
14	300	8.6	2025	2150	2030	2065	2210	2480
	350	8.6	2360	2510	2365	2405	2580	2890
	400	8.6	2700	2870	2705	2750	2950	3305

*Critical aisle width equals 2.209 times the radius of relative stiffness.
†k of subgrade; disregard increase in k due to subbase.
Notes: Assumed load width = 300 in.; allowable load varies only slightly for other load widths. Allowable stress = 1/2 flexural strength.

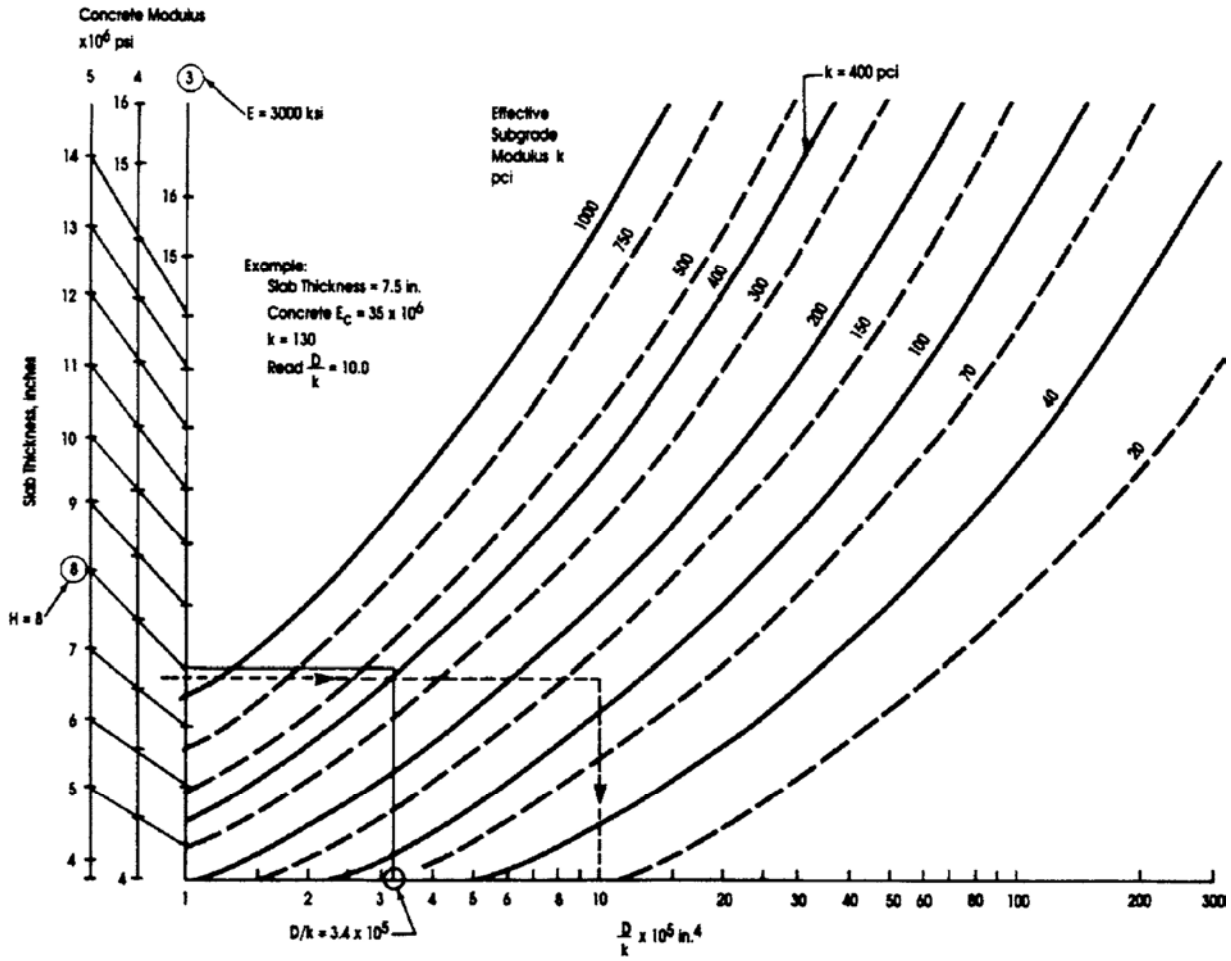


Fig. A2.1—Subgrade and slab stiffness relationship, used with WRI design procedure.

A2.2, and A2.3. The procedure starts with Fig. A2.1, where a concrete modulus of elasticity E, slab thickness H, and modulus of subgrade reaction k are assumed or known. For example, taking

E = 3000 ksi
Thickness = 8 in. (trial value)
Subgrade modulus k = 400 lb/in.^{3}

Figure A2.1 gives the relative stiffness parameter D/k = 3.4 × 10^5 in.^{4}; the procedure then uses Fig. A2.2.

Wheel contact area = 28 in.^{2}
Diameter of equivalent circle = $\sqrt{(28 \times 4)/\pi}$ = 6 in.
Wheel spacing = 45 in.

This gives the basic bending moment of 265 in.-lb/in. of width/kip of wheel load for the wheel load using the larger design chart in Fig. A2.2. The smaller chart in the figure gives the additional moment due to the other wheel as 16 in.-lb/in. of width kip of wheel load.

Moment = 265 + 16 = 281 in.-lb/in./kip
(Note that in.-lb/in. = ft-lb/ft)
Axle load = 14.6 kips
Wheel load = 7.3 kips
Design moment = 281 × 7.3 = 2051 ft-lb/ft

Then, from Fig. A2.3:
Allowable tensile stress = 190 psi
Solution: slab thickness H = 7-7/8 in.

When the design thickness differs substantially from the assumed thickness, repeat the procedure with a new assumption of thickness.

A2.3—The WRI thickness selection for aisle moment due to uniform loading

The procedure for the check of tensile stress in the top of the concrete slab due to this loading uses Fig. A2.1 and A2.4. Figure A2.3 is a part of Fig. A2.4, separated herein for clarity of procedure.

The procedure starts as before with determination of the term D/k = 3.4 × 10^5 in.^{4} It then goes to Fig. A2.4 as follows:

Aisle width = 10 ft = 120 in.
Uniform load = 2500 lb/ft^2 = 2.5 kips/ft^2
Allowable tension = MOR/SF = 190 psi

Find the solution by plotting up from the aisle width to D/k, then to the right-hand plot edge, then down through the uniform load value to the left-hand edge of the next plot, then horizontally to the allowable stress and down to the design thickness.

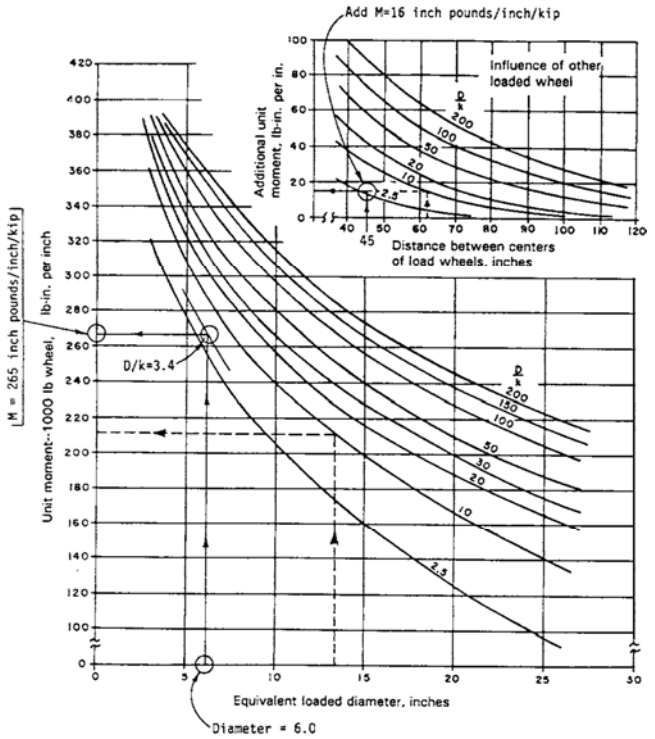


Fig. A2.2—Wheel loading design chart used with WRI procedure.

