

Design of Slabs on Grade

Reported by ACI Committee 360

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This document presents information on the design of slabs on grade, primarily industrial floors and the slabs adjacent to them. The report addresses the planning, design, and detailing of the slabs. Background information on design theories is followed by discussion of the soil support system, loadings, and types of slabs. Design methods are given for plain concrete, reinforced concrete, shrinkage-compensating concrete, and post-tensioned concrete slabs, followed by information on shrinkage and curling problems. 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); shrinkage; shrinkage-compensating concrete; slabs; **slabs on grade;** **soil mechanics;** shrinkage; warping.

CONTENTS

Chapter 1-Introduction, pg. 360R-2

- 1.1-Purpose and scope
- 1.2-Work of Committee 360 and other relevant committees
- 1.3-Work of non-ACI organizations
- 1.4-Design theories for slabs on grade
- 1.5-Overview of subsequent chapters

Chapter 2-Slab types and design methods, pg. 360R-4

- 2.1-Introduction
- 2.2-Slab types

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- 2.3-Design and construction variables
- 2.4-Design methods
- 2.5-Fiber-reinforced concrete (FRC)
- 2.6-Conclusion

Chapter 3-Soil support systems for slabs on grade, pg. 360R-8

- 3.1-Introduction
- 3.2-Soil classification and testing
- 3.3-Modulus of subgrade reaction
- 3.4-Design of the slab support system
- 3.5-Site preparation
- 3.6-Inspection and site testing of soil support
- 3.7-Special problems with slab on grade support

Chapter 4-Loads, pg. 360R-15

- 4.1-Introduction
- 4.2-Vehicle loads
- 4.3-Concentrated loads
- 4.4-Uniform loads
- 4.5-Line and strip loads
- 4.6-Unusual loads
- 4.7-Construction loads
- 4.8-Environmental factors
- 4.9-Factors of safety
- 4.10-Summary

Chapter 5-Design of plain concrete slabs, pg. 360R-19

- 5.1-Introduction

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- 5.2-Portland Cement Association (PCA) design method
- 5.3-Wire Reinforcement Institute (WRI) method
- 5.4-Corps of Engineers (COE) design method

Chapter 6-Design of slabs with shrinkage and temperature reinforcement, pg. 360R-20

- 6.1-Introduction
- 6.2-Thickness design methods
- 6.3-Subgrade drag equation
- 6.4-Reinforcement location

Chapter 7-Design of shrinkage-compensating concrete slabs, pg. 360R-21

- 7.1-Introduction
- 7.2-Thickness determination
- 7.3-Typical reinforcement conditions
- 7.4-Design implications
- 7.5-Maximum and minimum reinforcement requirements
- 7.6-Other considerations

Chapter 8-Design of post-tensioned slabs on grade, pg. 360R-27

- 8.1-Notation
- 8.2-Definitions
- 8.3-Introduction
- 8.4-Applicable design procedures
- 8.5-Data needed for design of reinforced slabs
- 8.6-Design for slabs on expansive soils
- 8.7-Design for slabs on compressible soil
- 8.8-Maximum spacing of post-tensioning tendons in normal weight concrete

Chapter 9-Reducing the effects of slab shrinkage and curling, pg. 360R-32

- 9.1-Introduction
- 9.2-Drying and thermal shrinkage
- 9.3-Curling and warping
- 9.4-Factors that affect shrinkage and curling
- 9.5-Compressive strength and shrinkage
- 9.6-Compressive strength and abrasion resistance
- 9.7-Removing restraints to shrinkage
- 9.8-Subgrade and vapor barriers
- 9.9-Distributed reinforcement to reduce curling and number of joints
- 9.10-Thickened edges to reduce curling
- 9.11-Relation between curing and curling
- 9.12-Warping stresses in relation to joint spacing
- 9.13-Warping stresses and deformation
- 9.14-Effect of eliminating contraction joints with post-tensioning or shrinkage-compensating concrete
- 9.15-Summary and conclusions

Chapter 10-References, pg. 360R-39

- 10.1-Recommended references
- 10.2-Cited references

Appendix, pg. 360R-41

- A1-Design examples using the PCA method
- A2-Slab thickness design by WRI method
- A3-Design examples using COE charts
- A4-Slab design using post-tensioning
- A5-Shrinkage-compensating concrete examples

CHAPTER 1-INTRODUCTION

1.1-Purpose and scope

Consistent with the mission of ACI Committee 360, this report presents state-of-the-art information on the design of slabs on grade. In this context, design is defined as the decision-making process of planning, sizing, detailing, and developing specifications generally preceding construction. 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 report, Committee 360 defines *slab on grade* as:

a slab, continuously supported by ground, whose total loading when uniformly distributed would impart a pressure to the grade or soil that is less than 50 percent of the allowable bearing capacity thereof. The slab may be of uniform or variable thickness, and it may include stiffening elements such as ribs or beams. The slab may be plain, reinforced, or prestressed concrete. The reinforcement or prestressing steel may be provided for the effects of shrinkage and temperature or for structural loading.

This report covers the design of slabs on grade for loads caused by material stored directly on the slab or on storage racks, as well as static and dynamic loads associated with handling equipment and vehicles. Other loads, such as loads on the roof transferred through dual purpose rack systems are also covered. ACI Committee 360 considers use of the information presented in this report reasonable for slabs on grade which support structural loads provided the loading limit of the above definition is satisfied.

In addition to design of the slab for these loadings, the report discusses subgrade-subbase, shrinkage and temperature effects, cracking, curling or warping, and other items affecting the design. Although the same general principles are applicable, the report does not specifically address the design of highways, airport pavements, parking lots, and mat foundations.

1.2-Work of ACI Committee 360 and other relevant committees

1.2.1 ACI 360 mission-Since several engineering disciplines and construction trades deal with slabs on grade, several ACI committees are involved, directly and indirectly. Before the formation of Committee 360, no

ACI committee was specifically charged to cover design. Consequently, ACI 360 was formed with this mission:

Develop and report on criteria for design of slabs on grade, except highway and airport pavements

1.2.2 ACI Committee 302-ACI Committee 302 develops recommendations on the construction of floor slabs. ACI 302.2R, gives basic information, guidelines, and recommendations on slab construction. It also contains information on thickness and finishing requirements for different classes of slabs.

1.2.3 ACI Committee 325-ACI Committee 325 is concerned with structural design, construction, maintenance, and rehabilitation of concrete pavements. The committee documents include ACI 325.1R on construction and ACI 325.3R on foundation and shoulder design.

1.2.4 ACI Committee 318-Although ACI 318 does not specifically mention slabs on grade, the commentary (ACI 318R) notes the exclusion of the soil-supported slabs from various requirements in ACI 318 unless such slabs transmit structural loads. Chapter 13 of ACI 318R states: "... Also excluded are soil-supported slabs such as 'slab on grade' which do not transmit vertical loads from other parts of the structure to the soil." The 318 commentary Chapter 7 on shrinkage and temperature reinforcement states that its provisions "... apply to structural floor and roof slabs only and not to soil-supported slabs, such as 'slab on grade.'"

1.2.5 ACI Committee 332-ACI Committee 332 develops information on the use of concrete in residential construction. Slabs on grade are important elements in such construction. However, residential slabs generally do not require detailed design unless poor soil conditions are encountered. Residential slabs placed on poor soils, such as expansive soils, and those slabs that support unusual or heavy loads, require more thorough evaluation of soil properties and their interaction with the slab structure.

1.2.6 ACI Committee 336-ACI Committee 336 is concerned with design and related considerations of foundations which 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 grade.

1.2.7 ACI Committee 330-ACI Committee 330 monitors developments and prepares recommendations on design, construction, and maintenance of concrete parking lots. While the principles and methods of design in this ACI 360 report are applicable to parking lot pavements, the latter have unique considerations that are covered in ACI 330R, which includes design and construction as well as discussions on material specifications, durability, maintenance, and repair of parking lots.

1.3-Work of non-ACI organizations

Numerous contributions to knowledge of slabs on

grade come from organizations and individuals outside of the American Concrete Institute. The United States Army Corps of Engineers, the National Academy of Science, and the Department of Housing and Urban Development have developed guidelines for floor slab design and construction. Several industrial associations, such as the Portland Cement Association, the Wire Reinforcement Institute, the Concrete Reinforcing Steel Institute, the Post-Tensioning Institute, as well as several universities and consulting engineers have studied slabs on grade and developed recommendations on their design and construction. In addition, periodicals such as *Concrete Construction* have continuously disseminated information for the use of those involved with slabs on grade. In developing this report, Committee 360 has drawn heavily from these contributions.

1.4-Design theories for slabs on grade

1.4.1 Introduction-Stresses in slabs on grade result from both imposed loads and volume changes of the concrete. The magnitude of these stresses depends upon 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 grade 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 Westergaard¹ proposed the so-called "corner formula" for stresses. Although the observations in the first road test with rigid pavements seemed to be in reasonable 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.^{1,2,3} 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. The subgrade is assumed to act as a linear spring, with a proportionality constant k with units of pressure (pounds per square inch) per unit deformation (in inches). The units are commonly abbreviated as pci. This is the constant now recognized as the coefficient of subgrade reaction, more commonly called the modulus of soil reaction 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 take care of the effects of the slab curling above the subgrade. However, although a proper choice of the modulus of subgrade reaction was found to be 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 accumulated to indicate that the behavior of many subgrades may be close to that of an elastic and isotropic solid. Two characteristic constants, typically the modulus of soil deformation and Poisson's ratio, are 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.⁴ He suggested that the design should be based on a criterion of limited deformation under load. However, the design procedures for rigid pavements based on this theory were never developed enough 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 been developed.

All of the preceding theories are limited to consideration of behavior in the linear range, where deflections, by assumption, are proportional to applied loads. Lösberg^{5,6} 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 grade 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:

- the elastic-isotropic solid
- the thin elastic slab
- the thin elastic-plastic slab.

Two models used for the subgrade are the elastic-isotropic solid and the so-called Winkler subgrade. Existing design theories are based on various combinations of these models. The methods presented in this report are generally graphical, plotted from computer-generated solutions of selected models. Design theories need not be limited to these combinations. As more sophisticated analyses become available, other combinations may well become more practical.

In developing a reliable theory for the design of slabs on grade, major attention should be devoted to modeling the subgrade. Most currently used theoretical design methods for the rigid pavements use the Winkler model, and a number of investigators report good agreement between observed response of rigid pavements and the prediction based on that model. At the same time, the elastic-isotropic solid model can, in general, predict more

closely the response of real soils.

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 grade. Solution of the governing equations by conventional methods is feasible only for simplified models, where the slab and the subgrade are assumed to be continuous and homogeneous. However, a real slab on grade 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 grade in general, and particularly those with discontinuities. Various models have been proposed to represent the slab.⁷ 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, its use in typical designs has been limited. Microcomputers may enhance its usage and that of other numerical methods in the future.

1.5-Overview of subsequent chapters

Chapter 2 identifies types of slabs on grade and appropriate design methods. **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. Chapters 5 through 9 provide information on design methods and the related parameters needed to complete the design. Design examples in the appendix illustrate application of selected design methods.

CHAPTER 2-SLAB TYPES AND DESIGN METHODS

2.1-Introduction

This chapter identifies and briefly discusses the common types of slab-on-grade construction and the design methods appropriate for each (**Table 2.1**). The underlying theory, critical pressures, and construction features intrinsic to each method are identified. Methods presented are those attributed to the Portland Cement Association,⁸ Wire Reinforcement Institute,⁹ United States Army Corps of Engineers,¹⁰ Post-Tensioning Institute¹¹ and ACI 223.

As stated in the basic definition of **Section 1.1**, a slab on grade is one whose total loading, uniformly distributed, would impart a pressure to the grade or soil that is less than 50 percent of the allowable bearing capacity thereof. There are, of course, exceptions such as where the soil is highly compressible and allowable bearing pressures are extremely low. Such situations are covered in literature of the Post-Tensioning Institute.

Slab on grade is an all-encompassing term that in-

cludes slabs for both heavy and light industrial usage, commercial slabs, apartment slabs, single-family dwelling slabs, and others. Although the term also includes parking lot slabs and paving surfaces, these are not specifically covered in this report.

2.2-Slab types

The six types of construction for slabs on grade identified in [Table 2.1](#) are:

- a) Plain concrete slab
- b) Slab reinforced for shrinkage and temperature only
- c) Shrinkage-compensating concrete with shrinkage reinforcement
- d) Slab post-tensioned to offset shrinkage
- e) Slab post-tensioned and/or reinforced, with active prestress
- f) Slab reinforced for structural action

Slab thickness design methods appropriate for each type are also shown in [Table 2.1](#). Slab Types A through E are designed with the assumption that applied loadings will not crack the slab. For Type F the designer anticipates that the applied loadings may crack the slab.

2.2.1 Type A, plain concrete slab-The design of this slab involves determining its thickness as a plain concrete slab without reinforcement; however, it may have strengthened joints. It is designed to remain uncracked due to loads on the slab surface. Plain concrete slabs do not contain any wire, wire fabric, plain or deformed bars, post-tensioning, or any other type of 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 these plain concrete slabs. To reduce drying shrinkage cracks, the spacing of contraction and/or construction joints is limited. PCA⁸ recommends joint spacings from 2 to 3 ft for each inch of slab thickness.

2.2.2 Type B, slab reinforced for shrinkage and temperature only-These slabs are normally constructed using ASTM C-150 Type I or Type II cement. Thickness design is the same as for plain concrete slabs, and the slab is assumed to remain uncracked due to loads placed on its surface. Shrinkage cracking is controlled by a nominal or small amount of distributed reinforcement placed in the upper half of the slab, and therefore joint spacings can be greater than for Type A slabs.

