Design of Slabs-on-Ground

Reported by ACI Committee 360

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This document presents information on the design of slabs-on-ground, primarily industrial floors. The report 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 problems. Advantages and disadvantages of each of these slab designs 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, however, it is unrealistic to expect crack-free and curl-free floors. Consequently, every owner should be advised by both the designer and contractor that it is normal to expect some amount of cracking and curling on every project, and that such occurrence does not necessarily reflect adversely on either the adequacy of the floor's design or the quality of its construction. Design examples appear in an appendix.

Keywords: concrete; 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; shrinkage; warping.

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CHAPTER 1—INTRODUCTION

1.1—Purpose and scope

This guide presents state-of-the-art information on the design of slabs-on-ground. Design is defined as 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 it is 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 may be of uniform or variable thickness, and it may include stiffening elements such as ribs or beams. The slab may be unreinforced, reinforced, or post-tensioned concrete. The reinforcement steel may be provided to limit the crack widths resulting from shrinkage and temperature restraint and the applied loads. Post-tensioning steel may be provided to minimize cracking due to shrinkage and temperature restraint and to resist the applied loads.

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 loads on the roof transferred through dual-purpose rack systems, are also mentioned.

In addition to design, 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, and mat foundations.

1.2—Work of ACI Committee 360 and other relevant committees

1.2.1 ACI Committee 360 develops and reports on criteria for design of slabs-on-ground, with the exception of highway and airport pavements, parking lots, and mat foundations.

1.2.2 ACI Committee 302 develops recommendations for construction of slab-on-ground and suspended-slab floors for industrial, commercial, and institutional buildings. ACI 302.1R provides guidelines and recommendations on materials and slab construction.

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

1.2.4 ACI Committee 325 addresses the structural design, construction, maintenance, and rehabilitation of concrete pavements.

1.2.5 ACI Committee 332 develops information on the use of concrete for one- and two-family dwellings and multiple single-family dwellings not more than three stories in height as well as accessory structures (residential). Where a residential slab-on-ground is placed, only loadings from pedestrian and passenger vehicles are expected. The slab should be continuously supported throughout and placed on suitable soil or controlled fill where little volume change is expected. Where these conditions are not met, a residential slab-on-ground should be designed specifically for the application.

1.2.6 ACI Committee 336 addresses design and related considerations of foundations that support and transmit substantial loads from one or more structural members. The design procedures for mat foundations are given in ACI 336.2R. Mat foundations are typically more rigid and more heavily reinforced than common slabs-on-ground.

1.2.7 ACI Committee 330 monitors developments and prepares recommendations on design, construction, and maintenance of concrete parking lots. Parking lot pavements have unique considerations that are covered in ACI 330R, which includes design and construction- and discussions on material specifications, durability, maintenance, and repair.

1.2.8 ACI Committee 544 provides measurement of properties of fiber-reinforced concrete (FRC); a guide for specifying proportioning, mixing, placing, and finishing steel FRC; and design considerations for steel FRC.

1.3—Work of non-ACI organizations

Numerous contributions of slabs-on-ground 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. Several industrial associations, such as 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 on their design and construction. In addition, periodicals such as *Concrete International* and *Concrete Construction* have continuously disseminated information for the use of those involved with slabs-on-ground.

1.4—Design theories for slabs-on-ground

1.4.1 *Introduction*—Stresses in slabs-on-ground result from both applied loads and volume changes of the soil and concrete. The magnitude of these stresses depends on factors such as the degree of continuity, subgrade strength and uniformity, method of construction, quality of construction, and magnitude and position of the loads. In most cases, the effects of these factors can only be evaluated by making simplifying assumptions with respect to material properties and soil-structure interaction. The following sections briefly review some of the theories that have been proposed for the design of soil-supported concrete slabs.

1.4.2 *Review of classical design theories*—The design methods for slabs-on-ground are based on theories originally developed for airport and highway pavements. An early attempt at a rational approach to design was made around 1920, when Westergaard (1926) proposed the so-called "corner formula" for stresses. Although the observations in the first road test with rigid pavements seemed to be in agreement with the predictions of this formula, its use has been limited.

Westergaard developed one of the first rigorous theories of structural behavior of rigid pavement in the 1920s (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 the deflection of the slab. This is 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 is the constant now recognized as the coefficient (or modulus) of subgrade reaction. Extensive investigations of structural behavior of concrete pavement slabs performed in the 1930s at the Arlington Virginia Experimental Farm and at the Iowa State Engineering Experiment Station showed good agreement between observed stresses and those computed by the Westergaard 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 a proper choice of the modulus of subgrade reaction was essential for good agreement with respect to stresses, there remained much ambiguity in the methods for experimental determination of that correction coefficient.

Also in the 1930s, considerable experimental information was accumulated that 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, in 1943, proposed the layered-solid theory of structural behavior for rigid pavements (Burmister 1943) and suggested that the design be based on a criterion of limited deformation under load. The design procedures for rigid pavements based on this theory, however, were not sufficiently developed for use in engineering practice. The lack of analogous solutions for slabs of finite extent (edge and corner cases) was a particular deficiency. Other approaches based on the assumption of a thin elastic slab of infinite extent resting on an elastic, isotropic solid have also been developed. The preceding theories are limited to consideration of behavior in the linear range, where deflections are proportional to applied loads. Lösberg (Lösberg 1978; Pichaumani 1973) later proposed a strength theory based on the yield-line concept for ground-supported slabs, but the use of strength as a basis for the design of the slab-on-ground is not common.

All existing theories can be grouped according to models used to simulate the behavior of the slab and the subgrade. Three different models are used for the slab:

- Elastic-isotropic solid;
- Thin elastic slab; and
- Thin elastic-plastic slab.

The two models used for the subgrade are:

- Elastic-isotropic solid; and
- Winkler.

The Winkler subgrade models the soil as linear springs so that the reaction is taken proportionally to the slab deflection. Existing design theories are based on various combinations of these models. The methods included in this guide are generally graphical, plotted from computer-generated solutions of selected models. Design theories need not be limited to these combinations. While the elastic-isotropic model provides closer prediction for the response of real soils, the use of the Winkler model is almost universally 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 plate resting on an elastic subgrade is often used to represent the slab-on-ground. Solving the governing equations by conventional methods is feasible only for simplified models where the slab and subgrade are assumed to be continuous and homogeneous. In reality, however, 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 quite 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; Pichaumani 1973). Typically, these models use combinations of various elements, such as elastic blocks, rigid blocks, and torsion bars, to represent the slab. The subgrade is usually modeled by linear springs (the Winkler subgrade) placed under the nodal joints. While 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—Overview of subsequent chapters

Chapter 2 identifies types of slabs-on-ground and provides a table with the advantages and limitations of each slab type. Chapter 3 discusses the role of the subgrade and outlines methods for physical determination of the modulus of subgrade reaction and other needed properties. Chapter 4 presents a discussion of various loads. Chapter 5 discusses joint design. Chapters 6 through 11 provide information on design methods and the related parameters needed to complete the design. Chapter 12 presents special requirements for slabs in refrigerated facilities. Chapter 13 covers the design methods used to reduce the effect of drying, shrinkage, and curling. References are listed in Chapter 14. Design examples in the Appendixes illustrate the application of selected design methods.

1.6—Further research

There are many areas where additional research is needed. Some of these areas are:

- Developing concrete mixture proportions that have low shrinkage characteristics but are still 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;
- Soil properties and how they may change over time under load repetitions, long-term loading, or both;
- Establishing an allowable differential deflection between the tops of the slab on each side of the joint and spacing between slab joint edges to minimize spalling due to lift truck traffic; and
- Recommended joint spacing using FRC.

CHAPTER 2—SLAB TYPES

2.1—Introduction

This chapter identifies and briefly discusses the common types of slab-on-ground construction. The term "slab-on-ground" is the preferred nomenclature although, in practice, the term "slab-on-grade" is often used to mean the same thing. Slab-on-ground is a general term that includes interior slabs subject to loadings as described in Chapter 4. These include industrial, commercial, residential, and related applications. Although the term might also include parking lot and roadway pavements, these are not specifically addressed in this guide.

2.2—Slab types

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

- Unreinforced concrete slab;
- Slabs reinforced to limit crack widths due to shrinkage and temperature restraint and applied loads. These slabs consist of the following:
  - –Mild steel bar, wire reinforcement, or fiber reinforcement, all with closely spaced joints; and
  - –Continuously reinforced (sawcut contraction joint-free floors);
- Slabs reinforced to prevent cracking due to shrinkage and temperature restraint and applied loads. These slabs consist of the following:
  - –Shrinkage-compensating concrete; and
  - –Post-tensioned;
- Structural slabs (ACI 318).

2.2.1 *Unreinforced concrete slab*—The design of this type of slab involves determining its thickness as a plain concrete slab without reinforcement; however, it may have joints strengthened with steel dowels. It is designed to remain uncracked between joints due to loads on the slab surface and restraint to concrete volumetric changes. Unreinforced concrete slabs do not contain high-volume macropolymeric fibers, wire fabric, steel fibers, plain or deformed bars, post-tensioning, or any other type of steel reinforcement. The cement normally used is portland cement Type I or II (ASTM C 150). The effects of drying shrinkage and uniform subgrade support on slab cracking are critical to the performance of unreinforced concrete slabs. Design methods for unreinforced slabs are provided in Chapter 6.

2.2.2 *Slabs reinforced for crack width control*—Thickness design can be the same as for unreinforced concrete slabs, and the slab is assumed to remain uncracked due to loads placed on its surface. Shrinkage crack width (if cracking occurs) for slabs constructed with portland cement is controlled by a nominal quantity of distributed reinforcement placed in the upper 1/3 of the slab. The primary purpose of the reinforcement is to limit the width of any cracks that may form between the joints. Bar or wire reinforcement should be stiff enough so that it can be accurately located in the upper 1/3 of the slab. Slabs may be reinforced with reinforcing bars, welded wire reinforcement sheets, steel fibers, or macropolymeric fibers.

Bars or welded wire reinforcement can be used to provide moment capacity 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 strength should be sized by conventional reinforced concrete theory as described in ACI 318. Using the methods in ACI 318 with high steel reinforcement stresses, however, may lead to unacceptably wide crack widths. Currently, building codes do not support the use of fiber reinforcement to provide moment capacity in cracked sections.

Reinforcement, other than post-tensioning or the reinforcement used in a shrinkage-compensating slab, does not prevent cracking. Typically, the most economical way to obtain increased load-carrying capacity is to increase the thickness of the slab. Design methods for slabs reinforced for limiting crack width can be found in Chapters 6, 7, and 10.

2.2.3 *Slabs reinforced to prevent cracking*—Post-tensioned slabs and shrinkage-compensating slabs are typically designed not to crack. Some incidental minor cracking may still occur, however. The reinforcement is used to prevent the slab from cracking. For shrinkage-compensating slabs, the slab is designed unreinforced, and the reinforcement is designed to prestress the expanding slab to resist the later shrinkage and temperature restraint. For post-tensioned slabs, the reinforcement is typically designed to resist the shrinkage and temperature restraint and the applied loads.

Shrinkage-compensating concrete slabs are produced either with a separate component admixture or with ASTM C 845 Type K cement, which is expansive. This concrete does shrink, but first expands to an amount intended to be slightly greater than its drying shrinkage. Distributed reinforcement is used in the upper 1/3 of the slab to limit the initial slab expansion and to prestress the concrete. Reinforcement should be rigid and supported so that it can be positively positioned in the upper 1/3 of the slab. The slab should be isolated from fixed portions of the structure, such as columns

Table 2.1—General comparison of slab types

Slab type	Advantages	Disadvantages
Unreinforced concrete	• Simple to construct • Generally costs less 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	• Sawcut contraction joint spacing may be further apart than an unreinforced slab • Reinforcement is used to limit crack width • May reduce the long-term loss of flatness and levelness if reinforcement is continuous through the joints	• May have higher cost 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	• Sawcut contract joints can be eliminated where sufficient reinforcement is used • Eliminates sawcut contraction joint maintenance. Curling is reduced to heavy reinforcement and elimination of joints • Changes to flatness and levelness are minimized	• Requires relatively heavy (at least 0.5%) continuous reinforcement placed near the top of the slab • 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 due to increased spacing of the joints reducing the total amount of joints and 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. Joints should be detailed for expansion • Contractor should have experience with this type of concrete
Post-tensioned	• Construction or sawcut contraction joint spacings 100 to 500 ft (30 to 150 m) • Most shrinkage cracks can be avoided • Eliminates sawcut contraction joints and their maintenance • Negligible slab curl if tendons are draped near joint ends • Improved long-term flatness and levelness • Decreased slab thickness or increased load capacity • Resilience against overloading • Advantages in poor soil conditions	• More demanding installation • Contractor should have experience with post-tensioning or employ a consultant with experience with post-tensioning • Inspection essential to ensure proper placement and stressing of tendons • Uneconomical for small areas • Need to detail floor penetrations and perimeter for movement • Impact of cutting tendons should be evaluated for post-construction penetrations
Steel fiber-reinforced concrete	• Increased resistance to impact and fatigue loadings when compared to slabs reinforced with bars or mesh • Simple to construct	• Concrete containing steel fibers may require adjustments to standard concrete mixing, placement, and finishing procedures • Some fibers will be exposed on the surface of the slab • Floors subjected to wet conditions may not be suitable candidates for steel fiber because fibers close to the surface and in water-permeable cracks will rust
Polymeric fiber-reinforced concrete	• Helps reduce plastic shrinkage cracking • Simple to construct • High-volume macropolymeric fibers provide increased resistance to impact and fatigue loadings, similar to steel fiber • Polymeric fibers do not corrode	• Micropolymeric fibers do not help in controlling drying shrinkage cracks • Joint spacing for micropolymeric fiber-reinforced slabs are the same as unreinforced slabs
Structural slabs reinforced for building code requirements	• Slabs can carry structural loads such as mezzanines • Sawcut contraction joints can be reduced or eliminated where sufficient reinforcement is used	• Slab design should comply with ACI 318 • Slab may have numerous fine or hairline cracks if reinforcement stresses are sufficiently low

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

Design methods for slabs reinforced to prevent cracking can be found in Chapters 8 and 9.

2.2.4 *Structural slabs*—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 unacceptably wide crack widths. For an unreinforced (plain) structural concrete slab, the requirements of ACI 318, Chapter 22, "Structural Plain Concrete," should be used.

2.3—General comparison of slab types

Table 2.1 provides general advantages and disadvantages for the various slab types discussed in Section 2.2. This table can assist the designer with selecting the slab type that is most appropriate for the particular project.

2.4—Design and construction variables

Design and construction of slabs-on-ground involves both technical and human factors. The 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 involve the workers' abilities, feedback to evaluate the construction process, and conformance to proper maintenance procedures for cracking, curling, shrinkage, and other conditions. These and other factors should be considered when designing a slab (Westergaard 1926).

2.5—Conclusion

There is no single design technique recommended for all applications. Rather, there are a number of identifiable construction concepts and a number of design methods. Each combination should be selected based on the requirements of the specific application.

CHAPTER 3—SOIL SUPPORT SYSTEMS FOR SLABS-ON-GROUND

3.1—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. The 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 capacity and not easily susceptible to be affected by climatic changes. Slab-on-ground failures can occur because a proper support system was not achieved. This chapter addresses issues related to the support system of the slab-on-ground, including:

- Geotechnical engineering reports;
- 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 those aspects of the support system necessary for the slab-on-ground to perform as intended.

The slab support system consists of a subgrade, usually a base, and sometimes a subbase, as illustrated in Fig. 3.1. Crushed rock, gravels, or coarse sands, which 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; however, they may also be used as base materials. Soils in the subgrade are generally the ultimate supporting materials, but bedrock, competent or weathered, may also be encountered. If 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, as such, 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.

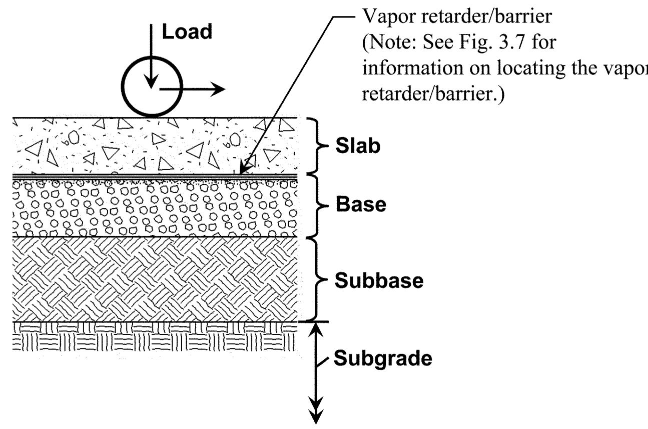


Fig. 3.1—Slab support system terminology.

3.2—Geotechnical engineering reports

3.2.1 *Introduction*—Geotechnical engineering investigations are now commonly performed for most building projects to supply subsurface site information for design and construction and to meet building code requirements. The primary purpose of these field investigations is to supply information for the design and construction of the building foundation elements. Within the geotechnical engineering report, slab-on-ground support is frequently discussed, and subgrade drainage and preparation recommendations are given. Even if slab support is not discussed in detail, information given within such 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.

3.2.2 *Boring or test pit logs*—Descriptions given on boring or test pit logs are useful because they give information on the texture of the soils encountered and their moisture condition and relative density, if noncohesive; or consistency, if cohesive. Field test results, such as the standard penetration test (ASTM D 1586) in blows per 6 in. (150 mm) interval values, are presented on these logs. The location of the water table at the time the boring is made and depths to shallow bedrock are also denoted on the log. 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, as well as the Atterberg limits. Also, the soil is classified, as will be discussed in Section 3.3.

3.2.3 *Report evaluations and recommendations*—In many cases, the geotechnical engineer writing the report has not been given complete information on the design requirements of the slab-on-ground. Evaluations and recommendations relative to the existing subgrade material, its compaction, and resulting supporting capability can be included in the report, and should be evaluated against the actual design requirements. Suggestions can also be given by the geotechnical engineer for possible 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 geotechnical engineer is generally knowledgeable about the use and experience with 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, the use of the facility and the proposed floor elevation should always be given to the geotechnical engineer; however, information concerning the type and magnitude of anticipated loads, environmental conditions of the building space, levelness and flatness criteria for the floors, and floor-covering requirements should also be conveyed to the geotechnical engineer. In some cases, it may be beneficial for the geotechnical engineer to visit local buildings or other facilities of the client having 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 slab-on-ground.

3.3—Subgrade classification

The soil that will be beneath the slab-on-ground should be identified and classified to estimate its suitability as a

Table 3.1—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
		Gravel 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.

subgrade, although it may meet the criteria for a subbase or even a base material. The Unified Soil Classification System is predominantly used in the United States and is referred to in this document. Table 3.1 provides information on classification groups of this system and some important criteria for each soil group. Visual procedures (ASTM D 2488) can be used, but more reliable classifications can be made using laboratory test results (ASTM D 2487). For example, the plasticity chart of Table 3.2 is used to classify the fine-grained soils.

The following tests and test methods are helpful in the proper classification of soils:

1. Moisture content: ASTM D 2216;
2. Specific gravity: ASTM D 854;
3. Liquid limits: ASTM D 4318; and
4. Plastic limit: ASTM D 4318.

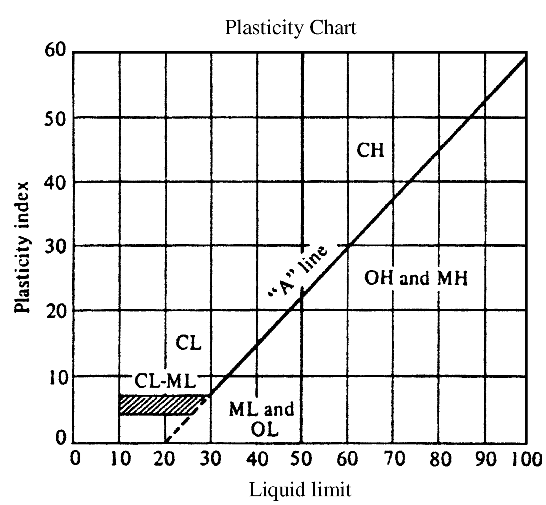
The standard Proctor compaction test (ASTM D 698) and modified Proctor compaction test (ASTM D 1557) are not strictly classification tests, but their moisture-density relationships are very useful in accessing a soil subgrade or subbase. A more detailed listing of the ASTM standards is given in Chapter 14.

3.4—Modulus of subgrade reaction

3.4.1 *Introduction*—Design methods listed in Chapter 2, including Westergaard's (Westergaard 1923, 1926) pioneering work on rigid pavement analysis, employ the modulus of subgrade reaction as a single property to represent the supporting capacity to be used in design. 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 deformation of a soil is nonlinear and is not a fundamental soil property. A typical nonlinear relationship between a normal compressive load and the resulting deformation for an area is depicted in Fig. 3.2. The type of soil structure, density, moisture content, and

Table 3.2—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] sieve size), 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[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM[a] d / u	Silty gravels, gravel-sand-silt mixtures		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
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits below "A" line with P.I. greater than 7	
	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		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a] d / u	Silty sands, sand-silt mixtures		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
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
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			

[a] 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.
[b] 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.

prior loading determine the load-deformation relationship. The relationship also depends on the width of the loaded area, 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 D 1196 have been used to estimate the subgrade modulus.

3.4.2 *Plate load field tests*—Determination of the modulus of subgrade reaction on representative subgrade in place with a 30 in. (760 mm) diameter bearing plate, which is recommended by ASTM D 1196, is time-consuming and

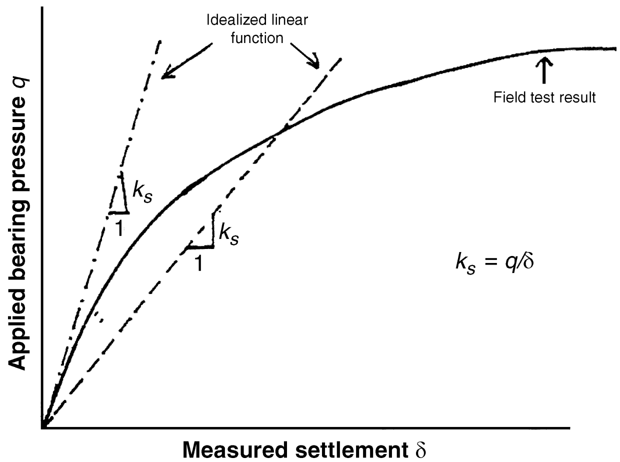


Fig. 3.2—Plate load-deformation diagram.

expensive. Several days are generally needed 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. This is illustrated in Fig. 3.2.

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 in reporting a k value. Typically, these tests are made directly on an unconfined natural or compacted subgrade or on a thickness 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, these 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 cost and time required and the probable results.

3.4.3 *American Association of State Highway Officials (AASHTO) approach*—For rigid pavements, AASHTO has 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

$$k \text{ (lb/in.}^3) = M_R \text{ (psi)/19.4 (in.-lb units)}$$

$$k \text{ (kN/m}^3) = M_R \text{ (kPa)} \times 2.03 \text{ (SI units)}$$

The resilient modulus is a measure of the assumed elastic property of soil taking into account its nonlinear characteristics. It is defined as the ratio of the repeated axial deviator stress to the recoverable axial strain. It is widely recognized as a method for characterizing pavement materials. Methods for the determination of M_R are described in AASHTO Test Method T307. The value of M_R can be evaluated using a correlation with the older and more common California bearing ratio (CBR) test value (ASTM D 1883) by the following empirical relationship (Heukelom and Klomp 1962)

$$M_R \text{ (psi)} = 1500 \times \text{CBR (in.-lb units)}$$

$$M_R \text{ (kPa)} = 10{,}342 \times \text{CBR (SI units)}$$

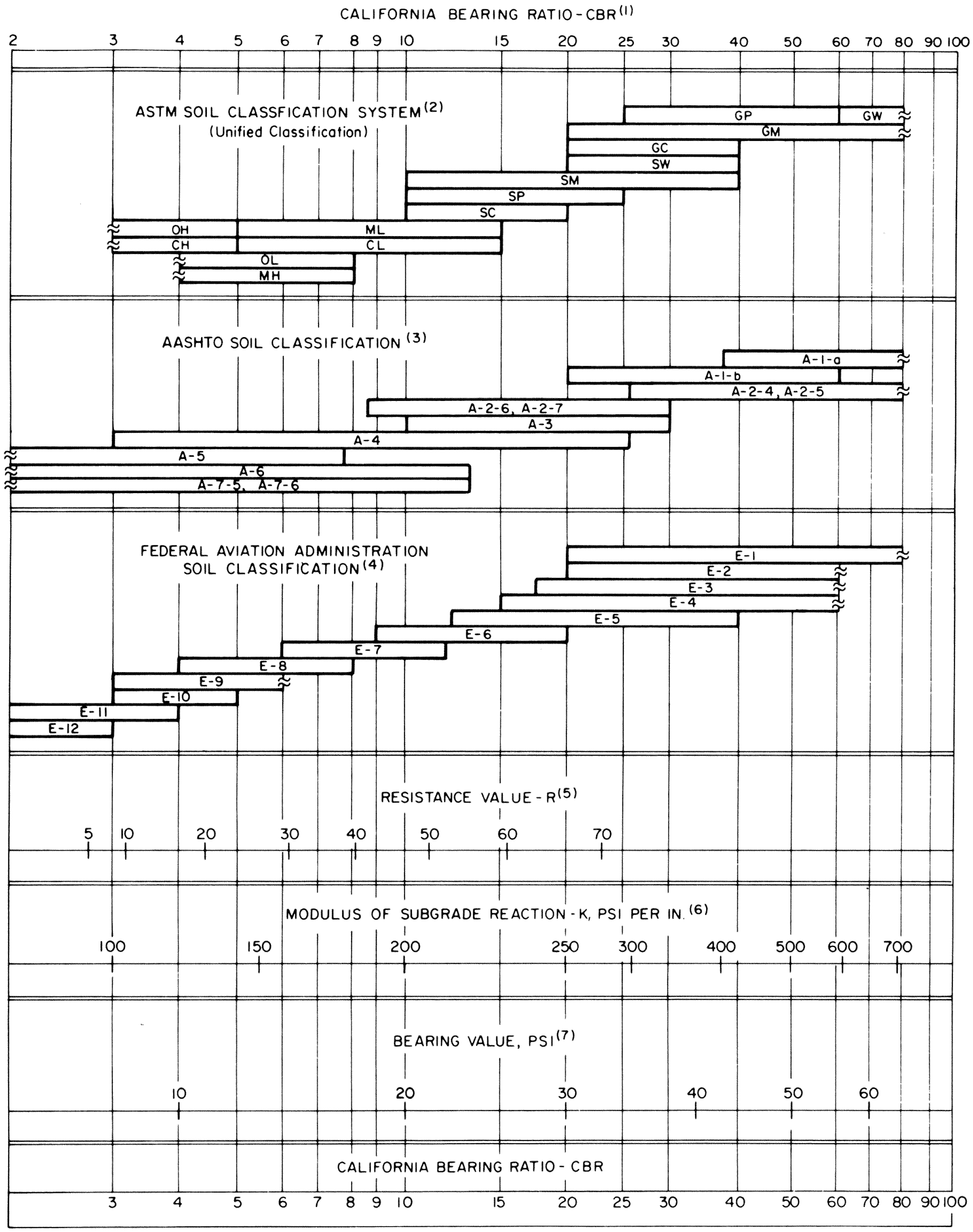
This approximate relationship has been used extensively for fine-grained soils having a soaked, saturated 96-hour CBR value of 10 or less (Heukelom and Klomp 1962). Correlations of M_R with soil properties such as clay content, Atterberg limits, and moisture content have also been developed.

The effective k value used for design as recommended by AASHTO for rigid pavements is dependent 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 AASHTO "Guide for the Design of Pavement Structures" 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 high values. It is recommended that the nomograph relationships contained in Fig. 3.3 be used to validate the results of correlated k values derived from AASHTO correlations.

3.4.4 *Other approaches*—Empirical relations between soil classification type, CBR, and k values have been developed by the Corps of Engineers, and this is illustrated by Fig. 3.3. These relationships are usually quite conservative. All of these test methods and procedures have been developed for pavements and 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.

3.4.5 *Influence of moisture content*—The moisture content of a fine-grained soil affects the modulus of subgrade reaction k, both at the time of testing and throughout the service life of the slab. 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 the clay minerals present. In general, the higher the moisture content, the lower the supporting capacity, but the relationship is unique for each type of soil. The more uniform the moisture content and dry density, the more uniform the support. Thus, providing good site surface drainage and drainage of the subgrade is very important. Experience has shown that high water tables and broken water or drain lines have caused slab-on-ground failures.

Laboratory tests can be performed to evaluate the influence of moisture by molding test specimens to various uniform



(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. 3.3—Approximate interrelationships of soil classifications and bearing values (Portland Cement Association 1988). (Note: 1 psi/in. = 0.271 kPa/mm; 1 psi = 6.90 kPa.)

moisture contents and dry densities. This is more practical than attempting to find the influence of moisture by field tests. Various test procedures, such as CBR, unconfined compression, and triaxial shear, can be followed. Moisture and density ranges chosen for testing should match those anticipated in the field.

3.4.6 *Influence of soil material on modulus of subgrade reaction*—Soils found at a building site are capable of providing a range of subgrade support. This is again illustrated by Fig. 3.3. 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. Stabilization methods can be used, but they will also have a limited range of effectiveness. In addition, drainage conditions can change the support capacity of most soils, but this can be most significant for clays and silts. Frost action can also reduce the support capacity of soils containing silt. Thus, the correlation between soil classification and supporting capacity is useful for estimating the range of capability but should be adjusted for expected site conditions.

3.4.7 *Uniformity of support*—The design charts of PCA, WRI, and Corps of Engineers (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, however, is not achieved in practice because of differences in composition, thickness, moisture content, slab curling, and subgrade density. If the joint recommendations given in Fig. 5.6 are followed, however, 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. On some projects, a well-constructed subgrade has been compromised by utility trenches that were poorly backfilled. After the slab has been installed, 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) has caused cracks in concrete slabs, even when design and construction precautions were taken. Inspection and testing of controlled fills should be mandatory. The lack of uniformity of support is a cause of slab cracks. The importance of providing uniform support cannot be overemphasized.

3.4.8 *Influence of size of loaded area*—The k value, if 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. Although this may be sufficient for the analyses of floor slabs subjected to relatively small concentrated loads, 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) could load and consolidate soils to depths of 30 ft (9.1 m) or more if fills have been used to develop the site. Settlement of slabs is not uncommon on sites where fills have been used to produce dock height floors or promote area drainage. The degree of settlement experienced under such a loading condition 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 evaluations may include the performance of soil test borings and 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, which account for long-term consolidation settlements under sustained heavy distributed loads.

3.4.9 *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 will be subject to load cycling. Applications of stresses from 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.

3.5—Design of slab-support system

3.5.1 *General*—After the subgrade soils have been classified, the general range of their k values can be approximated from Fig. 3.3. Adjustments may be made on the basis of local experience and expected seasonal changes as well as expected construction conditions.

With this information, a decision can be made whether to use the existing subgrade in its in-place condition, 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. Therefore, 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 a problem of 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; however, the selection of these materials has distinct advantages. Not only do they improve the modulus of subgrade reaction and produce more uniform support, but they also provide an all-weather working surface to speed construction during inclement weather.

3.5.2 *Economics and simplified design*—A prerequisite for the proper design of a slab-support system is identification of the subgrade material and conditions to which it will be exposed. Without this knowledge, neither the modulus of subgrade nor the potential volume change can be estimated. 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, and, therefore, the k value need not be known exactly. 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. The risk of slab failure increases the more the design is based on assumed conditions, but there are occasions where a simplified design approach is justified. These decisions are a matter of engineering judgment and economics.