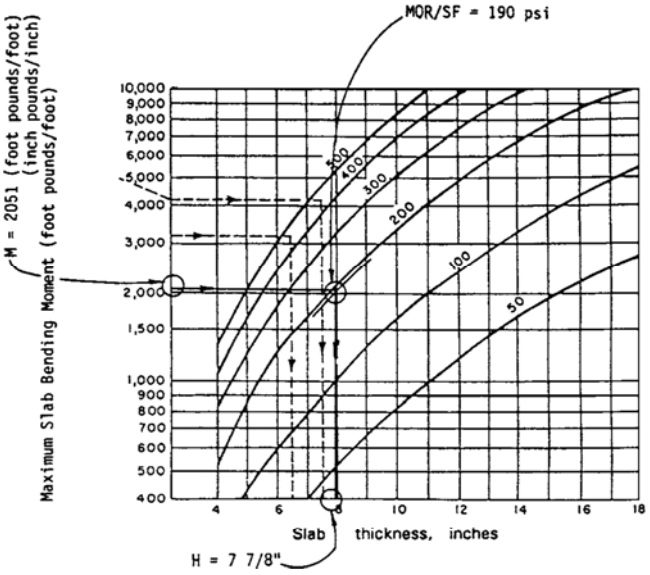


Fig. A2.3—Slab tensile stress charts used with WRI design procedure.

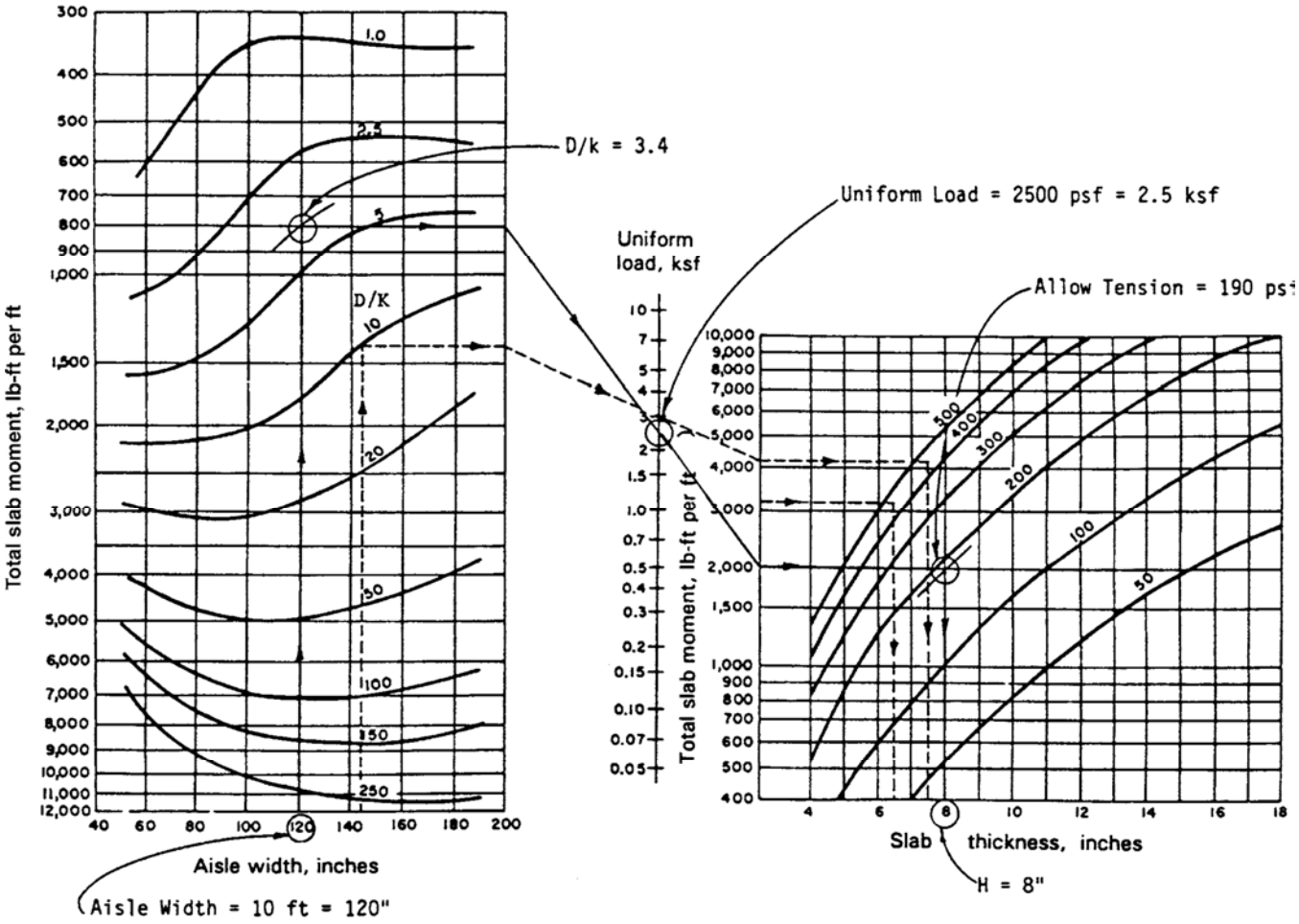


Fig. A2.4—Uniform load design and slab tensile stress charts used with WRI design procedure.

Solution: thickness = 8.0 in.

Again, when the design thickness differs substantially from the assumed value, repeat the process until reasonable agreement is obtained.

APPENDIX 3—DESIGN EXAMPLES USING CORPS OF ENGINEERS' (COE) CHARTS

A3.1—Introduction

The following examples show the determination of thickness for a slab-on-ground using the procedures published by the COE. The procedures appear in publications issued by the Departments of Defense (1977), the Department of the Army (1984, 1987) and the Department of the Air Force (1987). The examples presented are in inch-pound units. A table for converting the examples to SI units, along with an example of the process, is provided at the end of the Appendixes.

The procedure is based on limiting the tension on the bottom of the concrete at an interior joint of the slab. The loading is generalized in design index categories (Table A3.1). The procedure uses an impact factor of 25%, a concrete modulus of elasticity of 4000 ksi, and a factor of safety of approximately 2. The joint transfer coefficient has been taken as 0.75 for this design chart (Fig. A3.1).

The six categories shown in Table A3.1 are commonly used. Figure A3.1 shows 10 categories.

Categories 7 through 10 for exceptionally heavy vehicles are not covered in this guide.

A3.2—Vehicle wheel loading

This example selects the thickness of the concrete slab for a vehicle in design index Category IV (noted as Design Index 4 in Fig. A3.1). The vehicle parameters are needed to select the design index category from Table A3.1. Use of the design chart is illustrated assuming the following:

Loading: DI IV (Table A3.1)
Materials: concrete
Modulus of elasticity E = 4000 ksi
Modulus of rupture = 615 psi (28-day value)
Modulus of subgrade reaction k = 100 lb/in.^{3}
Solution: required thickness = 6 in. is determined from the design chart (Fig. A3.1) by entering with the flexural strength on the left and proceeding along the solid line.