Joint spacings can be computed using the subgrade drag equation ([Chapter 6](#)) for a pre-selected amount of steel for shrinkage and temperature control; however, the amount of reinforcement area or steel stress is usually computed from a predetermined joint spacing.

The primary purpose of the reinforcement in the Type B slab is to hold tightly closed any cracks that may form between the joints. The reinforcement must be stiff enough so that it can be accurately located in the top half of the slab. Reinforcement does not prevent the

cracking, nor does it add significantly to the load-carrying capacity of a Type B slab. Committee 360 believes that the best way to obtain increased flexural strength is to increase the thickness of the slab.

2.2.3 Type C, shrinkage-compensating concrete slabs-

The shrinkage compensating-concrete used in these slabs is produced either with a separate admixture or with ASTM C-845 Type K cement which contains the expansive admixture. This concrete does shrink, but first it expands an amount intended to be slightly greater than its drying shrinkage. Distributed reinforcement for temperature and shrinkage equal to 0.15 to 0.20 percent of the cross-sectional area is used in the upper half of the slab to limit the initial slab expansion and to restrain the slab's subsequent drying shrinkage.

Reinforcement must be stiff enough that it can be positively positioned in the upper half of the slab. The slab must be isolated from fixed portions of the structure, such as columns and perimeter foundations, with a compressible material that allows the slab to expand.

Type C slabs are designed to remain uncracked due to loads applied to the slab surface. Thickness design is the same as for Type A and B slabs, but joints can be spaced farther apart than in those slabs. Design concepts and details are explained in ACI 223.

2.2.4 Type D, slabs post-tensioned to offset shrinkage-

Post-tensioned slabs are normally made with ASTM C 150 Type I or Type II cement, following thickness design procedures like those for Types A, B, and C. As explained in literature of the Post-Tensioning Institute,⁹ post-tensioning permits joint spacing at greater intervals than for Type A, B, and C slabs. However, special techniques and sequences of post-tensioning the tendons are required.

The effective coefficient of friction (explained in [Chapter 6](#)), is critical to design of Type D slabs. Joint spacing and amount of post-tensioning force required to offset later shrinkage and still leave a minimum compressive stress are explained in [Chapter 8](#) and [Reference 11](#).

2.2.5 Type E, slabs post-tensioned and/or reinforced, with active prestress-Type E slabs are designed to be uncracked slabs, following PTI procedures,¹¹ using active prestress, which permits the use of thinner slabs. Reinforced with post-tensioning tendons and/or mild steel reinforcement, Type E slabs may incorporate monolithic beams (sometimes called ribs) to increase rigidity of the section.

The Type E slab may be designed to accept structural loadings, such as edge loadings from a building superstructure, as well as to resist the forces produced by the swelling or shrinking of unstable soils.

2.2.6 Type F, slabs reinforced for structural action-

Unlike the previously described slab types, the Type F slab is designed with the assumption that it is possible for the slab to crack under loads applied to its surface. Previously cited design methods,⁸⁻¹¹ are only appropriate up to the level of loading that causes the cracking stress of the concrete to be reached. Beyond this cracking level,

Table 2.1-Slab types with design methods suitable for each

TYPE OF SLAB CONSTRUCTION	DESIGN METHODS					
	PCA	WRI	COE	PTI	ACI 223	
TYPE A, PLAIN CONCRETE, no reinforcement						Thickness selection
						Related details
TYPE B, REINFORCED for shrinkage and temperature						Thickness selection
						Related details
TYPE C, SHRINKAGE-COMPENSATING CONCRETE with shrinkage reinforcement						Thickness selection
						Related details
TYPE D, POST-TENSIONED for crack control						Thickness selection
						Related details
TYPE E, POST-TENSIONED and/or reinforced, with active prestress						Thickness selection
						Related details
TYPE F, REINFORCED for structural action						Thickness selection
						Related details

conventional reinforced concrete design methods should be used.

Type F slabs are typically built with portland cement, Types I or II, and are reinforced with conventional mild steel in the form of deformed bars or substantial wire fabric. One or two layers of reinforcement may be used; however, the steel must be carefully positioned according to design requirements. Since cracking is anticipated, joint spacings, usually set for crack control, are not critical, but they must be set to accommodate the construction process.

2.3-Design and construction variables

Design and construction of slabs on grade involves both technical and human factors. The technical factors include loadings, support system, joint types and spacings, the design method, the slab type, the concrete mix, and the construction process. Human factors involve the workers' abilities, feedback to evaluate the construction process, and anticipated maintenance procedures to compensate for cracking, curling, shrinkage, and other conditions.

These and other factors should be considered in planning a slab. It is important to consider not just one or two items, but to look judiciously at the full set of interactive variables?

2.4-Design methods

2.4.1 Introduction-Five basic slab design methods are discussed in this report:

* The Portland Cement Association (PCA)

method'

- The Wire Reinforcement Institute (WRI) method'
- The The United States Army Corps of Engineers (COE) method¹⁰
- The Post-Tensioning Institute (PTI) method'
- The shrinkage-compensating concrete method (ACI 223)

Structurally active reinforcement and fiber reinforcement are also used in slabs on grade, but separate design methods for them are not presented here.

All five methods have been used successfully, and Committee 360 considers all of the methods to be acceptable. The common objective of all the methods is to minimize cracking and produce the required flatness and serviceability (see ACI 302).

The design engineer has many choices when planning a slab on grade,^{12,13} as outlined in Table 2.1. Each method includes recommendations for joint type and spacing. The modulus of subgrade support and friction between the slab and its supporting grade are the two most important parameters that tie slab types and design methods together. Multiple combinations of concepts and methods on one job are not uncommon. Committee 360 believes there is no single correct or incorrect decision, but rather several combinations of slab type and design method, each with its own critical features. Each will produce a successful slab on grade if these features are properly handled.

2.4.2 Portland Cement Association (PCA) method- This slab design method, attributed to the Portland

Cement Association, is a thickness selection process: in chart form for wheel loading, rack, and post loading; and in tables for uniform loading (see examples in Appendix A1). Reinforcement is not required and is frequently not used. When used, it is placed in the slab for crack control, temperature effects and, in the case of dowels, for load transfer at joints.

The design is based on a computerized solution by Packard¹⁴ and uses influence charts by Pickett and Ray¹⁵ with the concept of equivalent single wheel loading centrally located at the interior of the slab.⁷ The slab analyzed has a radius of three times the radius of relative stiffness^l.*

$$l = \sqrt[4]{\frac{E h^3}{12 (1 - \mu^2) k}}$$

The effect of slab discontinuities beyond this limit is not included in the charts. PCA suggests that the slab be strengthened at the joints to account for lack of continuity. This is commonly done by thickening at edges or by use of smooth dowels or tie bars.

2.4.3 Wire Reinforcement Institute (WRI) method- This method presents design nomographs for slab thickness determination⁷ based on solutions using a discrete element computer model for the concrete slab as a continuum on a Winkler foundation? The slab is represented by rigid bars for slab flexure, by torsion bars for slab twisting, and by elastic joints for plate bending. Continuous support is provided by elastic spring constants at all joints. Design variables are the modulus of elasticity of the concrete the modulus of subgrade reaction, diameter of the loaded area, the spacing of the wheels, the concrete's modulus of rupture and the selected factor of safety. The WRI method provides solutions for wheel loading and for uniform loading with a variable aisle width. There is an additional aisle solution by Rice.¹⁷ The WRI approach graphically accounts for the relative stiffness between grade support and concrete slab in the determination of moments in the slab. Only loadings on the interior of the slab are considered. (See examples in Appendix A2.)

2.4.4 Corps of Engineers (COE) method-The Corps of Engineers method^{10,18} is based on Westergaard's formulae for edge stresses in the concrete slab. In this approach, the ability to support the load using both the unloaded slab and the loaded slab at the edge or joint in question is included. The joint transfer coefficient accounts for this action. The coefficient value used by the COE method is 0.75; thus the load support is reduced by 25 percent at the joint. The COE method uses a concrete modulus of elasticity of 4000 ksi, a Poisson's ratio of 0.20, an impact factor of 25 percent, and a safety factor of approximately 2. Variables in the nomographs are modulus of rupture, subgrade modulus, and the load. Loading

is handled by placing loads in categories and by using a design index category. This index internally fixes the value for wheel area, wheel spacing, axle loading and other constants. The safety factor is also built into the nomograph.

Appendix A3 illustrates the method and Table A3.1 shows the index categories.

2.4.5 Post-Tensioning Institute (PTI) method-The Post-Tensioning Institute manual¹¹ for the analysis and design of slabs with applied post-tensioning forces develops strength requirements in terms of moments and shears. While post-tensioning is the intended technique, deformed steel bars, welded wire fabric, or a combination of tendons and reinforcing steel can also be used.

The design procedure is intended for slabs lightly reinforced against shrinkage effects, for slabs reinforced and stiffened with ribs or beams, and for structural slabs. Slabs supported on unstable soils are also covered. In this situation, it is the supporting soil itself that may cause a loading on the slab.

The PTI method is based on a number of soil parameters and a number of structural parameters and their interaction. Some key parameters are climate, differential soil movement, a moisture stability index (known as the Thornthwaite moisture index), slab length and width, beam spacings, applied loadings, and the depth and width of the stiffening beams (also known as ribs). One section of the PTI manual presents an equation-based procedure for calculation of stresses caused by concentrated loadings on the interior of the slab perimeter. It is based on the theory of beams on elastic foundations.⁷ Its use is illustrated in Appendix A4.

2.4.6 ACI Committee 223 shrinkage-compensating concrete method (ACI 223)-This design method is unlike the previous four in that it does not deal directly with the slab thickness required for loads placed on the surface of the slab, which must be handled by one of the other methods shown in Table 2.1. Rather, it deals with the critical aspects of concrete mix expansion and shrinkage. ACI 223 specifies the proper amount of reinforcement, in the form of reinforcing steel, and its location within the depth of the slab for specific values of anticipated expansion and shrinkage. Requirements for expansion joints are stated, as are joint spacings.

2.5-Fiber-reinforced concrete (FRC)

The use of fiber reinforcement in slabs on grade is increasing. Fiber materials in use include steel, polypropylene, polyester, and polyethylene. While the design concepts used for other material options are also used for FRC slabs on grade, the potential increases in composite material properties, such as flexural strength and flexural fatigue endurance, are taken into consideration. References 20,21, and ACI 544.4R provide additional information.

2.6-Conclusion

There is no single design technique that the

* The radius of relative stiffness in inches is found by taking the fourth root of the results found by dividing the concrete plate stiffness by the subgrade modulus k .

committee recommends for all applications. Rather, there are a number of identifiable construction concepts and a number of design methods. Each combination must be selected based on the requirements of the specific application.

CHAPTER 3-SOIL SUPPORT SYSTEMS FOR SLABS ON GRADE

3.1 Introduction

Design of the slab on grade involves the interaction of the slab and the soil support system to resist moments and shears induced by the applied loads. Therefore, the properties of both the concrete and the soil are important. This chapter discusses soil support of the slab on grade only, including:

- types and properties of soil
- site testing for modulus of subgrade reaction
- range of values for the subgrade modulus
- how to compact and stabilize soils

Foundation design is an independent topic, not included in this document.

The soil support system usually consists of a base, a sub-base and a subgrade, as illustrated in Fig. 3.1. If the existing soil has the required strength and properties to support the slab, the slab may be placed directly on the existing subgrade. However, the existing grade is not normally at the correct elevation or slope. Therefore, some cut or fill is required with the best of site selections.

3.2-Soil classification and testing

There are many standards by which soils are classified. The Unified Soil Classification System is used in this document. Table 3.2.1 provides information on this classification system and some important properties of each soil class. For complete details, see ASTM D 2487.

The nature of the soil must be identified in order to determine its suitability as either a base, a subbase, or a

subgrade material. Various laboratory tests can be performed in order to identify the soil. Soil classification is based primarily on grain size and the Atterberg limits as indicated in Table 3.2.2.

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

1. Sample preparation - ASTM D 421
2. Moisture content - ASTM D 2216
3. Specific gravity - ASTM D 854
4. Material larger than #4 Sieve - ASTM C 127
5. Liquid limits - ASTM D 4318
6. Plastic limit - ASTM D 4318
7. Shrinkage limit - ASTM D 427
8. Sieve analysis - ASTM D 422
9. Standard Proctor density - ASTM D 698
10. Modified Proctor density - ASTM D 1557

A more detailed listing of the ASTM standards is given in Chapter 10.

3.3-Modulus of subgrade reaction

3.3.1 Introduction-Design methods listed in Chapter 2, including Westergaard's pioneering work, use the modulus of subgrade reaction to account for soil properties in design. The modulus, also called the modulus of soil reaction, is a spring constant that depends on the kind of soil, the degree of compaction, and the moisture content. The general procedure for static non-repetitive plate load tests outlined in ASTM D 1196 provides guidance in the field determination of the subgrade modulus. However, it is not specifically oriented to the determination of modulus of subgrade reaction using a 30 in. diameter plate for the test. Therefore, a brief description of the procedure is given in Sec. 3.3.2.

3.3.2 Procedure for the field test-Remove loose material from the surface of the grade or subgrade for an area 3 to 4 feet in diameter. Place a thin layer of sand or plaster of paris over this area to assure uniform bearing under the load plates. Then place three 1-in.-thick steel plates, 30,24, and 18 inches in diameter, stacked concentrically pyramid fashion on this surface. Rotate the plates on the bearing surface to assure complete contact with the subgrade.

Attach a minimum of three dial gages to 18-ft deflection beams spanning across the load plates. Position the three dial gages on the top of the 30-in. plate, 120 degrees apart, to record the plate deflection. Generally, a heavy piece of construction equipment can provide the 8000-lb load required for the test. Place a hydraulic jack on the center of the load plates and apply a proof load of approximately 700 to 800 lb to produce a deflection of approximately 0.01 in. Maintain this load until the settlement is stabilized; then release the load and reset the dial gages to zero.