Compounding safety factors may produce an overly conservative design. Inclusion of cumulative safety factors 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 safety factor is normally accounted only in the allowable flexural stress in the concrete slab.

3.5.3 *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, however, is extremely important for serviceable slab performance.

3.6—Site preparation

3.6.1 *Introduction*—Initially, the top layer of soil should be stripped of all organic material, debris, and frozen material. Normally, to produce a uniform support, the surface is stripped, tilled, and recompacted before the subbase is placed. Both hard and soft pockets of soil should be located by proof rolling or other means, removed, and replaced by 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 capacity. Sinkholes, expansive soils, highly compressible materials, or other subgrade problems, however, can influence the performance of the slab and should be examined in detail.

3.6.2 *Proof rolling*—As is discussed in ACI 302.1R, proof rolling usually refers to the use of a loaded vehicle driven in a grid pattern over the subgrade in an effort to locate soft and compressible areas at or close to the surface. This should be a part of the process for quality assurance for the soil-support system, and should be set forth 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 of the surface. Rutting normally indicates excess moisture at the surface. Pumping indicates subgrade soils wet of the optimum moisture 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; however, guidelines are given in ACI 302.1R. If 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, then proof rolling may not be able to detect any soft or compressible areas under the surface. On some projects, proof rolling is employed three times: after stripping (before any fill is placed); after the fill has been installed; and after the base course is placed. To locate suspected deeper soft areas or buried debris, borings, test pits, resistivity, or other procedures may be needed. Proof rolling should be scheduled to permit remedial work to be performed without interfering with the construction schedule.

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

Weak subgrade material can be stabilized by the addition of chemicals that are combined with the soil, as shown in Table 3.3. 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. The maximum dry density and optimum moisture content values vary with texture and plasticity. This is illustrated by Fig. 3.4 for standard Proctor tests (ASTM D 698) on eight different soils.

Because the modified Proctor test (ASTM D 1557) uses a higher level of energy, the maximum dry density will be higher and the optimum moisture content will be lower than that of the standard Proctor test. Furthermore, the difference will vary with the texture and plasticity of the soil. This is shown in Fig. 3.5.

Table 3.3—Soil stabilization with chemical admixtures

Admixture	Quantity, % 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.	Cohesive soil is pulverized so that at least 80% will pass No. 4 (4.75 mm) sieve, mixed with cement, moistened to between optimum and 2% wet, compacted to at least 95% maximum density and cured for 7 or 8 days while moistened with light sprinkling or protected 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	3 to 5% bitumen in the form of cutback asphalt emulsion, or liquid tars for sandy soils. 6 to 8% asphalt emulsions and light tars for fine-grained materials. For coarse-grained soils, antistrip compounds are added to promote particle coating by bitumen.	Soil is pulverized, mixed with bitumen, solvent is aerated, and mixture compacted. 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.
Calcium chloride	1/2 to 1-1/2%	Normally applied at a rate of approximately 0.5 lb/yd^2 (0.27 kg/m^2) area. Dry chemical is blended with soil aggregate mixture, water added, and mixtured compacted at optimum moisture by conventional compaction procedures.	Used as dust palliative. Stabilized mixture of gravel-soil binder calcium chloride forms wearing surface in some secondary roads.	Retards rate of moisture evaporation from the stabilized mixture, tends to reduce soil plasticity. Greatest effect in sodium clays with capacity for base exchange. Lowers freezing point of soil water, decreasing loss in strength from freezing and thawing.
Lime	4 to 8%. Fly ash, between 10 and 20%, may be added to increase pozzolanic reaction.	Lime is spread dry, mixed with soil by pulvimixers or discs, moisture compacted 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).

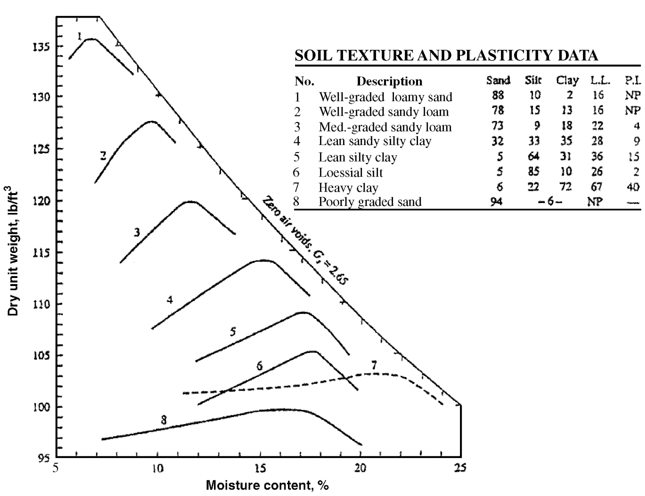


Fig. 3.4—Standard Proctor curves for various soils. (Note: 1 lb/ft^3 = 0.1571 kN/m^3.)

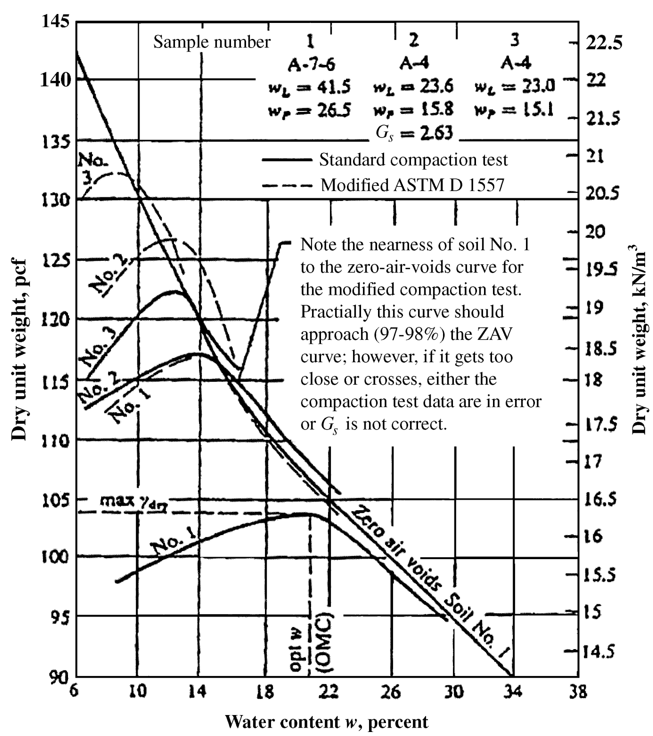


Fig. 3.5—Standard and modified Proctor curves.

Specifications are frequently adopted to control 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, however, 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 type of soil, uniformity of soil, 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, a moisture content range is also specified, for example, within ±3% of the optimum moisture content of the appropriate test. Higher moisture contents, from optimum moisture content to 4% above it, are frequently used to minimize volume changes.

3.6.4 *Subbase and base materials*—For many slabs-on-ground, the existing subgrade will provide adequate support.

Generally, the materials listed in Fig. 3.3 that yield a standard modulus of subgrade reaction above 100 lb/in.^{3} (3000 kN/m^3) can be used. Highly compressible organic materials (OL) should be avoided, and high-plasticity clays (CH) may cause heave or swell problems. Much of the variation in support capacity is the result of compaction and moisture content; for example, the k value for a 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. Thus, the use of a subbase in combination with a base course usually represents an economical alternative for construction on a poor subgrade with an expensive base course material. The subbase may be composed of stabilized subgrade soil, a fill of better quality soil, sand, crushed rock, reclaimed crushed concrete or asphalt pavement, or some local material that has properties that satisfy the requirements of the project.

Normally, the materials selected for base materials are alluvial sands and gravels (S or G) or crushed rock. These materials are easily compacted and have high strengths and low compressibilities. If 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 the support of the slab and the overall k value depends on the type and thickness of the base material, as is depicted in Fig. 3.6. Data for specific designs should be based on an analysis of laboratory and site-testing results. If an open-graded, crushed rock is used, 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.

3.6.5 *Stabilization of base and subbase*—Base and subbase materials are often densified by mechanical compaction to improve the k value. The relative cost of possible alternatives, such as chemical stabilization of the subgrade, use of high-quality base courses, or providing a thicker slab, should be considered.

The mechanical compaction of clay and silt is measured as a percent of standard Proctor density (ASTM D 698) or modified Proctor density (ASTM D 1557). Minimum dry densities typically specified for these materials are from 90 to 95% of the maximum dry densities of the standard and modified tests, respectively.

3.6.6 *Grading tolerance*—Usually, compliance with the initial rough- and fine-grading tolerance is based on level surveys 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 302.1R, Chapter 4.

3.6.7 *Vapor retarder/barrier* — Because 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. Manufacturers of these coverings 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 3.7 provides guidance.

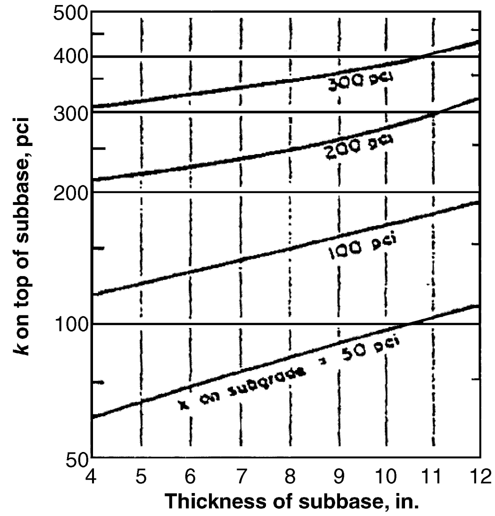
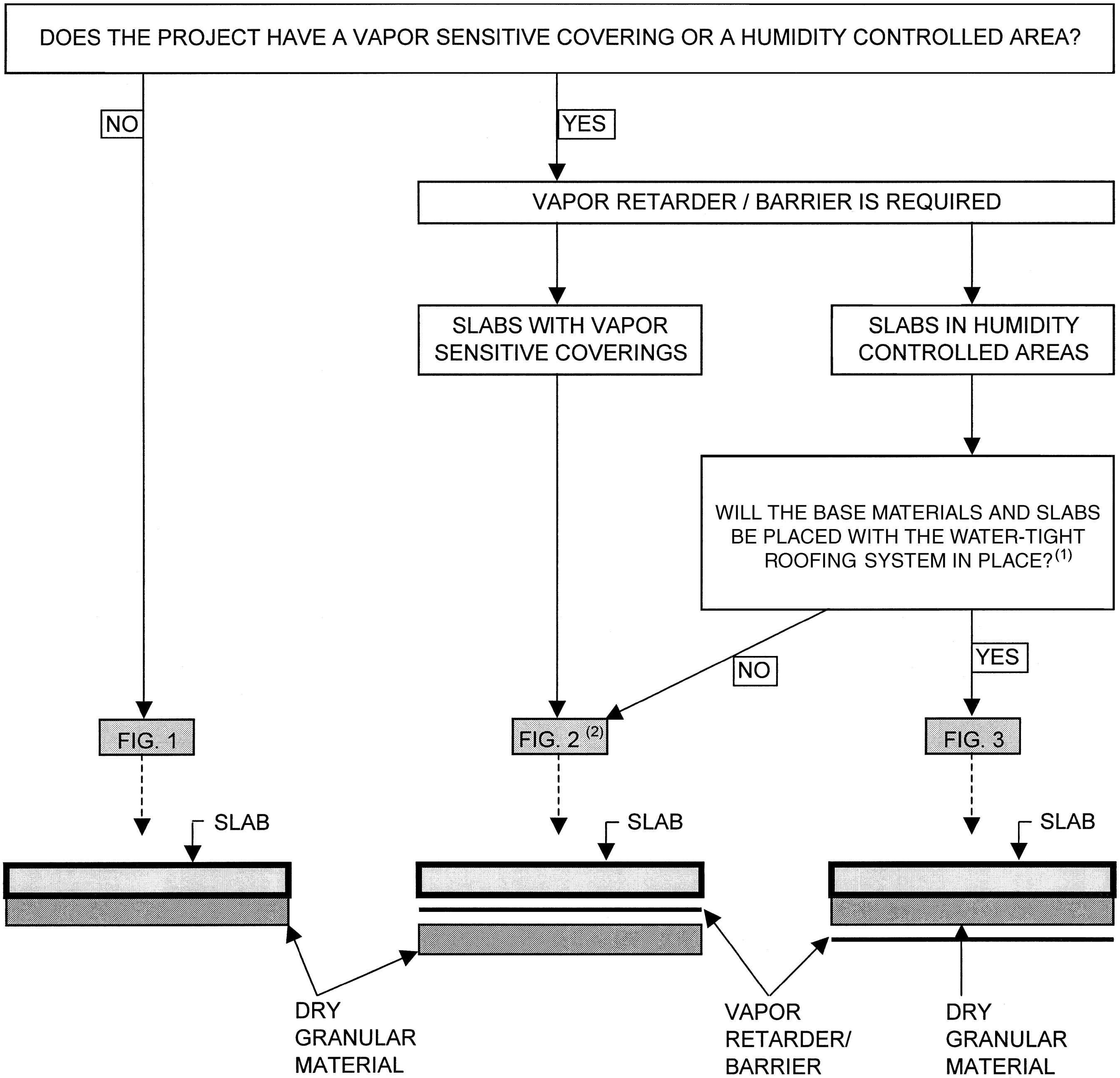


Fig. 3.6—Effect of subbase thickness on design modulus of subgrade reaction. (Note: 1 pci = 0.2714 MN/m^3; 1 in. = 25.4 mm.)

Excess water in the slab not taken up by chemical action will evaporate through the top of the slab until equilibrium is reached with ambient humidity. Additionally, moisture can transpire from the subgrade and through the slab. If 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. If hydrostatic forces can occur, they must be included in the slab design considerations. The amount of flow will depend on the amount of head and the width, length, and frequency of the joints and cracks in the concrete. If the base material is saturated or near saturation and there is no head, moisture can still be transmitted 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. Although vapor retarders/barriers can substantially reduce vapor transmission through slabs, some water vapor will transpire through the slab if 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 humidity and from warm to cold temperatures. The temperature of the soil subgrade 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 of plants and lawns adjacent to the slabs-on-ground, or from



NOTES:

(1) IF GRANULAR MATERIAL IS SUBJECT TO FUTURE MOISTURE INFILTRATION, USE FIG. 2.
(2) IF FIGURE 2 IS USED, A REDUCED JOINT SPACING, A LOW SHRINKAGE MIX DESIGN, OR OTHER MEASURES TO MINIMIZE SLAB CURL WILL LIKELY BE REQUIRED.

Fig. 3.7—Decision flowchart to determine if a vapor retarder/barrier is required and where it is to be placed.

broken pipes in the subgrade. Because there can be a variety of sources of moisture, there is likely to be 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 D 4263 will detect the presence of moisture coming through the slab, but it will not yield a rate of moisture movement. A quantitative test method, ASTM F 1869, uses a desiccant calcium chloride beneath an impermeable dome over a small slab area to calculate the moisture emission rate. These test results, however, may be misleading if the ambient air conditions do not represent those for in-service conditions. ASTM F 1869 requires 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. In addition, the test has been found to measure only the moisture in the top 1/2 in. (13 mm) of the slab, and cannot detect moisture below a depth of 3/4 in. (19 mm). To better quantify moisture in slabs, ASTM F 2170 was developed for the use of relative humidity probes.

Drainage of the subgrade and the selection of subgrade materials will have a great influence on the performance of vapor retarders/barriers. Also, protection of the vapor retarders/barriers from damage during construction can significantly influence the retarder/barrier's effectiveness. Vapor retarders/barriers have been reported to affect the behavior of the concrete 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 emission from the slab surface.

3.7—Inspection and site testing of slab support

Inspection and testing are required to control the quality of the subgrade and subbase construction and to determine if it conforms to the project specifications. Before construction begins, the subgrade soils and any subbase or base-course materials should be sampled, tested in the laboratory, and the results evaluated. In general, particle size (ASTM D 422), plasticity (ASTM D 4318), and laboratory compaction tests (ASTM D 698 or 1557) are performed on soils and soil-aggregate mixtures. For cohesionless and free-draining soils and aggregates gradation, determination of maximum relative density (ASTM D 4253) and minimum relative density (ASTM D 4254) and calculation of relative density may be appropriate. After compaction, the in-place density to calculate the percent compaction can be determined in the field by any of several methods: drive cylinder (ASTM D 2937); sand cone (ASTM D 1556); water balloon (ASTM D 2167); or nuclear densometer (ASTM D 2922 and D 3017). Although the sand cone test is the most accepted method, the nondestructive nuclear density method is advantageous because it is accomplished in a few minutes, and the results are available at the end of the field test. Because this allows the field density and moisture contents to be used to control construction, the nuclear density method is widely used and accepted. 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, a series of calibration tests are run. The moisture readings must also be checked against field moisture tests (ASTM D 566). Testing frequency is related to the uniformity of the materials being 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 the installation of the remainder of the slab-on-ground system. Minimum testing requirements should be established for each project. These should provide a reasonable test interval to be taken in each lift. These tests are relatively inexpensive and easy to perform.

3.8—Special slab-on-ground support problem

Placement of slabs on topsoil should generally be avoided because of their low shear strength and high compressibility. Project specifications generally require that the building site be stripped of all topsoil.

Expansive soils are defined as fine-grained soils, as shown in Tables 3.1 and 3.2. Soils with a plasticity index of 20 or higher have a potential for significant volume change. A geotechnical engineer should examine the soil data and recommend appropriate options. Potential problems can be minimized by proper slab designs, stabilization of the soil, or by preventing moisture migration through the slab. Failure to manage the problem 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 changes of volume, and consequently heave due to the growth of ice lenses when subjected to freezing cycles and loss of support due to saturation upon thawing. Three conditions must be present for this problem to occur:

- Freezing temperature in the soil;
- Water table close to the frost level to provide water for the formation of ice lenses; and
- 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/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 4—LOADS

4.1—Introduction

This chapter 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 combination of the following loads and effects:

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

Slabs should be designed for the most critical combination of these loadings, considering such variables that produce the maximum stress. The PCA guide for selecting the most critical or controlling design considerations for various loads (Packard 1976) is presented in Fig. 4.1. Because a number of factors, such as slab thickness, concrete strength, subgrade stiffness, and loadings, are relevant, cases where several design considerations may control should be investigated thoroughly.

Other potential problems, 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 that were once considered uniform loads may now be stored in narrow-aisle pallet racks that produce concentrated loads. The environmental

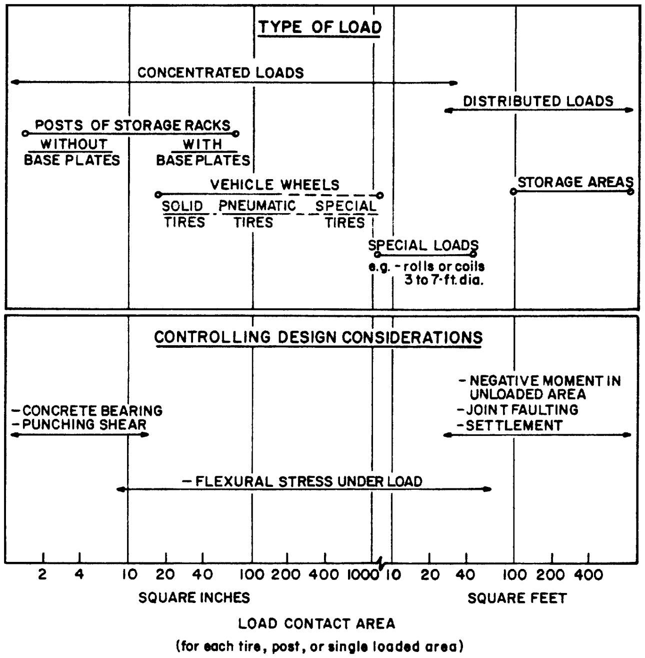


Fig. 4.1—Controlling design considerations for various types of slab-on-ground loading (Packard 1976). (Note: 1 in.^{2} = 645.2 mm^2; 1 ft^2 = 0.09290 m^2.)

exposure of the slab-on-ground is also a concern. These effects include subgrade volume changes (shrink/swell soils) and temperature changes. Normally, thermal effects may be minimized by constructing the slab after the building is enclosed. Many slabs, however, are placed before building enclosure. Therefore, the construction sequence is important in determining whether transient environmental factors should be considered in the design. Finally, thermal effect due to in-service conditions should be considered.

4.2—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. The Industrial Truck Association (1985) has compiled representative load and geometry data for lift truck capacities up to 20,000 lb (89 kN) (Table 4.1).

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. If vehicle details are unknown or if the lift truck capacity is expected to change in the future, the values in Table 4.1 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 helps the designer to quantify fatigue. Whether these values are predictable or constant during the service life of a slab should also be considered.

Table 4.1—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

Note: The concentrated reaction per tire is calculated by dividing the total axle load reaction by the number of tires on that axle. Figures givens are for standard trucks. The application of attachments and extended high lifts may increase these values. In such cases, the manufacturer should be consulted. 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.

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), while steel-cord tire pressures range from 90 to 120 psi (0.6 to 0.8 MPa). The Industrial Truck Association found that the standard solid and cushion solid rubber tires have floor contact areas that may be based on internal pressures between 180 and 250 psi (1.2 to 1.7 MPa) (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).

Dual tires have an effective contact area greater than the actual contact area of the two individual tires. There are charts available to determine this effective contact area (Packard 1976). A conservative estimate of this effective contact area, however, can be made using the contact area of the two tires and the area between the contact area. If it is not known whether the vehicle will have 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 and prevent edge spalling as a vehicle travels across the joint. Refer to Chapter 5 for more information and joint details.

4.3—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 the use of 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 will affect the deeper soil layers), and racks with long-term loading, the effect of the long-term soil settlement should also be considered in the design of the slab. Cracking can also be caused by early installation of rack systems that may restrain the slab 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.

The concentrated load variables that affect design of the slab-on-ground are:

- 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 or post 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 over the plate area.

4.4—Distributed loads

In many warehouse and industrial buildings, materials are stored directly on the slab-on-ground. The flexural stresses in the slab are usually less than those produced by concentrated loads. The design should prevent negative moment cracks in the aisles and prevent excessive settlement. For the higher load intensities, distributed loads that cover a large plan area (which will affect the deeper soil layers), and long-term uniform loads, the effect of the differential soil settlement should also be considered in the design of the slab. The effect of a lift truck operating in the 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;
- Duration of load;
- 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).

4.5—Line and strip loads

A line or strip load is a uniform load distributed over a relatively narrow area. A load may be considered to be a line or strip load if its width is less than 1/3 of the radius of relative stiffness of the slab. When the width approximates this limit, the slab should be reviewed for stresses produced by line loading and uniform load. If the results are within 15% of one another, the load should be taken as uniform. Partition loads, bearing walls, and roll storage are examples of this load type. For higher load intensities and long-term loading, the effect of differential soil settlement should also be considered 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 and duration of load;
- Width, length of loaded area, and if 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 (this is especially important when the line load crosses perpendicular to a joint or is directly adjacent and parallel to a joint).

4.6—Unusual loads

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

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

The load variables, however, will be similar to those for the load types previously discussed in this chapter.

4.7—Construction loads

During the construction of a building, various types of equipment may be located on the newly placed slab-on-ground. The most common construction loads are pickup trucks, scissor lift concrete trucks, dump trucks, hoisting equipment and cranes used for steel erection, tilt wall erection, and setting equipment. In addition, the slab may be subjected to other loads, such as scaffolding and material pallets. Some of these loads can exceed the design limits and, therefore, the construction load case should be anticipated, particularly relative to early-age concrete strength. Also, limiting of construction loads near the free edges or corners of slabs should be considered. 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 reference to local transportation laws or to the AASHTO standards. Off-road construction equipment may exceed these limits, but in most cases, depending on local custom, construction equipment will not exceed the legal limits of the Department of Transportation. Figure 4.2 gives values of contact area for wheel loads that can be used for design.

4.8—Environmental factors

Flexural stresses produced by thermal changes, expansive soils, and moisture changes in the slab (affecting curling due to the different shrinkage rates between the top and bottom of the slab) should be considered in the overall design. These

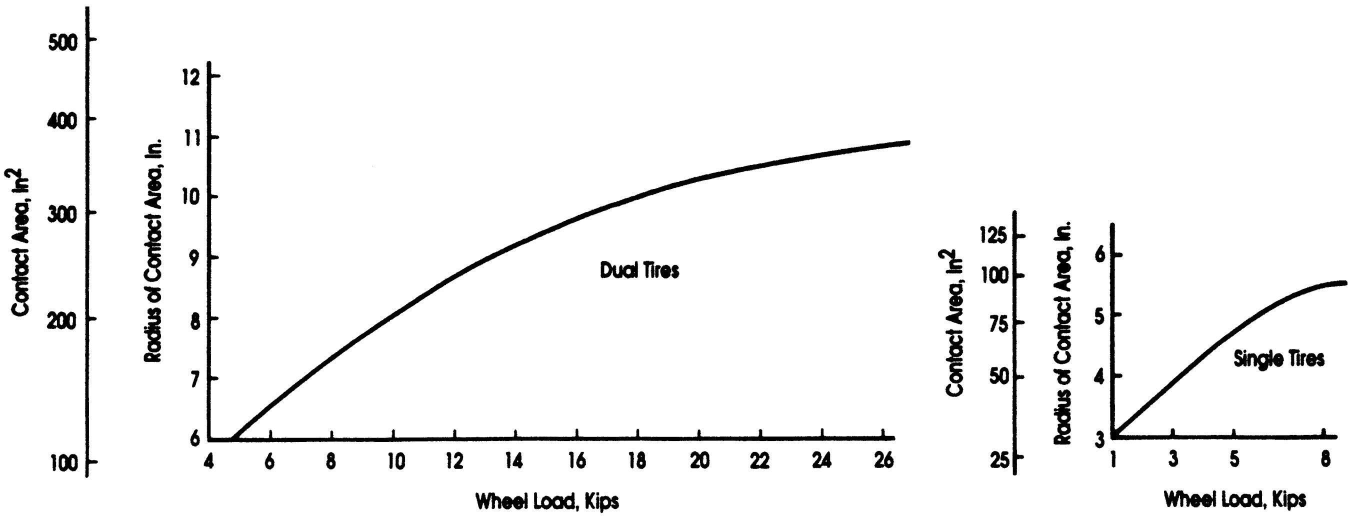


Fig. 4.2—Tire contact area for various wheel loads. (Note: 1 in. = 25.4 mm; 1 in.^{2} = 645.2 mm^2; 1 kip = 4.448 kN.)

Table 4.2—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+
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	—

*When a line load is considered to be a structural load due to building function, appropriate building code requirements must be followed.

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 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. There are several variables that would affect how short a slab this would be, but 30 times the slab thickness is a generally conservative joint spacing for most conditions. Built-in restraints (such as foundation elements, edge walls, and pits) should be avoided. Reinforcement should be provided at such restraints to limit the width of the cracks in the slab. Thermal and moisture effects are discussed further in Chapter 13, and expansive soils are discussed further in Chapter 9.

4.9—Factors of safety

Slabs-on-ground are distinguished from other structural elements by unique serviceability requirements. Some of these serviceability requirements minimize cracking and curling, increase surface durability, optimize joint locations and type of joints for joint stability (the differential deflection of the adjacent slab panels edges as wheel loads cross the joint) and maximize long-term flatness and levelness. Because the building codes primarily provide guidance to prevent catastrophic failures that would affect the public safety, the factors of safety for serviceability, while inherent in building codes, are not directly addressed as are those for strength. If 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 the load case.

The factor of safety to minimize the likelihood of a serviceability failure is selected by the designer. Some of the items the designer should consider in selecting the factor of safety are the following:

- Consequences of serviceability failure, including lost productivity, lost beneficial use, and the costs for repairing areas in an active facility. For example, crack frequency should be minimized and crack widths should be limited for facilities such as pharmaceutical and food processing facilities;
- Concrete mixture proportion and its shrinkage characteristics (shrinkage should be tested and minimized to reduce linear drying shrinkage and curling);
- Humidity-controlled environment that will increase linear drying shrinkage and curling of the slab;
- Subgrade smoothness and planeness to minimize restraint as linear drying shrinkage takes place;
- Spacing and type of joints;
- Geotechnical investigation to determine the shallow and deep properties of the soil;
- Number of load repetitions to allow consideration of fatigue cracking;
- Impact effects; and
- Storage racks installed at an early stage, which will restrain linear drying shrinkage.

Some commonly used safety factors are shown in Table 4.2 for the various types of slab loadings. Most range from 1.7 to 2.0, although factors as low as 1.4 are used for some conditions.

A moving vehicle subjects the slab-on-ground to the effect of fatigue. Fatigue strength is expressed as the percentage of

Table 4.3—Stress ratio versus allowable load repetitions (PCA fatigue curve)*

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

*_Thickness Design for Concrete Highway and Stress Pavements_, EB109.01P, Portland Cement Association, Skokie, Ill. (1984).

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 according to PCA (2001). Table 4.3 shows various load repetitions for a range of stress ratios. The safety factor is the inverse of the stress ratio.

CHAPTER 5—JOINTS

5.1—Introduction

Joints are used in slab-on-ground construction to limit the frequency and width of random cracks caused by volume changes. Generally, if 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. If the joint layout is not provided, the contractor should submit a detailed joint layout and placing sequence for approval by the designer before proceeding with construction.

Every effort should be made to avoid tying the slab to any other element of the structure. Restraint from any source,

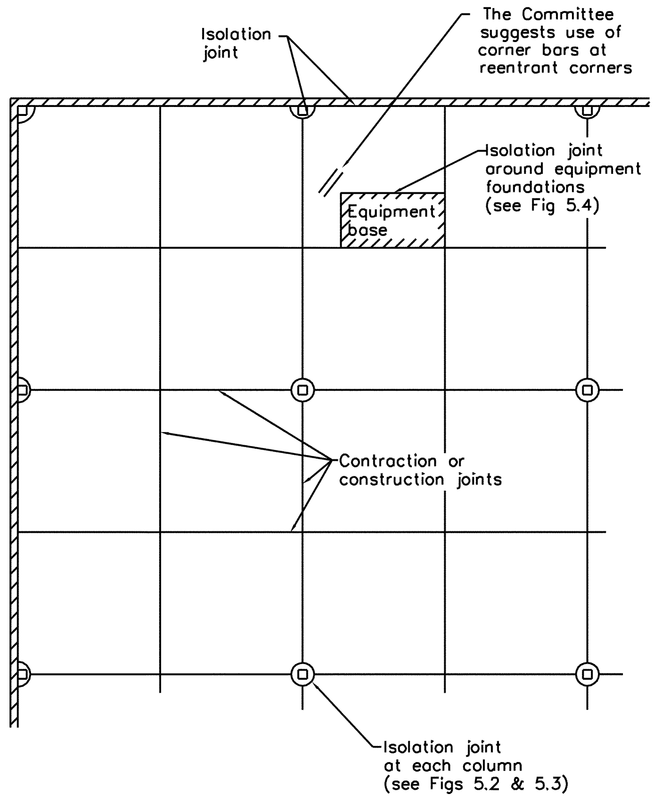


Fig. 5.1—Appropriate locations for joints.

whether internal or external, will increase the potential for random cracking.

Three types of joints are commonly used in concrete slabs-on-ground: isolation joints, sawcut contraction joints, and construction joints. Appropriate locations for isolation joints and sawcut contraction joints are shown in Fig. 5.1. With the designer's approval, construction joint and sawcut contraction joint details can be interchanged. Joints in topping slabs should be located directly over joints in the base slab and, if 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, additional joints should be considered 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 stress 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 an unreinforced topping slab.