A3.3—Heavy lift truck loading

This example selects the thickness of the concrete slab for a lift truck, assuming the following:

Loading: axle load 25,000 lb
Vehicle passes: 100,000
Concrete flexural strength: 500 psi
Modulus of subgrade reaction k = 300 lb/in.^{3}

Figure A3.2 shows the design curve. Enter at the flexural strength with 500 psi on the left. From there, proceed with the following steps: go across to the intersection with the curve of k = 300; go down to the line representing the axle load; go across to the curve for the number of vehicle passes; and finally, go down to find the final solution for the slab thickness of 5-1/4 in.

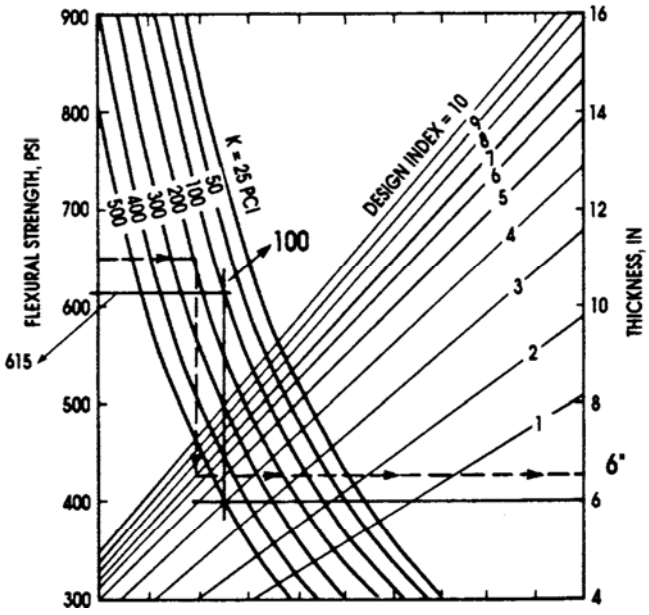


Fig. A3.1—COE curves for determining concrete floor thickness by design index.

Table A3.1—Design index categories used with the COE slab thickness selection method

Category	I	II	III	IV	V	VI
Capacity, lb	4000	6000	10,000	16,000	20,000	52,000
Design axle load, lb	10,000	15,000	25,000	36,000	43,000	120,000
No. of tires	4	4	6	6	6	6
Type of tire	Solid	Solid	Pneumatic	Pneumatic	Pneumatic	Pneumatic
Tire contact area, in.^{2}	27.0	36.1	62.5	100	119	316
Effect contact pressure, psi	125	208	100	90	90	95
Tire width, in.	6	7	8	9	9	16
Wheel spacing, in.	31	33	11.52.11	13.58.13	13.58.13	20.79.20
Aisle width, in.	90	90	132	144	144	192
Spacing between dual wheel tires, in.	—	—	3	4	4	4

APPENDIX 4—SLAB DESIGN USING POST-TENSIONING

This chapter includes:

- Design example: using post-tensioning to minimize cracking; and
- Design example: equivalent tensile stress design.

A4.1—Design example: Using post-tensioning to minimize cracking

Assume post-tensioned (PT) strip 500 x 12 ft.

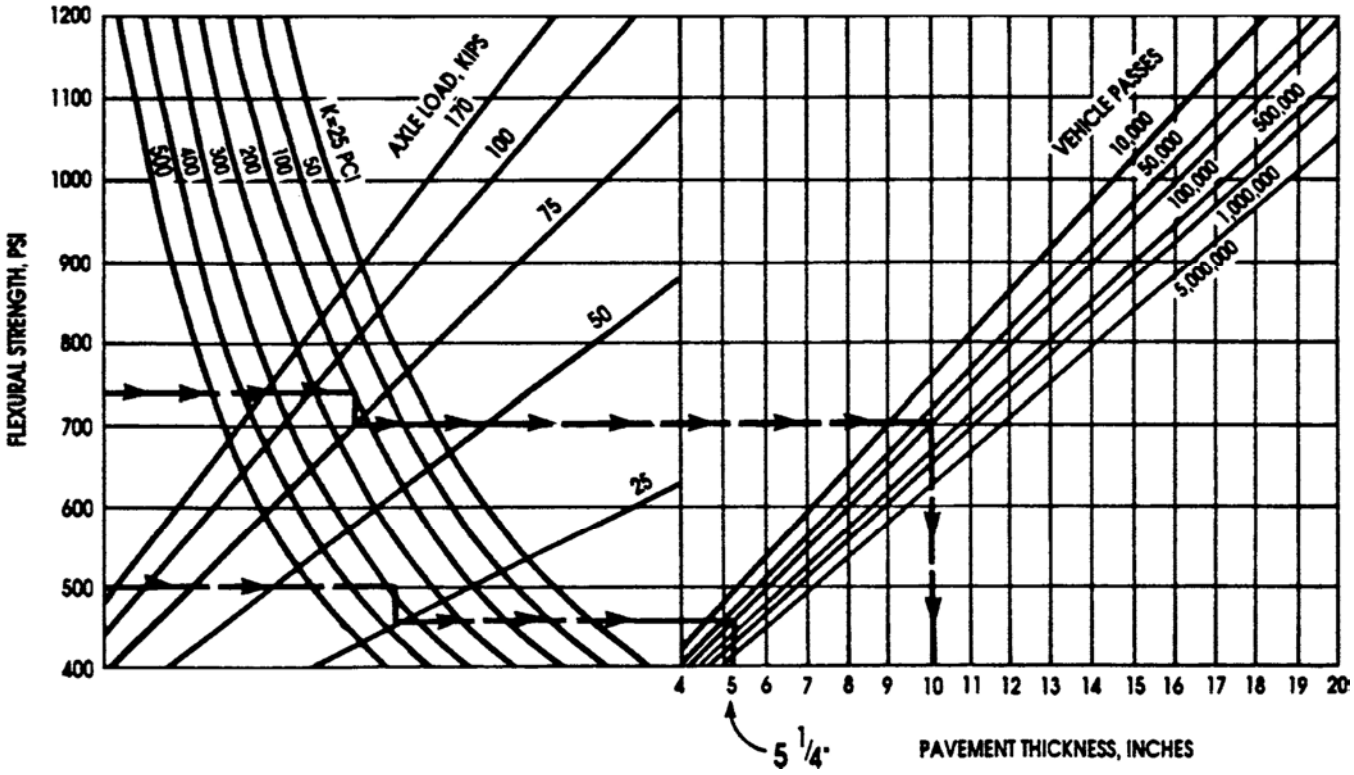


Fig. A3.2—The COE design curves for concrete floor slabs with heavy forklift traffic.

Determine minimum residual (effective) compression after all losses.

Calculate post-tensioning requirement for minimum residual compression (P/A), assume 250 psi:

Assume slab thickness: 6 in.

Calculate P-T requirement to overcome the subgrade friction using Eq. (A4-1).

Assume subgrade friction factor: 0.5.

$$P_r = W_{slab}\frac{L}{2}\mu = \frac{6\text{ in.}}{12\text{ in./ft}} \times 150/\text{ft}^3 \times \frac{500\text{ ft}}{2} \times 0.5 = 9375\text{ lb/ft} \quad \text{(A4-1)}$$

Calculate final effective force in PT tendon (friction and long-term losses).

Assume P_e = 26,000 lb.

Calculate the required spacing of the PT tendons using Eq. (A4-2).

$$S_{ten} = \frac{P_e}{f_p WH + P_r} = \frac{26{,}000\text{ lb}}{250\text{ psi} \times 12\text{ in.} \times 6\text{ in.} + 9375\text{ lb/ft}} \quad \text{(A4-2)}$$

$$= 0.95\text{ ft (11.4 in.)}$$

Use 11 in. to provide more than 250 psi compression. Twelve inch spacing provides a compression of approximately 230 psi, which may be adequate. Use groups of two cables 22 in. on center (or groups of three at 33 in. on center)

The type and magnitude of loading and other serviceability criteria determines the final spacing.