After this preparation, the test is performed by applying a series of loads and recording the settlement of the plates. Generally, three load increments are sufficient.

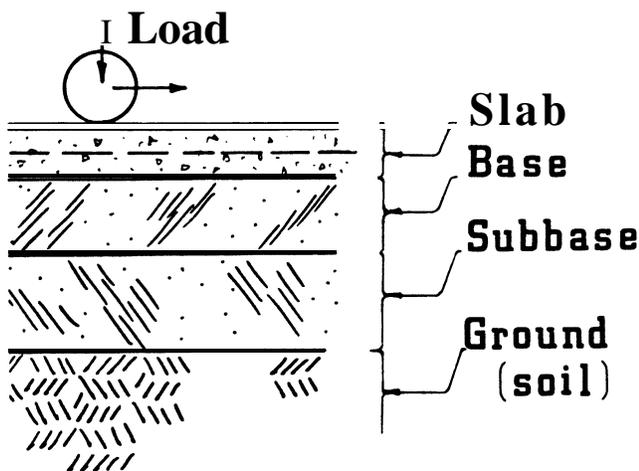


Fig. 3. 1-Soil system support terminology

Table 3.2.1-Unified soil classification system, from Reference 22

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 3 inches, and basing fractions on estimated weights)				GROUP SYMBOL	TYPICAL NAMES	
COARSE GRAINED SOILS (More than half of material is larger than No. 200 sieve*)	GRAVELS More than half of coarse fraction is larger than No. 4 sieve*	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well graded gravels, gravel-sand mixtures, little or no fines	
			Predominantly one size or a range of sizes with some intermediate sizes missing	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	
		GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see CL below.)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures	
			Plastic fines (for identification procedures see ML below)	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures	
	SANDS More than half of coarse fraction is smaller than No. 4 sieve*	CLEAN SANDS (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	SW	Well graded sands, gravelly sands, little or no fines	
			Predominantly one size or a 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)	Non-plastic fines (for identification procedures see ML below)	SM	Silty sands, poorly graded sand-silt mixtures	
			Plastic fines (for identification procedures see CL below)	SC	Clayey sands, poorly graded sand-clay mixtures	
Identification procedures on fraction smaller than no. 40 sieve						
FINE GRAINED SOILS (more than half of material is smaller than No. 200 sieve*)	SILTS AND CLAYS, liquid limit less than 50	DRY STRENGTH (crushing characteristics)	DILATANCY (reaction to shaking)	TOUGHNESS (consistency near plastic limit)	GROUP SYMBOL	TYPICAL NAMES
		None to slight	Quick to slow	None		
		Medium to high	None to very slow		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Slight to medium	Slow		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

* NOTES: All sieve sizes here are US. standard. The No. 200 sieve is about the smallest particle visible to the naked eye. For visual classifications, the 1/4-in. size may be used as equivalent for the No.4 sieve size. BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

The load should be maintained until the rate of settlement, an average recorded by dial gages is less than 0.001 in. per minute. The data should then be plotted on a load deflection graph and the modulus of subgrade reaction *k* determined. The value of *k* is calculated as 10 divided by the deformation produced by a 10 psi load. (A 7070-lb load produces 10 psi on a 30-in. plate.) If the dial gages are not zeroed before the test is run, an adjustment to the curve is required to make it intersect the origin as shown in Fig. 3.3.2. The calculation for *k* is also shown.

3.3.3 Modified modulus of subgrade reaction-A

modified modulus of subgrade reaction, based on a 12-in.-diameter plate test, can also be used to design slabs on grade. The modified modulus test is less expensive to perform, and the value for a given soil is twice that of the standard modulus.

3.3.4 Influence of moisture content-The moisture content of a fine-grained soil affects the modulus of subgrade reaction both at the time of testing and during the service life of the slab. For example, if the field test for a modulus of subgrade reaction is performed on a clay stratum with a liquid limit (LL) less than 50 and a

Table 3.2.2- Laboratory classification criteria for soils from Reference 22

Major Divisions	Group Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines		
		GP	Poorly graded gravels, gravel-sand mixture, little or no fines		
		GM ^a	d	Silty gravels, gravel-sand-silt mixtures	
			u	Clayey gravels, gravel-sand-clay mixtures	
		Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	SW	Well-graded sands, gravelly sands, little or no fines	
			SP	Poorly graded sands, gravelly sands, little or no fines	
	Sands with fines (Appreciable amount of fines)	SM ^a	d	Silty sands, sand-silt mixtures	
			u	Clayey sands, sand-clay mixtures	
		Determining percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: More than 5 percent fines to 12 percent 5 to 12 percent			$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 Not meeting all gradation requirements for GW
					Atterberg limits below "A" line or P.I. less than 4 Atterberg limits below "A" line with P.I. greater than 7
	Atterberg limits above "A" line or P.I. less than 4 Atterberg limits above "A" line with P.I. greater than 7			$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3 Not meeting all gradation requirements for SW	
				Limits plotting in hatched zone with P.I. between 4 and 7 are <i>borderline</i> cases requiring use of dual symbols	
Fine-grained soils (More than half material is smaller than No. 200 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		
				Plasticity Chart 	

^aDivision 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.

^bBorderline

GW-GC, well-graded gravel-sand mixture with clay binder.

example:

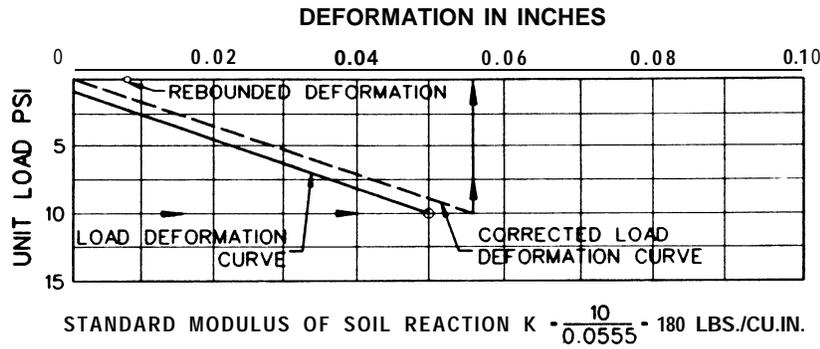


Fig. 3.3.2-Load-deformation plot for the plate field test

moisture content of 15 percent, the value of k will be higher than if the same test is performed with the material at a 23 percent moisture content.

Table 3.3.4 shows the approximate effect of moisture content on the value of the modulus of subgrade reaction for various types of soil. The following example shows how to use Table 3.3.4.

Assume that a test for the modulus k is performed on a clay stratum (LL less than 50) when the moisture content is 23 percent. From the data k is calculated to be 300 lb per cu in. (pci). What should the design value be if the long term value of the moisture content of the soil under the slab reduces to 15 percent? Using correction factors in Table 3.3.4

$$k(\text{design}) = 300 \times \frac{0.85}{0.65} = 392 \text{ lb per cu in.}$$

Conversely, if the moisture content at the time of the test is 15 percent and the projected moisture content during the life of the slab is 23 percent, the adjustment the test value would be:

$$k(\text{design}) = 300 \times \frac{0.65}{0.85} = 230 \text{ lb per cu in.}$$

3.3.5 Influence of soil material on modulus of subgrade reaction - Fig. 3.3.5 shows the general relationship between the soil classification and the range of values for the modulus of subgrade reaction. The figure also shows a general relationship between the California bearing ratio (CBR), modified modulus of subgrade reaction, and standard modulus of subgrade reaction which is the basis for slab on grade design.

The design examples in the appendix show the influence that the modulus of subgrade reaction has on the required slab thickness. Obvious design options are to improve the soil through such approaches as additional compaction, chemical stabilization or site drainage.

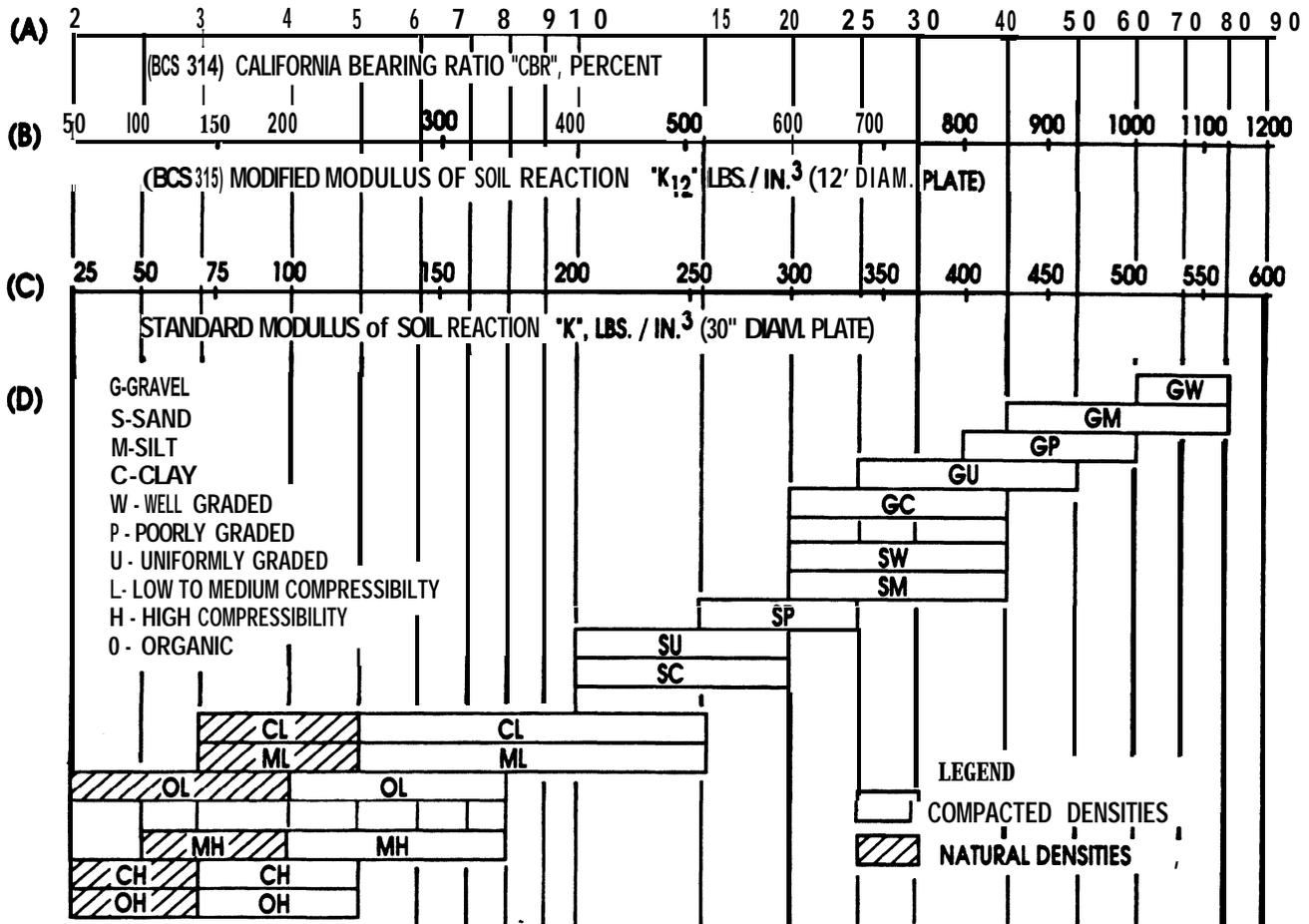
Under actual job conditions, the soil profile is generally made up of many layers of different materials, with the influence of the base and subbase predominant. Engineering judgement is required to select approximate values used during design. During construction verify the chosen value by on site testing before placing slabs.

3.4-Design of the slab support system

3.4.1 General-After the soils have been classified, the general range of k values can be approximated from Table 3.3.5. With this information, a decision can be made to densify the soil, improve the base material with

Table 3.3.4-Moisture content correction factors, from Reference 22

TYPE OF MATERIAL	Moisture content at time of test, percent of dry weight						
	5-10	11-14	15-18	19-21	22-24	25-28	over 28
Silts and clays LL < 50	0.35	0.50	0.65	0.75	0.85	1.0	1.0
Silts and clays LL > 50	0.25	0.35	0.50	0.63	0.75	0.85	1.0
Clayey sand or Clayey gravel	0.75	0.9	1.0	1.0	1.0	1.0	1.0



Note: Comparison of soil type to 'K', particularly in the "Land • Hm Groups, should generally be made in the lower range of the soil type.

Fig. 3.3.5-Interrelationship of soil classification and strengths (from Reference 23)

sand or gravel fill, or use the existing material in its in-situ condition.

Normally there is a wide range of soils across the site. The soil support system is rarely uniform. Therefore, some soil work is generally required to provide a more uniform surface to support the slab. The extent of this work, such as the degree of compaction or the addition of a sand-gravel base, is generally a problem of economics. Selection of soils in the wellgraded gravel (GW) and poorly graded gravel (GP) groups as a base material may appear costly. However, the selection of these materials has distinct advantages. Not only do they provide a superior modulus of subgrade reaction, but they also tend to speed construction during inclement weather.

3.4.2 Economics and simplified design-Certainly not all projects will require all of the data discussed above. On projects where the slab performance is not critical, engineering judgement should be exercised to reduce costs. A prime prerequisite for the proper design of a slab support system is soils identification. Without this knowledge, the modulus of subgrade reaction is unknown and potential volume change cannot be determined. With knowledge of soil classification, the engineer can select

an appropriate *k* value and design for the specific soil conditions.