5.1.1 *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. The joint

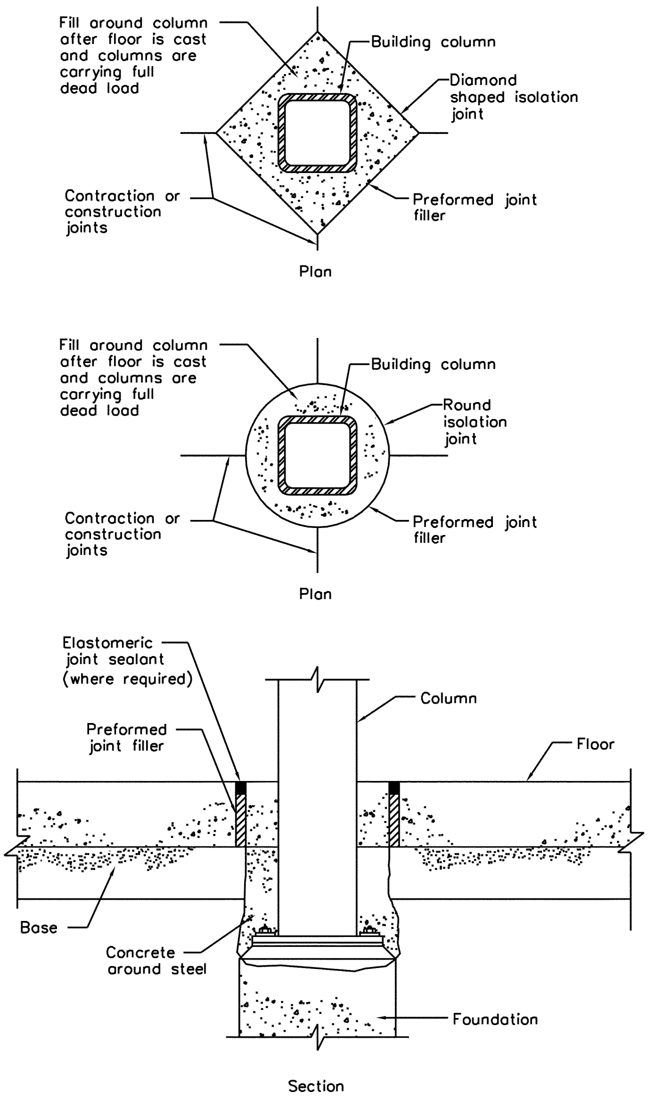


Fig. 5.2—Typical isolation joints at tube columns.

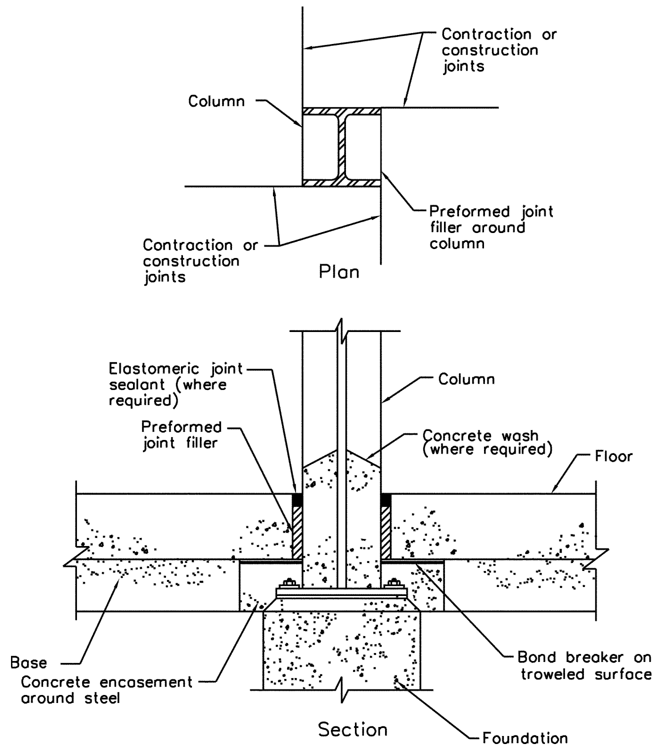


Fig. 5.3—Alternate (pinwheel) isolation joint and suggested concrete fill at wide flange columns.

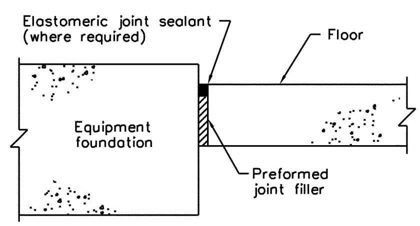


Fig. 5.4—Typical isolation joint around equipment foundation.

material should extend the full depth of the slab 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. Two methods of producing a relatively uniform depth of joint sealant are as follows:

1. Score both sides of the preformed filler at the depth to be removed using a saw. Insert the scored filler in the proper location and remove the top section after the concrete hardens by using a screwdriver or similar tool.

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.

Alternatively, a premolded joint filler with a removable top portion can be used. Refer to Fig. 5.2 and 5.3 for typical isolation joints around columns. Figure 5.4 shows an isolation joint at an equipment foundation.

Isolation joints for slabs using shrinkage-compensating concrete should be treated as recommended in ACI 223.

5.1.2 *Construction joints*—Construction joints are placed in a slab to define the extent of the individual placements, generally in conformity with a predetermined joint layout. If concreting is ever interrupted long enough for the placed concrete to harden, a construction joint should be used.

In areas not subjected to traffic, a butt joint may be adequate. In areas subjected to wheeled traffic, heavy loads, or both, joints with dowels are recommended (Fig. 5.5). A keyed joint is not recommended for load transfer 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 or bulkheads can be wood, metal, or precast concrete; they should be placed at the proper elevation with the necessary support required to keep the bulkheads straight, true, and firm during the entire placing and finishing procedure. If dowels are required, provisions should be made along the bulkhead to ensure proper alignment during construction and finishing operations. Dowel alignment devices should be rigidly attached to the bulkhead with nails

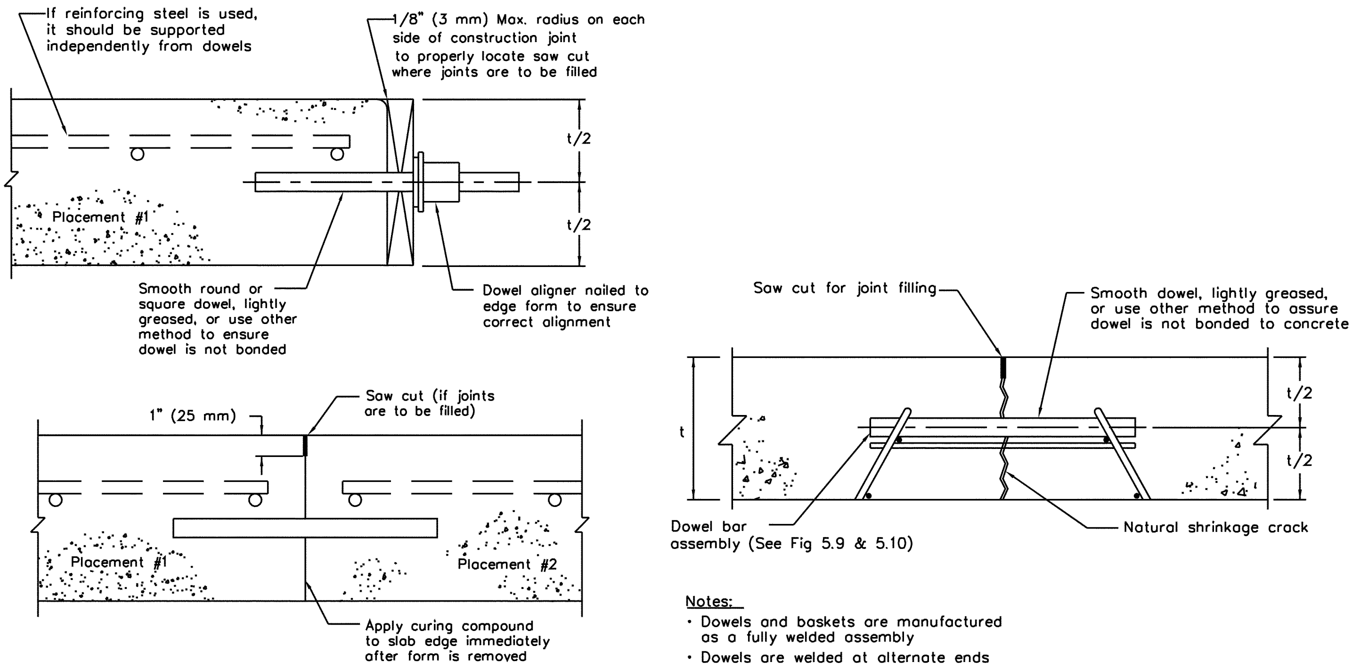


Fig. 5.5—Typical doweled joints.

or screws. These devices allow the dowel to be inserted through the bulkhead while maintaining the proper alignment of the dowel parallel to the surface and each other and perpendicular to the joint face. The dowels should be inserted into the dowel alignment device just prior to concreting operations to minimize disturbance during construction; refer to Fig. 5.5.

All construction joints should be internally vibrated at frequent intervals to properly consolidate and densify the concrete at the joint and around the dowels. Vibratory screeds, laser-guided screeds, and hand-rodding techniques do not provide sufficient internal vibration. This is particularly imperative when large, top-size coarse aggregate concrete is used adjacent of armored joints, round and square dowels, or diamond-shaped load plates.

5.1.3 *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, as shown in Fig. 5.1.

The following factors are normally considered when selecting spacing of sawcut contraction joints:

- Method of slab design;
- Thickness of slab;
- Type, amount, and location of reinforcement;
- Shrinkage potential of the concrete (cement type and quantity; aggregate size and gradation, quantity, and quality; *w/cm*; type of admixtures; and concrete temperature);
- Base friction;
- Floor slab restraints;
- Layout of foundations, racks, pits, equipment pads, trenches, and similar floor discontinuities;
- Environmental factors such as temperature, wind, and humidity; and
- Methods and quality of concrete curing.

As previously indicated, establishing slab joint spacing, thickness, and reinforcement requirements is the responsibility of the designer. The specified 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, Fig. 5.6 provides recommendations for joint spacing based on shrinkage values as determined by ACI 209R.

Sawcut contraction joints should be continuous across intersecting joints, not staggered or offset. The exception to this rule would be the column isolation joint, shown in Fig. 5.3. 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 5.7 shows one method that can be used to minimize slab cracking at reentrant corners of loading docks.

Plastic or metal inserts are not recommended for creating a sawcut contraction joint in any exposed floor surface that will be subjected to wheeled traffic.

Sawcut contraction joints in industrial and commercial floors are usually formed by sawing a continuous slot in the slab to form a weakened plane below which a crack will form (Fig. 5.8). Further details on sawcutting of joints are given in Section 5.3.

5.2—Load-transfer mechanisms

Doweled joints (Fig. 5.5) are recommended when positive load transfer is required, unless a sufficient amount of post-tensioning force is provided across the joint to transfer the

Table 5.1—Dowel size and spacing for round, square, and rectangular dowels*

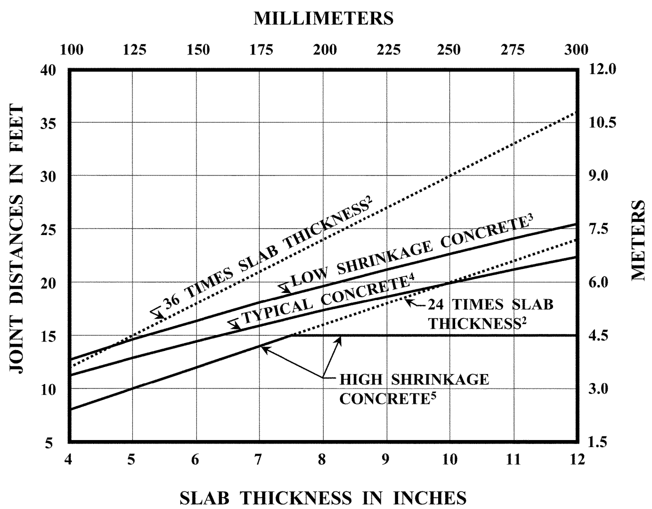
Slab depth, in. (mm)	Dowel dimensions,† in. (mm)			Dowel spacing center-to center, in. (mm)		
	Round	Square	Rectangular‡	Round	Square	Rectangular
5 to 6 (130 to 150)	3/4 x 14 (19 x 360)	3/4 x 14 (19 x 360)	3/8 x 2 x 12 (9 x 51 x 300)	12 (300)	14 (360)	19 (480)
7 to 8 (180 to 200)	1 x 16 (25 x 410)	1 x16 (25 x 410)	1/2 x 2-1/2 x 12 (13 x 64 x 300)	12 (300)	14 (360)	18 (460)
9 to 11 (230 to 280)	1-1/4 x 18 (32 x 460)	1-1/4 x 18 (30 x 450)	3/4 x 2-1/2 x 12 (19 x 64 x 300)	12 (300)	12 (300)	18 (460)

*ACI Committee 325 (1956); Walker and Holland (1998).

†Total dowel length includes allowance made for joint opening and minor errors in positioning dowels.

‡Rectangular plates are typically used in sawcut contraction joints.

Notes: Table values based on a maximum joint opening of 0.20 in. (5 mm). Dowels must be carefully aligned and supported during concrete operations. Misaligned dowels may lead to cracking.



NOTES:

1. Joint spacing recommendations based on reducing the curling stresses to minimize mid- panel cracking (Walker-Holland 2001). See discussion in Section 5.2 for joint spacing for aggregate interlock.
2. Joint spacing criteria of 36 and 24 times the slab thickness which has been utilized in the past is shown for reference.
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. 5.6—Recommended joint spacing for unreinforced slabs.

shear. Dowels force concrete on both sides of a joint to deflect approximately equally when subjected to a load and help prevent damage to an exposed edge when the joint is subjected to wheeled traffic. Table 5.1 provides recommended dowel sizes and spacing for round, square, and rectangular dowels.

For dowels to be effective, they should be smooth, aligned, and supported so they will remain parallel in both the horizontal and the vertical planes during the placing and finishing operation. All dowels should have sawn and deburred end edges. Properly aligned, smooth dowels allow the joint to open as concrete shrinks.

Dowel baskets (Fig. 5.9 and 5.10) should be used to maintain alignment of dowels in sawcut contraction joints and alignment devices, similar to what is shown in Fig. 5.5, should be incorporated into the bulkhead of construction joints. In exterior slabs, wet conditions, or corrosive environments, the designer should consider corrosion protection for the dowels. Round

Table 5.2—Dowel size and spacing for diamond-shaped load plates (Walker and Holland 1998)

Slab depth, in. (mm)	Diamond load plate dimensions, in. (mm)	Diamond load plate spacing center-to-center, in. (mm)
5 to 6 (130 to 150)	1/4 x 4-1/2 x 4-1/2 (6 x 110 x 110)	18 (460)
7 to 8 (180 to 200)	3/8 x 4-1/2 x 4-1/2 (9 x 110 x 110)	18 (460)
9 to 11 (230 to 280)	3/4 x 4-1/2 x 4-1/2 (19 x 110 x 110)	20 (510)

Note: Table values based on maximum joint opening of 0.20 in. (5 mm). Construction tolerances required make it impractical to use diamond-shaped load plates in sawcut contraction joints.

dowels should be placed no closer than 12 in. (300 mm) from the intersection of any joints because the maximum movement caused by curling and dry shrinkage occurs at this point, and the corner of the slab may consequently crack.

Diamond-shaped load plates (a square plate turned so that two corners align with the joint) can be used to replace dowels in construction joints. The diamond shape allows the slab to move horizontally without restraint when the slab shrinkage opens the joint (Fig. 5.11). Table 5.2 provides the recommended size and spacing of diamond-shaped load plates. Square and rectangular dowels cushioned on the vertical sides by a compressible material also permit horizontal movement parallel and perpendicular to the joint (Fig. 5.12). These types of load-transfer devices are useful in other slab types where the joint should have load-transfer capability while allowing some differential movement in the direction of the joint, such as might be necessary in post-tensioned and shrinkage-compensating concrete slabs, or in slabs with two-directional doweling (Schrader 1987, 1991; PTI 2000; Ringo and Anderson 1992; Metzger 1996; Walker and Holland 1998; American Concrete Paving Association 1992). These types of load-transfer devices may be placed within 6 in. (150 mm) of a joint intersection (Fig. 5.12 and 5.13).

Less effective as a 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. The designers that choose to use aggregate interlock as the load-transfer mechanism at joints are cautioned that, for unreinforced concrete slabs, the joint spacings recommended in Fig. 5.6 are intended to minimize the potential for midpanel out-of-joint random cracking, and

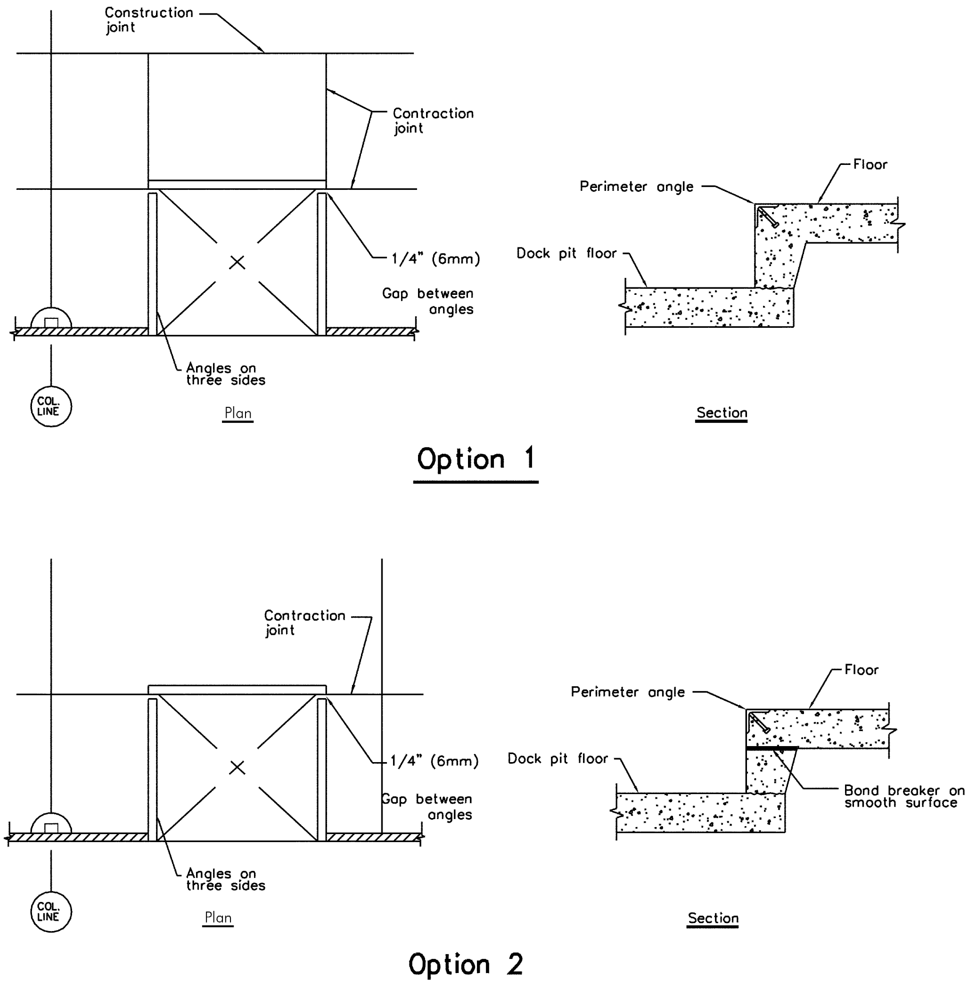


Fig. 5.7—Joint details at loading dock.

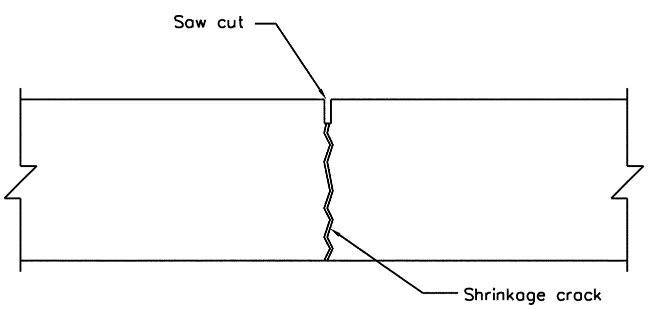


Fig. 5.8—Sawcut contraction joint.

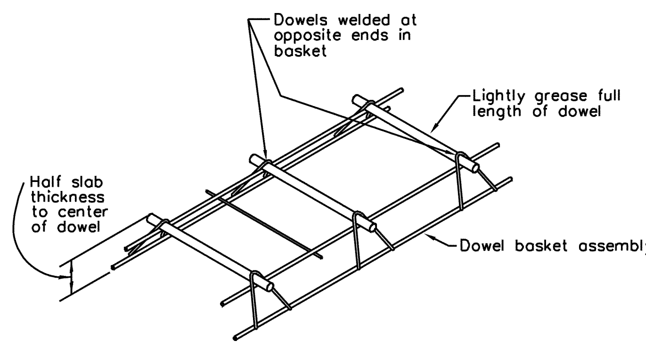


Fig. 5.9—Dowel basket assembly.

are independent of load transfer requirements at joints. Further, not all joints activate uniformly, resulting in some joint opening widths that are larger than might normally be anticipated. 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 along with the type of wheel loadings on the slab. Furthermore, if 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 wheeled 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 slabs that were 7 and 9 in. (180 and 230 mm) thick. The test slabs were constructed using 1-1/2 in. (38 mm) maximum-size aggregate, were fully supported on various base materials, and loaded using repetitive applications of a 9000 lb (40 kN) and 16 in. (440 mm) diameter pads centered 9 in. (230 mm) from the joints. Among the findings were the following:

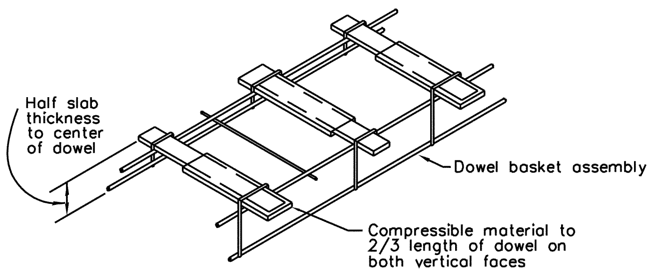


Fig. 5.10—Rectangular load plate basket assembly.

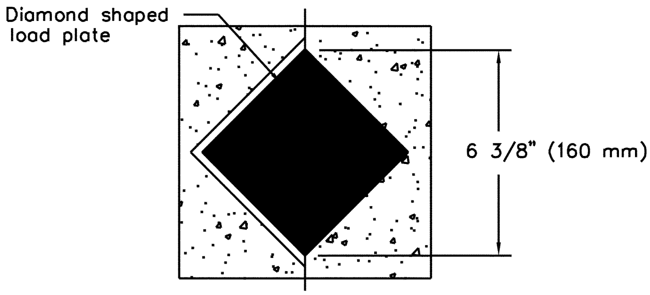


Fig. 5.11—Diamond-shaped load plate at construction joint.

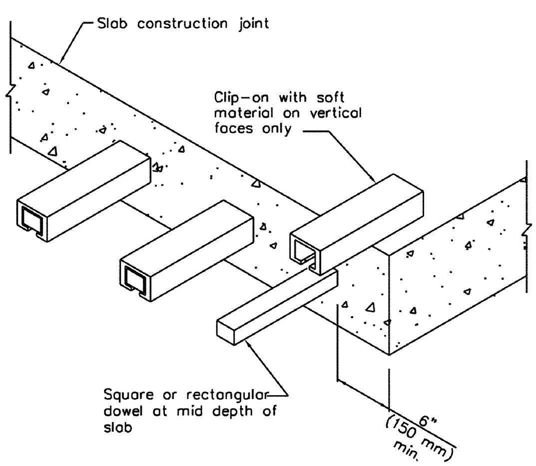


Fig. 5.12—Doweled joint detail for movement parallel and perpendicular to the joint.

1. Joint effectiveness for 7 in. (180 mm) thick slabs is reduced to 60% at an opening width of 0.025 in. (0.6 mm);

2. Joint effectiveness for 9 in. (230 mm) thick slabs is reduced to 60% at an opening width of 0.035 in. (0.9 mm);

3. 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

4. Joint effectiveness increased with increased aggregate particle angularity.

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%) through sawcut contraction joints, in combination with joint spacings shown in Fig. 5.6, 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 requirements:

1. Joints should be spaced per Fig. 5.6;

2. The reinforcement should be placed above mid-depth but low enough that the sawcut will not cut the reinforcement;

3. A construction or a smooth doweled sawcut contraction joint should be placed at a maximum of 125 ft (38 m). This will force activation at these joints if the other joints with the deformed reinforcement do not activate;

4. An early-entry saw should be used to cut all sawcut contraction joints; and

5. 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 will increase 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 5.3 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

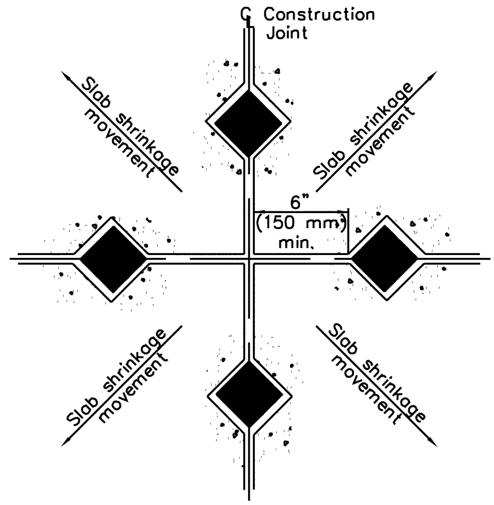


Fig. 5.13—Diamond-shaped load plates at slab corner.

Table 5.3—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.

slab is less than 3%. This percentage is relatively minor when compared with the potential impact of variations in the restraint stresses due to the different coefficients of subgrade friction (Fig. 13.3) and curling stresses.

Round, square, and rectangular smooth dowels for slab-on-ground installation should meet the requirements of ASTM A 36 or A 615. The diameter or cross-sectional area, length, shape, treatment for corrosion resistance, and specific location of dowels as well as the method of support should be specified by the designer. Refer to Tables 5.1 and 5.2 and Fig. 5.9 through 5.13.

For long post-tensioned floor strips and floors using shrinkage-compensating concrete with long joint spacing, care should be taken to accommodate significant slab movements. In most instances, post-tensioned slab joints are associated with a jacking gap. The filling of jacking gaps should be delayed as long as possible to accommodate shrinkage and creep. In traffic areas, armor plating of the joint edges is recommended (Fig 5.14). A doweled joint detail at a jacking gap in a post-tensioned slab (PTI 1996, 2000) is shown in Fig 5.15.

5.3—Sawcut contraction joints

The following three families of tools can be 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 will vary 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, thus creating the desired visual effect.

Conventional wet-cut saws are gasoline-powered and, with the proper blades, are capable of cutting joints up to 12 in. (300 mm) depth or more. Both types of dry-cut tools can use either 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). Early-entry dry-cut saws, however, which can cut to a maximum depth of 4 in. (100 mm), are now available. The timing of the early-entry process allows joints to be in place before development of significant tensile stresses in

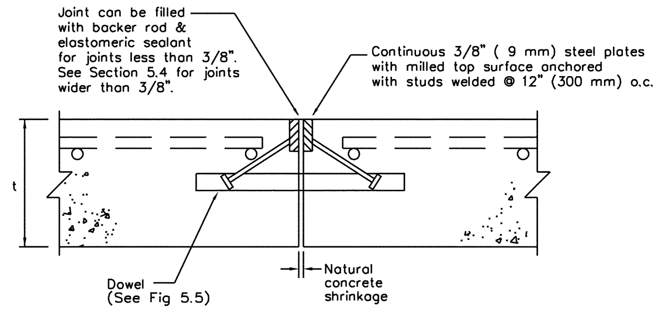


Fig. 5.14—Typical armored construction joint detail.

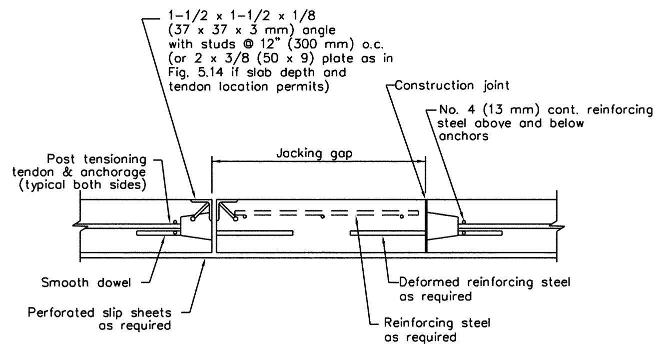


Fig. 5.15—Typical doweled joint detail for post-tensioned slab.

the concrete; this increases the probability of cracks forming at the joint when sufficient stresses are developed in the concrete. Care should be taken to make sure that the early-entry saw does not ride up over hard or large coarse aggregate. The highest coarse aggregate should be notched by the saw to ensure the 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 depth of sawcut using a wet conventional saw should be at least 1/4 of the slab depth or a minimum of 1 in. (25 mm), whichever is greater. The depth of sawcut using an early-entry dry-cut saw should be 1 in. (25 mm) minimum 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 are requiring that the slab be cut the following day to 1/4 of the slab depth to deepen the 1 in. (25 mm) nominal early-entry sawcut and ensure that the joint is activated. Restricted joint activation using a nominal 1 in. (25 mm) sawcut is a particular concern in doweled joints, where the dowels may restrain the movement of the slab. For this situation, square or rectangular dowels cushioned on the vertical sides by a compressible material are available in dowel basket assemblies and can reduce this restraint (Fig. 5.10 and 5.12).

For slabs that contain steel fibers, the sawcut using the conventional saw should be 1/3 of the slab depth. Typically, experience has shown that, 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 1-1/2 ± 1/4 in. (38 ± 6 mm) for higher fiber concentrations up to a 9 in. (230 mm) thick slab. 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 start building up in the concrete as it sets and cools. If 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.

5.4—Joint protection

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

For wheeled 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 angles or plates. 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 D 2240. Refer to Section 5.5 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, or slabs cast with shrinkage-compensating concrete, it is recommended that the joints be protected with back-to-back steel angles (Fig. 5.15) or bars, as shown in Fig. 5.14. It is critical that the top surfaces of the angles or bars used to armor be true. Milling may be required to produce a flat surface if conventional rolled shapes or bar stock is used for this purpose. Steel-armored joints less than 3/8 in. (9 mm) in width can be sealed with an elastomeric sealant as described in ACI 504R. Armored joints where width is 3/8 in. (9 mm) or greater should be filled full depth with semirigid epoxy or polyurea joint filler, or with a joint filler with an integral sand extender to provide a smooth transition for wheel traffic.

Construction and sawcut contraction joints that are unstable will not retain any type of joint filler. Joints are unstable if 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/concrete interface to failure. Joint edge protection provided by supportive filler is increased when load-transfer provisions are incorporated in the joint design.

5.5—Joint filling and sealing

Where there are wet conditions, hygienic and dust control requirements, and the slab is not subjected to wheel traffic, contraction and construction joints can be filled with joint filler or an elastomeric joint sealant. Joints subjected to wheeled traffic should be treated as discussed in Section 5.4.