When there is rack loading with post far apart or other concentrated loading spaced sufficiently far apart as to not significantly influence each other, then check with the Westergaard Eq. (7-4)

$$f_b = 0.316\left[\frac{P}{h^2}\log(h^3) - 4\log\left(\sqrt{1.6a^2 + h^2} - 0.675h\right) - \log(k) + 6.48\right]$$

where f_b is the tensile stresses at the bottom of the concrete slab; P is the concentrated load; h is the slab thickness; a is the radius of a equivalent circular load contact area; and k is the modulus of subgrade reaction.

Assume:

P = 15,000 lb;
h = 6 in;
a = 4.5 in. (base plate 8 x 8 in.);
k = 150 lb/in.^{3}; and
f_b = 545 psi.

Cracking of concrete: $7.5 \times \sqrt{f_c'}$ = 474 psi

Post-tensioning to provide necessary precompression of: 545 – 474 = 71 psi

Post-tensioning providing 250 psi is adequate.

In the case of two or more placements post-tensioned together across the joint and creating a continuous slab, use the following:

Case 1: Multiple (12) strips 30 ft wide post-tensioned partially in the 30 ft direction before placing the adjacent strip. Final stress ties all strips together on the end.

When calculating the force to overcome the subgrade friction, consider the total width of all strips to be (12 x 30 = 360 ft).

Case 2: First place a section of 200 ft, stressed partially, and then place and stress the other section of 160 ft.

When calculating the force to overcome the subgrade friction, use the following criteria:

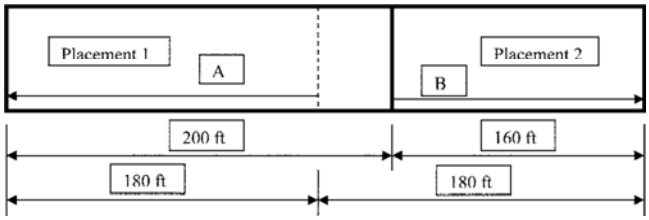
Placement 1: Formula

$$W_{slab} \times \frac{L}{2} \times \mu$$

$$\frac{L}{2} = \frac{360}{2} = 180 \text{ ft} = A$$

Placement 2:

$$\frac{L}{2} = 160 \text{ ft} = B$$



The tendons in Placement 1 have to overcome maximum friction based on 180 ft length at the critical section at the center of the combined length (dashed line).

The tendons in Placement 2 have to overcome maximum friction based on 160 ft length at the critical section at the joint between Placement 1 and 2 (pulling Placement 2 toward Placement 1).

A4.2—Design example: Equivalent tensile stress design

Determine the reduction in slab thickness of a 6 in. thick unreinforced slab when using post-tensioning.

Assume a modulus of rupture of $9\sqrt{f'_c}$ with a factor of safety of 2 was used to design the unreinforced 6 in. thick slab. Then, the allowable tension stress for 4000 psi concrete is

$$\frac{9\sqrt{4000 \text{ psi}}}{2} = 285 \text{ psi}$$

When the P-T force provides an effective residual compression of 150 psi (selected for this example) with the tendons in the center of the slab, then the allowable tensile stress due to the bending moments is 150 psi + 285 psi = 435 psi.

The moment strength of the slab is given by

$$M = f_t S = f_t \frac{bh^2}{6}$$

Equate the moment capacity of the unreinforced slab to the post-tensioned slab

$$285 \text{ psi} \frac{b(6 \text{ in.})^2}{6} = 435 \text{ psi} \frac{bh^2}{6}$$

$$h = \sqrt{\frac{285 \text{ psi}(6 \text{ in.})^2}{435 \text{ psi}}} = 4.85 \text{ in.}$$

Increase the 150 psi residual compressive stress to use a 4 in. thick slab or reduce it to use a 5 in. thick slab.

APPENDIX 5—DESIGN EXAMPLE USING SHRINKAGE-COMPENSATING CONCRETE

A5.1—Introduction

ACI 223 discusses the material in this appendix in greater detail. Slab design using this material is divided into three parts.

Part 1:

Select slab thickness by using Appendixes 1, 2, or 3. This follows the assumption that the slab is being designed to remain essentially uncracked due to external loading.

Follow this by designing the concrete mixture and the reinforcing steel to compensate for subsequent drying shrinkage. Because the net result of initial expansion and later shrinkage is to be essentially zero, do not consider prestress.

Part 2:

Selecting the appropriate amount of reinforcement is a critical part of the design. The reinforcement can be nonprestressed steel, as illustrated in this appendix, or post-tensioning tendons. Place the reinforcement in the top 1/3 to 1/4 of the slab (ACI 223).

Part 3:

Determine the required prism expansion to ensure shrinkage compensation, as this leads concrete mixture design. This is shown in Section A5.2. Expansion of the length of the slab is also determined.

A5.2—Example selecting the optimum amount of reinforcement to maximize the compressive stress in the concrete where the slab thickness, the joint spacing, and prism expansion are known

This example is the typical design case for a slab-on-ground. The following data are known:

Slab thickness = 6.0 in.
Joint spacing = 100 ft
Prism expansion = 0.05% (ASTM C878/C878M)
Coefficient of subgrade friction = 0.30 (for two sheets of polyethylene)

The slab is assumed to dry on the top surface only; therefore, the volume-surface ratio = 6.0 in.

Ignore the small eccentricity due to the reinforcement being located near the top of the slab and the eccentricity due to the subgrade friction.

Determine the optimum amount of reinforcement that produces the maximum tension in the reinforcement which causes the maximum compressive stress in the slab. Designing the reinforcement to produce this maximum

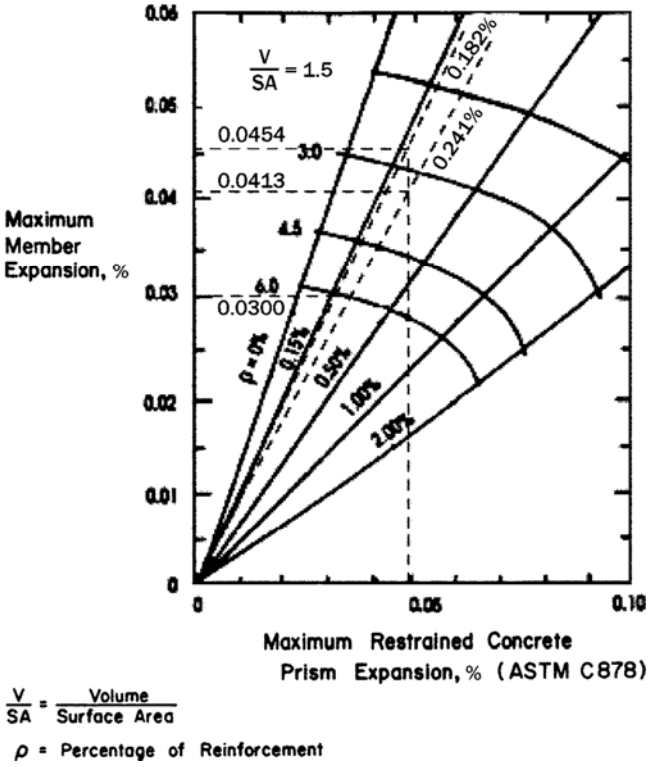


Fig. A5.1—Prediction of member expansion from prism data (ACI Committee 223 1970).