For small projects, it may be advantageous to assume a low *k* factor and add a selected thickness of crushed stone to enhance the safety factor rather than performing an expensive soil analysis. Use of the modified modulus of subgrade reaction test rather than the standard modulus test can also reduce costs. Risk of slab failure at an earlier age increases as the design is rationalized but there are occasions where the simplified design approach is justified. These decisions are a matter of engineering judgment and economics.

Compounding safety factors is a common error. Inclusion of safety factors in the modulus of subgrade reaction, the applied loads, the compressive strength of the concrete, the flexural strength of the concrete and the number of load repetitions will produce an expensive design. The safety factor is normally contained in the flexural strength of the concrete and is a function of the number of load repetitions (see Sec. 4.9).

3.5-Site preparation

3.5.1 Introduction-Prior to soil compaction, the top

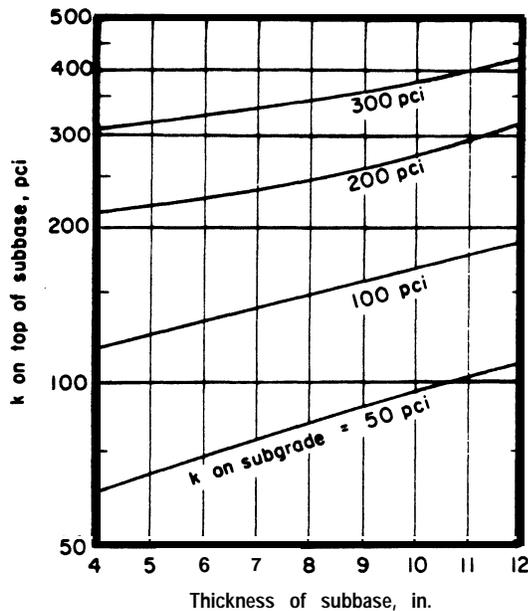


Fig. 3.5.3-Effect of selected fill on modulus of subgrade reaction (from Reference 14)

layer of soil must be stripped of all humus and frozen material. Both hard and soft pockets of soil material should be removed and recompacted to provide a uniform support for the base, subbase or concrete slab. See ACI 302.1R for additional information.

When a thick combination of base and subbase is provided, sinks, holes, expansive soils, highly compressible materials, or any other problems that can influence the life of the slab must be examined. Normally, the surface is stripped and recompacted before the subbase is placed.

3.5.2 Subgrade stabilization -There are many methods of improving the performance of the soil system by densification and drainage (see list in the U.S. Navy's Design Manual.²⁴) Generally, for slab on grade, the soil is densified by using rolling equipment such as sheepsfoot, rubber tire, or vibratory rollers. The degree of compaction is normally measured and controlled by ASTM D 698 (standard) or D 1557 (modified) Proctor density curves.

Another densification method used to improve the entire building site is preloading. A surcharge is placed over the building site in order to decrease the voids in the original soil system. This procedure not only reduces total and differential settlement for the overall structure but also improves the modulus of subgrade reaction.

Drainage of the soil is an effective approach to densification. The site is drained by ditches, tunnels, pervious fills, or subsoil drains. This reduces ground water pressure and increases effective stresses in the soil system.

Chemical methods listed in Table 3.5.2 can also be used to stabilize soil. Generally, portland cement, lime, calcium chloride, or bitumen is mixed into the soil substrate, and the mixture is recompacted. Less common than densification stabilization, chemical stabilization is a viable procedure, especially with expansive soils.

3.5.3 Base and subbase material-The base and subbase frequently comprise a thick stratum used to bring the surface of the soil support system to a uniform elevation under the slab. The subbase is usually a good economical fill material, with the base being a thinner layer of more expensive material having a superior value of modulus of subgrade reaction.

Often the existing subgrade may be a satisfactory base material. Generally the materials listed in Fig. 3.3.5 that yield a standard modulus of subgrade reaction above 125 pci, can be used. The soils below this value, as well as the low compressibility organic material (OL) and high compressibility silt (MH) are to be avoided. Note in Fig. 3.3.5 that k for soil type CL (low compressibility clay) ranges from 70 to a high of 250. Much of this variation is a product of the degree of compaction and/or moisture content of the soil.

Frequently, a selected fill used as a base material bears on a weaker subgrade. Normally, these selected materials are from the G and S (gravel and sand) classification. How they affect k values depends on both the type and thickness of the material. A typical effect of selected fill on k values is shown in Figure 3.5.3. Data for specific designs should be based on laboratory analysis and site testing.

3.5.4 Stabilization of base and subbase-Weak base material can be stabilized by the addition of chemicals that are mixed or combined with the soil, as shown in Table 3.5.2. Lime and calcium are also used to lower the plasticity index of subgrades, subbases, and base materials. For silty soils, portland cement may be effective. It is recommended that a geotechnical expert plan, supervise, and analyze the soil conditions before chemical stabilization is used.

Base and subbase material are often densified by mechanical compaction with a subsequent improvement in the k value. The relative cost of options such as chemical stabilization 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). Nominal targets for these materials are from 90 to 95 percent of the modified Proctor density. Estimates of k values resulting from this and other compactive efforts can be projected from laboratory CBR values, as shown in Fig. 3.3.5. The depth of compacted lifts varies with soil type and compaction equipment, but in most cases should be 6 to 9 inches (150-225 mm). Granular soils are most responsive to vibratory equipment and cohesive soils respond best to sheepsfoot and rubber-tired rollers.

3.5.5 Grading tolerance- Initial rough grade tolerance should be ± 0.1 ft (30 mm). After the forms are set, final grading and compaction should be completed prior to slab placement. The final elevation of the base material should be no more than $\frac{1}{4}$ in. above or $\frac{1}{2}$ in. below the design grade.

Table 3.5.2-Soil stabilization with chemical admixtures

ADMIXTURE	QUANTITY, % BY WEIGHT OF STABILIZED SOIL	PROCESS	APPLICABILITY	EFFECT ON SOIL PROPERTIES
PORTLAND CEMENT	Varies from about 2½ 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 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 surface should be added to provide abrasion resistance. Not applicable to plastic clays.	Unconfined compressive strength increased up to about 1000 psi. Decreases soil plasticity. Increases durability in 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 grain materials. For coarse grain soils antistripping 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 for 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	½ TO 1 ½%	Normally applied at rate of about 0.5 lb/sq yd area. Dry chemical is blended with soil-aggregate mixture, water added, and mixture compacted at optimum moisture by conventional compaction procedures.	Used as dust palliative. Stabilized mix 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%. Flyash, between 10 and 20%, may be added to increase pozzolanic action.	Lime is spread dry, mixed with soil by pulvi-mixers 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 about 500 psi.

3.6-Inspection and site testing of soil support

To control the quality of the soils work, inspection and testing are required. As the soil support system is placed, the soil classification of the fill material should already have been determined and the in-place density should be checked. The in-place density as a percent of standard or modified Proctor density should be verified using a nuclear density meter (ASTM D 2922) or by the sand cone method (ASTM D 1556).

After the controlled fill is placed, the surface of the base should be checked for in-situ k values. Higher in-situ k values offer an opportunity for thinner slabs. Lower values require a thicker slab or indicate a lower effective factor of safety with a decrease in slab life.

Testing frequency is related to the work quality. Substandard work may require more testing. The over-all quality of the work can be controlled by statistical analysis similar to that used in Sections 2.3.1 and 2.3.2 of ACI 318 to maintain quality control of the concrete. A rea-

sonable target is to be 90 percent certain that 85 percent of the work meets or exceeds minimum specifications. Fig. 3.6 can be used to evaluate achievement of this target.

For example, if the minimum specified modified Proctor density is 90 percent, and the first six tests furnished by the soils technician are as follows: 93, 92, 94, 93, 88 and 95 percent; then the average of these values is 92.5 percent. The spread is from 88 percent to 95 percent, or 7 percent. When this spread is plotted on Fig. 3.6 (point A), it falls below the line for six tests, and fails. Therefore, one cannot be 90 percent certain that 85 percent of the compaction work will meet the specified minimum. If six tests yield values of 91, 95, 95, 96, 93 and 95 percent modified Proctor, then the average is 94.1 percent and the spread is 5 percent. When this is plotted on Fig. 3.6 (point B) it is above the control line for six tests, and therefore the compaction meets the target.

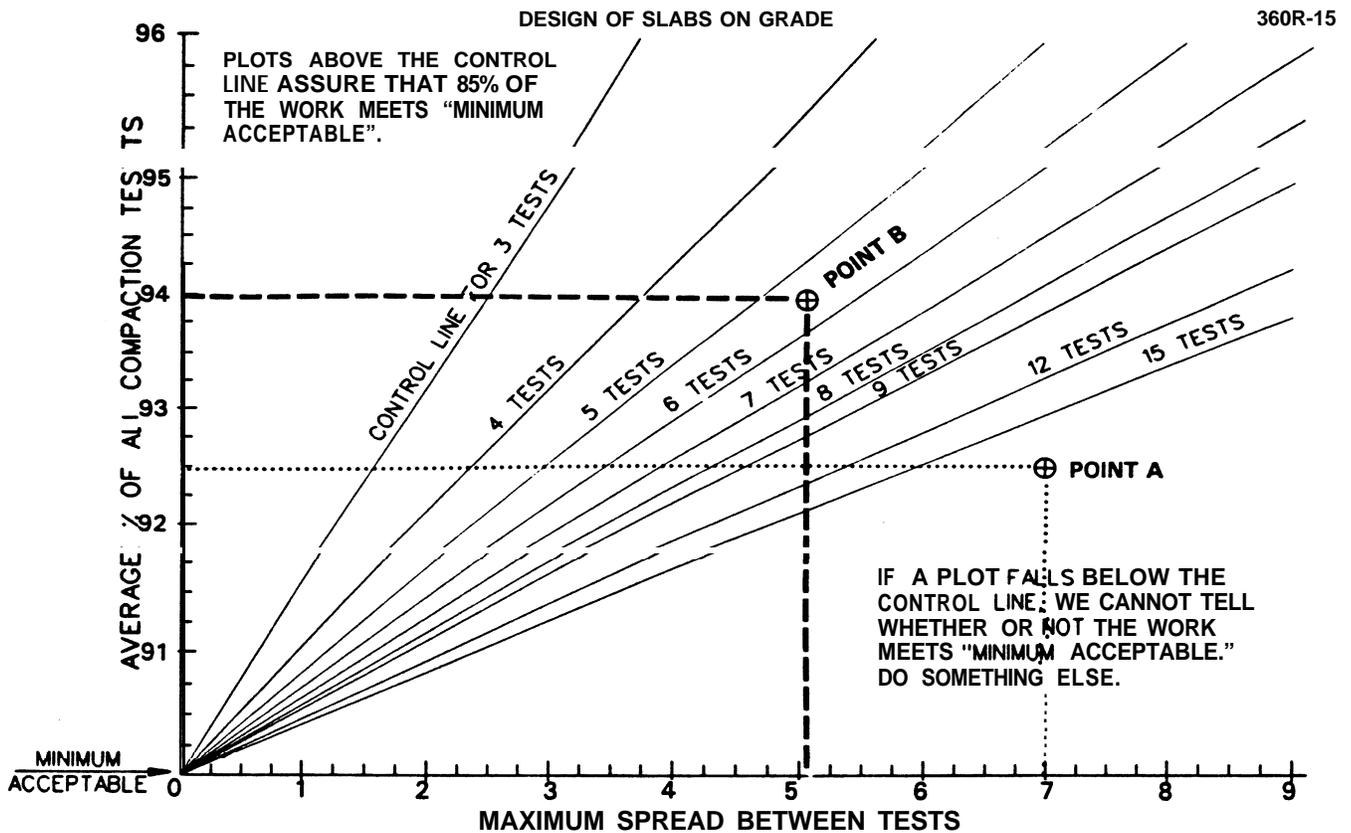


Fig. 3.6-Evaluation of control test results for soil compaction

3.7-Special slab on grade support problems

Placement of slabs on topsoil should generally be avoided. In extreme cases where it is unavoidable, special precautions and approaches must be undertaken, as described in Reference 25.

Expansive soils are defined as fine grained soils, as shown in Tables 3.2.1 and 3.2.2. As a general rule, any soil with a plasticity index of 20 or higher has 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 under the slabs. Failure to manage the problem can and often will result in early slab failure.

Frost action may be critical to silts, clays, and some sands. These soils can experience large changes of volume when subjected to freezing cycles. Three conditions must be present for this problem to occur:

- Freezing temperature in the soil
- Water table close enough to the frost level to form ice lenses
- A soil that will act as a wick to 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 occur at building perimeters,

under freezer areas, and under ice skating rink floors.²⁶

CHAPTER 4-LOADS

4.1-Introduction

This chapter describes loadings and load conditions commonly applied to concrete slabs on grade. Appropriate factors of safety and the variables that control load effects are described. Where vertical forces from a superstructure are transmitted through the slab on grade to the soil, requirements of the applicable building code must also be followed.