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 that will be subjected to vehicular traffic unless protected with steel armored edges. Refer to ACI 504R for more information on elastomeric sealants.

5.5.1 *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, if the building is equipped with an HVAC system, it should be run for 2 weeks before joint filling. This is especially important where joint fillers are used in traffic-bearing joints because such materials have minimal extensibility. If the joint should be filled before most of the shrinkage has occurred, separation should be expected 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. If construction traffic dictates that joints be filled early, provisions should be made to require that the contractor return at a pre-established date to complete the necessary work using the same manufacturer's product. Earlier filling will result in greater separation and will lead to the need for more substantial correction; this separation does not indicate a failure of the filler.

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 joint filling.

There should be an understanding between 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.

5.5.2 *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 depth in sawcut joints. Joints should be suitably cleaned to provide optimum contact between the filler or sealant and bare concrete. Vacuuming is recommended rather than blowing the joint out with compressed air. Dirt, debris, sawcuttings, curing compounds, and sealers should be removed. 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 6—DESIGN OF UNREINFORCED CONCRETE SLABS

6.1—Introduction

The thickness of unreinforced concrete slabs is determined as a plain concrete slab without reinforcement. Although the effects of any welded wire reinforcement, plain or deformed bars, post tensioning, steel fibers, or any other type of reinforcement are not considered, joints may be reinforced for load transfer across the joint. The slab is designed to remain uncracked due to loads applied on the slab surface. Normally, a safety factor of 1.4 to 2.0 is used 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 11). Consequently, cracking, joint instability, and surface character problems are considered to be serviceability issues and are 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 will continue as long as water, heat, or both, are being released to the surroundings. Moreover, because the drying-and-cooling rates at the top and bottom of the slab are dissimilar, the shrinkage will vary with the depth, causing the as-cast shape to be distorted and reduced in volume. Resistance to formation of this distorted shape introduces internal stresses in the concrete that, if unrelieved, may cause the concrete to crack.

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 (out-of-joint) random cracks and to maintain adequate joint stability. The slab's anticipated live loading will govern its thickness and cross-joint shear transfer requirements, while shrinkage considerations will dictate the maximum joint spacing.

Application of present technology permits only a reduction in cracking and curling due to restrained shrinkage, not their elimination. 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.

A jointed, unreinforced slab-on-ground design seeks to optimize the finished floor's serviceability by attempting to influence the shrinkage cracks to develop beneath the sawcut contraction joints. In industrial construction, this can result in a floor slab that will be susceptible to relative movement of the joint edges and joint maintenance problems when exposed to wheeled traffic. If 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 wheeled traffic. Refer to Section 5.2 for additional information.

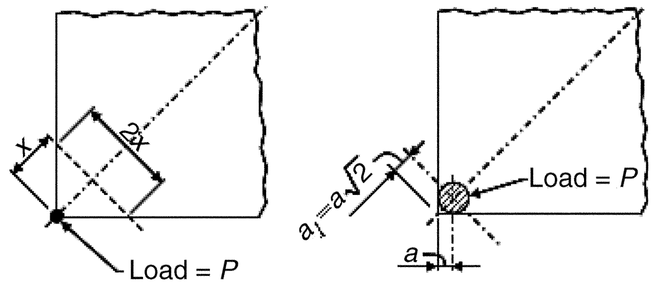


Fig. 6.1—Corner load on slab-on-ground.

6.2—Thickness design methods

If the slab is loaded uniformly over its entire area and is supported by uniform subgrade, stresses will be due solely to restrained volumetric changes; however, most slabs are subjected to nonuniform loading. In warehouses, for example, 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 pattern of loading.

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 given herein 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. 6.1). At small 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 width of the section of slab at right angles to the bisector of the corner angle. For a 90-degree corner, the width of this section is $2x$, and the bending moment per unit width of slab is

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

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

$$f_t = \frac{M}{S} = \frac{P/2}{h^2/6} = \frac{3P}{h^2} \tag{6-1}$$

This equation will give reasonably close results only in the immediate vicinity of the slab corner, and only if 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. 6.1), 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

$$f_t = \frac{3P}{h^2}\left[1 - \left(\frac{a\sqrt{2}}{L}\right)^{0.6}\right] \tag{6-2}$$

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

$$L = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}} \tag{6-3}$$

where E = elastic modulus of concrete, psi (Pa); μ = Poisson's ratio for concrete—approximately 0.15; and k = 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, 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

$$f_b = 0.316\frac{P}{h^2}[\log(h^3) - 4\log(\sqrt{1.6a^2 + h^2} - 0.675h) - \log(k) + 6.48] \tag{6-4}$$

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

$$f_b = 0.572\frac{P}{h^2}[\log(h^3) - 4\log(\sqrt{1.6a^2 + h^2} - 0.675h) - \log(k) + 5.77] \tag{6-5}$$

For Eq. (6-4) and (6-5), 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.^{3}). log is base 10 log.

In the event that the flexural tensile stress in the slab, as given by the previous equations, exceeds the allowable flexural tensile stress on the concrete, it is necessary to increase the thickness of the slab, increase the 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. Its centroid should be no closer to the neutral axis than that of the tension concrete that it replaces.

Loads distributed over partial areas—In addition to concentrated loads, it may be that uniform loads distributed over partial areas of slabs will 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

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

where

M_c = slab moment 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 width of the aisle cannot always be predicted exactly, Rice suggested 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:

- PCA method (Section 6.2.1);
- WRI method (Section 6.2.2); and
- COE method (Section 6.2.3).

Each of these methods, the evolution of which is described in Chapter 1 and previously, seek to avoid live load-induced cracks through the provision of adequate slab cross section by using an adequate safety factor against rupture. The PCA and WRI methods only address live loads imposed on the slab's interior, while 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. ACI 117 does provide tolerances for slabs-on-ground, and both the slab designer and the contractor should consider these tolerances. Specifying a minimum thickness may be appropriate. Design examples in Appendixes 1, 2, and 3 show how to use all three methods.

6.2.1 *PCA 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 on the surface of the slab, but are not adjacent to free edges.

6.2.1.1 *Wheel loads*—Slabs-on-ground are subjected to various types, sizes, and magnitudes of wheel loads. Lift-truck loading is a common example, where loads from wheels are transmitted to the slab. 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 1 shows use of the PCA design charts for wheel loadings.

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

6.2.1.3 *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. The 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 1) 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.

6.2.1.4 *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.

6.2.2 *Wire Reinforcement Institute (WRI) design method*

6.2.2.1 *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.

6.2.2.2 *Wheel loads*—Slabs-on-ground subjected to wheel loadings are discussed in Section 6.2.1.1. 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.

6.2.2.3 *Concentrated loads*—WRI charts do not cover concentrated loads directly. It is possible, however, to determine the equivalent wheel loading that represents a concentrated loading and thereby using the wheel load charts for this purpose.

6.2.2.4 *Uniform loads*—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 6.2.2.1, the width of the aisle and the magnitude of the uniform load are variables in this method.

6.2.2.5 *Construction loads*—Various 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 loading expressed in terms of wheel loads or uniform loads.

6.2.3 *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 covered.

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 contact area of each wheel, 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.

6.3—Shear transfer at joints

Recent analysis (Walker and Holland 1999) shows that edge curl is a principal concern governing the spacing of sawcut contraction joints in slabs-on-ground. 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 wheeled traffic. Positive load-transfer devices, such as dowels, should be used for joints subjected to wheeled traffic where the joint is expected to open more than 0.035 in. (0.9 mm). Chapter 5 contains an expanded discussion of jointing of slabs-on-ground and protection of the joints. PCA (2001) provides extended consideration of the effectiveness of shear transfer at joints.

6.3.1 *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 5 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.

6.4—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 recommended in Fig. 5.6 and as discussed in Chapter 5. Refer also to Chapter 13, Section 13.8.

CHAPTER 7—DESIGN OF SLABS REINFORCED FOR CRACK-WIDTH CONTROL

7.1—Introduction

Slabs-on-ground are designed and their thickness selected to prevent cracking due to external loading, as discussed in Chapter 6. Slab thickness calculations are based on the assumption of an uncracked and unreinforced slab. Steel reinforcement may be used in slabs-on-ground to improve performance of the slab under certain conditions. These include:

- Limiting width of shrinkage cracks;
- Use of longer joint spacings than unreinforced slabs; and
- Providing moment capacity and stability at cracked sections.

The use of reinforcement will not prevent cracking, but will actually increase crack frequency while reducing crack widths. Properly proportioned and positioned, reinforcement will limit crack widths such that the cracks will not affect slab serviceability.

7.2—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 6 may all be applied identically to the design of reinforced slabs-on-ground by simply ignoring the presence of the reinforcement.

7.3—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 minimum steel ratio of 0.5% (PCA 2001) of the slab cross-sectional area is recommended. The reinforcement should be located as close to the slab top surface as possible while maintaining minimum concrete coverage over the reinforcement.

7.4—Reinforcement for moment capacity

Reinforcement for moment capacity (WRI 2001) provides a cracked, reinforced section equivalent to the uncracked, plain concrete section. This design requires the joint spacing to be as shown in Fig. 5.6, and the reinforcement is to be discontinuous at the joints. For steel located at mid-depth

$$A_s = \frac{4.4 \times \mathrm{MOR} \times h}{f_s} \text{ (in.-lb units)} \qquad (7\text{-}1)$$

$$A_s = \frac{370 \times \mathrm{MOR} \times h}{f_s} \text{ (SI units)}$$

where

A_s = cross-sectional area of steel, in.^{2}/ft (mm^2/m) of slab;

h = slab thickness, in. (mm);

f_c' = compression strength of concrete, psi (MPa);

f_y = yield strength of reinforcement, psi (MPa);

f_s = 75% of f_y maximum. (Note: using high steel reinforcement stresses may lead to unacceptable wide crack widths. The designer may want to consider using less than 75% of f_y to limit the width of the cracks.), psi (MPa); and

MOR = modulus of rupture for the concrete, as used for unreinforced design, generally taken as $9\sqrt{f_c'}$, psi ($0.75\sqrt{f_c'}$, MPa); may range from 7 to $11\sqrt{f_c'}$, psi (0.58 to $0.91\sqrt{f_c'}$, MPa).

7.5—Reinforcement location

Reinforcement for crack-width control only should be at or above mid-depth of the slab-on-ground, never below mid-depth. A common practice is to specify that the steel have 1.5 to 2 in. (38 to 51 mm) cover below the top surface of the concrete. Reinforcement for moment capacity should be at the centroid of the tensile area of the uncracked concrete section.

CHAPTER 8—DESIGN OF SHRINKAGE-COMPENSATING CONCRETE SLABS

8.1—Introduction

This chapter deals with concrete slabs-on-ground constructed with shrinkage-compensating concrete made with cement conforming to ASTM C 845. The design procedure differs significantly from that for conventional concrete with ASTM C 150 portland cement and blends conforming to ASTM C 595.

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 an 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 will reduce the expansion strains, but ideally, a residual compressive stress will remain in the concrete, thereby minimizing shrinkage cracking and curling. How shrinkage-compensating concrete differs from conventional concrete with respect to these volume changes is explained in Sections 8.1.1 and 8.1.2.

8.1.1 *Portland-cement and blended-cement concrete*—The shortening of portland-cement and blended-cement concrete due to shrinkage is restrained by reinforcement and friction between the ground and the slab. This shortening may occur at an early age with the friction restraint stressing the concrete in excess of its early tensile strength, thereby cracking the slab.

As drying shrinkage continues, cracks open wider. This may present maintenance problems, and if the crack width exceeds 0.035 in. (0.9 mm), aggregate interlock (load transfer) becomes ineffective. Refer to Section 5.2 for additional information on aggregate interlock. Cracking due to shrinkage restraint may be limited by closer joint spacing, additional distributed reinforcement, or post-tensioning.

8.1.2 *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 C 845 rather than ASTM C 150 or C 595. 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 both the expansion during curing and subsequent shortening due to drying shrinkage. Figure 8.1 illustrates the typical length-change characteristics of shrinkage-compensating and portland-cement concrete prism specimens tested in accordance with ASTM C 878 (ACI 223).

In shrinkage-compensating concrete, the expansion is restrained internally by the bonded reinforcement, which is placed in tension. As a result of this expansive strain, compression is developed in the concrete, which in turn is relieved by drying shrinkage and some creep. With shrinkage-compensating concrete, it is intended that the restrained expansion be greater than the resultant long-term shrinkage, as shown in Fig. 8.2, so the concrete will remain in compression. The minimum recommended amount of concrete expansion for slabs-on-ground, measured in accordance with ASTM C 878, is 0.03%.

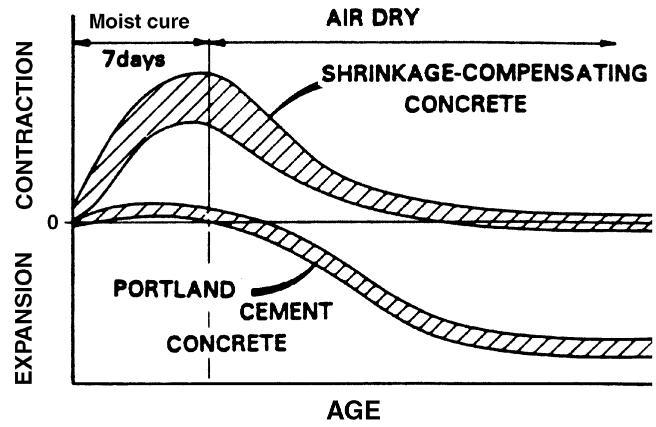


Fig. 8.1—Typical length change characteristics of shrinkage-compensating and portland-cement concretes (ACI Committee 223 1970).

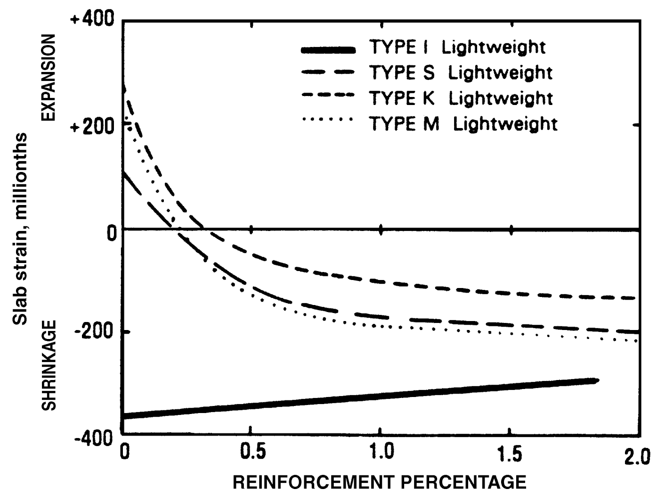


Fig. 8.2—Effect of reinforcement on shrinkage and expansion at an age of 250 days (American Concrete Institute 1980).

8.2—Thickness determination

For a slab-on-ground cast with shrinkage-compensating concrete, the determination of the slab thickness required by imposed loading is similar to that used for other slab designs. The PCA, WRI, and COE methods are all appropriate. They are discussed in Chapter 5 and illustrated in Appendixes 1, 2, and 3. Appendix 5 illustrates other design considerations specific to the use of the shrinkage-compensating concrete.

8.3—Reinforcement

8.3.1 *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 one to two have been found satisfactory. High restraint will induce a high compressive stress in the concrete but provide little shrinkage compensation. Wherever possible, the design should specify the reinforcement recommended in ACI 223.

8.3.2 *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 yield strength of the reinforcement. When procedures outlined in ACI 223 are followed, however, a reinforcement ratio less than the aforementioned minimum may be used.

8.3.3 *Effect of reinforcement location*—The location of the steel is critical to both slab behavior and internal concrete stress. ACI 223 recommends that reinforcement be positioned 1/3 of the depth from the top. The function of the top reinforcement is to balance the restraint provided by the subgrade, in addition to providing elastic restraint against expansion. Caution is advised when using smaller percentages of reinforcement because lighter gage material may be more difficult to position and maintain in the top portion of the

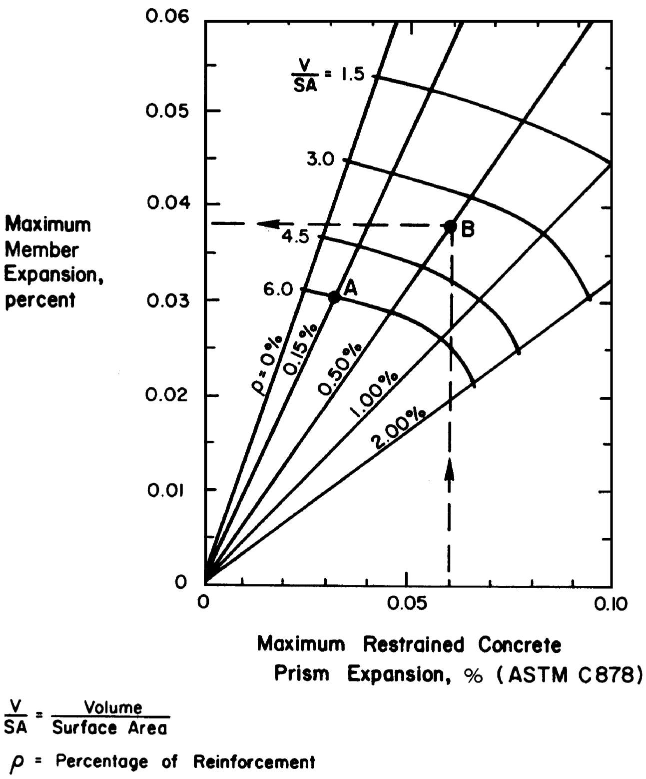


Fig. 8.3—Slab expansion versus prism expansion for different volume-surface ratios and reinforcement percentages (from ACI 223).

slabs. Stiffer, more widely spaced reinforcement permits lower reinforcement percentages to be used satisfactorily. This is typically achieved with ASTM A 497 deformed wire reinforcement or ASTM A 615 deformed bars, widely spaced. Other deformed bar reinforcement is also acceptable, such as reinforcement defined in ASTM A 996 and A 706.

8.3.4 *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 yield strength of the reinforcement. To prevent concrete from shrinking more than the restrained expansion, lighter percentages of steel are recommended. Should high steel ratios be required for structural design conditions, higher expansion levels in the concrete, as measured by ASTM C 878 prisms, would be required.

The required level of ASTM C 878 prism expansion strains can be determined by using Fig. 8.3. The figure shows the relationship between prism expansions, internal reinforcement percent, volume-surface relationship, and resulting concrete slab expansions. The figure enables one to estimate the anticipated member shrinkage strains using the volume-surface ratio for different slabs and different reinforcement percentages. If the resulting slab expansions are greater than the resulting shrinkage strains for a given volume-surface relationship, then full shrinkage compensation is obtained. 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 C 878 is 0.03% (Russell 1973).

8.3.5 *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,* and environmental and other conditions. 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.

Therefore, 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 C 878. For a given volume-surface ratio and a minimum standard prism expansion level (verified with trial batch data), internal restraint levels provided by less than 0.15% steel in a typical 6 in. (150 mm) slab can be used (ACI 1980). If the slab expansion is greater than the shrinkage strain for a surface-volume ratio of 6:1, using Russell's data (American Concrete Institute 1980), full compensation can be achieved. Circumferential curves depicting shrinkage strains for volume-surface ratios for other slab thicknesses are also shown in Fig. 8.4.

Care should be exercised when using low reinforcement ratios. If light reinforcement is used, it may accidentally be depressed into the bottom third 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. If tests and design calculations are not used, the minimum 0.15% reinforcement is often specified.

8.4—Other considerations

8.4.1 *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

[*]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, while 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.

quarter of the slab. Both reinforced and plain slabs, as well as fiber-reinforced slabs, displayed this behavior.

8.4.2 *Prism and slab expansion strains and stresses*—Because the reinforcement percentage varies, the ASTM C 878 restrained concrete prism test is used to verify the expansive potential of a given mixture. Figure 8.2 may then be used to determine the amount of slab expansion (strain) using the known prism expansion value and the percent of reinforcement in the slab.

With the use of Fig. 8.2, the amount of internal compressive force acting on the concrete can be estimated knowing the maximum member (slab) expansion and the percent of internal reinforcement in the slab.

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

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 D 1621 or D 3575 should be used.

If a slab is allowed to expand only at one end during initial expansion, the width of the isolation joint (in inches) should be equal to two times the anticipated slab expansion, as taken from Fig. 8.3, and multiplied by the length of the longest dimension of the slab (in inches). For a 100 x 120 ft (30 x 37 m) slab with expansion strain of 0.00035:

$$\text{Joint width} = 2 \times 120 \times 12 \times 0.00035\ (2 \times 36.6 \times 1000 \times 0.00035)$$

$$= 1.008 \text{ in. } (25.60 \text{ mm})$$

Use 1 in. (25 mm) thick joint material if 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.

8.4.4 *Construction joints*—ACI 223 states that with the use of shrinkage-compensating concrete, slabs may be placed in areas as large as 16,000 ft^2 (1500 m^2) without joints. 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. Further explanation and details are found in ACI 223.

8.4.5 *Placing sequence*—For slabs-on-ground, the placement sequence should allow the 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.

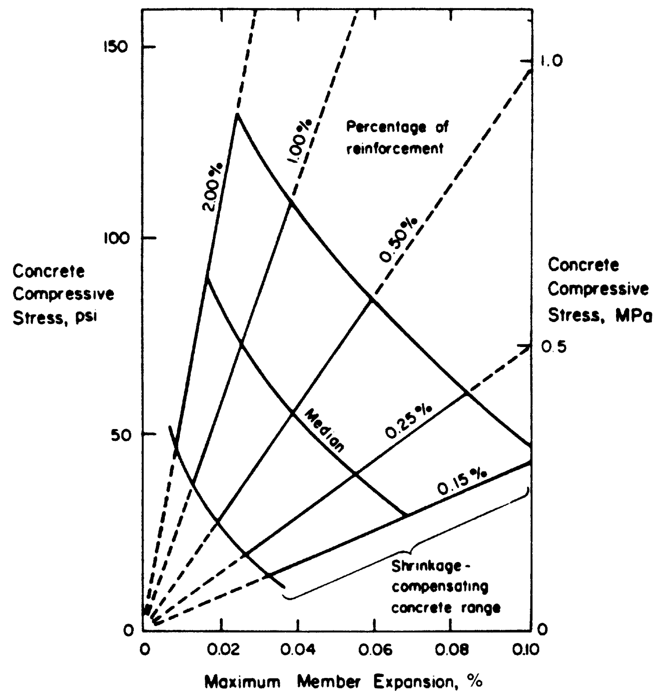


Fig. 8.4—Calculated compressive stresses induced by expansion (from ACI 223).

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 C 878 should occur before placing adjacent slabs when a slab is not free to expand on two opposite ends. Examples of placement patterns are shown in ACI 223. Checkerboarded placements should not be used unless a compressible joint material is placed between the slab before concrete placement as per Section 8.4.3.

Before establishing the placement sequence, results of expansion testing per ASTM C 878 should be considered. A minimum level of prism expansion of 0.04% is recommended for slabs-on-ground. Higher expansion results would 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.

8.4.6 *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 a new finished floor elevation may be the most frequent reasons for using overlays. The two types of overlays—bonded and nonbonded—are covered in ACI 302.1R as Class 6 and Class 7 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. Typical bonded overlays are used to improve surface abrasion resistance with the use of a wear-resistant aggregate. At times, more ductile materials, such as graded iron, are employed in bonded overlays to improve the abrasion resistance and impact resistance of the floor surface.

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. Further, base slabs that contain cracks that move due to slab motion will often reflect cracks into the topping. Therefore, these cracks should be repaired. If the base slab contains shrinkage-compensating concrete, the portland-cement concrete bonded topping should be applied at least 10 days after the base slab is placed. 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 will negate the expansion action of the topping, leading to cracking or possibly delamination.

CHAPTER 9—DESIGN OF POST-TENSIONED SLABS-ON-GROUND

9.1—Notation

A = area of gross concrete cross section, in.^{2} (mm^2)
A_b = bearing area beneath tendon anchor, in.^{2} (mm^2)
A'_b = maximum area of portion of supporting surface that is geometrically similar to and concentric with loaded area, in.^{2} (mm^2)
A_{bm} = total area of beam concrete, in.^{2} (mm^2)
A_c = activity ratio of clay
A_o = coefficient in Eq. (9-11)
A_{ps} = area of prestressing steel, in.^{2} (mm^2)
A_{sl} = total area of slab concrete, in.^{2} (mm^2)
B = constant used in Eq. (9-11)
B_w = assumed slab width, in. (mm)
b = width of individual stiffening beam, in. (mm)
C = constant used in Eq. (9-11)
C_Δ = coefficient used to establish allowable differential deflection used in Eq. (9-23)
CGC = geometric centroid of gross concrete section
CGS = center of gravity of prestressing force
C_p = coefficient in Eq. (9-41) for slab stress due to partition load—function of k_s
CR = prestress loss due to creep of concrete, kips (kN)
c = distance between CGC and extreme cross-section fibers, in. (mm)
E_c = long-term or creep modulus of elasticity of concrete, psi (MPa)
ES = prestress loss due to elastic shortening of concrete, kips (kN)
E_s = modulus of elasticity of soil, psi (MPa)
e = eccentricity of post-tensioning force (perpendicular distance between CGS and CGC), in. (mm)
e_m = edge moisture variation distance, ft (m)
e_n = base of natural (Naperian) logarithms
f = flexural concrete stress (tension or compression), ksi (kN/mm^2)
f_B = section modulus factor for bottom fiber
f_{bp} = allowable bearing stress under tendon anchorages, psi (MPa)
f_c = allowable concrete compressive flexural stress, psi (MPa)
f'_c = 28-day concrete compressive strength, psi (MPa)
f'_{ci} = concrete compressive strength at time of stressing tendons, psi (MPa)
f_{cr} = concrete modulus of rupture, flexural tension stress that produces first cracking, psi (MPa)
f_p = minimum average residual prestress compressive stress, psi (MPa)
f_{pi} = allowable tendon stress immediately after stressing, psi (MPa)
f_{pj} = allowable tendon stress due to tendon jacking force, psi (MPa)
f_{pu} = specified maximum tendon tensile stress, psi (MPa);
f_{py} = specified yield strength of prestressing steel, psi (MPa)
f_T = section modulus factor for top fiber
f_t = allowable concrete flexural tension stress, psi (MPa)
g = moment of inertia factor
H = thickness of uniform thickness foundation, in. (mm)
h = total depth of stiffening beam, measured from top surface of slab to bottom of beam (formerly d, changed for consistency with ACI 318), in. (mm)
I = gross concrete moment of inertia, in.^{4} (mm^4)
k = depth-to-neutral axis ratio; also abbreviation for kips
k_s = soil subgrade modulus, lb/in.^{3} (N/mm^3)
L = total slab length (or total length of design rectangle) in direction being considered (short or long), perpendicular to W, ft (m)
L_L = long length of design rectangle, ft (m)
L_S = short length of design rectangle, ft (m)
M_{cs} = applied service moment in slab on compressible soil, ft-kips/ft (kNm/m)
M_L = maximum applied service load moment in long direction (causing bending stresses on short cross section) from either center lift or edge lift swelling condition, ft-kips/ft (kNm/m)
M_{max} = maximum moment in slab under load-bearing partition, ft-kips/ft (kNm/m)
M_{ns} = moment occurring in the no-swell condition, ft-kips/ft (kNm/m)
M_S = maximum applied service load moment in short direction (causing bending stresses on long cross section) from either center lift or edge lift swelling condition, ft-kips/ft (kNm/m)
N_T = number of tendons
n = number of stiffening beams in cross section of width W
P = uniform unfactored service line load (P) acting along entire length of perimeter stiffening beams representing weight of exterior building material and that

portion of the superstructure dead and live loads that frame into exterior wall. P does not include any portion of foundation concrete, lb/ft (N/m)

P_e = effective prestress force after losses due to elastic shortening, creep, and shrinkage of concrete, and steel relaxation, lb (N)

PI = plasticity index

P_i = prestress force immediately after stressing and anchoring tendons, kips (kN)

P_r = resultant prestress force after all losses (including those due to subgrade friction), kips (kN), see 9.8.1 and 9.8.6

P_r = post-tensioning force required to overcome subgrade friction, lb/ft (N/m), see 9.5.2

q_{allow} = allowable soil-bearing pressure, lb/ft^2 (N/m^2)

q_u = unconfined compressive strength of soil, lb/ft^2 (N/m^2)

RE = prestress loss due to steel relaxation, kips (kN)

r_1 = area ratio

S = interior stiffening beam spacing, ft (m) If beam spacings vary, average spacing may be used if ratio between largest and smallest spacing does not exceed 1.5. If ratio between largest and smallest spacing exceeds 1.5, use $S = 0.85 \times$ (largest spacing);

S_b = section modulus with respect to bottom fiber, in.^{3} (mm^3)

SG = prestress loss due to subgrade friction, kips (kN)

SH = prestress loss due to concrete shrinkage, kips (kN)

S_t = section modulus with respect to top fiber, in.^{3} (mm^3)

S_{ten} = tendon spacing, ft (m)

t = slab thickness in a ribbed (stiffened) foundation, in. (mm)

V = controlling service load shear force, larger of V_S or V_L, lb/ft (N/m)

V_{cs} = maximum service load shear force in slab on compressible soil, kips/ft (kN/m)

V_L = maximum service load shear force in long direction from either center lift or edge lift swelling condition, kips/ft (kN/m)

V_{ns} = service load shear force in no-swell condition, kips/ft (kN/m)

V_S = maximum service load shear force in the short direction from either center lift or edge lift swelling condition, kips/ft (kN/m)

v = service load shear stress, psi (MPa)

v_c = allowable concrete shear stress, psi (MPa)

W = foundation width (or width of design rectangle) in direction being considered (short or long), perpendicular to L, ft (m), see 9.8.3 and 9.8.4

W = slab strip width, 12 in./ft (1000mm/m), see 9.5.2;

W_{slab} = foundation weight, lb (kg)

W_{slab} = self-weight of the foundation slab, lb/ft^2 (N/m), see 9.5.2

y_m = maximum differential soil movement or swell, in. (mm)

α = slope of tangent to tendon, radians

β = relative stiffness length, approximate distance from edge of slab to point of maximum moment, ft (m)

Δ = expected service load differential deflection of slab, including correction for prestressing, in. (mm)

Δ_{allow} = allowable differential deflection of slab, in. (mm)

Δ_{cs} = differential deflection in slab on compressible soil, in. (mm)

Δ_{ns} = differential deflection in no-swell condition, in. (mm)

Δ_o = expected service load differential deflection of slab (without deflection caused by prestressing), in. (mm)

Δ_p = deflection caused by prestressing, in. (mm)

δ = expected settlement, reported by geotechnical engineer, occurring in compressive soil due to total load expressed as uniform load, in. (mm)

μ = coefficient of friction between slab and subgrade

9.2—Definitions

Selected terms and expressions that appear in Chapter 9 are defined and explained.

allowable differential deflection—the amount of slab deflection that can be tolerated by the type of superstructure supported by the slab or equipment operating on the slab.

differential deflection distance—the total slab length may not be the proper distance over which to evaluate the acceptability of the expected differential deflection. Analysis of the locations of maximum and minimum deflections shows that several such locations may occur in longer (or wider) slabs; that is, the slabs experience multimodal bending (Thompson and Anderson 1968). All such bending, however, occurred within a distance of 6β from the edge of the slab. Using a length of L or 6β, whichever is smaller, when determining the allowable differential deflection, will limit the deflection to an acceptable amount, and this length is called the differential deflection distance.

differential soil movement y_m—this is the expected vertical movement of the perimeter soil due to type and amount of clay mineral, its initial wetness, the depth of the zone within which the moisture varies, and other factors (PTI 2004). The differential soil movement will often be greater than the allowable deflection.

edge moisture variation distance e_m—also known as the edge moisture penetration distance, e_m is the distance measured inward from the edge of the slab over which the moisture content of the soil varies. An increasing moisture content at increasing distances inside the slab perimeter is indicative of a center lift condition, whereas a decreasing moisture content indicates an edge lift condition.

lift conditions—several terms refer to the shape of a slab or the stresses generated within a slab during the transition period from the as-cast shape to the intermediate or long-term shape. If the moisture content of the soil beneath the slab changes after construction of the slab, it will distort into either a center lift condition (also termed “center heave” and “doming”) or an edge lift condition (also called “edge heave” and “dishing”). The center lift condition is a long-term condition and occurs either when the soil beneath the interior of the slab becomes wetter and expands, when the soil around the perimeter of the slab dries and shrinks, or a combination of both. The center lift moment is caused by the slab conforming to the doming configuration and is the strength required to resist this change in shape. The moment is usually expressed as a negative moment.