compression force minimizes the tension stress due to the subgrade friction when the slab shrinks. For these given data, the optimum reinforcement is No. 4 at 18 in. (A_s = 0.131 in.^{2}/ft, ρ = 0.182%). Determine this optimum reinforcement by a few iterations of the following procedure. The final iteration is shown below:

Determine the force in the reinforcement without subgrade restraint. For No. 4 at 18 in., ρ = 0.182% and from Fig. A5.1, the slab expansion is ε_{exp} = 0.0454% or 0.000454 in./in. The stress in the reinforcement is

$$\varepsilon_{exp} \times E_s = 0.000454 \text{ in./in.} \times 29{,}000{,}000 \text{ psi} = 13{,}200 \text{ psi}$$

Subgrade friction force is

$$W_{slab} \times \frac{\text{joint spacing}}{2} \times \mu = 75 \text{ psf} \times \frac{100 \text{ ft}}{2} \times 0.30 = 1130 \text{ lb/ft}$$

Because the subgrade friction varies over the length of the slab, use the average force, which is

$$\frac{1130 \text{ lb/ft}}{2} = 565 \text{ lb}$$

The equivalent steel area is

$$\frac{565 \text{ lb/ft}}{13{,}200 \text{ psi}} = 0.0428 \text{ in.}^2\text{/ft}$$

The equivalent reinforcement ratio in percent is

$$\rho_{equ} = \frac{0.131 \text{ in.}^2\text{/ft} + 0.0428 \text{ in.}^2\text{/ft}}{6 \text{ in.} \times 12 \text{ in./ft}} \times 100 = 0.241\%$$

From Fig. A5.1, the slab expansion with subgrade restraint is ε_{exp_equ} = 0.0413% or 0.000413 in./in.

From Fig. A5.1, the slab shrinkage with subgrade restraint is ε_{sh_equ} = 0.0300% or 0.000300 in./in.

The force in the reinforcement after the concrete shrinkage has occurred is

$$A_s(\varepsilon_{exp\_equ} - \varepsilon_{sh\_equ})E_s = 0.131 \text{ in.}^2\text{/ft}(0.000413 \text{ in./in.} - 0.0003 \text{ in./in.}) \times 29{,}000{,}000 \text{ psi} = 429 \text{ lb/ft}$$

This tension force causes the maximum compressive stress in the slab due to the reinforcement and helps reduce the tension stress due to the subgrade restraint.

APPENDIX 6—DESIGN EXAMPLES FOR STEEL FRC SLABS-ON-GROUND USING YIELD LINE METHOD

A6.1—Introduction

These examples show the design of a slab-on-ground containing steel FRC. This design procedure is iterative and involves assumption of a slab thickness, determination of a residual strength factor, and determination of the reasonableness of the residual strength factor. Select an appropriate fiber type and quantity rate to meet the residual strength factor.

a = radius of circle with area equal to that of the base plate, in. (mm)
E = elastic modulus of concrete, psi (MPa)
f_c' = concrete cylinder compressive strength, psi (MPa)
f_r = concrete modulus of rupture, psi (MPa)
h = slab thickness, in. (mm)
k = modulus of subgrade reaction, lb/in.^{3} (N/mm^3)
L = radius of effective stiffness, in. (mm)
Mn = negative bending moment strength of the slab, tension at top slab surface, in.-k (N-mm);
Mp = positive bending moment strength of the slab, tension at bottom slab surface, in.-k (N-mm)
P_{ult} = ultimate load strength of the slab, kip
$R_{e,3}$ = residual strength factor (JSCE SF4)
S = slab section modulus, in.^{3}/in. (mm^3/mm)
ν = Poisson's ratio for concrete (approximately 0.15)

A6.2—Assumptions and design criteria

Slab thickness h = 6 in. (150 mm)
Concrete compressive strength (cylinder) f_c' = 4000 psi (27.5 MPa)
Concrete rupture modulus f_r = 550 psi (3.79 MPa)
Concrete elastic modulus E = 3,600,000 psi (25,000 MPa)
Poisson's ratio ν = 0.15
Modulus of subgrade reaction, k = 100 lb/in.^{3} (0.027 N/mm^3)
Storage rack load = 15 kips (67 kN)
Base plate = 4 x 6 in. (10 x 15 cm)

A6.2.1 *Calculations for a concentrated load applied a considerable distance from slab edges*

The radius of relative stiffness is given by
$L = [E \times h^3/(12(1 - \nu^2)k)]^{0.25}$
$= [3{,}600{,}000 \times 63/(12(1 - 0.15^2)100]^{0.25}$
$= 28.5$ in.

The section modulus of the slab is
$S = 1$ in. $\times h^2/6 = 1 \times 6^2/6 = 6$ in.^{3}/in.

The equivalent contact radius of the concentrated load is the radius of a circle with area equal to the base plate.

$$a = (\text{base plate area}/3.14)^{0.5} \quad \text{(A6-1)}$$

$= (24/3.14)^{0.5} = 2.8$ in.

A concentrated load applied a considerable distance away from slab edges should not exceed the ultimate load strength of the slab:

$$P_{ult} = 6(1 + (2a/L)) \times (Mp + Mn) \quad \text{(A6-2)}$$

where
$Mp = f_r \times R_{e,3}/100 \times S$
$Mn = f_r \times S$

Combining Mp and Mn
$Mp + Mn = f_r \times S \times (1 + R_{e,3}/100)$

Select a factor of safety of 1.5 for this example
$Mp + Mn = f_r \times S \times (1 + R_{e,3}/100)/1.5$

Solving Eq. (A6-2)
$15 = 6\ (1 + 2 \times 2.8/28.5) \times (Mp + Mn)/1.5$

The minimum required bending moment strength of the slab for the applied load is

3.13 in.-k/in. $= Mp + Mn$

The stresses due to shrinkage and curling can be substantial. For the purpose of this example, select 200 psi. This translates into an additional moment of 1.2 in.-k/in. (6.0 in.^{3}/in. $\times$ 200 psi) to account for shrinkage and curling stresses. This stress varies depending on the factor of safety and other issues, including mixture proportion, joint spacing, and drying environment.

Using Eq. (A6-3) to solve for the required residual strength factor $R_{e,3}$

$$3.13 \text{ in.-k/in.} + 1.2 \text{ in.-k/in.} = f_r \times S \times (1 + R_{e,3}/100) \quad \text{(A6-3)}$$

$R_{e,3} \geq [(4.33 \times 1000/550/6.0) - 1.0]100$
$R_{e,3} \geq 31$

Residual load factors for various fiber types and quantities are available from steel fiber manufacturers' literature. Use laboratory testing for quality control to verify residual strength factors on a project basis. The quantity of steel fibers to provide the residual strength factor shown in this example ranges from 33 to 50 lb/yd^3 (20 to 30 kg/m^3), depending on the properties (length, aspect ratio, tensile strength, and anchorage) of the fiber.

A6.2.2 *Calculations for post load applied adjacent to sawcut contraction joint*

Assuming 20% of the load is transferred across the joint (Meyerhof 1962), the load for a concentrated load applied adjacent to a sawcut contraction joint should not exceed

$$0.80 \times P_{ult} = 3.5(1 + (3a/L)) \times (Mp + Mn)/1.5 \quad \text{(6-4)}$$

Solving Eq. (A6-4),
$0.80 \times 15 = 3.5(1 + 3 \times 2.8/28.5) \times (Mp + Mn)/1.5$

The minimum required bending moment strength of the slab for the applied load is 3.97 in.-k/in. $= Mp + Mn$.