Concrete slabs are usually subjected to some combination of the following:

- Vehicle wheel loads
- Concentrated loads
- Line and strip loads
- Uniform loads
- Construction loads
- Environmental effects including expansive soil
- Unusual loads, such as forces caused by differential settlement

Slabs must be designed for the most critical combination of these loading conditions, considering such variables as the maximum load, its contact area, and load spacing. The Portland Cement Association guide for selecting the most critical or controlling design considerations for var-

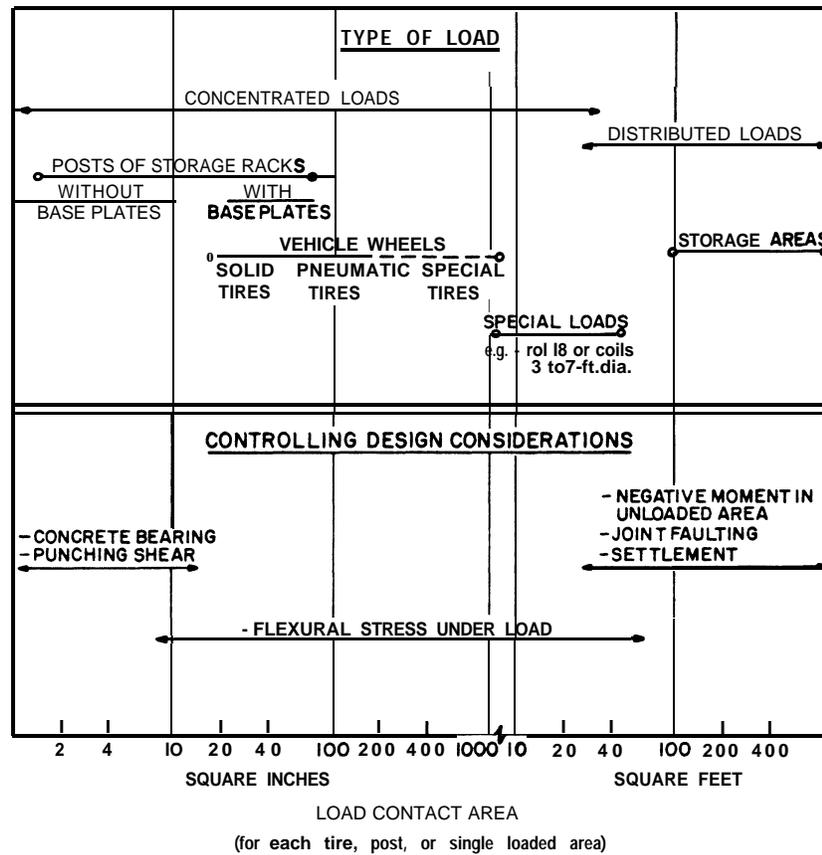


Fig. 4.1—Controlling design considerations for various types of slab on grade loadings (from Reference 14)

ious load types¹⁴ is presented in Fig. 4.1. Since a number of factors such as slab thickness, concrete strength, subgrade stiffness, compressibility, and loadings are relevant, areas where several design considerations may control should be investigated thoroughly.

Other potential problems such as load conditions which change during the life of the structure and those encountered during construction¹³ must also be considered. For example, material handling systems today make improved use of the building volume. Stacked pallets which were once considered uniform loads may now be stored in narrow-aisle pallet racks which produce concentrated loads. Critical loading conditions may change, and load magnitudes may increase due to the storage of denser materials or the use of new handling equipment.¹³ In either case, the actual loading during the life of the structure and its grade slab may differ significantly from the original design assumptions.

The environmental exposure of the slab on grade is also a concern. Normally, thermal effects are not considered since the slab is usually constructed after the building is enclosed. However, with the use of strip placement, more and more slabs are being placed prior to building enclosure. The construction sequence is therefore important in determining whether or not environmental factors should be considered in the design. This is discussed in greater detail in Chapter 9.

4.2—Vehicle loads

Most vehicular traffic on industrial floors consists of lift trucks and distribution trucks with payload capacities as high as 70,000 lb. The payload and much of a truck's weight are generally carried by the wheels of the loaded axle. The Industrial Truck Association²⁷ has compiled representative load and geometry data for lift truck capacities up to 20,000 lb (Table 4.2). The contact area between tire and slab must also be included in the analysis for larger lift trucks with pneumatic or composition tires.¹³

Vehicle variables affecting the thickness selection and design of slab on grade include the following:

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

The axle load, wheel spacing, and contact area are a function of the lift truck or vehicle specifications. If vehicle details are unknown, the values in Table 4.2 may be adopted. 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 must also be considered.

The contact area of a single tire can be approximated by dividing the tire load by the tire pressure.¹⁴

This is somewhat conservative since the effect of tension in the tire wall is not included. Assumed pressures are variable; however, pneumatic tire pressures range from 80 to 100 psi, while steel cord tire pressures range up to 120 psi. 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.²⁸

Dual tires spread the load over an area greater than the actual contact area of the two individual tires. An area equal to that of the two tires and the area between them is a conservative estimate.²⁹ The rectangular area between the tires has a length equal to the distance between tires and a width equal to the diameter of the single tire 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.

4.3-Concentrated loads

Because of increasing building costs, there has been a trend toward more efficient use of warehouse space. This has led to narrower aisles, higher material stacking, and the use of automated stacking equipment. Material storage racks may be higher than 80 ft and may produce concentrated post loads of 40,000 lb or more. For the higher racks, these loads may well exceed the vehicle wheel loads and thus control the thickness selection.

Table 4.2-Representative axle loads and wheel spacings for various lift truck capacities (from Reference 27)

Truck rated capacity, lb	Total axle load static reaction, lb	Center to center of opposite wheel tires, in.
2,000	5,600-7,200	24-32
3,000	7,800-9,400	26-34
4,000	9,800-11,600	30-36
5,000	11,600-13,800	30-36
6,000	13,600-15,500	30-36
7,000	15,300-18,100	34-37
8,000	16,700-20,400	34-38
10,000	20,200-23,800	37-45
12,000	23,800-27,500	38-40
15,000	30,000-35,300	34-43
20,000	39,700-43,700	36-53

The concentrated reaction per tire is calculated by dividing the total axle load reaction by the number of tires on that axle. Figures given are for standard trucks. The application of attachments, extended high lifts, etc., may increase these values. In such case, 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.

In some designs where these racks also support the building's roof the rack posts themselves are primary structural elements. Appropriate requirements of the building code, including mandatory safety or load factors, must be followed.

The concentrated load variables which affect design of the slab on grade are:

- Maximum or representative post load
- Spacings between posts and aisle width
- Area of contact between post or post plate and slab

Material handling systems are a major part of the building layout and are generally 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 stress effect of the concentrated load.

4.4-Uniform loads

In many warehouse and industrial-buildings, materials are stored directly on the slab on grade. The flexural stresses in the slab are usually less than those produced by concentrated loads. The design must endeavor to prevent negative moment cracks in the aisles and to prevent excessive settlement. 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. However, the individual cases are always considered in the design.

For uniform loads, the variables affecting the design of slab on grade are:

- Maximum load intensity
- Width and length of loaded area
- Aisle width
- Presence of a joint located in and parallel to the aisle

Loads for randomly stacked materials are not normally predictable, nor are they constant during the service life of a slab. Therefore, the slab should be designed for the most critical case. The maximum moment in the center of an aisle is a function of aisle width as well as other parameters. For a given modulus of subgrade reaction, modulus of rupture, and slab thickness, there is an aisle width that maximizes the center aisle moment. This critical aisle width is important in the design. Wider aisles are generally less critical.

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 one-third of the radius of relative stiffness (see [Sec. 2.4.2](#)). When the width approximates this limit, the slab should be reviewed for stresses produced by line loading as well as uniform load. If the results are within 15 percent of one another, the load should be taken as uniform. Partition loads, bearing walls, and roll storage are examples of this load type.

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

- Maximum load intensity
- Width and length of loaded area

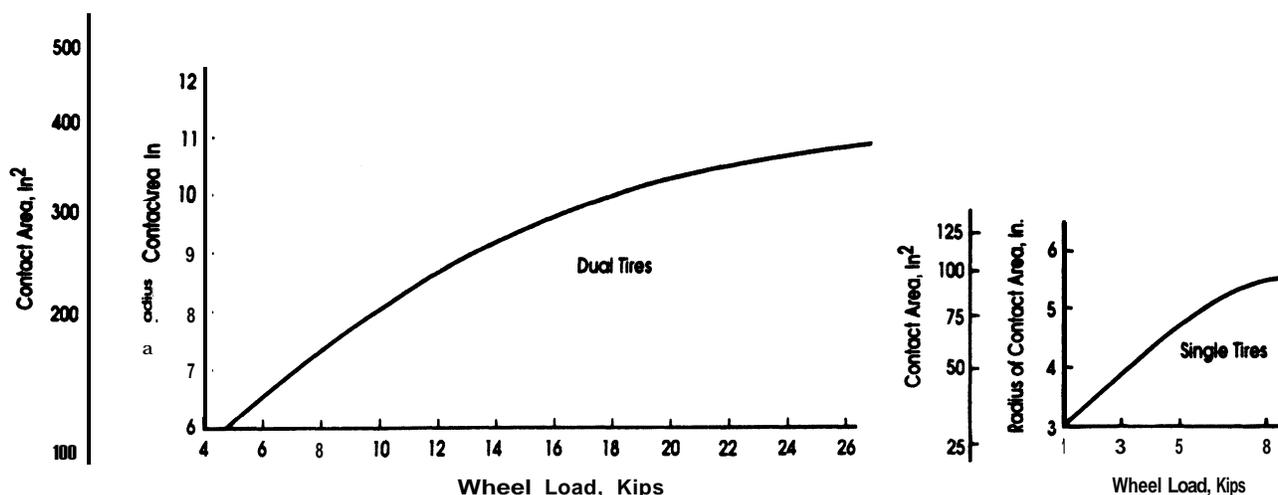


Fig. 4.7-Tire contact area for various wheel loads

- Aisle width
- Presence of a joint in and parallel to the aisle
- Presence of parallel joints on each side of the aisle

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:

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

However, the load variables and the factor of safety 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 grade. The most common construction loads are pick-up trucks, concrete trucks, dump trucks, and hoisting equipment. In addition, the slab may be subjected to other loads such as scaffolding and material pallets. Some of these loads can exceed the functional design limits, and their effects should be anticipated.

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 American Association of State Highway and Transportation Officials standards. Off-road construction equipment may exceed these limits, but in most cases, construction equipment will not exceed the legal limits of the state. Fig. 4.7 gives values of contact area for wheel loads that can be used for design.

4.8-Environmental factors

Stresses and load effects produced by thermal and moisture changes must be considered in the overall de-

sign. These effects are of particular importance for exterior slabs and for slabs constructed before the building is enclosed. Curling caused by these changes increases the flexural stress due to the reduction in subgrade support. Generally, the restraint stresses can be ignored in short slabs, since the subgrade does not significantly restrain the short-slab movement due to uniform thermal expansion, contraction, or drying shrinkage. Built-in restraints, such as foundation elements, edge walls, and pits should be avoided. Environmental factors are discussed further in Chapter 9.

4.9-Factors of safety

The factor of safety for a slab on grade is never dictated by a building code. The designer selects it on the basis of experience.¹³ The safety factor accounts for a number of items including:

- Ratio of modulus of rupture to the tensile bending stress caused by imposed loadings
- Influence of shrinkage stresses
- Number of load repetitions
- Fatigue and impact effects

A critical factor in the performance of a slab is the number of vehicles crossing a slab edge or joint. Shrinkage stresses and impact are usually less significant in the design, but shrinkage is important to performance since it causes cracking, curling, dishing, and subsequent strength loss. Shrinkage stresses and the relationship of subgrade drag and joint spacing are discussed in Chapters 6 and 9.

A moving vehicle subjects the slab on grade to the effect of fatigue. Fatigue strength is expressed as the percentage of the static tensile strength that can be supported for a given number of load repetitions. As the ratio of the actual flexural stress to the modulus of rupture decreases, the slab can withstand more load repetitions before failure. For stress ratios less than 0.50, concrete can be subjected to unlimited load repetitions according to PCA.⁸ Table 4.9.1 (taken in part from Reference 8) shows various load repetitions for a range of stress ratios.

The safety factor is the inverse of the stress ratio.

Commonly applied safety factors are shown in Table 4.9.2 for the various types of slab loadings. Most range from 1.7 to 2.0, although factors as low as 1.4 are applied for some conditions. For more substantial concentrated structural loads, Reference 30 recommends safety factors ranging as high as 3.9 to 4.8. These higher

Table 4.9.1-Allowable load repetitions for various stress ratios (from Reference 52)

Stress Ratio	Allowable Repetitions	Stress Ratio	Allowable Repetitions
0.51	400,000	0.69	2,500
0.52	300,000	0.70	2,000
0.53	240,000	0.71	1,500
0.54	180,000	0.72	1,100
0.55	130,000	0.73	850
0.56	100,000	0.74	650
0.57	75,000	0.75	490
0.58	57,000	0.76	350
0.59	42,000	0.77	270
0.60	32,000	0.78	210
0.61	24,000	0.79	160
0.62	18,000	0.80	120
0.63	14,000	0.81	90
0.64	11,000	0.82	70
0.65	8,000	0.83	50
0.66	6,000	0.84	40
0.67	4,500	0.85	30
0.68	3,500		

Table 4.9.2-Factors of safety used in design for 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 a 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.

values are for special circumstances where the slab is considered to be governed by requirements for plain concrete. Higher values may also be applicable where settle-

ment controls or where rack layouts **are not coordinated** with the area layout.

4.10-Summary

Externally-applied loads and environmental **factors** that affect the design of slabs on grade are **not as clearly** defined as they are for structural elements subjected to usual building loads. However, since slab **distress is** caused by external loadings as well as environmental effects, it is important to account for these factors accurately.

CHAPTER 5-DESIGN OF PLAIN CONCRETE SLABS

5.1-Introduction

Slabs on grade are frequently designed as plain concrete slabs where reinforcement, if used in any form, serves in a manner other than providing strength to the uncracked slab. The amounts of reinforcement used, as well as joint spacings, are to control cracking and to prevent the cracks from gaping or faulting.^{8,30}

The purpose of the plain concrete slab on grade is to transmit loadings from their source to the subgrade with minimal distress. Design methods cited consider the strength of the concrete slab based on its uncracked and unreinforced properties.

Three methods available for selecting the thickness of the plain slab on grade are described in this chapter:

- The Portland Cement Association (PCA) method
- The Wire Reinforcement Institute (WRI) method
- The Corps of Engineers (COE) method

The PCA and WRI methods are for interior loadings while the COE method is for edge or joint loading cases only. Design examples in Appendices A1, A2, and A3 show how to use all three methods.