Conversely, the edge lift condition is, in general, a seasonal or short-term condition that occurs when the soil beneath the perimeter becomes wetter than the soil beneath the interior of the slab, causing the edges to rise or heave. The edge lift moment is caused by the slab conforming to the dishing configuration and is the strength required to resist this change in shape. This moment is usually expressed as a positive moment.

relative stiffness length β—the distance from the edge of the slab at which the maximum moment occurs. The maximum moment does not occur at the point of actual soil-slab separation, but at some distance further inward the interior. The location of the maximum moment can be closely estimated by the Eq. (9-22) to calculate β, a length that depends on the relative stiffness of the soil and the stiffened slab.

The moment increases rapidly from the edge of the slab until it reaches a maximum at approximately a distance of β. The magnitude of the moment then begins to reduce toward the midpoint of the slab. For slabs 48 ft (15 m) long or less, the amount of this reduction depends on the slab length. For slabs longer than 48 ft (15 m), the increased length results in no significant changes in moment. Further, the maximum shear forces develop at or near the perimeter of the slab, within one β-length from the edge of the slab.

ribbed and stiffened slab—a slab of uniform thickness that has been stiffened against deflection by incorporating ribs or beams cast monolithically with the slab, such as a series of T-beams. The addition of the ribs greatly increases the moment of inertia of the concrete cross section, thereby increasing the ability of a section to resist deflection.

9.3—Introduction

Slabs-on-ground may be prestressed using bonded or unbonded tendons that are post-tensioned and anchored after the concrete has obtained 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;
- Avoidance of shrinkage cracks and significant reduction of cracking through active prestress;
- Lower life-cycle cost, fewer joints to maintain, and higher durability due to precompression;
- Enhanced serviceability and no facility down time for repairs of joints;
- Better preservation of floor flatness and levelness by minimizing the number of joints and joint curling;
- Decreased slab thickness;
- Increased load capacity;
- Resilience and recovery capability from overloading; and
- Reduction of superstructure cracking.

Regarding the recovery capability of post-tensioned concrete slabs, it is not likely that a ground-supported slab can be deflected sufficiently to exceed the yield strength of the prestressing steel, which means that cracks due to overload are likely to close up after the load is removed. Competent construction supervision and coordination is required for post-tensioned slabs. The post-tensioning system (PTI 2006) used for industrial floor applications should meet PTI "Specifications for Unbonded Tendons" (PTI 2000). 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. Slab penetrations made and drilled anchorage devices placed after construction should be coordinated with tendon locations to avoid severing tendons. This can be done using metal detectors or similar devices to locate the tendons in an existing slab.

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) prestressed residential ground-supported slab were reported (Thompson and Anderson 1968). These tests and previous experience with completed construction led to the first general approval for the use of prestressed post-tensioned ground-supported slabs throughout the United States in June 1968 by HUD. The only requirement placed on the use of this method of reinforcement was that a rational design be provided by a registered professional engineer. Since June 1968, millions of square feet of ground-supported concrete slabs for residential, commercial, and industrial applications have been constructed using post-tensioned prestressed concrete.

9.4—Applicable design procedures

The preplanning of the slab-on-ground design criteria is of utmost importance. The following list of needed information is not exclusive:

- Slab geometry—dimensions, thickness, self weight;
- Slab usage—industrial, residential;
- Surface requirements—flatness and levelness, floor covering;
- Loading—concentrated, uniform, line, lift truck loads;
- Rack layout—determined or not, base plate size;
- Service life—expected service life, cost of interruptions for maintenance;
- Soil conditions—soil properties, base and subgrade;
- Method of construction—strip or section placement, placement sequence;
- Construction conditions—building enclosure, time factor; and
- Concrete—mixture proportion, aggregates, construction method, and equipment.

A careful consideration of these criteria apply to the design of any slab-on-ground and helps determine the most appropriate type of slab and design method.

9.4.1 *Thickness design*—The required thickness of post-tensioned slabs may be determined by the PCA, WRI, and COE methods described in Chapter 6 and illustrated in Appendixes 1, 2, and 3. This is done by simply increasing the permissible tensile stress of the concrete by the net precompression from the prestressing force.

9.4.2 *Crack-control design*—This design is normally used for slabs with light loading, usually with no rack post loading. Post-tensioning is used instead of reinforcing steel or close jointing to compensate for concrete shrinkage and temperature effects. The minimum required post-tensioning force is calculated to provide some residual compression over tension resulting from subgrade drag.

9.4.3 *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 following these steps:

- Determine slab geometry, placement sizes, estimate slab thickness (normally 5 to 10 in. [125 to 250 mm]);
- Calculate the subgrade drag and the friction losses in the post-tensioning tendons;
- Estimate long-term losses to arrive at the final effective prestress force. For floors subjected to large temperature changes, the effects of temperature on the concrete should also be considered in determining the final effective post-tensioning force;
- 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;
- Verify that the actual total superimposed stresses and deflections do not exceed the allowable values. Depending on the results a modification of the slab thickness and or slab placement, layout may be necessary.

9.4.4 *Post-Tensioning Institute (PTI) method*—In 2004, the PTI published a third edition of a document (PTI 2004) containing recommendations for establishing the strength and serviceability requirements primarily 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 the unique advantages of post-tensioning as the primary reinforcement for a ribbed and stiffened slab. A stiffened slab is reinforced to provide sufficient strength and deflection control in swelling and compressible soil conditions. The uncracked section modulus in a post-tensioned analysis enhances stiffness and flexural stress control—two of the most important factors associated with slab-on-ground design.

Sections 9.7, 9.8, and 9.9 present PTI equations for the determination of the moment, deflection, and shear requirements 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 then used in the regression analysis that developed the design equations (PTI 2004).

Some of the equations contain variables with an exponent carried out to three digits, which result from the analysis described previously. It should not imply an accuracy of the results, considering that this method provides theoretical results based on ideal conditions. Any site-specific circumstances like climate, trees, slopes with cut and fill, and surface water drainage conditions should be taken into account by the engineer, and can change the results considerably. Moisture-sensitive soils should be stabilized by minimizing exchanges in moisture content.

9.5—Slabs post-tensioned for crack control

9.5.1 *Design methods*—For lightly loaded slabs 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, combined with residual compression from post-tensioning, will provide a considerable load capacity of the slab.

9.5.2 *Post-tensioning force required*—The post-tensioning force P_r (lb/ft [N/m]) required to overcome the subgrade friction can be calculated by the equation

$$P_r = W_{slab}\frac{L}{2}\mu \tag{9-1}$$

where

W_{slab} = 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 = slab length in the direction being considered, ft (m); and

μ = coefficient of friction between slab and subgrade.

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

- 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, the restraint due to the beams should be taken into consideration.

The residual compressive force after all prestress losses and the subgrade friction losses should be determined by the engineer based on the slab geometry, loading, and usage. The following prestress levels have been used satisfactorily:

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 over 400 ft (120 m) long:	200 to 250 psi (1.4 to 1.7 MPa)

The friction losses, elastic shortening, and the long-term losses in the tendons can be calculated 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, lightly reinforced for crack control, is given by the equation

$$S_{ten} = \frac{P_e}{f_p WH + P_r} \tag{9-2}$$

where

P_e = effective prestress force per tendon, lb (N);

f_p = minimum average residual prestress (required compressive stress), psi (MPa);

W = slab unit strip width, 12 in./ft (1000 mm/m);

H = slab thickness, in. (mm); and

P_r = from Eq. (9-1), lb/ft (N/m).

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

9.5.4 *Tendon stressing*—The stressing sequence should be adjusted to the project requirements. While one-time stressing may be used for most residential slabs after the concrete has reached sufficient strength to transfer the force from the anchorages, a more gradual force introduction may be needed for industrial floors. Two stressing stages may be necessary to prevent early shrinkage cracks from appearing. Typically, the initial (partial) stressing should be completed within 24 hours after concrete placement.

9.5.5 *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 for 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 keep it tied. This enhances the durability of the traffic joint and eliminates a need for more severe measures such as dowels or armored joint.

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

9.6.1 *Design methods*—The thickness design methods listed in Section 9.4.1 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. Also, the provisions of Section 9.5 apply accordingly to control cracking and to help determine the placement and tendon layouts.

9.6.2 *Safety factors*—The use of multiple safety factors should be avoided. The post-tensioning reinforcement provides reserve capacities, and in no case should safety factors greater than those for nonprestressed slabs be used. Cracking under the concentrated load can be permissible for prestressed slabs, and it can be taken into account by using structural design requirements of ACI 318.

9.6.3 *Subgrade friction reduction*—Refer to Section 9.5.2 for the recommended range of friction coefficients. To fully use the advantages of post-tensioning 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. A thin layer of sand choker under the slab can also reduce the subgrade friction. It is difficult to place concrete directly on the thin layer of sand. This should be considered for thicker slabs only and with special care during concrete placement.

9.6.4 *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. As the lengths between the joints are long and the shortening of the slabs should not be restricted, a few considerations should be noted.

9.6.4.1 *Strip placements*—Normally, every other strip is placed, and the adjacent slabs are placed in a second phase. This allows the initial slabs to shorten in their long (and short) direction before the adjacent slab is placed. 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 be left open as long as practical to allow the majority of shortening to occur before the placement strip is closed. Also, stage stressing reduces this long-direction shortening as the young concrete is only partially loaded. The elastic shortening decreases with the age of concrete at load transfer.

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

9.6.5 *Special considerations*—The prestressing tendons should be located within the upper middle 1/4 of slab thickness while maintaining a proper concrete cover. Occasionally, the engineer may determine that the tendons need to be in another location based on other considerations. A higher position of the tendons in the slab will reduce the risk of surface cracking, and a lower position will reduce the potential of cracking under concentrated loads.

The tendons can be supported on support bars, especially when one-way post-tensioning is used. Support bars can also serve as crack-control reinforcement. Special slab-on-ground chairs of the required height are used to ensure that the location of the tendons will remain unchanged during concrete placement.

9.7—Residential slabs with post-tensioned reinforcement for structural action

9.7.1 *Soil properties*—The designer should have the following information about soil properties:

- Allowable soil bearing pressure q_{allow}, lb/ft^2 (N/m^2);
- Edge moisture variation distance e_m (ft [m]), center lift, and edge lift;
- Differential soil movement y_m (in. [mm]), center lift, and edge lift; and
- Slab-subgrade friction coefficient μ.

9.7.2 *Structural data and materials properties*—The necessary data for design is: slab length L, ft (m); beam spacing S, ft (m); overall depth h, in. (mm); width b (in. [mm]) of the stiffening beams; and the perimeter loading P, lb/ft (N/m).

Required material properties:

- Specified 28-day compressive strength of concrete f_c', psi (MPa);
- Type, grade, and strength of the prestressing steel; and
- Type and grade of nonprestressed reinforcement.

9.7.3 *Design stresses for the concrete*—The following stresses are used when designing by the PTI method:

Allowable tensile stress

$$f_t = 6\sqrt{f_c'} \quad \text{(in.-lb)} \tag{9-3}$$

$$f_t = 0.5\sqrt{f_c'} \quad \text{(SI)}$$

Allowable compressive stress

$$f_c = 0.45 f_c' \tag{9-4}$$

Estimated tensile cracking stress

$$f_{cr} = 7.5\sqrt{f_c'} \quad \text{(in.-lb)} \tag{9-5}$$

$$f_{cr} = 0.62\sqrt{f_c'} \quad \text{(SI)}$$

Allowable shear stress

$$v_c = 1.7\sqrt{f_c'} + 0.2 f_p \quad \text{(in.-lb)} \tag{9-6}$$

$$v_c = 0.14\sqrt{f_c'} + 0.2 f_p \quad \text{(SI)}$$

Allowable concrete bearing stress at anchorages:

At service load

$$f_{bp} = 0.6 f_c' \sqrt{\frac{A_b'}{A_b}} \le f_c' \tag{9-7}$$

At transfer

$$f_{bp} = 0.8 f_{ci}' \sqrt{\frac{A_b'}{A_b} - 0.2} \le 1.25 f_{ci}' \tag{9-8}$$

Allowable stresses in prestressing steel:

Allowable stress due to tendon jacking force

$$f_{pj} = 0.8 f_{pu} \le 0.94 f_{py} \tag{9-9}$$

Allowable stress immediately after prestress transfer

$$f_{pi} = 0.7 f_{pu} \tag{9-10}$$

9.8—Design for slabs on expansive soils

The equations presented will determine the moment, deflection, and shear requirements for slabs cast on expansive soils. Equation (9-11) through (9-20) determine the flexural strength requirements; Eq. (9-22) through (9-27) determine the deflection requirements; and Eq. (9-28) through (9-30) determine the shear requirements. Slabs designed by the PTI method should meet these requirements. The designer may select either nonprestressed reinforcement, post-tensioned reinforcement, or a combination of both, to meet the strength requirements. Appendix 4 presents a design example.

9.8.1 *Moments*—Center lift design moment in the long direction (flexural strength requirement of the section across the long direction) is given by Eq. (9-11)

$$M_L = A_o[B(e_m)^{1.238} + C] \quad \text{(in.-lb)} \tag{9-11}$$

$$M_L = 0.445 A_o[4.35 B(e_m)^{1.238} + C] \quad \text{(SI)}$$

where

$$A_o = \frac{1}{727}[(L)^{0.013}(S)^{0.306}(h)^{0.688}(P)^{0.534}(y_m)^{0.193}] \quad \text{(in.-lb)} \tag{9-12}$$

$$A_o = \frac{1}{36{,}000}[(L)^{0.013}(S)^{0.306}(h)^{0.688}(P)^{0.534}(y_m)^{0.193}] \quad \text{(SI)}$$

and for

$$0 \le e_m \le 5 \qquad B = 1, C = 0 \quad \text{(in.-lb)} \tag{9-13}$$

$$0 \le e_m \le 1.53 \qquad B = 1, C = 0 \quad \text{(SI)}$$

$$e_m > 5 \qquad B = \left(\frac{y_m - 1}{3}\right) \le 1.0 \quad \text{(in.-lb)} \tag{9-14}$$

$$e_m > 1.53 \qquad B = \left(\frac{y_m - 25.4}{76.2}\right) \le 1.0 \quad \text{(SI)}$$

$$C = \left[8 - \frac{P - 613}{255}\right]\left[\frac{4 - y_m}{3}\right] \ge 0 \quad \text{(in.-lb)} \tag{9-15}$$

$$C = \left[8 - \frac{P - 8940}{3720}\right]\left[\frac{102 - y_m}{76.2}\right] \geq 0 \quad \text{(SI)}$$

Center lift design moment in the short direction (flexural strength requirement of the section across the short direction) is given by Eq. (9-16):

For $L_L/L_S \geq 1.1$,

$$M_s = \left[\frac{58 + e_m}{60}\right] M_L \quad \text{(in.-lb)} \qquad (9\text{-}16)$$

$$M_s = \left[\frac{17.7 + e_m}{18.3}\right] M_L \quad \text{(SI)}$$

For $L_L/L_S < 1.1$,

$$M_S = M_L \qquad (9\text{-}17)$$

Edge lift design moment in the long direction (flexural strength requirement of the section across the long direction) is given by Eq. (9-18)

$$M_L = \frac{(S)^{0.10}(he_m)^{0.78}(y_m)^{0.66}}{7.2(L)^{0.0065}(P)^{0.04}} \quad \text{(in.-lb)} \qquad (9\text{-}18)$$

$$M_L = \frac{(S)^{0.10}(he_m)^{0.78}(y_m)^{0.66}}{54(L)^{0.0065}(P)^{0.04}} \quad \text{(SI)}$$

Equation (9-19) gives edge lift design moment in short direction (flexural strength requirement of the section across the short direction):

For $L_L/L_S \geq 1.1$,

$$M_s = h^{0.35}\left[\frac{19 + e_m}{57.75}\right] M_L \quad \text{(in.-lb)} \qquad (9\text{-}19)$$

$$M_s = 0.322h^{0.35}\left[\frac{5.79 + e_m}{17.6}\right] M_L \quad \text{(SI)}$$

For $L_L/L_S < 1.1$,

$$M_S = M_L \qquad (9\text{-}20)$$

Concrete flexural stresses produced by the applied service moments can be calculated with the following

$$f = \frac{P_r}{A} \pm \frac{M_{L,S}}{S_{t,b}} \pm \frac{P_r e}{S_{t,b}} \qquad (9\text{-}21)$$

The resultant concrete flexural stresses f must be limited to f_t in tension and f_c in compression.

9.8.2 *Differential deflection*—Allowable and expected deflections can be determined from actual section properties. The relative stiffness distance β for both long and short direction can be calculated using Eq. (9-22)

$$\beta = \frac{1}{12}\sqrt[4]{\frac{E_c I}{E_s}} \quad \text{(in.-lb)} \qquad (9\text{-}22)$$

$$\beta = \frac{1}{1000}\sqrt[4]{\frac{E_c I}{E_s}} \quad \text{(SI)}$$

Differential deflection distance—Either L or 6β, whichever is shorter, should be used in determining allowable deflections.

Equation (9-23) can be used to obtain allowable differential deflections for center lift, long and short directions

$$\Delta_{allow} = \frac{12(L \text{ or } 6\beta)}{C_\Delta} \quad \text{(in.-lb)} \qquad (9\text{-}23)$$

$$\Delta_{allow} = \frac{1000(L \text{ or } 6\beta)}{C_\Delta} \quad \text{(SI)}$$

Allowable differential deflection for edge lift, long and short directions, is given by Eq. (9-24)

$$\Delta_{allow} = \frac{12(L \text{ or } 6\beta)}{C_\Delta} \quad \text{(in.-lb)} \qquad (9\text{-}24)$$

$$\Delta_{allow} = \frac{1000(L \text{ or } 6\beta)}{C_\Delta} \quad \text{(SI)}$$

C_Δ may be selected from the following table, which presents sample C_Δ values for various types of superstructures.

Sample values of C_Δ

Material	Center lift	Edge lift
Wood frame	240	480
Stucco or plaster	360	720
Brick veneer	480	960
Concrete masonry units	960	1920
Prefab roof trusses	1000	2000

Expected differential deflection without prestressing, center lift, long and short directions, can be calculated from Eq. (9-25)

$$\Delta_o = \frac{(y_m L)^{0.205}(S)^{1.059}(P)^{0.523}(e_m)^{1.296}}{380(h)^{1.214}} \quad \text{(in.-lb)} \qquad (9\text{-}25)$$

$$\Delta_o = \frac{9.0(y_m L)^{0.205}(S)^{1.059}(P)^{0.523}(e_m)^{1.296}}{(h)^{1.214}} \quad \text{(SI)}$$

Expected differential deflection without prestressing, edge lift, long and short directions, can be calculated from Eq. (9-26)

$$\Delta_o = \frac{(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(y_m)^{0.76}}{15.9(h)^{0.85}(P)^{0.01}} \quad \text{(in.-lb)} \qquad (9\text{-}26)$$

$$\Delta_o = \frac{22.8(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(y_m)^{0.76}}{(h)^{0.85}(P)^{0.01}} \quad \text{(SI)}$$

Additional slab deflection is produced by prestressing if the prestressing force is applied at any point other than the CGS

$$\Delta_p = \frac{P_e e \beta^2}{2E_c I} \quad \text{(in.-lb)} \qquad (9\text{-}27)$$

$$\Delta_p = \frac{P_e e \beta^2}{2E_c I}(10^6) \quad \text{(SI)}$$

9.8.3 *Shear*—Expected service shear per foot (meter) of structure: center lift condition, short direction, can be calculated from Eq. (9-28)

$$V_S = \frac{1}{1350}[(L)^{0.19}(S)^{0.45}(h)^{0.20}(P)^{0.54}(y_m)^{0.04}(e_m)^{0.97}] \quad \text{(in.-lb)} \quad (9\text{-}28)$$

$$V_S = \frac{1}{126}[(L)^{0.19}(S)^{0.45}(h)^{0.20}(P)^{0.54}(y_m)^{0.04}(e_m)^{0.97}] \quad \text{(SI)}$$

Center lift condition, long direction, can be calculated from Eq. (9-29)

$$V_S = \frac{1}{1940}[(L)^{0.09}(S)^{0.71}(h)^{0.43}(P)^{0.44}(y_m)^{0.16}(e_m)^{0.93}] \quad \text{(in.-lb)} \quad (9\text{-}29)$$

$$V_S = \frac{1}{373}[(L)^{0.09}(S)^{0.71}(h)^{0.43}(P)^{0.44}(y_m)^{0.16}(e_m)^{0.93}] \quad \text{(SI)}$$

Edge lift condition, long and short direction, can be calculated from Eq. (9-30)

$$V_S = V_L = \frac{(L)^{0.07}(h)^{0.4}(P)^{0.03}(e_m)^{0.16}(y_m)^{0.67}}{3.0(S)^{0.015}} \quad \text{(in.-lb)} \qquad (9\text{-}30)$$

$$V_S = V_L = \frac{(L)^{0.07}(h)^{0.4}(P)^{0.03}(e_m)^{0.16}(y_m)^{0.67}}{5.5(S)^{0.015}} \quad \text{(SI)}$$

9.8.3.1 *Applied service load shear stress* v—Only the beams are considered in calculating the cross-sectional area resisting shear force in a ribbed slab:

Ribbed foundations

$$v = \frac{VW}{nhb} \qquad (9\text{-}31)$$

Uniform thickness foundations

$$v = \frac{V}{12H} \quad \text{(in.-lb)} \qquad (9\text{-}32)$$

$$v = \frac{V}{1000H} \quad \text{(SI)}$$

Compare v to v_c. If v exceeds v_c, shear reinforcement in accordance with ACI 318 should be provided. Possible alternatives to shear reinforcement include:

- Increasing the beam depth;
- Increasing the beam width; and
- Increasing the number of beams (decreasing the beam spacing).

9.8.4 *Uniform thickness conversion*—Once the ribbed foundation has been designed to satisfy moment, shear, and differential deflection requirements, it may be converted to an equivalent uniform thickness foundation with thickness H, if desired. The following equation for H should be used for the conversion

$$H = \sqrt[3]{\frac{I}{W}} \quad \text{(in.-lb)} \qquad (9\text{-}33)$$

$$H = \sqrt[3]{\frac{12I}{1000W}} \quad \text{(SI)}$$

9.8.5 *Other applications of design procedure*—This design procedure has other practical slab-on-ground applications besides construction on expansive clays, discussed as follows:

9.8.5.1 *Design of nonprestressed slabs-on-ground*—Equations (9-11), (9-16), (9-18), (9-19), (9-22), (9-23), (9-25), (9-26), and (9-28) through (9-30) predict the values of bending moment, shear, and differential deflection expected to occur using a given set of soil and structural parameters. These design values may be calculated for slabs reinforced with unstressed and stressed reinforcement. Once these design parameters are known, design of either type of slab can proceed. This report does not provide design procedures for non-post-tensioned slabs-on-ground. To conform to the same deflection criteria, however, non-post-tensioned slabs designed on the basis of cracked sections will need significantly deeper beam stems than post-tensioned slabs.

9.8.5.2 *Design of slabs subject to frost heave*—Applied moments, shears, and deflections due to frost heave can be approximated by substituting anticipated frost heave for expected swell of an expansive clay. The value of e_m for frost heave should be estimated from values comparable to those for expansive soils.

9.8.6 *Calculation of stress in slabs due to load-bearing partitions*—The equation for the allowable tensile stress in a slab beneath a bearing partition may be derived from beam-on-elastic foundation theory. The maximum moment directly under a point load, P (kips [kN]), in such a beam is

$$M_{max} = -\frac{P\beta}{4} \quad (9\text{-}34)$$

where

$$\beta = \left[\frac{4E_cI}{k_sB_w}\right]^{0.25} \le S \quad \text{(in.-lb)} \quad (9\text{-}35)$$

$$\beta = \frac{12}{1000}\left[\frac{4E_cI}{k_sB_w}\right]^{0.25} \le S \quad \text{(SI)}$$

with E_c = 1,500,000 psi (10,340 MPa), and k_s = 4 lb/in.^{3} (0.00109 N/mm^3)

$$\frac{I}{B_w} = \frac{B_wt^3}{12B_w} = \frac{t^3}{12} \quad (9\text{-}36)$$

$$\beta = \left[\frac{4(1{,}500{,}000)t^3}{4(12)}\right]^{0.25} = 18.8t^{0.75} \quad \text{(in.-lb)} \quad (9\text{-}37)$$

$$\beta = \frac{12}{1000}\left[\frac{4(10{,}340)t^3}{0.00109(12)}\right]^{0.25} = 0.506t^{0.75} \quad \text{(SI)}$$

Therefore

$$M_{max} = -\frac{18.8Pt^{0.75}}{4} = -4.7Pt^{0.75} \quad \text{(in.-lb)} \quad (9\text{-}38)$$

$$M_{max} = -\frac{0.506Pt^{0.75}}{4} = -0.127Pt^{0.75} \quad \text{(SI)}$$

The equation for applied tensile stress f is

$$f = \frac{P_r}{A} - \frac{M_{max}c}{I} \quad (9\text{-}39)$$

because

$$\frac{I}{c} = \frac{B_wt^3}{12}\left(\frac{2}{t}\right) = \frac{B_wt^2}{6} = \frac{12t^2}{6} = 2t^2 \quad \text{(in.-lb)} \quad (9\text{-}40)$$

$$\frac{I}{c} = \frac{B_wt^3}{12}\left(\frac{2}{t}\right) = \frac{B_wt^2}{6} = \frac{1000t^2}{6} = 167t^2 \quad \text{(SI)}$$

The applied tensile stress is

$$f = \frac{P_r}{A} - \frac{4.7Pt^{0.75}}{2t^2} = \frac{P_r}{A} - 2.35\frac{P}{t^{1.25}} = \frac{P_r}{A} - C_p\frac{P}{t^{1.25}} \quad \text{(in.-lb)} \quad (9\text{-}41)$$

$$f = \frac{P_r}{A} - \frac{0.127(1000)Pt^{0.75}}{167t^2} = \frac{P_r}{A} - 0.76\frac{P}{t^{1.25}} = \frac{P_r}{A} - C_p\frac{P}{t^{1.25}} \quad \text{(SI)}$$

For uniform thickness foundations, H can be substituted for t in Eq. (9-37), (9-38), and (9-41). The value of C_p depends on the assumed value of the subgrade modulus k_s. The following table illustrates the variation in C_p for different values of k_s

Type of subgrade	k_s, lb/in.^{3} (N/mm^3)	C_p
Lightly compacted, high plastic, compressible soil	4 (0.00109)	2.35 (0.760)
Compacted, low plastic soil	40 (0.0109)	1.34 (0.425)
Stiff, compacted, select granular or stabilized fill	400 (0.109)	0.74 (0.240)

If the allowable tensile stress $6\sqrt{f'_c}$ psi ($0.5\sqrt{f'_c}$ MPa) is exceeded by the results of the aforementioned analysis, a thicker slab section should be used under the loaded area, or a stiffening beam should be placed directly beneath the concentrated line load.

9.9—Design for slabs on compressible soil

Compressible soils are nonexpansive granular soils containing little or no clay whose allowable bearing capacity is 1500 lb/ft^2 (72,000 N/m^2) or less. The following equations will determine the moment, deflection, and shear requirements for slabs cast on compressible ground. The design engineer may reinforce the slab using either reinforcing steel, post-tensioning tendons, or a combination of both, to meet these strength requirements. Equations (9-42) and (9-45) determine the flexural strength requirements; Eq. (9-46) determines the deflection requirement; and Eq. (9-47) and (9-49) determine the shear requirements for design using either reinforcing steel or post-tensioning tendons.

9.9.1 *Moments—*

Moment in the long direction

$$M_{cs_L} = \left(\frac{\delta}{\Delta_{ns_L}}\right)^{0.50} M_{ns_L} \quad (9\text{-}42)$$

where

$$M_{ns_L} = \frac{(h)^{1.35}(S)^{0.36}}{80(L)^{0.12}(P)^{0.10}} \quad \text{(in.-lb)} \quad (9\text{-}43)$$

$$M_{ns_L} = \frac{(h)^{1.35}(S)^{0.36}}{788(L)^{0.12}(P)^{0.10}} \quad \text{(SI)}$$

$$\Delta_{ns_L} = \frac{(L)^{1.28}(S)^{0.80}}{133(h)^{0.28}(P)^{0.62}} \quad \text{(in.-lb)} \quad (9\text{-}44)$$

$$\Delta_{ns_L} = \frac{29.5(L)^{1.28}(S)^{0.80}}{(h)^{0.28}(P)^{0.62}} \quad \text{(SI)}$$

where δ = expected settlement, in. (mm), due to the total load expressed as a uniform load; reported by the geotechnical engineer.

Moment in the short direction

$$M_{cs_S} = \left(\frac{970 - h}{880}\right) M_{cs_L} \quad \text{(in.-lb)} \qquad (9\text{-}45)$$

$$M_{cs_S} = \left(\frac{24{,}600 - h}{22{,}400}\right) M_{cs_L} \quad \text{(SI)}$$

9.9.2 *Anticipated differential deflection—*

$$\Delta_{cs} = \delta e_n[1.78 - 0.103(h) - 1.65 \times 10^{-3}(P) + 3.95 \times 10^{-7}(P)^2] \quad \text{(in.-lb)} \qquad (9\text{-}46)$$

$$\Delta_{cs} = \delta e_n[1.78 - 4.06 \times 10^{-3}(h) - 1.13 \times 10^{-4}(P) + 1.86 \times 10^{-9}(P)^2] \quad \text{(SI)}$$

9.9.3 *Shear—*

Long direction

$$V_{cs_L} = \left[\frac{\delta}{\Delta_{ns_L}}\right]^{0.30} V_{ns_L} \qquad (9\text{-}47)$$

where

$$V_{ns_L} = \frac{(h)^{0.90}(PS)^{0.30}}{550(L)^{0.10}} \quad \text{(in.-lb)} \qquad (9\text{-}48)$$

$$V_{ns_L} = \frac{(h)^{0.90}(PS)^{0.30}}{1220(L)^{0.10}} \quad \text{(SI)}$$

Short direction

$$V_{cs_S} = \left[\frac{116 - h}{94}\right] V_{cs_L} \quad \text{(in.-lb)} \qquad (9\text{-}49)$$

$$V_{cs_S} = \left[\frac{2950 - h}{2390}\right] V_{cs_L} \quad \text{(SI)}$$

Appendix 4 provides an example of tendon selection along with the necessary tables.