As in the previous example, use an additional moment of 1.2 in.-k/in. to account for shrinkage. No curling stress exists at the edge. Using Eq. (A6-3) to solve for the required residual strength factor $R_{e,3}$
3.97 in.-k/in. + 1.2 in.-k/in. $= f_r \times S \times (1 + R_{e,3}/100)$
$R_{e,3} \geq [(5.17 \times 1000/550/6.0) - 1.0] \times 100$
$R_{e,3} \geq 57$

The quantity of steel fibers to provide the residual strength factor shown in this example range from 40 to 60 lb/yd^3 (25 to 35 kg/m^3), depending on the mixture proportion and all mixture constituents, including fiber type and quantity.

APPENDIX 7—CONSTRUCTION DOCUMENT INFORMATION

A7.1—Introduction

It is helpful when the design criteria are well established that it be shown on the drawings for future slab modifications. Below is an example design criteria of some of the more relevant loading information that could be shown on the drawings, along with some typical conceptual details that will help reduce the majority of serviceability performance problems that have been observed concerning slabs-on-ground.

A7.2—Example design criteria

The following is an example design criteria that could be placed on the drawing (Fig. A7.1) showing some of the various considerations possible:

SLAB-ON-GROUND DESIGN CRITERIA

1. MINIMUM REQUIRED MODULUS OF SUBGRADE REACTION FOR WIDE AREA RACK LOADING AND UNIFORM STORAGE LOADING..... 100 PCI

2. MINIMUM REQUIRED MODULUS OF SUBGRADE REACTION FOR LIFT-TRUCK LOADING..... 150 PCI

3. UNIFORM STORAGE LOAD .....925 PSF

4. LIFT-TRUCK FRONT AXLE LOAD .....15,500 LB (SINGLE WHEELS SPACED 33 IN.)

5. GENERAL RACK LOADING .....AS SHOWN BELOW

6. SLAB-ON-GROUND CONTRIBUTES TO THE RESISTANCE OF WIND AND SEISMIC UPLIFT FORCES FOR FOUNDATIONS.

7. SLAB-ON-GROUND IS USED AS A HORIZONTAL DIAPHRAGM TO LATERALLY STABILIZED BUILDING.

8. SLAB IS USED TO LATERALLY STABILIZE MASONRY WALLS. SEE DRAWINGS FOR LOCATIONS.

A7.3—Typical details

Most problems for slabs-on-ground can be related to improper details or not providing details. The typical slab-on-ground problem areas have been at doors, slab penetrations, reentrant corners, discontinuous joints, and lateral ties to the slab-on-ground. Below are some typical conceptual details that have been used successfully on projects.

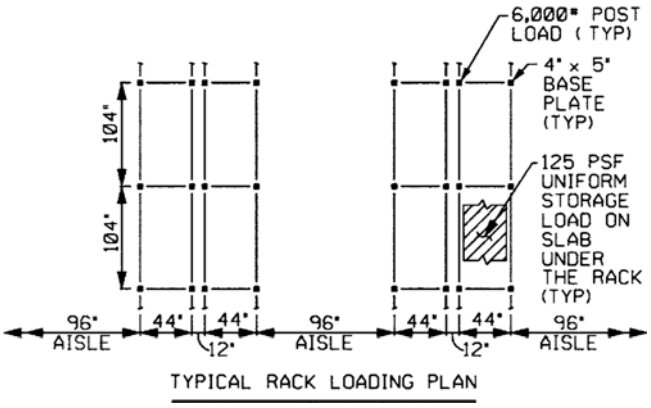
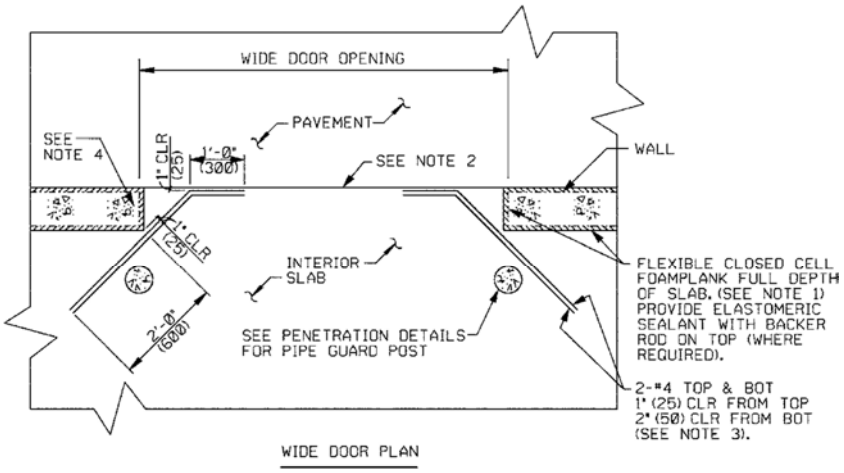


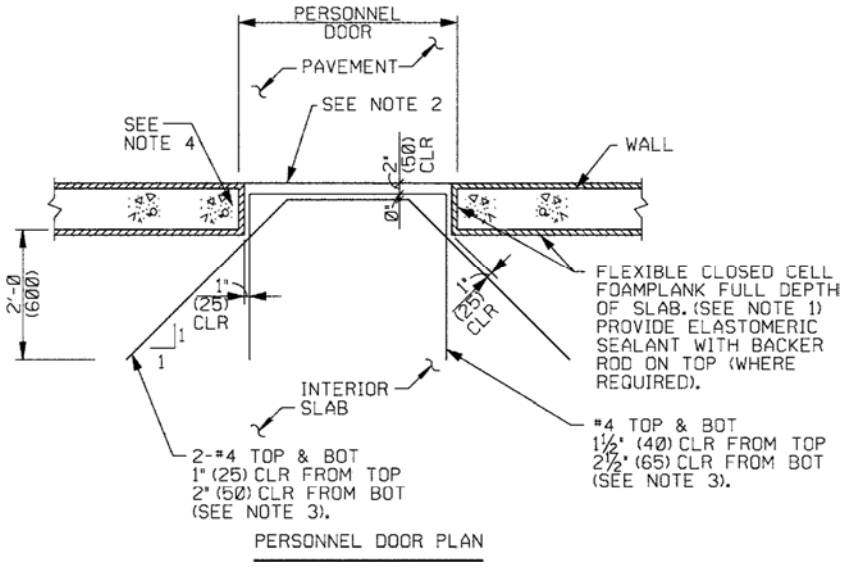
Fig. A7.1—Example design for typical rack loading plan.

Notes for Sections A7.3.1 to A7.3.4 are on p. 72.