5.2-Portland Cement Association (PCA) design method

The PCA method is based on Pickett's analysis? The variables used are flexural strength, working stress, wheel contact area and spacing, and the subgrade modulus. Assumed values are Poisson's ratio (0.15) and the concrete modulus of elasticity (4000 ksi). 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.

5.2.1 Wheel loads-Grade slabs are subjected to various types, sizes, and magnitudes of wheel loads. Lift-truck loading is a common example, where forces from wheels are transmitted to the slab. Small wheels have tire inflation pressures in the general range of 85 to 100 psi for pneumatic tires, 90 to 120 psi for steel cord tires, and 150 to 250 psi for solid or cushion tires. Large wheels have tire pressures ranging from 50 to 90 psi. Appendix A1 shows use of the PCA design charts for wheel loadings.

5.2.2 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. [Appendix Sec. A1.3](#) shows the PCA design charts used for concentrated loads as found in conventionally spaced rack and post storage.

5.2.3 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 and load placement. Consolidation of the subgrade is beyond the scope of this report. The PCA tables for uniform loads ([Appendix A1](#)) are based on the work of Hetenyi,¹⁹ 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.

5.2.4 Construction loads-The PCA method does not directly address construction loading. However, if such loading can be determined as equivalent wheel loads, concentrated loads or uniform loads, the same charts and tables can be used.

5.3-Wire Reinforcement Institute (WRI) design method

5.3.1 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 slab thickness), diameter of equivalent loaded area, distance between wheels, flexural strength, and working stress.

5.3.2 Wheel loads-Grade slabs subjected to wheel loadings were discussed in [Section 5.2.1](#). 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 A2](#) shows the use of the WRI design charts for wheel loadings.

5.3.3 Concentrated loads-WRI charts do not cover concentrated loads directly. It is possible, however, to determine the equivalent wheel loading which represents a concentrated loading and thereby use the wheel load charts for this purpose.

5.3.4 Uniform loads-WRI provides other charts ([Appendix A2](#)) 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 5.3.1](#), the width of the aisle and the magnitude of the uniform load are variables in this method.

5.3.5 Construction loads-Various construction loads such as equipment, cranes, ready-mix trucks, and pick-up trucks may affect slab thickness design. As with the PCA design method, these are not directly addressed by WRI. However, thickness design may be based on an equivalent loading expressed in terms of wheel loads or uniform

loads.

5.4-Corps of Engineers (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 grade. 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 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 percent, concrete modulus of elasticity of 4000 ksi, Poisson's ratio of 0.20, the contact area of each wheel, and the wheel spacings. The latter two are fixed internally for each index category.

[Appendix A3](#) illustrates the use of the design index category and the COE charts. Additional design charts (for pavements with unprotected corners and with protected corners) have been developed by the Corps of Engineers for pavements although they may be applied to slabs on grade in general.

CHAPTER 6-DESIGN OF SLABS WITH SHRINKAGE AND TEMPERATURE REINFORCEMENT

6.1-Introduction

Slabs on grade are designed and their thickness is selected to prevent cracking due to external loading as discussed in [Chapter 4](#). Slab thickness calculations are based on the assumption of an uncracked and unreinforced slab. Steel reinforcement-commonly plain or deformed welded wire fabric, bar mats, or deformed reinforcing bars-is sometimes used in slabs on grade to improve performance of the slab under certain conditions.

Even though the slab is intended to remain uncracked under service loading, the reinforcement is used to aid in crack control; to permit use of longer joint spacings, thereby reducing the number of joints; to increase load transfer ability at joints; and to provide reserve strength after shrinkage or temperature cracking occurs.

6.2-Thickness design methods

The methods described in [Chapter 5](#) may be used to determine the thickness and joint spacings of reinforced slabs on grade. The WRI and PCA methods are intended

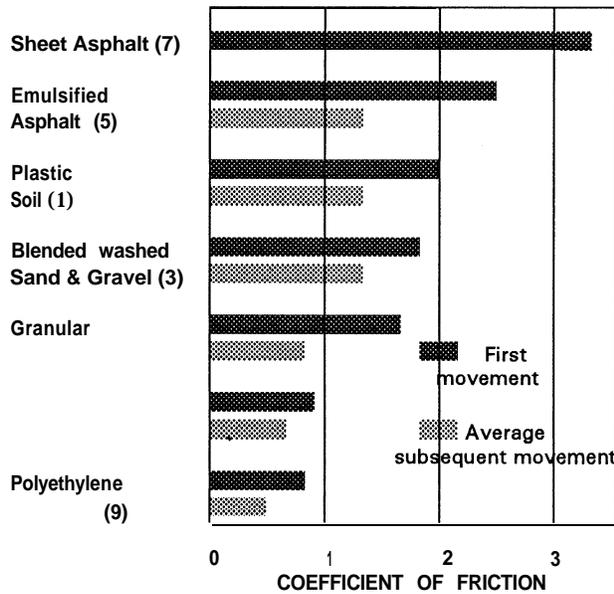


Fig. 6.3-Variation in values of coefficient of friction for 5-in. slabs on different bases and subbases (based on Reference 11)

for interior loading cases, while the COE method is intended for edge or joint loading cases. The required cross-sectional area of steel for shrinkage and temperature reinforcement is calculated using the subgrade drag theory formula explained in the following section.

6.3-Subgrade drag equation

The subgrade drag equation is frequently used to determine the amount of non-prestressed reinforcement to serve as shrinkage and temperature reinforcement and to control crack widths for slabs on grade. It does not apply when prestressing or fibers are used. The reinforcement selected by this equation is not intended to serve as flexural reinforcement.

$$A_s = \frac{F L w}{2 f_s} \tag{6-3}$$

where

- A_s = cross-sectional area in sq in. of steel per lineal ft
- f_s = allowable stress in the reinforcement, psi
- F = the friction factor (designated μ in Chapter 8)
- L = distance in ft between joints (the distance between the free ends of the slab that can move due to shrinkage contraction or thermal expansion)
- w = dead weight of the slab, psf, usually assumed to be 12.5 psf per in. of thickness

The value of 2 in the denominator is based on the assumption that the slab will shrink in such a manner that each end will move an equal distance towards the center. This is not always the case. The number 2 is not a safety

factor.

The friction factor varies from less than 1 to more than 2.5. A value of 1.5 is common. Additional values are shown in Fig. 6.3. Construction features that increase restraint will in effect alter and increase the friction factor.

A safety factor is provided in the allowable stress in the steel. The engineer makes a judgment as to the value of f_s . Commonly used values are $\frac{2}{3}$ to $\frac{3}{4}$ of the yield point of the steel. This allows the stress in the reinforcement to remain less than the proportional limit of the material, which is necessary for the reinforcement to function.

Applying the formula for an 8-in.-thick slab:

For $w = 100$ psf $F = 1.5$ $L = 20$ ft and $f_s = 30,000$ psi

$$A_s = (1.5 \times 20 \times 100)/(2 \times 30,000) = 0.05 \text{ in.}^2 \text{ per ft, on a } 20 \times 20\text{-ft unit}$$

This could be satisfied by WWF 12 x 12 W5 x W5, although for wire reinforcement a higher value for f_s would be acceptable.

If L were 40 ft, then the area A_s would be 0.10 in.² per ft on a 40 x 40-ft unit. This could be satisfied by #3 bars at 12 in. both ways (Grade 60) or WWF 12 x 12 W10 x W10 or by WWF 4 x 4 W4 x W4.

6.4-Reinforcement location

Shrinkage and temperature reinforcement should be at or above middepth of the slab on grade, never below middepth.

A common practice is to specify that the steel be 1.5 to 2 in. below the top surface of the concrete, or at $\frac{1}{3}$ the slab depth below the surface.

CHAPTER 7-DESIGN OF SHRINKAGE-COMPENSATING CONCRETE SLABS

7.1-Introduction

This chapter deals with concrete slabs on grade constructed with shrinkage-compensating cement conforming to ASTM C 845. The design procedure differs significantly from that for conventional concrete with ASTM C 150 portland cements 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 concretes. ACI 224R discusses this phenomenon in detail. Volume changes also occur with temperature changes. How shrinkage-compensating concretes differ from conventional concretes with respect to these volume changes is explained below.

7.1.1 Portland cement and blended cement concretes- The shortening of portland cement and blended cement concretes due to shrinkage is restrained by friction be-

tween 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 to 0.04 in., aggregate interlock (load transfer) becomes ineffective. Cracking due to shrinkage restraint may be limited by closer joint spacing, additional distributed reinforcement or post-tensioning.

7.1.2 Shrinkage-compensating concretes compared with conventional concretes-Shrinkage-compensating cement is also used to limit cracking.^{31,32,33} Shrinkage-compensating concrete is made with cement conforming to ASTM C 845 rather than ASTM C 150 or ASTM C 595. Therefore the volume change characteristics are different.

Shrinkage-compensating concrete undergoes an initial volume increase during the first few days of curing, then undergoes drying shrinkage similar to that of conventional concrete. This action provides early compression to restrained concrete due to the restraint of the mass, possible subgrade friction, perimeter edge restraint, and by embedded reinforcement.

In reinforced concrete which is free to expand, 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. The level of compressive stress is normally low enough to prevent overstressing of the reinforcement, and yet high enough to provide adequate concrete strain to offset subsequent negative creep and shrinkage strains.

The three basic differences between expansive concrete and normal concrete are:

- Early expansion instead of early shrinkage with shrinkage-compensating concrete
- Delayed shrinkage strain with shrinkage-compensating concrete
- A lower level of total residual shrinkage strain at later ages with shrinkage-compensating concrete

With shrinkage-compensating concrete, it is intended that the restrained expansion be greater than the resultant long-term shrinkage as shown in Fig. 7.1.2.1 and 7.1.2.2.

7.2-Thickness determination

For a slab on grade cast with shrinkage-compensating concrete, the determination of the slab thickness required by imposed loadings 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 Appendices A1, A2, and A3. Appendix A5 illustrates other design considerations peculiar to the use of the shrinkage-compensating concretes.

7.3-Typical reinforcement conditions

Table 7.3 shows typical reinforcement percentages for a 6-in. slab on grade. The compressive stress which re-

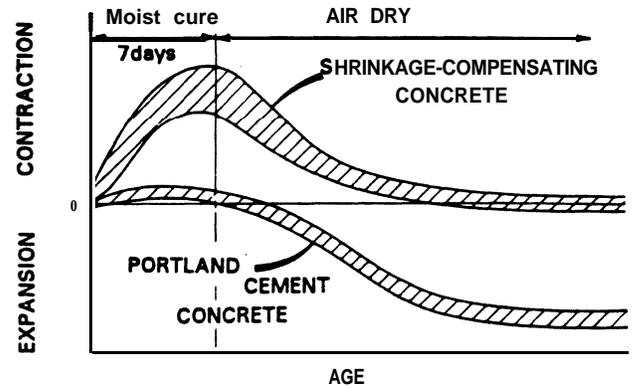


Fig. 7.1.2.1-Typical length change characteristics of shrinkage-compensating and portland cement concretes from Reference 31)

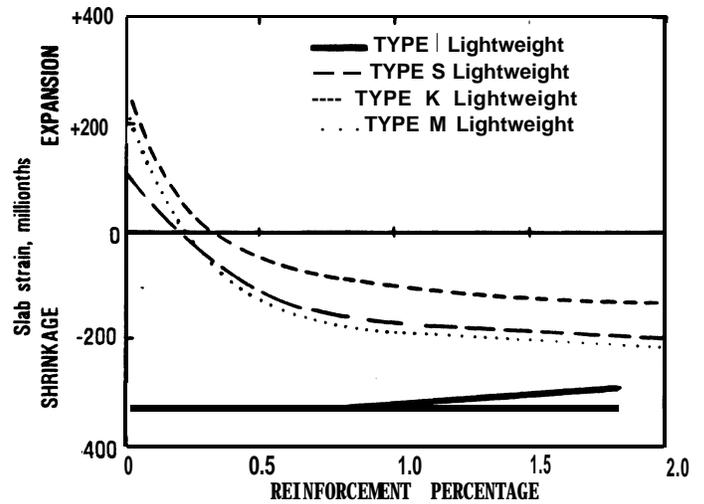


Fig. 7.1.2.2-Effect of reinforcement on shrinkage and expansion at an age of 250 days (from Reference 33)

sults when the concrete expands is predominantly a function of the subgrade restraint, reinforcement percentage, and reinforcement eccentricity. Using principles of prestressing and Fig. 7.3, the following maximum expansion can be calculated for the reinforcement percentages of Table 7.3, using concrete with 517 lb cement per cu yd and a water-cement ratio of 0.6:

Percent reinforcement	0.083	0.111	0.153
Percent expansion	0.042	0.0363	0.0293
Resulting stress in reinforcement, ksi	12.7	10.9	8.78

These stresses are not high enough to cause the reinforcement to yield, and therefore the total force in the concrete can be computed. The 0.6 w/c ratio is given for illustrative purposes only; this ratio typically is too high for shrinkage-compensating concrete slabs.