CHAPTER 10—FIBER-REINFORCED CONCRETE SLABS-ON-GROUND

10.1—Introduction

Polymeric 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. Polymeric fibers of nylon and polypropylene have been used to provide control of random plastic-shrinkage cracking. Steel fibers and some polymeric fibers have been used to provide random crack control in concrete after it reaches a hardened state. The combination of both products in concrete contributes both plastic and hardened state benefits. This chapter presents polymeric and steel fiber-reinforced concrete (FRC) material properties and design methods for FRC slabs-on-ground. The designer should understand that the performance of FRC slabs-on-ground is dependent 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.

10.2—Polymeric fiber reinforcement

Polymeric fibers are used to reinforce concrete against plastic shrinkage and drying shrinkage stresses. Fine monofilament (denier less than 100) or fibrillated polymeric fibers are typically added at low volume addition (LVA) rates of 0.1% or less of concrete volume for plastic shrinkage crack control. Macropolymeric fibers (denier greater than 1000) are typically added at high volume addition (HVA) rates of 0.3 to 1% by volume for drying shrinkage crack control.

The length of fibers used for slab-on-ground applications can range between 1/2 to 2.0 in. (13 to 51 mm).

10.2.1 *Properties of polymeric fibers*—The addition of polymeric fibers to concrete for plastic shrinkage crack control provides a mechanism that increases the concrete's tensile capacity 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. Microfibers provide support for the coarse aggregate and enhance the mixture uniformity. Some micropolymeric fibers can increase the fracture toughness of concrete slabs-on-ground in the hardened state.

The compressive, flexural, and tensile strength of concrete is not significantly altered by inclusion of micropolymeric fibers at quantity rates of 0.1% by volume or less. The flexural toughness of concrete may be increased significantly with macropolymeric fibers at quantities between 0.3 to 1.0% by volume. ASTM C 1399 provides a quantitative measure that is useful in the evaluation of the performance of polymeric FRC in the hardened state. The results of this test method can be used to optimize the proportions of FRC, to determine compliance with construction specifications and to evaluate FRC that is already in service.

More information on properties of polymeric fibers, including elastic moduli, tensile strengths, and specific gravities, is available in ACI 544.1R.

10.2.2 *Design principles*—The design principles for micropolymeric FRC are the same as those used for unreinforced concrete.

Macropolymeric fibers provide increased post-cracking residual strength to concrete slabs-on-ground. The same design principles in Section 10.3.3 can be used for macropolymeric FRC.

10.2.3 *Joint details*—Construction and sawcut contraction joint details and spacing for micropolymeric FRC are the same as those used for unreinforced concrete. Macropolymeric fibers at quantities between 0.3 to 1% by volume increase the post-cracking residual strength of the concrete.

This material behavior permits wider sawcut contraction joint spacing; however, load transfer stability at sawn contraction joints should be considered carefully at wider joint spacing.

10.3—Steel fiber reinforcement

Steel fibers are used to reinforce concrete slabs-on-ground to provide increased strain capacity, impact resistance, flexural toughness, fatigue endurance, and tensile strength (ACI 544.4R). Steel fibers are either 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 between 3/4 to 2-1/2 in. (19 to 64 mm).

10.3.1 *Properties of steel fibers*—Steel fibers for concrete reinforcement are short, discrete lengths of steel having an aspect ratio (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 A 820 provides classification for four 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 A 820 also established 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) cylinder 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 things. Long-term loading does not adversely influence the mechanical properties of steel FRC.

10.3.2 *Properties of steel FRC*—The properties of FRC in both 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 parameters used for design. ACI 544.3R should be consulted to ensure that proper mixing, placing, and finishing guidelines are met.

10.3.2.1 *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.

10.3.2.2 *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 the average crack widths. Steel FRC, when used in combination with conventional deformed or smooth continuous reinforcement, will have synergistic effects, and can be designed to share the applied tensile forces with the continuous reinforcement, thereby adding to the crack width opening control. The degree of crack width control is directly related to the fiber type and quantity.

10.3.2.3 *Flexural toughness (ductility)*—Flexural toughness of steel FRC is determined by testing beams in a laboratory using JSCE SF4 or ASTM C 1399. It is generally accepted that the presence of steel fibers in quantities < 0.5% by volume, as would be expected in most slabs-on-ground, will not affect the first crack strength (modulus of rupture) of concrete. Steel fibers, however, greatly affect the deformation characteristics of a beam after first crack. Toughness is a measure of the post-cracking energy-absorbing capacity of steel FRC, and is defined as the area under the test beam load-deflection curve. Residual strength factors $R_{e,3}$ and average residual strength (ARS), determined according to JSCE SF4 and ASTM C 1399, respectively, are used in slab-on-ground design. These factors represent an average value of load-carrying capacity of the test beam over a deflection interval. 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. Further discussions of these test methods can be found in ACI 544.2R, ASTM STP 169C, and ACI SP-155 (Stevens et al. 1995). The residual strength factor $R_{e,3}$ will be used in this document to represent the postcrack 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.

10.3.2.4 *Impact resistance*—The impact resistance of steel FRC has been determined to be as much as three to 10 times greater than that of 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.

10.3.2.5 *Fatigue resistance*—The fatigue strength at two million cycles for plain concrete is approximately 50% of the static rupture modulus. This is the basis for the well-known safety factor 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 is directly related to the mixture proportions and all mixture constituents, including fiber type and quantity.

10.3.2.6 *Shear resistance*—Steel FRC can provide higher punching shear resistance and anchor bolt pullout resistance as compared with plain concrete. The degree of shear resistance is directly related to the mixture proportion and all mixture constituents, including fiber type and quantity.

10.3.2.7 *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 (for consistency) exposed to freezing and thawing.

10.3.2.8 *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, steel fiber corrosion is limited to a 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 similar to conventional reinforcement but without causing spalling (Hoff 1987). Previous studies showed that crack widths of less than 0.004 in. (0.1 mm) do not allow corrosion of steel fibers passing the crack (Morse and Williamson 1977); however, more recent studies show that crack widths up to 0.02 in. (0.5 mm) have no adverse effect on corrosion of steel fibers. If cracks wider than 0.02 in (0.5 mm) are limited in depth, the consequences of this localized corrosion may not be structurally significant.

10.3.3 *Thickness design methods*—Three methods available for selecting the thickness of steel FRC slabs-on-ground are described in this section:

- PCA, WRI, and COE thickness design methods;
- Elastic method;
- Yield line method;
- Nonlinear finite modeling;
- Combined steel FRC and bar reinforcement.

These design methods are dependent on steel FRC attaining a minimum level of ductility. In addition, suggested performance levels are provided in Table 10.1 for various floor loading conditions. These values represent a compilation of performance values obtained from trade literature.

10.3.3.1 *PCA/WRI/COE method*—The PCA/WRI/COE methods described in Chapter 6 may all be applied to the design of steel FRC slabs-on-ground. Using this approach, steel fiber reinforcement will be used for serviceability design issues such as temperature and shrinkage crack control, enhanced joint stability, and impact and fatigue resistance. For specific designs or steel fiber quantities, fiber manufacturers should be consulted.

10.3.3.2 *Elastic method*—Slabs-on-ground are designed and their thickness is selected to prevent cracking due to external loading, as discussed in Chapter 6, with the following modifications. Steel fibers are accounted for 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

Table 10.1—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

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

When steel fibers are added at high rates (> 0.5% by volume), the modulus of rupture may be increased.

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

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

This would be compared to a unreinforced slab that would have an allowable flexural strength of 0.50 × 570 psi = 285 psi.

10.3.3.3 *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. The use of plastic hinges permits the use of the full moment capacity of the slab and an accurate determination of its ultimate load capacity. Because the formation of plastic hinges is dependent on ductility, it is recommended that the minimum $R_{e,3}$ residual strength be greater than 30%. The results of recent tests (Beckett 1995) have 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 the location of the load with respect to the edges of the slab, which might be considered.

Case 1: *Central load on large slab*

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

For this case, the value of M_o can be expressed 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, the value of M_o can be expressed 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, the value of M_o can be expressed 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);
f_r = concrete modulus of rupture, psi (MPa);
h = slab thickness, in. (mm);
L = radius of relative stiffness, in. (mm);
M_n = negative moment capacity of the slab, tension at top slab surface, in.-lb (N-mm);
M_p = positive moment capacity of the slab, tension at bottom slab surface, in.-lb (N-mm);
P_o = ultimate load capacity 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 takes account of the ductility of steel FRC slabs-on-ground. The same safety factors as those given in Chapter 4 apply.

10.3.3.4 *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. The design process is typically iterative. Once final stresses are determined, a residual strength factor $R_{e,3}$ can be calculated to determine the appropriate steel fiber quantity.

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

10.3.4 *Joint details*—The three types of joints commonly used in concrete slabs-on-ground are isolation joints, sawcut contraction joints, and construction joints. Isolation and construction joints for SFRC floors should be designed as discussed in Chapter 5.

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

As mentioned in Chapter 5, sawcut contraction joints are usually located on column lines, and intermediate joints are located at predetermined spacings. In addition to the amount of steel fibers added to the mixture, the other factors listed in Section 5.1.3 should be considered when selecting the 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. When joint spacings greater than 24 to 36 times the slab thickness are required, higher quantities of steel fiber reinforcement will be required to ensure proper crack containment and shear-load transfer across the sawn joints. In addition to increased fiber quantities, items that need to be considered when increased sawcut contraction joint spacings are required are: blended aggregate gradation optimization (as recommended in Table 2.1 of ACI 544.1R), water reducers, adequate curing, a choker run base material, and a slip membrane. More information on actual case studies is available (Shashanni et al. 2000; Destree 2000).

The same families of tools as discussed in Chapter 5 can be used to sawcut joints in steel FRC slabs. The depth of sawcut using a conventional wet saw should be approximately 1/3 of the slab depth (depending on fiber type and quantity dosage).

Experience has shown that when timely cutting is done with an early-entry saw, the depth can be the same as for plain concrete for lower fiber concentrations, and preferably 1-1/2 ± 1/4 in. (38 ± 6 mm) for higher fiber concentrations up to a 9 in. (230 mm) thick slab. Longer waiting periods may be necessary for all types of sawing of steel FRC floors. Best results are achieved when sufficient time has elapsed so that fibers are cut by the blade and not pulled out of the slab surface.

CHAPTER 11—STRUCTURAL SLABS-ON-GROUND SUPPORTING BUILDING CODE LOADS

11.1—Introduction

There are cases where the slab-on-ground transmits vertical loads or lateral forces from other portions of the structure to the soil. For example, storage rack columns can be used to support the building roof, and there are times when a mezzanine is supported only by the slab-on-ground. These structural slabs should be designed in accordance with ACI 318.

11.2—Design considerations

Strength and serviceability are the two 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.

CHAPTER 12—DESIGN OF SLABS FOR REFRIGERATED FACILITIES

12.1—Introduction

This chapter describes the design considerations for concrete slabs in refrigerated buildings. The typical construction for a floor in a refrigerated building consists of a slab on a slip sheet on insulation on a vapor retarder/barrier on either a soil base or a subslab. Refer to Fig. 12.1.

The floor slab is considered a slab-on-ground. The slip sheet is typically a polyethylene film (6 mil [0.15 mm] minimum thickness) used as 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. When insulation is used, it is typically extruded polystyrene board, rigid polyurethane board, or cellular glass board insulation. The vapor retarder/barrier is under the insulation and a polyethylene film (10 mil [0.25 mm] minimum thickness), 45 mil (1.14 mm) EPDM, or bituminous materials in the form of liquid-applied coatings or composite sheets have been used. For refrigerated buildings, vapor retarders/barriers are always installed on the warm side of the insulation. Under the vapor retarder/barrier, there is either a soil base or a subslab. Many times, a subslab is installed for ease of insulated floor-system construction, or it may encase a grid of heating pipes or conduits. For refrigerated buildings with operating temperatures below freezing, an under-floor heating system is required to prevent the ground from freezing and heaving. The insulated floor system may also be installed over a structural slab supported by deep foundations such as piles.

12.2—Design and specification considerations

A floor slab installed over insulation is designed as a slab-on-ground. The slab type and method of design can be any one of the types as described in other chapters. Slab thickness and reinforcement design should follow same methods and guidelines as described elsewhere in this document. The differences and special considerations for slabs in refrigerated buildings are described in the following paragraphs.

12.2.1 *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, the strength of the insulation should be considered in a similar manner. The design methods in this report use the modulus of subgrade reaction to account for soil properties in design. Insulation also has a similar modulus to consider in slab-on-ground design.

Data provided by the manufacturer using ASTM D 1621 results should not be used in conjunction with the design methods presented in this chapter. Instead, the insulation should be treated like a subgrade and the modulus determined using the plate bearing test described in Chapter 3 (ASTM D 1196). The value of k is normally defined as the pressure to cause a 30 in. (760 mm) diameter plate to deflect

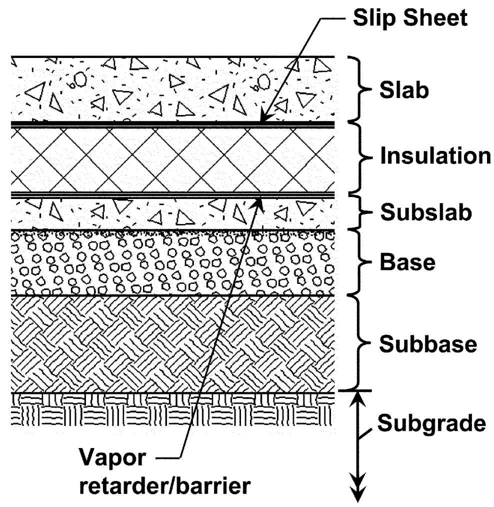


Fig. 12.1—Typical construction for refrigerated building slab-on-ground.

0.05 in. (1.3 mm). The COE, however, determines k for the deformation obtained under a 10 psi (0.07 MPa) load.

12.2.2 *Compressive creep*—Compressive loading on insulation causes deformation in the insulation. The deformation will increase if the load continues to be applied to the insulation. In addition to the instantaneous deformation described by the insulation modulus, there will be 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. Guidelines may vary from manufacturer to manufacturer.

12.2.3 *Reinforcement*—Slabs in refrigerated facilities do not require special considerations for reinforcement because of room temperature. The design of any reinforcement should follow methods described elsewhere in the document. If 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.

12.2.4 *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. This inadequacy is due to the temperature shrinkage in the slabs, which causes the joints to open wider, thus causing those joints to be ineffective in load transfer. Joints should be filled after refrigerated rooms are at operating temperature so as to allow the slab to contract and stabilize because of the temperature reduction. The colder the room or the greater the temperature reduction, the more the slab will contract. The slab will take 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 (embedding steel angles or 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 is performed less often because of the cold temperatures and because of the limited availability of products that work at these temperatures. Refer to Chapter 5 for more information on floor joints.

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

12.2.6 *Underslab tolerance*—The elevation tolerance for the soil base or for the subslab, if used, is important because the rigid board floor insulation will mirror the surface upon which it bears. The surface of the insulation typically cannot be adjusted. If a subslab is used, the surface should have either a smooth flat finish or a light steel-trowel finish. Irregularities in the soil base or subslab should be avoided because the insulation may bear on high points and be pushed up higher than the adjacent insulation board. A high point may also 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).

12.2.7 *Forming*—Typically, slab-on-ground forms are staked into the soil below. For a refrigerated building, however, this would not be acceptable because of the floor insulation and the vapor retarder/barrier. Forms for this type of floor are constructed with a form mounted vertically on a horizontal base, such as plywood. This L-shaped form is placed on the insulation and sandbagged to hold it in place.

12.3—Temperature drawdown

The temperature reduction for refrigerated rooms should be gradual to control cracking caused by differential thermal contraction and to allow drying to remove excess moisture from the slab after curing. A typical drawdown schedule might be as follows:

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

CHAPTER 13—REDUCING EFFECTS OF SLAB SHRINKAGE AND CURLING

13.1—Introduction

This chapter covers design methods used to reduce the effect of drying shrinkage and curling (warping) in slabs-on-ground. The material is largely based on Ytterberg's three articles (Ytterberg 1987). Further analysis and discussion can be found in Walker and Holland's article "The First Commandment for Floor Slabs: Thou Shalt Not Curl Nor Crack…(Hopefully)" (1999). For additional information on concrete shrinkage, refer to ACI 209R and references provided by Ytterberg (1987).

To be workable enough to be placed, virtually all concrete is produced with approximately twice as much water as is 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. Such moisture gradients are magnified by moist subgrades and by low humidity at the top surface. Evaporation of moisture from the top surface of a slab causes the upper half of the slab to shrink more than the lower half, although some shrinkage occurs in all three dimensions. 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) indicated curling stresses can easily range from 200 to 450 psi (1.4 to 3.1 MPa). Awareness of moisture content of subgrades and shrinkage potential of concrete should be given the same importance as compressive strength and slump testing of the slab concrete because neither of the latter two tests is a good indicator 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 may 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, resulting 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 reduction of calculated slab thickness. Walker and Holland (1999) have shown that under certain conditions, however, higher compressive strength can actually decrease load-carrying capacity due to increased curling stress. The higher strengths can improve durability; however, designers should look at 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 3. Excess moisture in the subgrade adds to the moisture gradient already present in slabs-on-ground and thereby increases slab curling.

Designers can take steps to reduce shrinkage cracking and curling through appropriate design and specification provisions. 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.

13.2—Drying and thermal shrinkage

Typical portland-cement concrete, along with shrinkage-compensating concrete, shrinks approximately 0.04 to 0.08% due to drying (PCA 1967). 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 that at which the slab was initially placed. It should be taken into account for any floor where the concrete is cast 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 °C to –18 °C) can shorten a 100 ft (30 m) slab by 0.46 in. (12 mm), assuming no subgrade restraint.

13.3—Curling and warping

Curling of concrete slabs at joints and cracks is directly related to drying shrinkage. Therefore, if an effort is made to reduce drying shrinkage, curling will also be reduced. The terms "curling" and "warping" are used interchangeably in this document, in conformance with ACI 116R, which defines them as follows:

curling—the distortion of an originally essentially linear or planar member into a curved shape, such as the warping of a slab to differences in temperature or moisture content in the zones adjacent to its opposite faces. (See also warping.)

warping—a deviation of a slab or wall surface from its original shape, usually caused by either temperature or moisture differentials or both within the slab or wall. (See also curling.)

Curling occurs at slab edges because of differential shrinkage. The upper part of the slab-on-ground almost always has the greatest shrinkage because the top surface is commonly free to dry faster, and the upper portion has a higher unit water content at the time of final set. A higher relative humidity in the ambient air at the upper surface will reduce the severity of curling even though the concrete may be a high shrinkage material. Curvature occurs over the whole slab panel, but per 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, which includes joints with or without positive load-transfer devices and cracks wider than hairline. Figures 13.1 and 13.2 show the curling effect in exaggerated fashion.

13.4—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 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 13.1 shows the cumulative effect of these eight

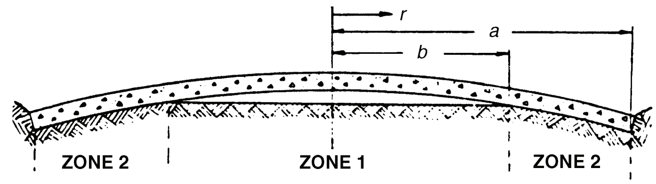


Fig. 13.1—Highway slab edges curl downward at edges during the day when the sun warms the top of the slab.

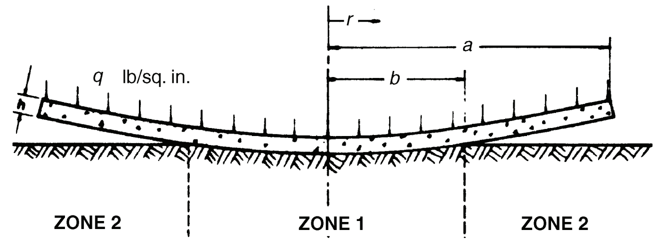


Fig. 13.2—Slabs indoors curl upward because of the moisture differential between top and bottom of the slabs.

factors, resulting in about a fourfold increase in drying shrinkage rather than a twofold increase if arithmetically added. The influence of four of these factors on water demand of the concrete is discussed.

13.4.1 *Effect of maximum size of coarse aggregate*—Table 13.1 shows that the use of 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 3/4 in. (19 mm) maximum-size aggregate as compared with 1-1/2 in. (38 mm) maximum-size aggregate. In addition to the water demand effect, aggregate generally acts to control (reduce) shrinkage by restraining the shrinkage of the cement paste. To minimize shrinkage of the cement paste, the concrete should contain the maximum practical amount of incompressible, clean aggregate.

In actual practice, the dry-rodded volume of the coarse aggregate is approximately 50 to 66% of the concrete volume if 1/2 in. (13 mm) maximum-size aggregate is used, but can be as high as 75% if 1-1/2 in. (38 mm) maximum-size aggregate is used (ACI 211.1). Use of 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.

13.4.2 *Influence of cement*—Table 13.1 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 inexpensively achieved using Type I or Type III cement because these cements usually give higher early strength than Type II, other things being equal. Designers should specify the type of cement to be used for slabs-on-ground. Type I and III cements can cause higher concrete shrinkage than Type II cement because of their physical and chemical differences. Thus, specifying minimum concrete compressive strength without regard to either cement type or relative cement mortar shrinkage can contribute to slab shrinkage and

Table 13.1—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%

curling. Because the quality of cement may vary from brand to brand and within brand, comparative cement mortar shrinkage tests (ASTM C 157) conducted before the start of a project are desirable.

13.4.3 *Influence of slump*—Table 13.1 shows that 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. This increase in shrinkage potential would be anticipated if the slump increase was due to additional water or admixtures that increase the shrinkage. If 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. Many factors should be specified and controlled to have a satisfactory slab with regard to shrinkage in the hardened state.

13.4.4 *Influence of water-reducing admixtures*—Water reductions of approximately 7% may be achieved with ASTM C 494 Type A water-reducing admixtures, but their effect on shrinkage and curling is minimal. Tremper and Spellman (1963) and others, however, have found that chloride-based admixtures of this type definitely increase shrinkage of the concrete.

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 reducers, will not always decrease shrinkage proportionally. In many cases, shrinkage is not changed significantly by the introduction of a water-reducing or high-range water-reducing admixture (Types A and F, ASTM C 494) or by a nominal decrease in slump from 5 to 3 in (130 to 76 mm). Designers should note that ASTM C 494 allows concrete manufactured with admixtures to have up to 35% greater shrinkage than the same concrete without the admixture.

13.5—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 reducer that increases shrinkage are the typical means 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 sufficient to transmit loads to the subgrade. A 60-day, 90-day, or longer strength, rather than a 28-day strength, might be considered for designing slab thickness. This would assume that the design loads would not be applied during the first 60 or 90 days.

Instead of using a high design strength to minimize slab thickness, designers might consider other alternatives, such as adding conventional reinforcement or post-tensioning. For another example, 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).

13.6—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 upper parts of slabs have a higher water content than the lower portion because of the gravity effect on concrete material before set takes place. Pawlowski et al. (1975) report that compressive strengths are always higher in the lower half of floors, and shrinkage is always higher in the upper half.

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 be used to improve abrasion resistance.

13.7—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 pitwalls to the floor slab. If 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.

In most industrial slabs-on-ground, it is desirable to reduce joints to a minimum because joints become 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 that they do not either reduce the slab thickness or restrain drying shrinkage.

Restraint parallel to joints due to conventional round dowels can be eliminated by the use of square, diamond-plate, or rectangular-plate dowel systems with formed voids or compressible isolation material on the bar/plate sides to allow transverse and longitudinal movement while transferring vertical load. Refer to Chapter 5 for more information.

13.8—Base and vapor retarders/barriers

A permeable base, with a smooth, low-friction surface helps reduce shrinkage cracking because it allows the slab to shrink with minimal restraint. A relatively dry base also allows some of the 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. If used, 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 3. One option more fully discussed in Chapter 3 is for the retarder/barrier to be covered 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, however, will improve constructibility and minimize damage. Nicholson (1981) showed that serious shrinkage cracking and curling can occur when concrete slabs are cast on an impervious base. If the base is kept moist by groundwater or if the slab is placed on a wet base, then this will increase the moisture gradient in the slab and will increase curl. If the aggregate material over the vapor retarder/barrier is not dry enough at concrete placement, however, it will not act as a blotter and can aggravate curling and moisture problems. Thus, in spite of the inherent problems with placing the concrete directly on the vapor retarder/barrier, it is better to do so if there is a chance that the aggregate blotter will not be relatively dry; refer to the discussion in Chapter 3. If crushed stone is used 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 will allow the slab-on-ground to shrink with minimum restraint.

If 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 bottom of the slab before the concrete sets.

Figure 13.3 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. For long (over 100 ft [30 m]) post-tensioned slabs, 0.5 might be used to account for variations in base elevation over longer distances.

13.9—Distributed reinforcement to reduce curling and number of joints

Because it is the upper part of a floor slab that has the greatest shrinkage, the reinforcement should be in the upper half of that slab so that the steel will restrain shrinkage of the concrete. 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 deformed wire or bar should have a minimum diameter of 3/8 in. (9 mm) to provide sufficient stiffness to prevent bending during concrete placement.

For unreinforced slabs, joint spacings of 24 to 36 times the slab thickness up to 18 ft (5.5 m) have generally produced acceptable results. A closer spacing, however, is more likely to accommodate the higher shrinkage concrete mixtures often encountered (refer to Fig. 5.6 for recommendations). If greater joint spacings than these are desirable to reduce maintenance, the designer should 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. The specified steel should be stiff enough and have a large enough spacing so that it is practical to expect the steel to be placed (and remain) in the upper half of the slab. Joint locations should be detailed on the slab construction drawings.

13.10—Thickened edges to reduce curling

Curling is greatest at corners of slabs, and corner curling is reduced as slab thickness increases (Child and Kapernick 1958). For example, 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 subjected to loading is a design strategy that accounts for the difference in mid-panel and edge load response in slabs of constant thickness. Edge curling may be reduced by thickening slab edges. The thickened edge contributes added weight and also reduces the surface area exposed to drying relative to the volume of concrete, both of which help to reduce upward curling. Free slab edges and edges at construction joints where positive load-transfer devices, such as dowels, are not provided should be thickened 50% with a gradual 1-in-10 slope.

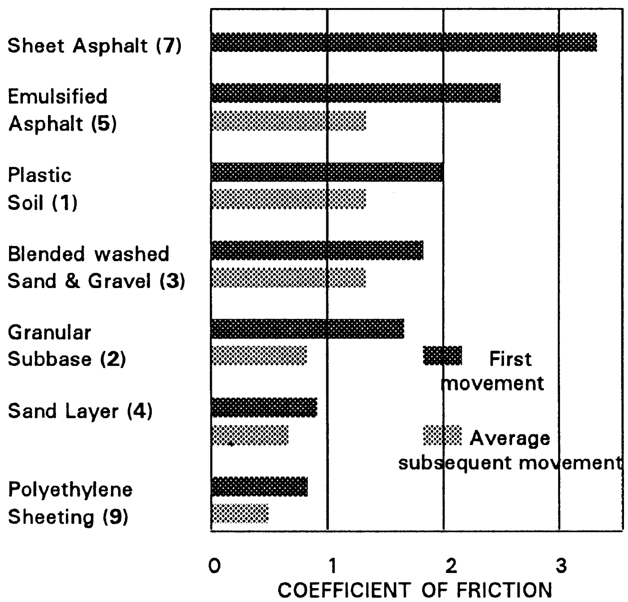


Fig. 13.3—Variation in values of coefficient of friction for 5 in. (125 mm) slabs on different bases (based on Design and Construction of Post-Tensioned Slabs on Ground*, Post-Tensioning Institute, Phoenix, Ariz. [2004]).*

Provided that the subgrade is smooth with a low coefficient of friction, as detailed in Section 13.8, then thickened edges should not be a significant linear shrinkage restraint; however, curling stresses would be increased somewhat.

13.11—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 will delay shrinkage and curling of enclosed slabs-on-ground.

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

Water curing can saturate the base and subgrade, creating a reservoir of water that may eventually transmit through the slab. This is also discussed in Chapter 3.

All curing methods have limited life spans when the concrete's top surface is exposed to wear. Thus, curing does not have the same effect as long-term high ambient relative humidity. Extended curing only delays curling; it does not reduce curling.

13.12—Warping stresses in relation to joint spacing

Several sources (Kelley 1939; Leonards and Harr 1959; Walker and Holland 1999) have shown that the warping stress increases as the slab length increases only up to a certain slab length. 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) 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 only slightly with changes in modulus of elasticity and modulus of subgrade reaction. Figure 13.4 shows both 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.

PCA (Spears and Panarese 1983) states that there will be a marked loss of effectiveness of aggregate interlock at sawcut contraction joints if the joints are too far apart. Positive load transfer using dowels or plates should be provided where joints are expected to open more than 0.035 in. (0.9 mm) for slabs subjected to wheel traffic (refer to Section 5.2 for additional information). Slabs may be more economical if 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. The lowest floor and fork lift truck maintenance cost may well be achieved 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.

13.13—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 assumption that slabs-on-ground would be fully supported by the subgrade when they warped from temperature gradients. This assumption is not correct.

When slabs-on-ground warp from temperature or moisture gradients, they are not fully supported by the subgrade, and unsupported edges suffer higher stresses than if they were supported. These factors can be taken into account as described in Walker and Holland (1999).

In 1938, Bradbury (1938) extended Westergaard's work with a working stress formula referred to as the Westergaard-Bradbury formula. This formula is still in use today (Packard 1976). In 1939, Kelley (1939) used the Westergaard-Bradbury formula to calculate the warping stresses shown in Fig. 13.5 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. 13.6, presented here for general understanding. The upper center set of curves in Fig. 13.6 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 while 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 form of computer modeling that permitted the slab to lift off the subgrade if the uplift force was greater than the gravity force. Figure 13.7 shows their vertical deflection curves for the same six cases of slabs whose warping stresses are shown in Fig. 13.6. The upward slab edge lift and downward slab center deflection shown in Fig. 13.7 is the usual case for slabs inside buildings. True temperature gradient is very 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. 13.6 and 13.7, no matter what the thickness. They also assumed a cold top and a hot slab bottom, which is not a usual temperature gradient, but it is a usual equivalent moisture gradient for slabs inside buildings with a very moist bottom and a very dry top.

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 is documented in Ytterberg's 1987 paper. Because the three commonly used slab thickness design methods (PCA, WRI, and COE) all are based on Westergaard's work and on the assumption that the 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 (referred to as the cantilever effect). The thickness 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 current slab thickness design methods permit thinner slabs as the modulus of subgrade reaction increases. The fact is, however, that a higher subgrade reaction modulus will increase 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 becomes stiffer, and resultant load-carrying capacity decreases for edge loadings; however, higher k with loadings away from the edges allows thinner slabs. Proper design should take all of these factors into account.

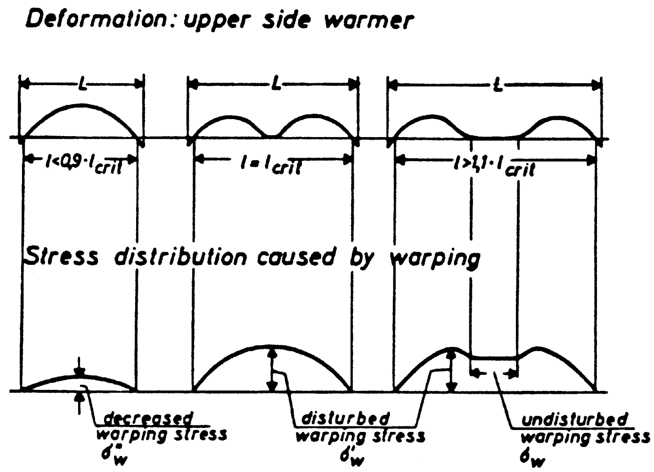


Fig. 13.4—Effect of slab length on warping and warping stress in an exposed highway slab (from Eisenmann [1971]).