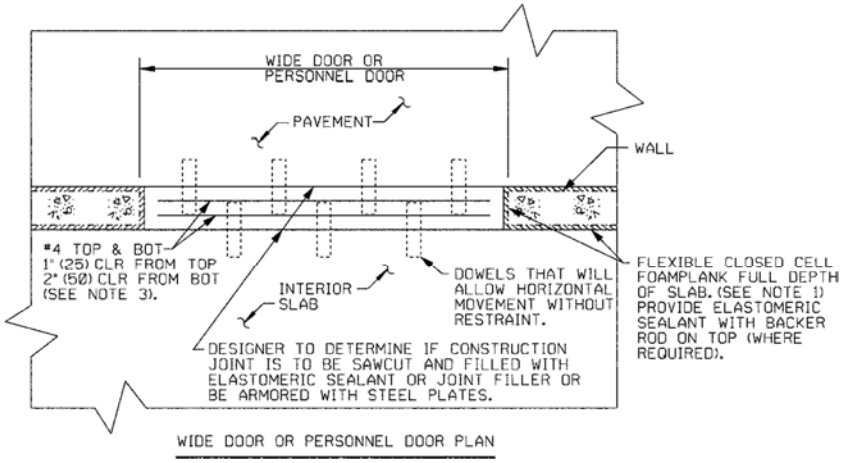
A7.3.1 *Door details*



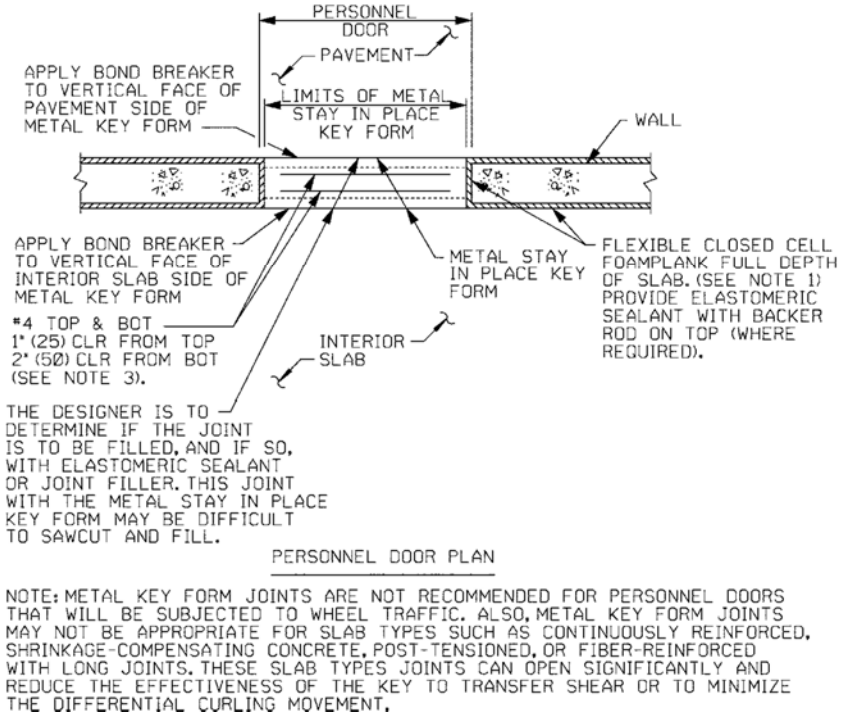
Wide door plan.



Personnel door plan.

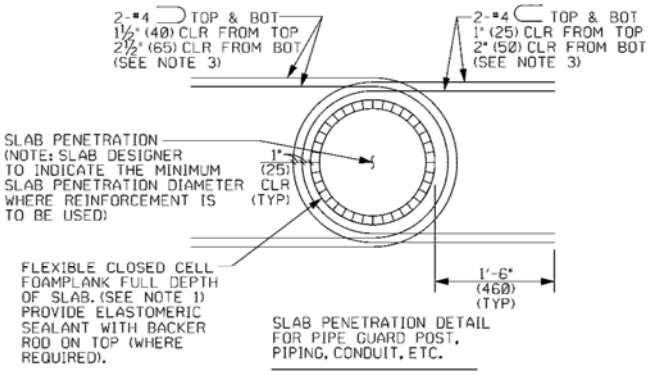


Wide door or personnel door plan.

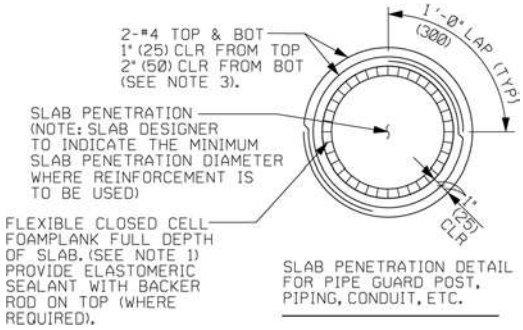


Personnel door plan.

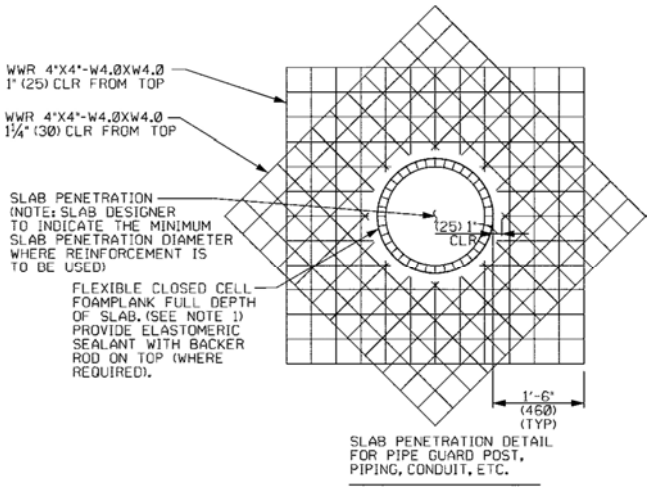
A7.3.2 *Slab penetrations*



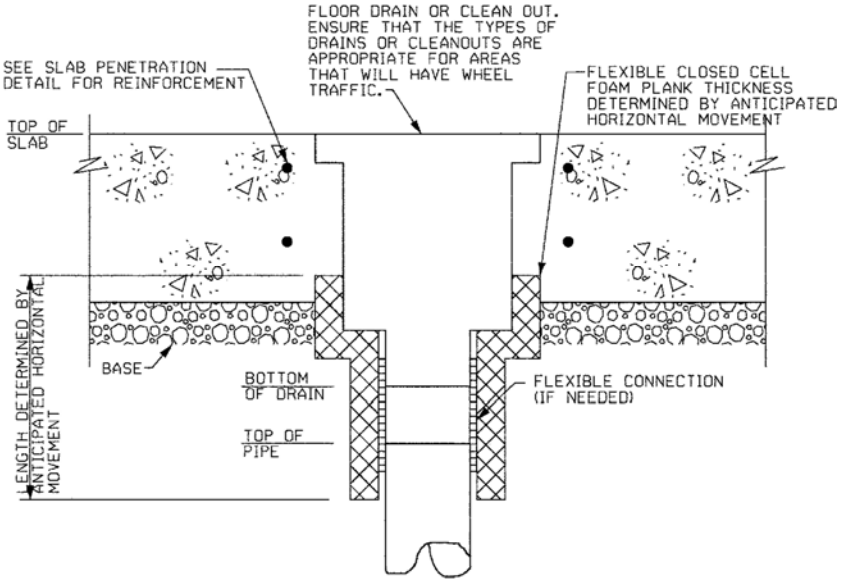
Slab penetration detail—Option 1.



Slab penetration detail—Option 2.

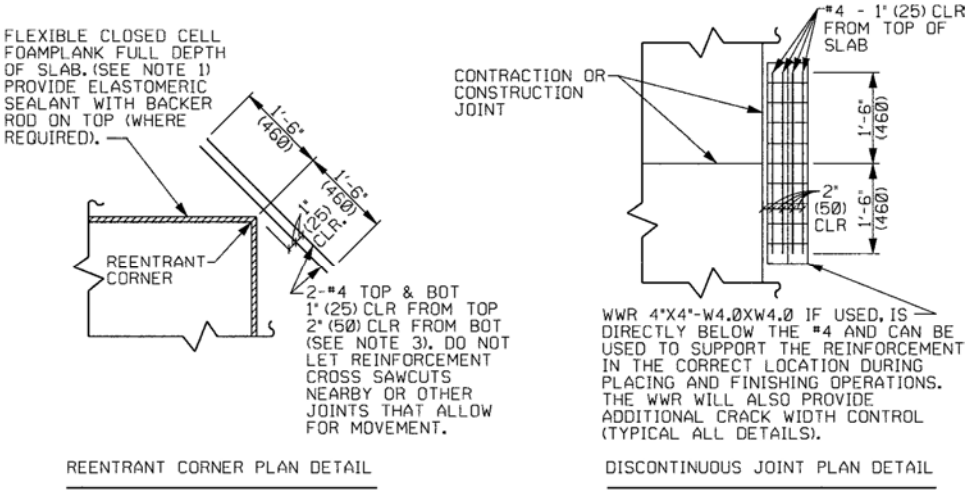


Slab penetration detail—Option 3.