7.3.1 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 one-third of the depth from the top. Caution is needed when using smaller percentages of reinforcement because lighter gage material may be more diffi-

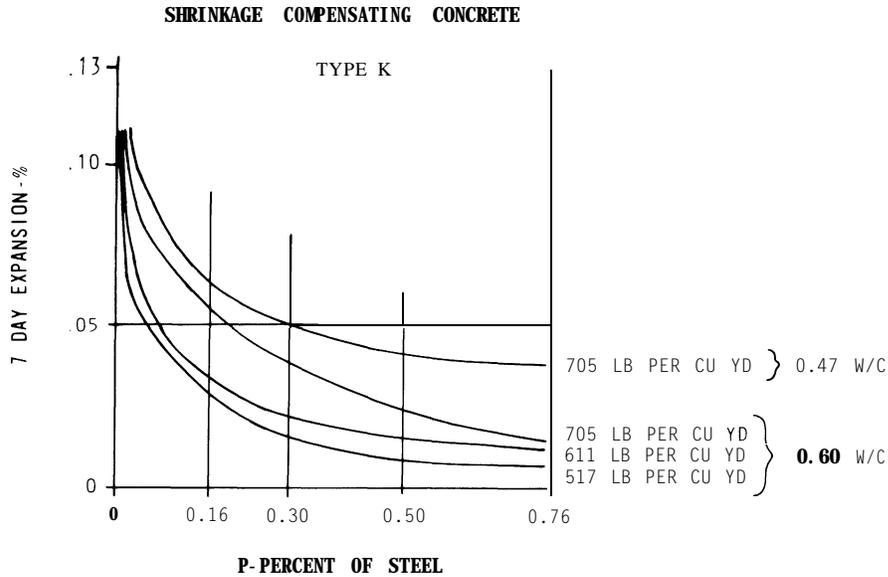


Fig. 7.3-Effect of degree of restraint on 7-day expansion (from Reference 31)

cult to position and maintain in the top portion of the slabs. Stiffer, more widely-spaced reinforcement permits lower reinforcement percentages to be used satisfactorily. This is typically achieved with ASTM A 497 deformed wire fabric or ASTM A 615 deformed bars, widely spaced. Other deformed bar reinforcement is acceptable, such as ASTM A 616, A 617, and A 706. ASTM A 185 plain wire fabric can be used if the bond and crack control for wide wire spacings are deemed adequate.

Fig. 7.3.1.1 and 7.3.1.2 show the resulting concrete stresses due to proper and improper placement of reinforcing steel. These values are taken from Table 7.3.1 for a 6-in. slab on grade with 0.08 percent steel using a 5.5-sack mix with a 0.6 w/c ratio.

In the example of Fig. 7.3.1.1, the steel is placed at ¼ the depth from the top. Stresses developed are representative of those common in practice and depend on the subgrade friction coefficient, taken here as 1.0 per unit length. It is important to note that compression is not developed on the top of the slab. The slab in Fig. 7.3.1.2 has reinforcement improperly located below the mid-depth at ¾ the depth from the top. A net tension value is developed at the top surface of the concrete. Cracking and curling are more likely in this case.

Table 7.3-Typical reinforcement for 6-in. slab on grade made with shrinkage compensating concrete (from Reference 34)

Reinforcement, A _s , sq in. per ft	12 x 12 D 6 x 6 0.06	12 x 12 D 8 x 8 0.08	12x12 D 11 x 11 0.11
Weight, lb per 100 sq ft	45	61	84
A _s /A _c	0.06/6 x 12	0.08/6 x 12	0.11/6 x 12
Percent steel	0.03	0.111	0.153

7.3.2 Effect of two layers of reinforcement-Fig. 7.3.2 shows the result of using two layers of reinforcement (one top and one bottom) with 0.15 percent reinforcement. The reinforcement is the same top and bottom, and both layers are placed 2 in. from the outer face of a 6-in. slab. Other values assumed are the same as in Section 7.3.1.

If concrete slabs made with expansive cement are designed with top and bottom reinforcement located symmetrically about the middepth, compression develops in the top and bottom of the slab due to the restrained expansion. When it shrinks the slab relieves some of the buildup precompression.

7.4-Design implications

In design applications for reinforced specimens, the flexural first-crack moment capacity for shrinkage-compensating concrete is about 15 to 20 percent higher than for portland cement concrete after drying shrinkage has occurred. This has been shown by Pfeiffer³² and confirmed by Russell.³³ Note that these higher relative strengths exist even after release of some of the precompression. The ultimate moment capacity is still the same since it is controlled by the reinforcement. Should the flexural first-crack capacity be used in the slab design, this increase in strength can be taken into account when using this type of concrete.

Table 7.3.1-Steel stresses and concrete pressures

Steel stress	13,000	10,900	8,940
Force, L, lb/ft	752	872	963
Pressure, psi	10.5	12.1	13.1

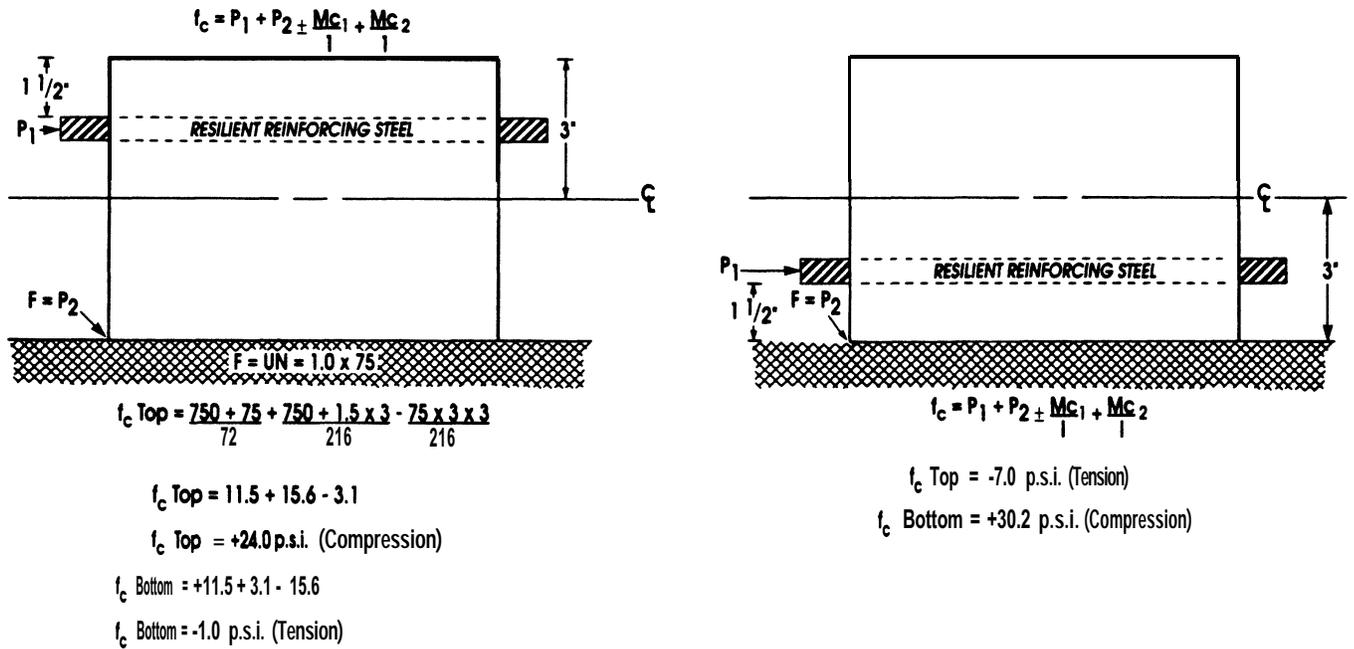


Fig. 7.3.1-Resulting stresses in 6-in. slab on grade at maximum expansion (from Reference 31). At left, reinforcement is correctly placed in top half of slab; on right, it is incorrectly placed in the bottom.

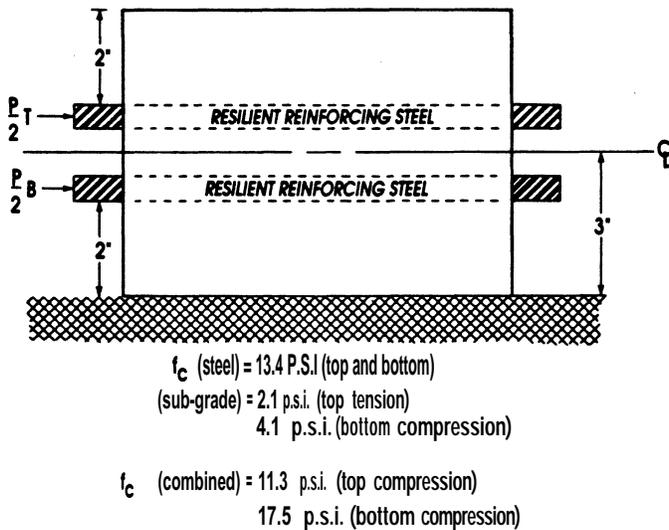


Fig. 7.3.2-Concrete stresses resulting in 6-in. slab when steel is placed in both top and bottom

7.5-Maximum and minimum reinforcement requirements

7.5.1 ACI 223 minimum recommendations-In 1977 ACI 223³¹ recommended a minimum of 0.15 percent reinforcement without testing for expansion of the concrete. This resulted from information contained in an earlier Committee 223 report³⁴ and the concept of induced compressive stress resulting from external and internal restraint against expansion. No specific attention was given to shrinkage potential as a function of the member size and shape.

Because of satisfactory applications reported with less than the above minimum,^{31, 33, 35} ACI 223 now al-

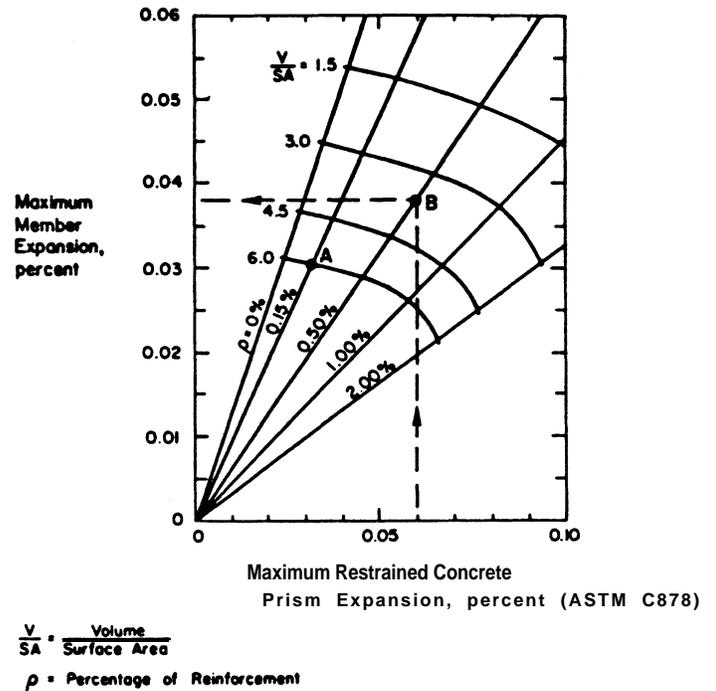


Fig. 7.5.2-Slab expansion versus prism expansion for different volume:surface ratios and reinforcement percentages (from ACI 223)

lows lower reinforcement ratios with expansion bar testing of the concrete mix design per ASTM C 878.

7.5.2 Maximum restraint levels-The objective of full shrinkage compensation is to attain restrained member expansive strains equal to or greater than the restrained shrinkage strains. Kesler cautioned³⁵ that the maximum

level of internal reinforcement should be approximately 0.6 percent, because at that point, restrained expansion strains equalled restrained shrinkage strains. To prevent concrete from shrinking more than the restrained expansion, lighter percentages of steel are recommended unless the strain capacity of mature concrete (approximately 100 μ in.) is taken into account. 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. 7.5.2. 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 which should be specified or verified in the lab with trial mixes.

7.5.3 Alternative minimum restraint levels—Russell concluded that restrained expansion should be equal to or greater than restrained shrinkage. The concrete shrinkage depends on aggregate, unit water content, and volume:surface ratios.* 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: (a) the potential shrinkage of the slab, and (b) the restrained prism expansion of the concrete mix measured according to ASTM C 878—typically 0.03 percent with concrete containing 517 lb cement per cu yd. 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 percent steel in a typical 6-in. slab can be used.³³ If the slab expansion is greater than the shrinkage strain for a surface:volume ratio of 6:1, using Russell's data (modified) from p. 225 of Reference 32, full compensation can be achieved. Circumferential curves depicting shrinkage strains for volume:surface ratios for other slab thicknesses are also shown in Fig. 7.5.2.

Care should be exercised when using low reinforcement ratios. If light mesh 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 plain wire fabric, the spacing should be not more than 14 in. longitudinally and 14 in. transversely, even though a wider spacing is easier for workers to step through. Deformed welded wire fabric can be spaced in the same manner as reinforcing bars. A gage can be inserted from the top of a slab during concrete placement to periodically check the location of the reinforcement.

If tests and design calculations are not used, then one may simply specify the minimum 0.15 percent reinforcement unless temperature conditions dictate otherwise.

7.6-Other considerations

7.6.1 Abrasion resistance—ACI 223 states that shrinkage-compensating cement concretes have approximately 30 percent higher surface abrasion resistance. Further research by the Portland Cement Association³³ has confirmed this finding.

7.6.2 Curvature benefits—Keeton³⁶ investigated portland cement concrete and shrinkage-compensating concrete slabs which were allowed to dry only from the top surface for one 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 concretes were greater at the top fibers than at the lower fibers of a slab on grade, setting up a convex profile which was the opposite of the concave profile of portland cement concrete slabs. This occurred despite having reinforcement located in the top quarter of the slab. Both reinforced and non-reinforced slabs, as well as fiber reinforced slabs, displayed this behavior.

7.6.3 Strain analysis—As drying occurs later, the resulting shrinkage strains are greater on the exposed top face than on the bottom face [Fig. 7.6.3 (B)]. This shrinkage behavior is similar to conventional concrete slabs and is represented by $S_{,,}$, the differential in strain between the top and bottom of the slab. With shrinkage even as late as one year at 20 percent relative humidity, the residual positive strains were still larger on the top surface than on the bottom portions of the slabs. Not only were slabs longer and wider than their as-cast dimensions, but the strains were larger at the top than the bottom after expansion and subsequent shrinkage [Fig. 7.6.3 (C)]. These laboratory data show reverse curling (doming) in properly installed, thick concrete grade slabs made with shrinkage-compensating concrete having an estimated ASTM C 878 restrained concrete prism value of 0.062 percent. Lower reversed curling values would be obtained with typically-used concretes having lower potential expansion values as measured by the standard prism expansion test.