ACI Committee 325 (1956) recommends that highway slabs-on-ground be designed for a 3 °F (1.7 °C) per in. (25 mm) daytime positive gradient (downward curl) and a 1 °F (0.6 °C) per in. (25 mm) nighttime negative gradient (upward curl). 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), according to Leonards and Harr (1959).

The Westergaard-Bradbury formula (Yoder and Witczak 1975) concluded that warping stress in slabs is proportional to the modulus of elasticity of concrete, and partially proportional to the modulus of elasticity of aggregates used in a particular concrete. Therefore, 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, if no hardener or topping is used, many low-modulus aggregates will not be as durable for some applications.

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

The total amount of drying shrinkage of concrete is magnified when it is placed in large blocks 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 portland-cement concrete slabs. This is because the intermediate sawcut contraction joints within the latter slabs will take up most of the shrinkage.

Post-tensioned and shrinkage-compensated slabs do not have intermediate sawcut contraction joints. Where vehicle traffic will cross construction joints in post-tensioned or shrinkage-compensated slabs-on-ground, the joints should

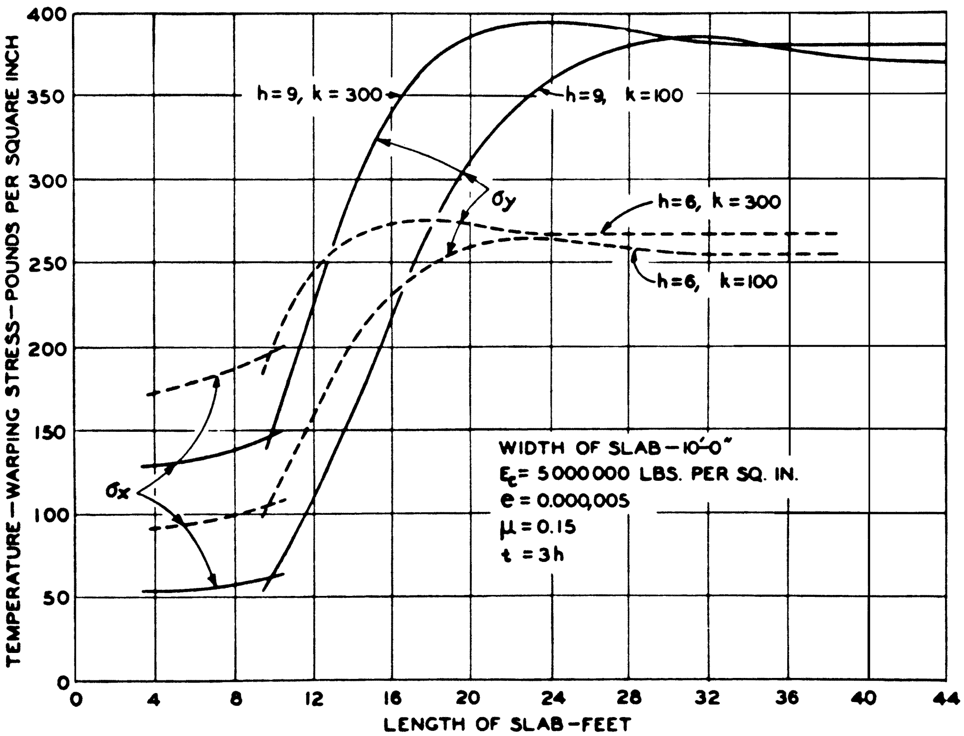


Fig. 13.5—Slab length increases beyond a certain amount do not increase warping stress in the slab interior (Kelly 1939). (Note: 1 psi = 0.006895 MPa; 1 ft = 0.3048 m.)

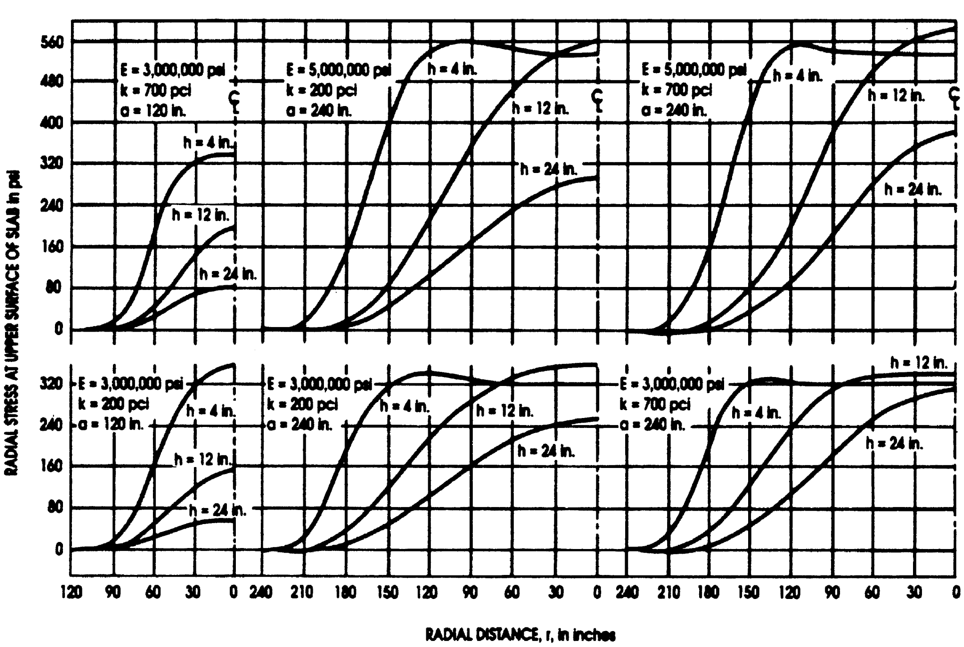


Fig. 13.6—Representative radial stresses for an effective temperature difference of 30 °F between top and bottom (Leonards and Harr 1959). (Note: 1 psi = 0.006895 MPa; 1 in. = 25.4 mm; 1 °F [ΔT] = 0.56 °C.)

be doweled, and the top edges of the construction joints should be protected with back-to-back steel bars or angles, epoxy-armored edges, or by other equally durable material.

13.15—Summary and conclusions

Designers of enclosed slabs-on-ground can reduce shrinkage cracking and shrinkage curling by considering the features that affect these phenomena. The following checklist indicates factors that should be addressed.

SUBGRADE CONDITIONS

- Before and during slab installation, check for smoothness, dryness, and permeability of the base and subgrade. Measure the base and subgrade moisture content.
- Do not use a vapor retarder/barrier unless required to control moisture transmission through the slab. If it is used, decide whether an aggregate blotter should be used over the vapor retarder/barrier.

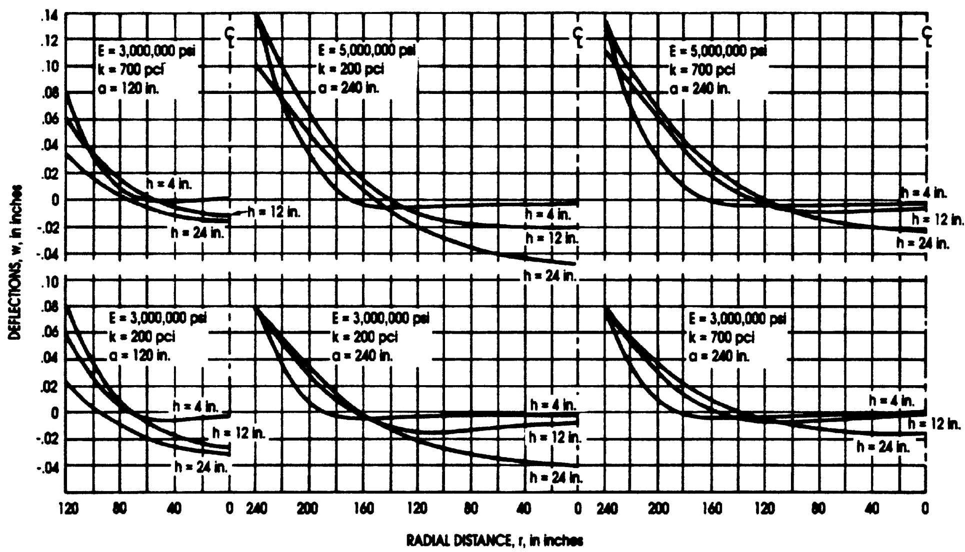


Fig. 13.7—Representative curling deflection curves for 20- and 40-ft slabs with an effective temperature difference of 30 °F between top and bottom (Leonards and Harr 1959). (Note: 1 psi = 0.006895 MPa; 1 in. = 25.4 mm; 1 °F [ΔT] = 0.56 °C; 1 lb/ft^3 = 0.2714 MN/m^3.)

DESIGN DETAILS

- Calculate the slab thickness, and consider thickening slab edges in terms of load-carrying ability and slab restraint.
- Where random cracking is acceptable, specify distributed reinforcement in the upper half of the slab to minimize or eliminate sawcut contraction joints. Shrinkage reinforcement is not needed in the bottom half of slabs-on-ground.
- If 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, diamond-plate, or rectangular-plate dowel systems that eliminate restraint longitudinally and transversely while transferring vertical load.
- Eliminate as many slab restraints as possible, and isolate those that remain.
- Specify largest practical size of base plate for rack posts. Include the base plate size in the slab thickness design process (baseplate thickness must be adequate to distribute post load over area of plate).
- Consider shrinkage-compensating concrete or post-tensioning as design options.

CONTROL OF CONCRETE MIXTURE

- Specify workable concrete with the largest practical maximum size of coarse aggregate and with aggregate gap-grading minimized.
- Specify concrete design strength and the age at which it is to be achieved. Consider using 60- or 90-day strengths in slab thickness design to permit the use of concrete with lower shrinkage than could be obtained with the same compressive strength at 28 days. Use lowest compressive strength and corresponding minimum cement content feasible; use mineral or metallic hardener or topping if 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.
- Consider plant inspection to perform the aforementioned testing and to monitor batching for uniformity (refer to ACI 311.5 for guidance).

CHAPTER 14—REFERENCES

14.1—Referenced standards and reports

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

American Concrete Institute

116R	Cement and Concrete Terminology
117	Standard Specifications for Tolerances for Concrete
201.1R	Guide for Making a Condition Survey 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
311.5	Guide for Plant Inspection and Field Testing of Ready-Mixed Concrete
318	Building Code Requirements for Structural Concrete

330R	Guide for Design and Construction of Concrete Parking Lots
350	Code Requirements for Environmental Engineering Concrete Structures
336.2R	Suggested Analysis and Design Procedures for Combined Footings and Mats
504R	Guide to Sealing Joints in Concrete Structures
544.1R	Fiber Reinforced Concrete
544.2R	Measurement and Properties of Fiber Reinforced Concrete
544.4R	Design Considerations for Steel Fiber Reinforced Concrete

ASTM International

A 36	Specification for Carbon Structural Steel
A 497	Specification for Steel Welded Wire Reinforcement, Deformed, for Concrete Reinforcement
A 615	Specification for Deformed and Plain Carbon Steel Bars for Concrete Reinforcement
A 706	Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A 820	Specification for Steel Fibers for Fiber-Reinforced Concrete
A 996	Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement
C 150	Specification for Portland Cement
C 157	Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete
C 494	Specification for Chemical Admixtures for Concrete
C 595	Specification for Blended Hydraulic Cements
C 845	Specification for Expansive Hydraulic Cement
C 878	Test Method for Restrained Expansion of Shrinkage-Compensating Concrete
C 1399	Test Method for Obtaining Average Residual-Strength of Fiber-Reinforced Concrete
D 422	Method for Particle-Size Analysis of Soils
D 566	Test Method for Dropping Point of Lubricating Grease
D 698	Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lpf/ft^3 (600 kN-m/m^3))
D 854	Test Method for Specific Gravity of Soil Solids by Water Pycnometer
D 1196	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
D 1556	Test Method for Density of Soil in Place by the Sand Cone Method
D 1557	Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lpf/ft^3 (2700 kN-m/m^3))
D 1586	Test Method for Penetration Test and Split-Barrel Sampling of Soils
D 1621	Test Method for Compressive Properties of Rigid Cellular Plastics
D 1883	Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils
D 2167	Test Method for Density and Unit Weight of Soil in Place by the Rubber Balloon Method
D 2216	Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
D 2240	Test Method for Rubber Property—Durometer Hardness
D 2487	Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
D 2488	Practice for Description and Identification of Soils (Visual-Manual Procedure)
D 2922	Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)
D 2937	Test Method for Density of Soil in Place by the Drive-Cylinder Method
D 3017	Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)
D 3575	Test Methods for Flexible Cellular Materials Made from Olefin Polymers
D 4253	Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table
D 4254	Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density
D 4263	Test Method for Indicating Moisture in Concrete by the Plastic Sheet Method
D 4318	Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
F 1869	Test Method for Measuring Moisture Vapor Emission Rate of Concrete Subfloor Using Anhydrous Calcium Chloride
F 2170	Test Method for Determining Relative Humidity in Concrete Floor Slabs Using in situ Probes

The above publications may be obtained from the following organizations:

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

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

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APPENDIX 1—DESIGN EXAMPLES USING PCA 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 are presented 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—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

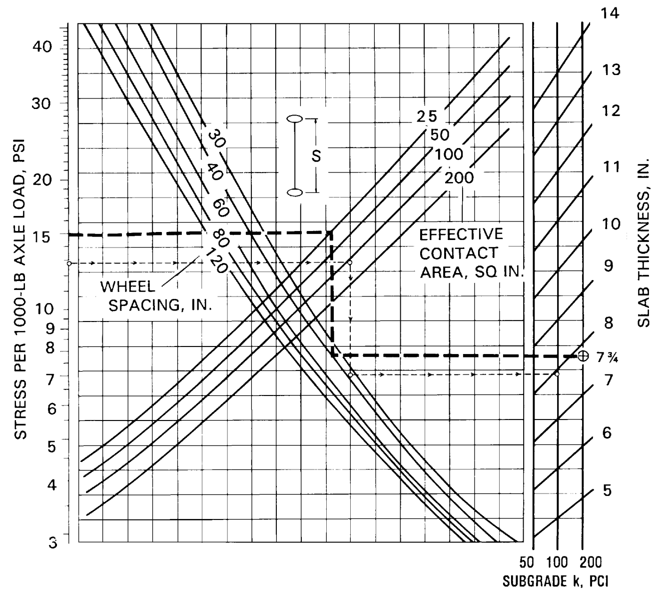


Fig. A1.1—PCA design chart for axles with single wheels.

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 contract area and for the equivalent load factor.

A1.3—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 by assuming the following:

Loading: post load = 15.5 kips
Plate contact area for each post = 36 in.^{2}
Long spacing y = 100 in.
Short spacing x = 40 in.

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

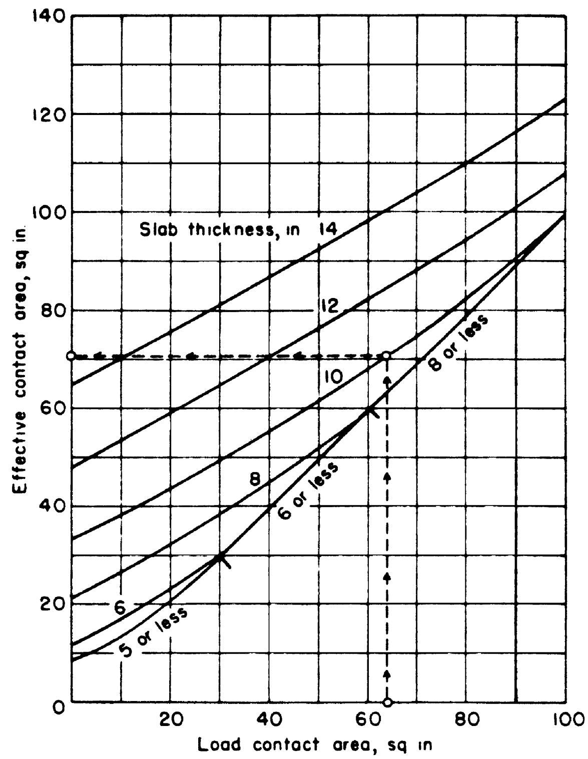


Fig. A1.2—Relationship between load contact area and effective load contact area.

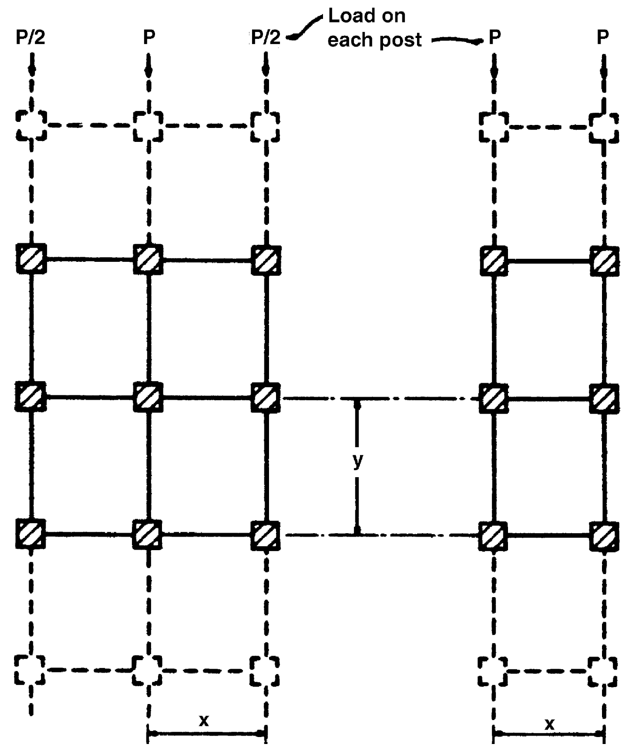


Fig. A1.4—Post configurations and loads.

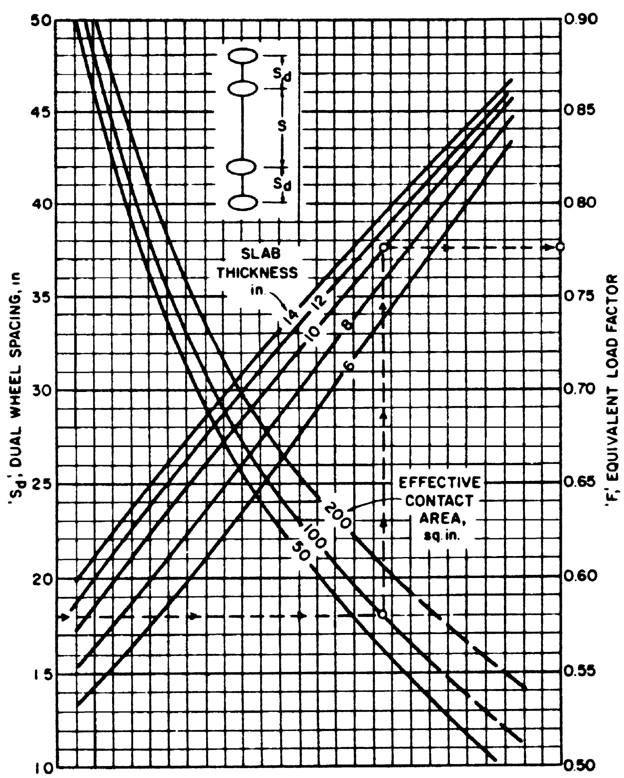


Fig. A1.3—PCA design chart for axles with dual wheels.

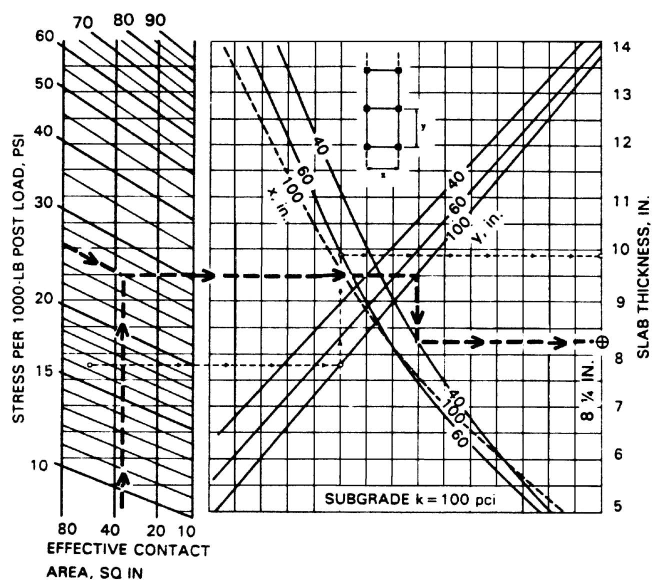


Fig. A1.5—PCA design chart for post loads where subgrade modulus is 100 pci.

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.

A1.4—Other PCA design information

Tables A1.1 and A1.2 are also included for uniform load applications. Examples of their uses may be found in Portland Cement Association (2001) and Ringo (1985).

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.

APPENDIX 2—SLAB THICKNESS DESIGN BY WRI METHOD

A2.1—Introduction

The following two examples show the determination of thickness for a slab-on-ground based on an unreinforced slab. A nominal quantity of distributed reinforcement can be placed in the upper 1/3 of the slab. The primary purpose of this reinforcement is to limit the width of any cracks (if they occur) that may form between the joints. The following examples are presented 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. The first situation is controlled by tension on the bottom of the slab, and the second is controlled by tension on the top of the slab. 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—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, 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

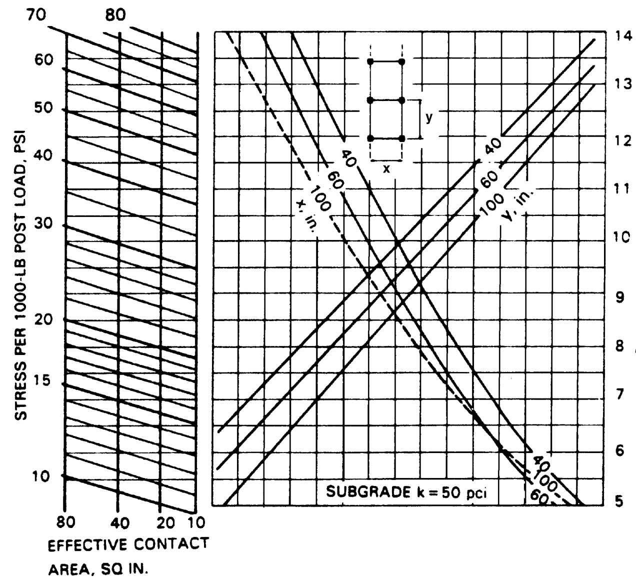


Fig. A1.6—PCA design chart for post loads where subgrade modulus is 50 pci.

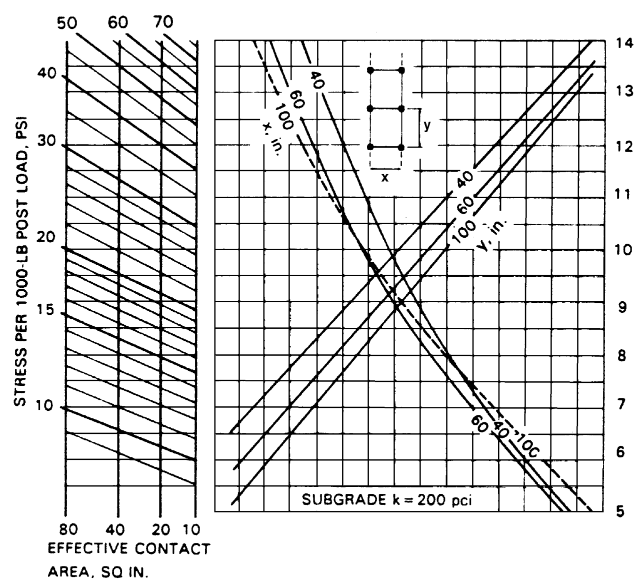


Fig. A1.7—PCA design chart for post loads where subgrade modulus is 200 pci.

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

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.

(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.

If the design thickness differs substantially from the assumed thickness, the procedure is repeated with a new assumption of thickness.

A2.3—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 $D/k = 3.4 \times 10^5$ 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

The solution is found 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.

Solution: thickness = 8.0 in.

Again, if the design thickness differs substantially from the assumed value, the process should be repeated until reasonable agreement is obtained.

APPENDIX 3—DESIGN EXAMPLES USING 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 procedure appears in publications issued by the Departments of Defense (1977), the Army (1984) and the Air Force (1987). The following examples are presented 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 safety factor 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 those most commonly used. Figure A3.1 shows a total of 10 categories.

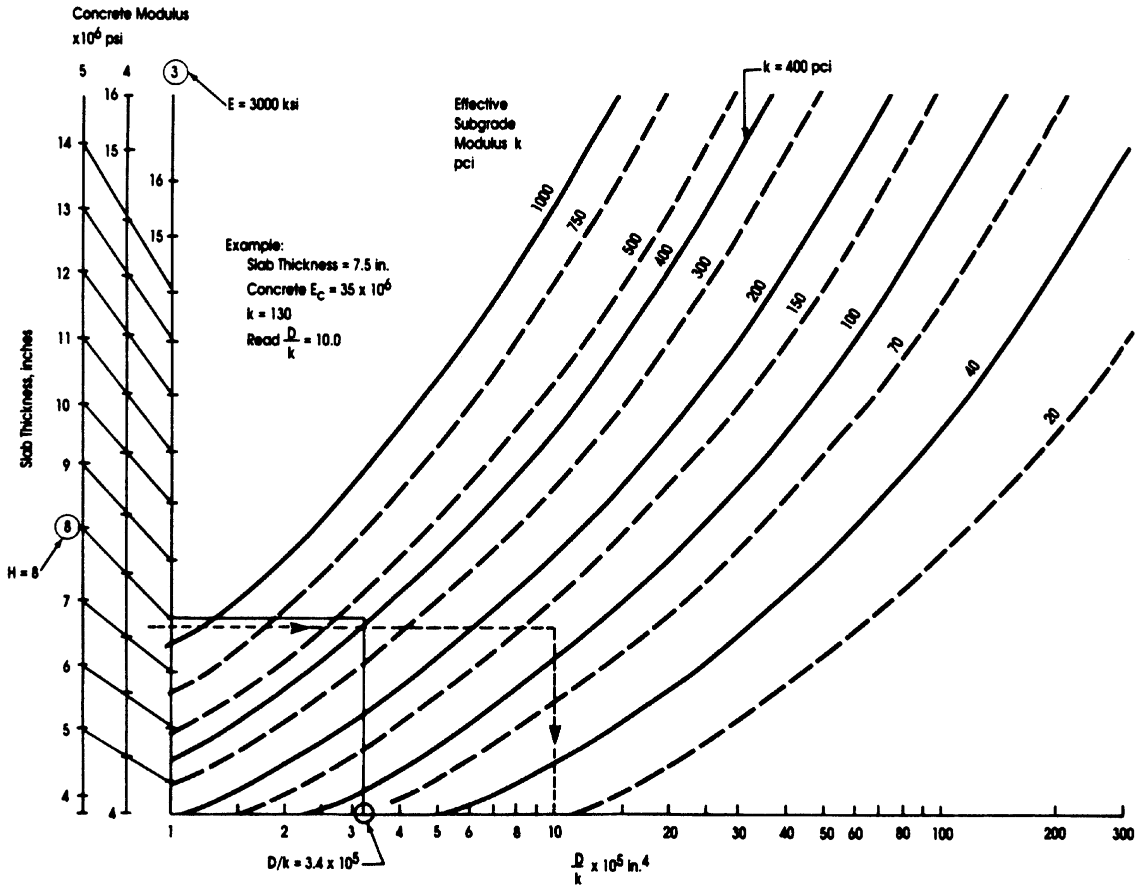


Fig. A2.1—Subgrade and slab stiffness relationship, used with WRI design procedure.

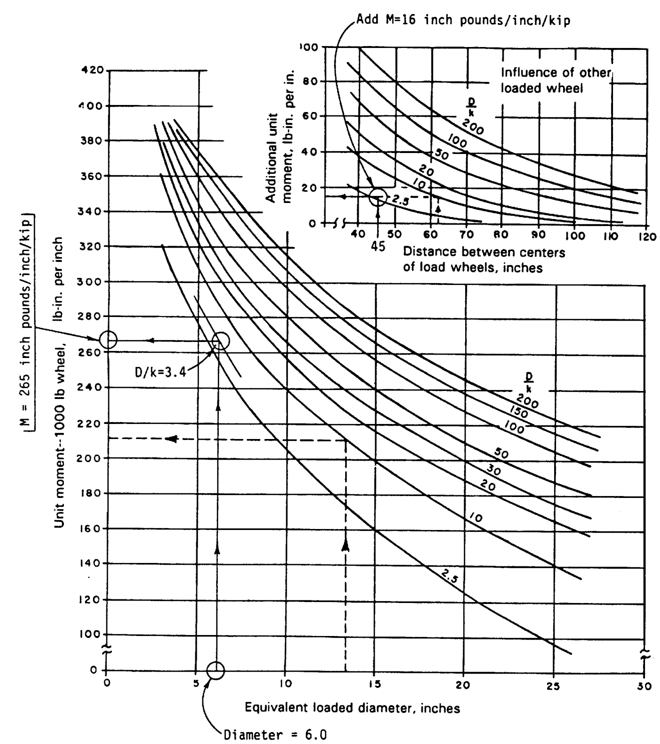


Fig. A2.2—Wheel loading design chart used with WRI procedure.

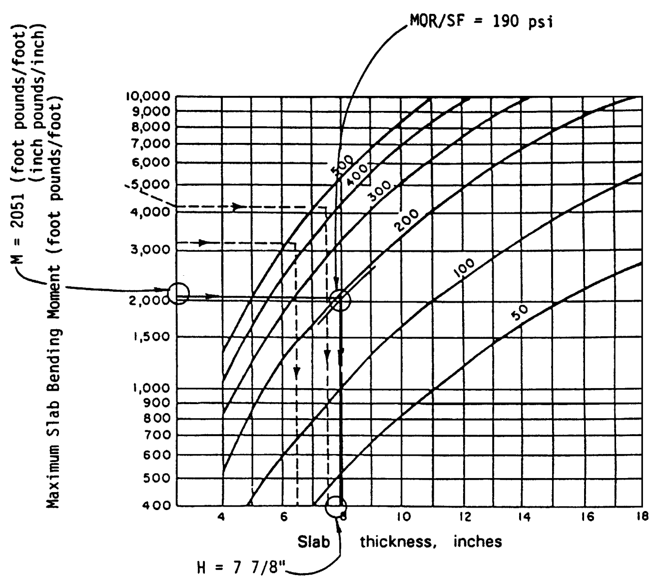


Fig. A2.3—Slab tensile stress charts used with WRI design procedure.