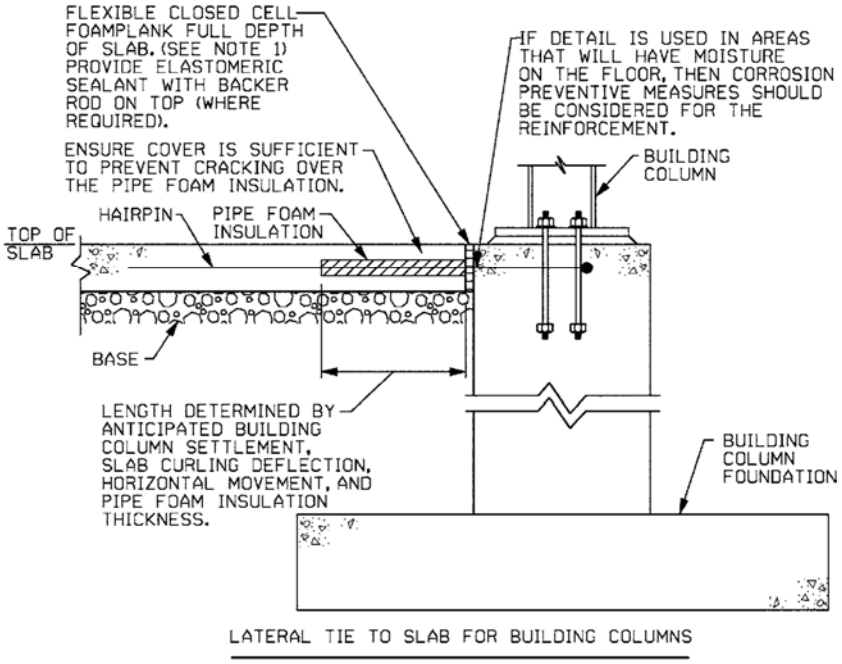
Floor drain and cleanout detail.

A7.3.3 *Reentrant corners and discontinuous joints*

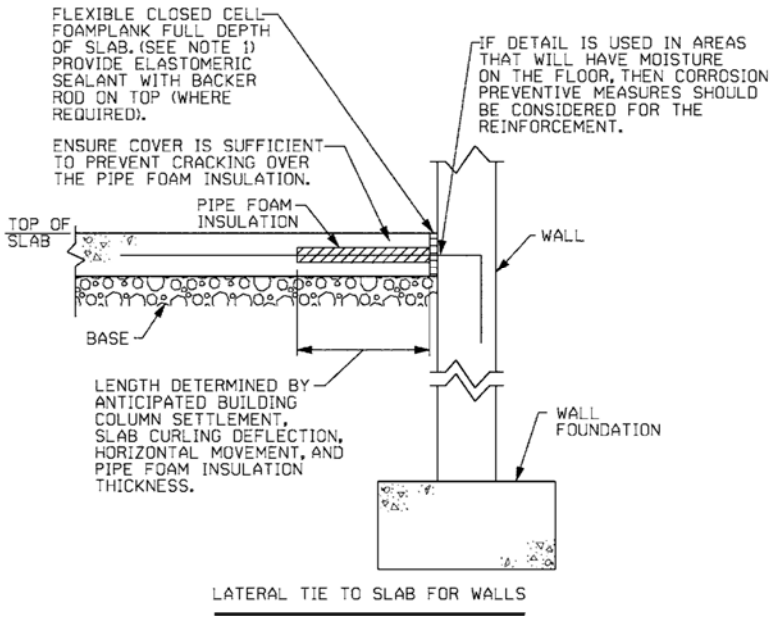


Reentrant corner and discontinuous joint detail.

A7.3.4 *Lateral ties to slab-on-ground*



Lateral tie to slab for building columns.



Lateral tie to slab for walls.

Notes for Sections A7.3.1 to A7.3.4.

1. FLEXIBLE CLOSED CELL FOAM PLANK THICKNESS OF ¼" (6) TO ½" (13) HAVE WORKED WELL FOR UNREINFORCED JOINTED SLABS. FOR CONTINUOUSLY REINFORCED, SHRINKAGE-COMPENSATING CONCRETE, POST-TENSIONED, AND FIBER-REINFORCED SLABS WITH LONG JOINTS, THE FOAM THICKNESS MAY HAVE TO BE DESIGNED THICKER TO ACCOMMODATE THE LARGER MOVEMENTS.
2. DESIGNER TO DETERMINE IF JOINT IS TO HAVE DOWELS, SAWCUT AND FILLED WITH ELASTOMERIC SEALANT OR JOINT FILLER, OR BE ARMORED WITH STEEL PLATES.
3. PROVIDE BOTTOM REINFORCEMENT FOR SLABS GREATER THAN OR EQUAL TO 6" (150) THICK.
4. THE DOOR JAMB SHOULD BE STOPPED ABOVE THE SLAB AND NOT BE ATTACHED TO THE SLAB. IF THE JAMB IS EXTENDED BELOW THE TOP OF THE SLAB OR ATTACHED TO THE SLAB, THE CONCRETE SLAB HORIZONTAL SHRINKAGE OR CURLING MOVEMENT MAY CAUSE THE DOOR JAMB TO BECOME MISALIGN AND CAUSE PROBLEMS FOR THE DOOR.

CONVERSION FACTORS

LENGTH

1 in. = 2.54 cm
1 cm = 0.39 in.
1 ft = 0.305 m
1 m = 3.28 ft
1 mile = 1.61 km
1 km = 0.62 miles

in. to m ..... multiply by 2.5
m to in ..... multiply by 0.4
ft to m ..... multiply by 2.5
oz to g ..... multiply by 3.3
oz to g ..... multiply by 28.3
g to oz ..... multiply by 0.035
lb to kg ..... multiply by 0.45
kg to lb ..... multiply by 2.2

VOLUME

1 fl oz = 29.57 mL
10 mL = 0.34 fl. oz
1 qt (32 fl. oz) = 946.35 mL
1 L = 1.06 U.S. qt
1 gal. (128 fl. oz) = 3.79 L
3.79 L = 1 U.S. gal.

oz to mL ..... multiply by 30
mL to oz ..... multiply by 0.03
qt to L ..... multiply by 0.95
L to qt ..... multiply by 1.06

1 in.^{3} = 16.39 cm^3
1 ft^3 = 1,728 in.^{3} = 7.481 gal.
1 yd^3 = 27 ft^3 = 0.7646 m^3

WEIGHT

1 oz = 28.3 g
10 g = 0.35 oz
1 lb = 0.45 kg
1 kg = 2.20 lb

oz to g ..... multiply by 28.3
g to oz ..... multiply by 0.035
lb to kg ..... multiply by 0.45
kg to lb ..... multiply by 2.2

TEMPERATURE

°C = (°F – 32)/1.8
°F = (1.8 × °C) + 32
1 °F/in. = 0.22 °C/cm

SPECIFIC WEIGHT

1 lb water = 27.7 in.^{3} = 0.1198 gal.
1 ft^3 water = 62.43 lb
1 gal. water = 8.345 lb

WATER-CEMENT RATIO

Multiply *w/c* by 11.3 to obtain gallons per bag

AREA

1 in.^{2} = 6.452 cm^2

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Guide to Design of Slabs-on-Ground

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