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

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

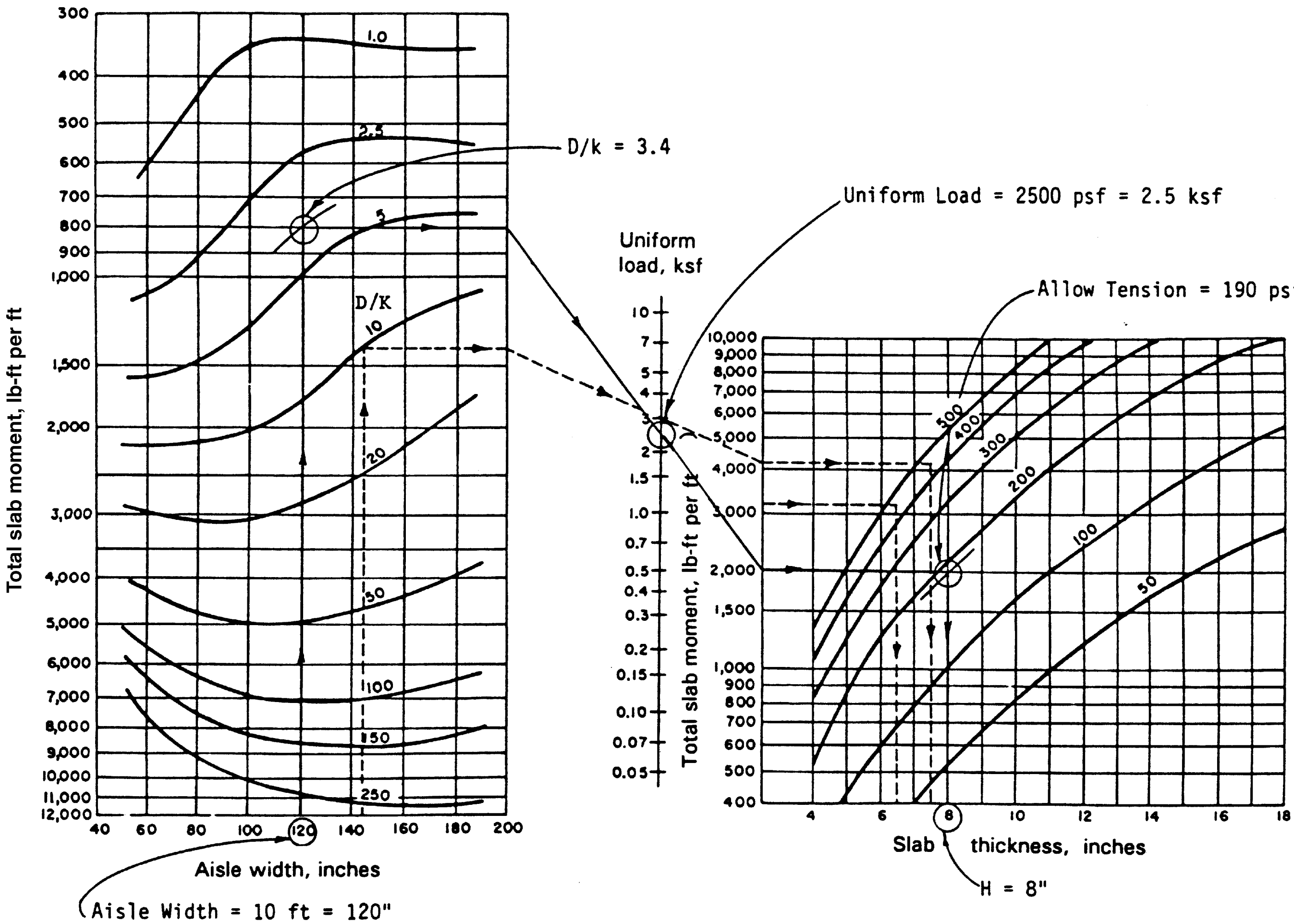


Fig. A2.4—Uniform load design and slab tensile stress charts used with WRI design procedure.

Fig. A3.1). A knowledge of the vehicle parameters is needed to select the design index category from Table A3.1. Use of the design chart is illustrated by 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 forklift loading

This example selects the thickness of the concrete slab for a forklift 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.

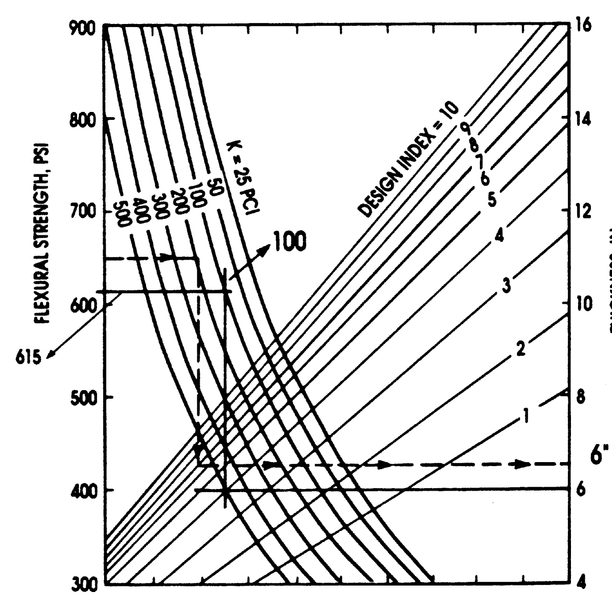


Fig. A3.1—COE curves for determining concrete floor thickness by design index.

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

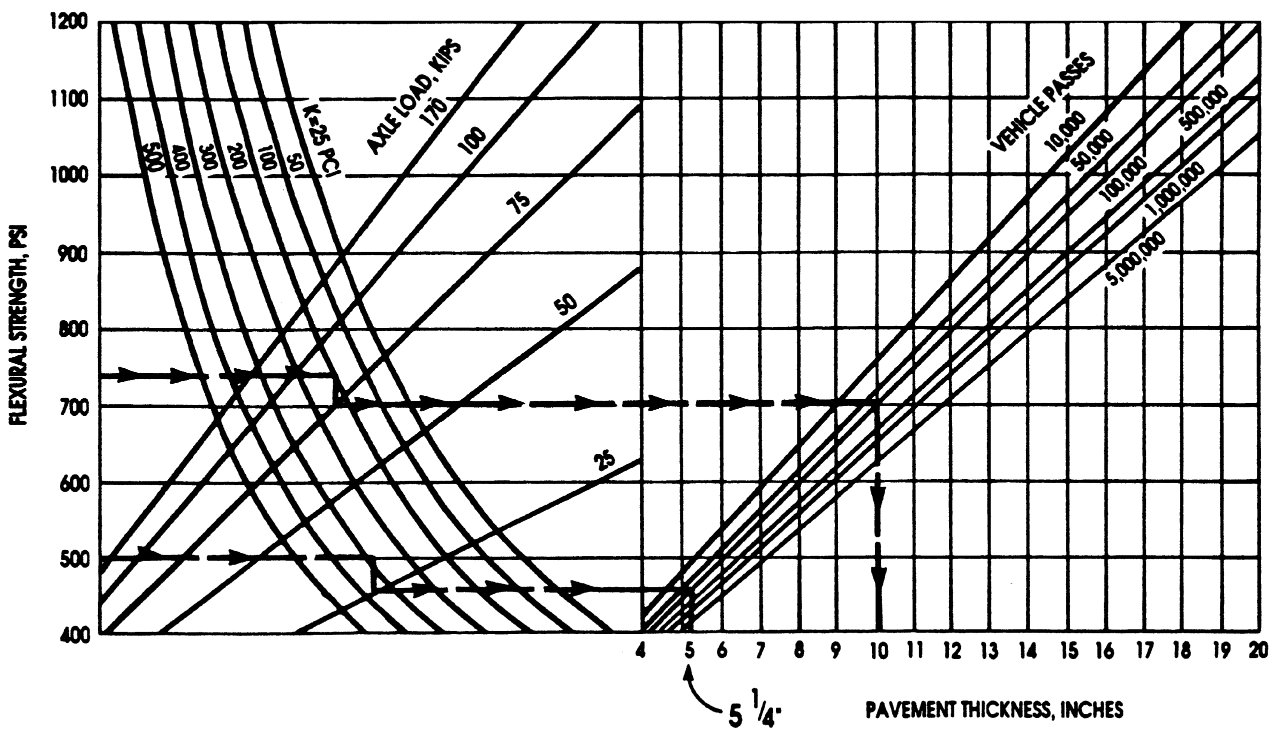


Fig. A3.2—COE design curves for concrete floor slabs with heavy forklift traffic.

APPENDIX 4—SLAB DESIGN USING POST-TENSIONING

This chapter includes:

- Design example: residential slabs on expansive soil;
- Design example: using post-tensioning to minimize cracking; and
- Design example: equivalent tensile stress design.

A4.1—Design example: Residential slabs on expansive soil

This design example is a three-story apartment house in Houston, Tex., with plan dimensions of 120 x 58 ft. It is built on expansive soil. Construction is stucco exterior, sheetrock interior, and gable truss roof. Design calculations are worked out as outlined in Section 9.7:

- Design soil values are provided by the geotechnical engineer;
- Design for the edge lift condition; and
- Design for center lift condition.

A4.1.1 *Design data including design soil values*

A. Loading

1)Perimeter loading = 2280 lb/ft

2) Live load = 40 lb/ft^2

B. Materials

1) Concrete: f'_c = 3000 psi

2) Concrete creep modulus of elasticity:

E_c = 1,500,000 psi

3) Prestressing steel: 270k 1/2 in. seven-wire strand

C. Design soil values

1) PI = 40

2) Edge moisture variation distance

e_m = 4.0 ft (center lift)

e_m = 5.0 ft (edge lift)

3) Differential swell

Y_m = 0.384 in. (center lift)

Y_m = 0.338 in. (edge lift)

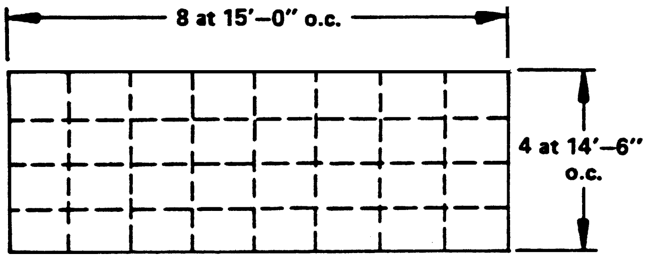


Fig. A4.1—Beam layout for apartment house example.

D. Assume spacing of stiffening beams as sketched in Fig. A4.1.

A4.1.2 *Design for edge lift*

A. Calculate approximate depth of stiffening beams where

$$d = (x)^{1.176} \text{ and}$$

$$x = \frac{(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(Y_m)^{0.76}}{12\Delta(P)^{0.01}}$$

1) Long direction: L = 120 ft; assume β = 10 ft, 6β = 60 ft. Governs

$$\Delta_{allow} = \frac{12(60)}{1700^*} = 0.424 \text{ in.}$$

$$x = \frac{(120)^{0.35}(15.00)^{0.88}(5.0)^{0.74}(0.338)^{0.76}}{12(0.424)(2280)^{0.01}}$$

$$x = 15.20;$$

$$d = (15.20)^{1.176}; = 24.54 \text{ in., say 26 in.}$$

2) Short direction: assume β = 10 ft; L = 58 ft < 6β. Therefore, 58 ft governs.

$$\Delta_{allow} = 12(58)/1700^* = 0.409 \text{ in.}$$

$$x = \frac{(58)^{0.35}(15)^{0.88}(5.0)^{0.74}(0.338)^{0.76}}{12(0.409)(2280)^{0.01}}$$

$$x = 12.21$$

$$d = (12.21)^{1.176} = 18.97 \text{ in.}$$

18.97 in. < 26 in. Use 26 in. for trial depth.

B. Check soil bearing pressure under beams

1) Allowable soil pressure

$$q_{allow} = 3400 \text{ lb/ft}^2 = 3.40 \text{ k/ft}^2$$

2) Applied loading
Slab = 120 × 58 × 0.333 × 0.150 = 347.65 kips
Beams = 9 × 58 × 1.0 × 1.833 × 0.150 = 143.52
Beams = 5 × 111 × 1.0 × 1.833 × 0.150 = 152.59
Perimeter = 2.280 × 35 = 811.68
Live load = 0.040 × 58 × 120 = 278.40
Total: 1733.84 kips

For 1.0 ft wide beams, the assumed spacing on Fig. A4.1 provides 1077 ft^2 of bearing area. The soil bearing pressure is then: w = 1733.84/1077 = 1.610 kips/ft^2

1.610 < 3.40, so soil bearing pressure is OK for the assumed beam layout.

C. Calculate section properties for full slab width

	Long direction	Short direction
Beam depth d, in.	26	26
Beam width b, in.	12	12
Number of beams	5	9
Total beam width nb, in.	60	108
Slab thickness t, in.	4	4
Moment of inertia I, in.^{4}	208,281	387,791
Section moduli, in.^{3}		
S_T	33,702	66,861
S_B	10,509	19,198
Cross sectional area, in.^{2}	4104	8136
Depth to neutral axis c_g, in.	–6.18	–5.80
Allowable concrete stress, ksi		
Tension	0.329	0.329
Compression	1.350	1.350
Tensile cracking stress, ksi	0.411	0.411

D. Calculate minimum number of tendons required

1) Number of tendons required for minimum average prestress of 50 psi. Stress in tendons immediately after anchoring:

$$f_{ps} = 0.7f_{pu} = (0.7)(270) = 189 \text{ ksi}$$

Stress in tendons after losses: f_{ps} = 189 – 30 = 159 ksi

$$N_T = \frac{(\text{minimum prestress}) \times (\text{area slab})}{1000 \times (\text{effective tendon stress}) \times (\text{area tendon})}$$

$$N_T(\text{long}) = \frac{(50 \text{ psi})(4103 \text{ in.}^2)}{(1000 \text{ lb/kip})(159 \text{ ksi})(0.153 \text{ in.}^2)} = 8.44$$

$$N_T(\text{short}) = \frac{(50 \text{ psi})(8136 \text{ in.}^2)}{(1000 \text{ lb/kip})(159 \text{ ksi})(0.153 \text{ in.}^2)} = 16.72$$

2) Number of tendons required to overcome slab-subgrade friction on polyethylene sheeting:

*The 1700 value is based on experience; refer to Chapter 9 for typical values.

Weight of beams and slab = 643.76 kips

$$N_T = 0.50\frac{(\mu)(W)}{(f_{ps})(\text{tendon area})}$$

$$N_T = 0.50\frac{(0.75)(643.76)}{(159)(0.153)} = 9.92 \text{ strands (each direction)}$$

3) Total number of tendons

N_T (long) = 8.44 + 9.92 = 18.36, use 19 tendons

N_T (short) = 16.72 + 9.92 = 26.64, use 27 tendons

4) Design prestress forces

Because maximum moments occur near the slab perimeter, friction losses will be minimal at points of maximum moments. Therefore, assume total prestressing force effective for structural calculations.

Long direction: $P_r = (19) \times 24.3k = 461.7$ kips

Short direction: $P_r = (27) \times 24.3k = 656.10$ kips

E. Calculate design moments

1) Long direction

$$M_l = \frac{(S)^{0.10}(de_m)^{0.78}(Y_m)^{0.66}}{7.2(L)^{0.0065}(P)^{0.04}}$$

$$M_l = \frac{(14.50)^{0.10}(26 \times 5.0)^{0.78}(0.338)^{0.66}}{7.2(120)^{0.0065}(2280)^{0.04}}$$

$M_l = 2.81$ ft-kips/ft

2) Short direction

$M_s = (d)^{0.35}\ [(19 + e_m)/\ 57.75](M_l)$

$M_s = (26)^{0.35}\ [(19 + 5.0)/57.75](2.81) = 3.65$ ft-kips/ft

A4.1.3 *Design for edge lift continued; service moments compared with design moments*

F. Calculate allowable service moments and compare with design moments

1) Long direction

a) Tension in bottom fiber

$(12 \times 58)M_t = S_B[(P_r/A) + f_t] - P_re$

$(12 \times 58)M_t = 10{,}509[(461.7/4104) + 0.329] - (461.7)(4.18)$

$M_t = 2710$ in.-kips/(12 × 58) = 3.89 ft-kips/ft

3.89 > 2.81 OK

b) Compression in top fiber

$(12 \times 58)M_c = S_T[f_c - P_r/A] - P_re$

$(12 \times 58)M_c = 33{,}702[1.350 - (461.7/4104)] - (461.7)(4.18)$

$M_c = 39{,}776$ in.-kips/(12 × 58) = 57.15 ft-kips/ft

57.15 > 2.81 OK

2) Short direction

a) Tension in bottom fiber

$(12 \times 120)M_t = 19{,}198\ [(656.1/8136) + 0.329] - (656.1)(3.80)$

$M_t = 5371$ in.-kips/(12 × 120) = 3.73 ft-kips/ft

3.73 > 3.65 OK

b) Compression in top fiber

$(12 \times 120)M_c = (66{,}861)\ [1.350 - (656.1/8136)] - (656.1)(3.80)$

$M_c = 82{,}377$ in.-kips/(12 × 120) = 57.21 ft-kips/ft

57.21 > 3.65 OK

G. Deflection calculations

1) Long direction

a) Allowable differential deflection

$$\beta = 1/12\sqrt[4]{\frac{E_cI}{E_s}} = 1/12\sqrt[4]{\frac{1{,}500{,}000 \times 208{,}281}{1000}} = 11.91 \text{ ft}$$

6β = 66.48 ft < 120 ft, so 6β governs

$\Delta_{allow} = 12\ (66.48)/800 = 0.997$ in.

b) Expected differential deflection

$$\Delta = \frac{(L)^{0.35}(S)^{0.88}(e_m)^{0.74}(Y_m)^{0.76}}{15.90(d)^{0.85}(P)^{0.01}}$$

$$\Delta = \frac{(120)^{0.35}(14.50)^{0.88}(5.0)^{0.74}(0.338)^{0.76}}{15.90(26)^{0.85}(2280)^{0.01}}$$

Δ = 0.296 in. 0.296 < 0.997 OK

2) Short direction

a) Allowable differential deflection

$$\beta = 1/12\sqrt[4]{\frac{1{,}500{,}000 \times 387{,}791}{1000}} = 12.94 \text{ ft}$$

6Δ = 77.64 > 58 ft, so 58 ft governs

$\Delta_{allow} = 12(58)/800 = 0.870$ in.

b) Expected differential deflection

$$\Delta = \frac{(58)^{0.35}(15)^{0.88}(5.0)^{0.74}(0.338)^{0.76}}{15.90(26)^{0.85}(2280)^{0.01}}$$

Δ = 0.236 in. 0.236 < 0.870 in. OK

Deflections for edge lift bending are much less than allowable in both long and short directions.

H. Shear calculations

1) Long direction

a) Expected shear

$$V_l = \frac{(L)^{0.07}(d)^{0.40}(P)^{0.03}(e_m)^{0.16}(Y_m)^{0.67}}{3.0(S)^{0.015}}$$

$$V_l = \frac{(120)^{0.07}(26)^{0.40}(2280)^{0.03}(5.0)^{0.16}(0.338)^{0.67}}{3.0(14.50)^{0.015}}$$

$$V_l = 1.300 \text{ kips/ft}$$

b) Permissible shear stress

$$v_c = 1.5\sqrt{f_c'} = 1.5\sqrt{3000} = 82.2 \text{ psi}$$

c) Total design shear stress

$$V = \frac{VW}{ndb} = \frac{(1.300)(58)(1000)}{(5)(12)(26)} = 48.33 \text{ psi}$$

$$48.33 < 82.2 \text{ psi OK}$$

2) Short direction

a) Expected shear

$$V_s = \frac{(58)^{0.07}(26)^{0.40}(2280)^{0.03}(5.0)^{0.16}(0.338)^{0.67}}{3.0(15)^{0.015}}$$

$$V_s = 1.235 \text{ kips/ft}$$

b) Total design shear stress

$$V = \frac{(1.235)(120)(1000)}{(9)(12)(26)} = 52.78 \text{ psi}$$

$$52.78 < 82.2 \text{ psi OK}$$

Shear stresses are OK in both short and long directions.

A4.1.4 *Design for center lift*

A. Calculate design moments

1) Long direction

$M_l = A_o[B(e_m)^{1.238} + C]$

$A_o = 1/727[(L)^{0.013}(S)^{0.306}(d)^{0.688}(P)^{0.534}(Y_m)^{0.193}]$

$A_o = 1/727[(120)^{0.013}(14.50)^{0.306}(26)^{0.688}(2280)^{0.534}(0.384)^{0.193}]$

$A_o = 1.612$

$0 \le e_m \le 5$

$e_m = 4.0 \quad B = 1.0 \quad C = 0$

$M_l = (1.612)(4.0)^{1.238} = 8.97$ ft-kips/ft

2) Short direction

$M_s = [(58 + e_m)/60]M_l$

$M_s = [(58 + 4.0)/60]8.97 = 9.27$ ft-kips/ft

B. Calculate allowable moments and compare with design moments

1) Long direction

a) Tension in top fiber

$(12 \times 58)M_t = S_T[(P_r/A) + f_t] + P_r e$

$(12 \times 58)M_t = (33{,}702)[(461.7/4104) + 0.329] + (461.7)(4.18)$

$M_t = 16{,}809$ in.-kips/$(12 \times 28) = 24.15$ ft-kips/ft

$24.15 > 8.97$ OK

b) Compression in bottom fiber

$(12 \times 58)M_c = S_B[f_c - (P_r/A)] + P_r e$

$(12 \times 58)M_c = 10{,}509[1.350 - (461.7/4104)] + (461.7)(4.18)$

$M_c = 14{,}935$ in.-kips/$(12 \times 58) = 21.46$ ft-kips/ft

$21.46 > 8.97$ OK

2) Short direction

a) Tension in top fiber

$(12 \times 120)M_t = 66{,}861[(656.1/8136) + 0.329] + (656.1)(3.80)$

$M_t = 29{,}882$ in.-kips/$(12 \times 120) = 20.75$ ft-kips/ft

$20.75 > 9.27$ ft-kips/ft

b) Compression in bottom fiber

$(12 \times 120)M_c = 19{,}198[1.350 - (656.1/8136)] + (656.1)(3.80)$

$M_c = 26{,}862$ in.-kips/$(12 \times 120) = 18.65$ ft-kips/ft

$18.65 > 9.27$ OK

Moment capacities exceed expected service moments for center lift loading in both long and short directions. By observation, deflection and shear calculations are within permissible tolerances. For a detailed discussion for slabs on expansive soils, see *Design and Construction of Post-Tensioned Slabs-on-Ground* (PTI 2004).

C. Tendon and beam requirements

1) Long direction: Use nineteen 1/2 in., 270k strands in slab. Two spaces at 3 ft 2 in. on center and 16 spaces at 3 ft 1-1/4 in. on center beginning 1 ft from each edge. Five beams, 12 in. wide, 26 in. deep, evenly spaced at 14 ft 3 in. on centers.

2) Short direction: Use twenty-seven 1/2 in., 270k strands in slab. Two spaces at 4 ft 6 in. and 24 spaces at 4 ft 6-1/2 in. on center beginning 1 ft from each edge. Nine beams, 12 in. wide, 26 in. deep, evenly spaced at 14 ft 10-1/2 in. on centers.

A4.2—Design example: Using post-tensioning to minimize cracking

Assume P-T strip 500 x 12 ft.

Determine minimum residual (effective) compression after all losses.

Calculate P-T 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. (9-1):

Assume subgrade friction factor: 0.5

$$P_r = W_{slab}\frac{L}{2}\mu = \frac{6\text{ in.}}{12\text{ in./ft}} \times 150\text{ lb/ft}^3 \times \frac{500\text{ ft}}{2} \times 0.5$$

$$= 9375\text{ lb/ft}$$

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

Assume P_e = 26,000 lb

Calculate the required spacing of the P-T tendons using Eq. (9-2)

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

$$= 0.95\text{ ft or } 11.4\text{ in.}$$

Use 11 in. to provide more than 250 psi compression. Twelve inch spacing would provide 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 will help determine the final spacing.

If 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. (6-4)

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

where

f_b = tensile stresses at the bottom of the concrete slab;
P = concentrated load;
h = slab thickness;
a = radius of a equivalent circular load contact area; and
k = 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

P-T to provide necessary precompression of: 545 – 474 = 71 psi

P-T providing 250 psi is adequate.

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

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

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

Case 2: A section of 200 ft is placed first, stressed partially, and then the other section of 160 ft is placed and stressed.

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

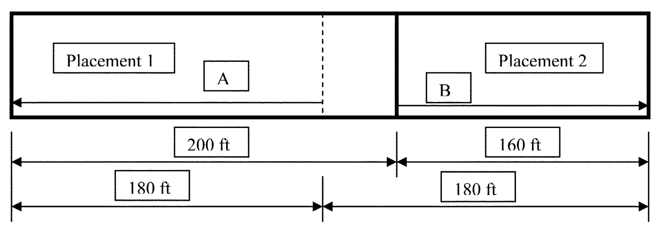
Placement 1: Formula

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

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

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).

A4.3—Design example: Equivalent tensile stress design

Determine the reduction in slab thickness of a 6 in. thick unreinforced slab if post-tensioning is used.

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

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

If the P-T force will provide 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 capacity 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

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

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

The 150 psi residual compressive stress could be increased to use a 4 in. thick slab or reduced so a 5 in. thick slab could be used.

APPENDIX 5—DESIGN EXAMPLES USING SHRINKAGE-COMPENSATING CONCRETE

A5.1—Introduction

The material presented in this appendix is discussed in greater detail in ACI 223. Slab design using this material is divided into three parts.

The first part is the selection of slab thickness, which can be done, for example, using Appendixes 1, 2, or 3. This follows the assumption that the slab is being designed to remain essentially uncracked due to external loading.

This is followed by design of 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, no prestress is to be considered.

The second part of the process—selection of the appropriate amount of reinforcement—is a critical part of the design. This reinforcement can be mild steel, as illustrated in this appendix, or post-tensioning tendons. ACI 223 recommends that the reinforcement be placed in the top 1/3 to 1/4 of the slab.

The third part of the design is the determination of the required prism expansion to ensure shrinkage compensation, which leads to the design for properties of the concrete mixture. This is shown in Section A5.2. Expansion of the length of the slab is also determined.

A5.2—Example with amount of steel and slab joint spacing predetermined

The thickness of the slab, the joint spacing, and the amount of steel have been set as follows:

Thickness = 6.0 in.

Amount of mild steel = 0.36 in.^{2}/ft, which is 0.5%

Joint spacing = 80 ft

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

For complete shrinkage compensation, the amount of expansion will be equal to the anticipated amount of shrinkage, which must be determined first. For this example, the shrinkage is assumed to be equal to the prism expansion.

Prism expansion = 0.046% = 0.00046 in./in.

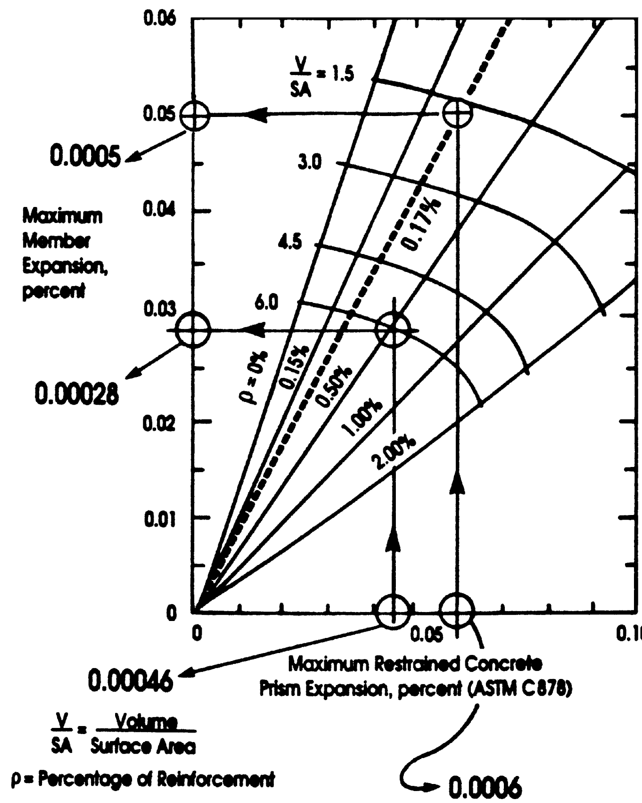


Fig. A5.1—Prediction of member expansion from prism data (ACI Committee 223 1970).

This expansion, as determined by ASTM C 878 from tests of the concrete mixture, is used with Fig. A5.1 to determine member expansion.

Member expansion = 0.028% or 0.00028 in./in.

This is to be combined with the joint spacing to determine the total slab motion and the required thickness of the joint filler. The design assumption is made that all motion occurs at one end of the slab.

Motion = 0.00028 × 80 × 12 = 0.269 in.

Use a joint filler that will compress at least twice this amount.

Filler thickness = 2 × 0.27 = 0.54 in.

ACI 223 discusses these features in greater detail.

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-grade containing steel FRC. The equations shown in this example can be found in *Technical Report* No. 34 (Concrete Society 1994). This design procedure is iterative and involves assumption of a slab thickness, determination of a residual strength toughness factor, and determining the reasonableness of the toughness factor. An appropriate fiber type and quantity rate is then selected to meet the toughness 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 capacity of the slab, tension at top slab surface, in.-k (N-mm);
Mp = positive bending moment capacity of the slab, tension at bottom slab surface, in.-k (N-mm)
P_{ult} = ultimate load capacity of the slab, kips
$R_{e,3}$ = residual strength factor (JSCE SF4)
S = slab section modulus, in.^{3}/in. (mm^3/mm)
μ = Poisson's ratio for concrete (approximately 0.15)

A6.2—Assumptions/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 μ 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

$L = [E \times h^3/(12\ (1 - \mu^2)k)]^{0.25}$
$= [3{,}600{,}000 \times 6^3/(12(1 - 0.15^2)100)]^{0.25}$
$= 28.5$ in.

The section modulus of the slab is

$S = 1$ in. $\times h^2/6 = 12 \times 6^2/6 = 6$ 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 = (base plate area / 3.14)$^{0.5}$ (Eq. (10-2))
= (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 capacity of the slab:

$P_{ult} = 6(1 + (2a/L)) \times (Mp + Mn)$ (Eq. (10-3))

where

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

Combining Mp and Mn

$$Mp + Mn = f_r \times S \times (1 + R_{e,3}/100) \qquad \text{(Eq. (10-4))}$$

A safety factor of 1.5 is selected for this example

$$Mp + Mn = f_r \times S \times (1 + R_{e,3}/100)/1.5$$

Solving Eq. (10-3)

$$15 = 6\ (1 + 2 \times 2.8/28.5) \times (Mp + Mn)/1.5$$

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

$$3.13 \text{ in.-k/in.} = Mp + Mn$$

It is known that stresses due to shrinkage and curling can be substantial. For the purpose of this example, an amount of 200 psi is selected. This translates into an additional moment of 1.2 in.-k/in. (6.0 in.^{3}/in. × 200 psi) to account for shrinkage and curling stresses. This stress can vary depending on the safety factor and other issues, including mixture proportion, joint spacing, and drying environment.

Using Eq. (10-4) to solve for the required residual strength factor $R_{e,3}$

3.13 in.-k/in. + 1.2 in.-k/in. = $f_r \times S \times (1 + R_{e,3}/100)$
$R_{e,3} \geq [(4.33 \times 1000/550/6.0) - 1.0]100$
$R_{e,3} \geq 31$

Residual load factors for various fiber types and quantities are available from steel fiber manufacturers' literature. Laboratory testing may be used 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 would be in the range of 15 to 33 lb/yd^3 (10 to 20 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

$$0.80 \times P_{ult} = 3.5(1 + (3a/L)) \times (Mp + Mn)/1.5 \qquad \text{(Eq. (10-5))}$$

Solving Eq. (10-5),

$$0.80 \times 15 = 3.5(1 + 3 \times 2.8/28.5) \times (Mp + Mn)/1.5$$

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

As in the previous example, an additional moment of 1.2 in.-k/in. is used to account for shrinkage. No curling stress exists at the edge. Using Eq. (10-4) 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 would be in the range of 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.

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