Field experience indicates a lack of expected normal curling (dishing) at construction joints. This behavior is unique to shrinkage-compensating concrete in contrast to portland cement concrete. Dimensions of the latter are always smaller than their as-cast dimensions. They are

* Volume:surface ratio mathematically expresses the drying surface or surfaces in comparison to the volume of a concrete member. Slabs on grade 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. slab drying on the top surface.

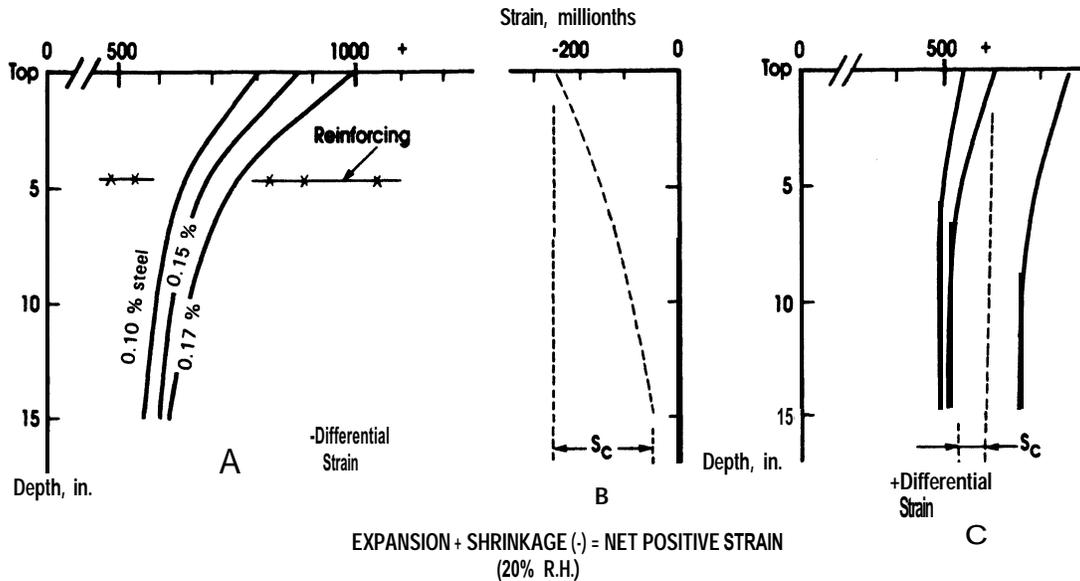


Fig. 7.6.3-Expansion and shrinkage behavior of 15-in.-thick slab on grade for reinforcement percentages of 0.10, 0.15, and 0.17 (from Reference 31)

also smaller on the top face than on the bottom face, leading to typical dishing of the as-cast plane surface. Additional curvature data are reported in two papers by Russell^{32,33} relative to single face drying and eccentric steel restraint without subgrade restraining conditions.

During the expansion phase, subgrade restraint causes the slabs to lift up at the midpoint but, because of the low modulus of elasticity and creep, the slab dead weight tends to keep them flat. Thus, subgrade restraint reduces bottom expansion strains and dead weight reduces top expansion strains.

7.6.4 Prism and slab expansion strains and stresses- Because the reinforcement percentage does vary, the ASTM C 878 restrained concrete prism test is used to verify the expansive potential of a given mix. Then Fig. 7.5.2 may 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. 7.6.4, 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.

Typical comparative strain levels at 250 days of drying for ASTM C 150 and ASTM C 845 cement concretes are shown in Fig. 7.1.2.2. Excessive reinforcement ratios provide less than full shrinkage compensation for the slab sections studied.

7.6.5 Expansion/isolation joints- Because a slab may be restrained externally on one side by a previously cast slab, the opposite side must 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 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 150 psi range at a compression of 50 percent of the original joint thickness. Polyethylene foam and expanded polystyrene are more compliant materials and will deform under the expansive strains if they have a 20 psi maximum compressive strength at 50 percent deformation, conforming to ASTM D 1621 or ASTM D 3575.

The width of the isolation joint in inches should be equal to two times the anticipated slab expansion as taken from Fig. 7.5.2, multiplied by the length of the longest dimension of the slab in inches. For a 100 x 120-ft slab with expansion strain of 0.00035:

$$\begin{aligned} \text{Joint width} &= 2 \times 120 \times 12 \times 0.00035 \\ &= 1.008 \text{ in.} \end{aligned}$$

Use 1-in.-thick joint material.

The material is then twice as thick as the deformation that is required due to expansion. This will assure adequate compressibility of the isolation joint material and also provide strain accommodation at one edge rather than distributing it between two opposite edges.

7.6.6 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 non-bonded-are covered in ACI 302 as Class 6 and Class 7 floors.

Bonded overlays are generally a minimum of $\frac{3}{4}$ in. thick, but thicknesses of 3 in. 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

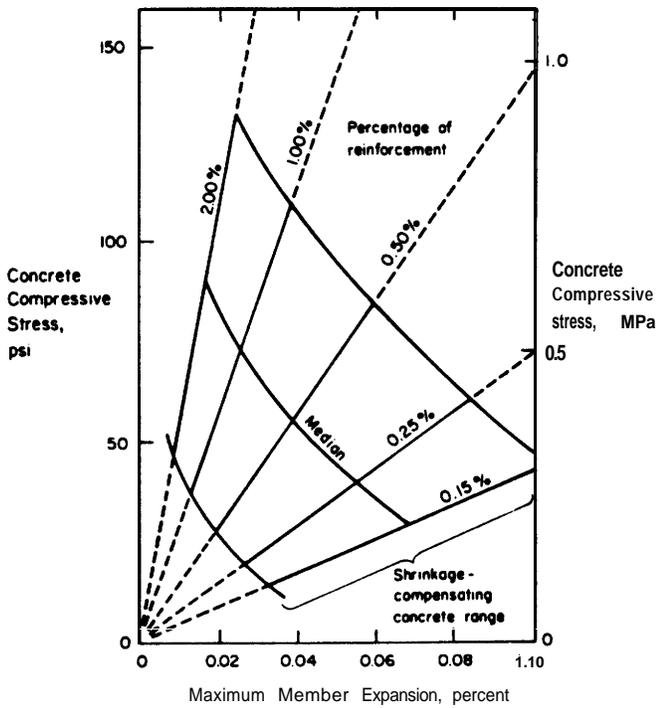


Fig 7.6.4-Calculated compressive stresses induced by expansion (from ACI 223)

as graded iron, are employed in bonded overlays to improve the abrasion resistance and impact resistance of the floor surface.

A deferred topping must contain joints to accommodate shrinkage strains. The base slab joints must be carefully continued through the topping or a crack will develop. Further, base slabs which contain cracks that must move due to slab motion will often reflect cracks into the topping. Therefore, joints in the topping should be located in the same position as the base slab cracks or joints.

If the base slab contains shrinkage-compensating concrete, the portland cement concrete bonded topping must be applied at least two weeks 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. If a well-bonded, low shrinkage topping is applied through the use of an absorption process, vacuum dewatering, low water-cement ratio (0.25 by weight), or similar method, no joints in addition to those in the base slab need be made in the topping.³²

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 8-DESIGN OF POST-TENSIONED SLABS ON GRADE

8.1-Notation

- A Area of gross concrete cross-section, in.²
- A_b Bearing area beneath a tendon anchor, in.²
- A_b' Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.²
- A_{bm} Total area of concrete in the beams, in.²
- A_C Activity ratio of clay
- A_{sl} Area of concrete in the slab, in.²
- b Width of an individual stiffening beam, in.
- c Centroid of prestressing force, in.
- $CEAc$ Cation Exchange Activity
- c_g Centroid of gross concrete section, in.
- d Depth of stiffening beam (measured from top surface of slab to bottom of beam), in.
- e Eccentricity of post-tensioning force, in.
- e_m Edge moisture variation distance, ft
- E_c Long-term or creep modulus of elasticity of concrete, psi
- E_s Modulus of elasticity of soil, psi
- f_B Section modulus factor for bottom fiber
- f_c Allowable concrete compressive stress, psi
- f_c' 28-day compressive strength of concrete, psi
- f_{ci}' Concrete compressive strength at time of stressing tendons, psi
- f_{bp} Allowable bearing stress under anchorages, psi
- f_{cr} Tensile cracking stress in concrete, psi
- f_p Minimum residual prestress or compressive stress, psi
- f_s Section modulus factor for bottom fiber
- f_T Section modulus factor for top fiber
- f_t Allowable tensile stress in concrete, psi
- g Moment of inertia factor
- I Gross moment of inertia, in.⁴
- k Depth-to-neutral-axis ratio; also kips
- L Total slab length in the direction being considered, ft
- M_{cs} Moment occurring as a result of constructing over compressible soil, ft-kips/ft
- $M_{cs\ell}$ Moment requirement in long direction for compressible soils, ft-kips/ft
- M_{css} Moment requirement in short direction for compressible soils, ft-kips/ft
- M_ℓ Design moment in the long direction, ft-kips/ft
- M_{ns} Moment occurring in the "no-swell" condition, ft-kips/ft
- M_s Design moment in the short direction, ft-kips/ft
- n Number of beams in a cross section section
- ${}_nM_t$ Negative bending moments including tension or compression in the extreme fibers, ft.-kips/ft
- ${}_pM_t$ Positive bending moments including tension or compression in the extreme fibers, ft.-kips/ft
- N_T Number of tendons
- P Perimeter loading on the slab, lb/ft
- P_r Prestressing force, kips
- PI Plasticity Index
- q_{allow} Allowable soil bearing pressure, psf

q_u	Unconfined compressive strength of the soil, psf
r_1	Area ratio
S	Beam spacing, ft
S_B	Section modulus with respect to bottom fiber, in. ³
S_T	Section modulus with respect to top fiber, in. ³
t	Slab thickness, in.
v	Design shear stress, psi
V	Design shear force, kips/ft
v_c	Permissible concrete shear stress, psi
V_{cs}	Shear force occurring as a result of constructing over compressible soil, kips/ft
$V_{cs\ell}$	Shear force occurring in long direction as a result of constructing over compressible soil, ft.-kips/ft
V_{css}	Shear force occurring in short direction as a result of constructing over compressible soil, ft.-kips/ft
V_{ns}	Shear force occurring in the "no swell" condition, kips/ft
V_s	Expected service shear in short direction, kips/ft
V_ℓ	Expected service shear in long direction, kips/ft
w	Slab width, ft
W_{slab}	Slab weight, lb
Y_m	Differential soil movement, in.
α	Angle of tendon inclination
β	Relative stiffness length, ft
Δ	Expected differential deflection of slab under service load, in.
Δ_{allow}	Allowable differential deflection of slab, in.
Δ_c	Correction to expected differential deflection due to prestressing, percent
Δ_{cs}	Differential deflection occurring as a result of constructing over compressible soil, in.
Δ_{ns}	Differential deflection occurring in the "no swell" condition, in.
Δ_o	Expected differential deflection without prestressing, in.
δ	Expected settlement occurring in compressive soil due to total load expressed as a uniform load, in.
μ	Slab subgrade friction coefficient

8.2—Definitions

Selected terms and expressions that appear in Chapter 8 are defined and explained below.

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. Addition of the ribs greatly increases the moment of inertia of the concrete cross-section, thereby increasing ability of a section to resist deflection.

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, or 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. This 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 toward the interior. The location of the maximum moment can be closely estimated by using Eq. (8-5) to calculate β , a length which depends upon 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 long or less, the amount of this reduction depends on the slab length. For slabs longer than 48 ft, the increased length does not offer further moment reduction in the mid-region of the overall slab length.

Further, the maximum shear forces develop at or near the perimeter of the slab, within one β -length from the edge of the slab.

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; i.e., the slabs experience multimodal bending.³⁷ However, it was found that all such bending 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 a tolerable amount, and this length is called the *differential deflection distance*.

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.

Differential soil movement. Y_m —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.” The differential soil movement will often be greater than the allowable deflection.

8.3-Introduction

Slabs on grade may be prestressed using unbonded tendons which 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 grade are:

- Increased joint spacings
- Decreased slab thickness

Post-tensioned concrete slabs have tremendous resilience. It is not likely that a ground-supported slab can be deflected sufficiently to exceed the yield strength of the steel. Thus, these slabs have an exceptional recovery capability.

However, there are several areas of caution for post-tensioned slabs. Anchors must have sufficient size and holding capacity, and tendons must be properly placed, stressed, and anchored. Slab penetrations made after construction must be properly located to avoid severing tendons.

Post-tensioning of ground-supported slabs was begun in the early 1960s. In March 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 prestressed residential ground-supported slab were reported.³⁷ 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 the U.S. Department of Housing and Urban Development. 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.

8.4-Applicable design procedures

8.4.1 Thickness design—The required thickness of post-tensioned slabs may be determined by the PCA, WRI, and COE methods described previously and illustrated in Appendices A1, A2, and A3. This is done by simply increasing the permissible tensile stress of the concrete by the net precompression from the prestressing

force.

8.4.2 Post-Tensioning Institute (PTI) method—In 1980, the Post-Tensioning Institute published a **manual**¹¹ containing recommendations for establishing the strength requirements for any reinforced concrete slab on either stable, expansive, or compressible soils. These strength requirements are applicable to either nonprestressed or prestressed reinforcement, or a combination of the two.

The PTI design procedure capitalizes on the unique advantages of post-tensioning as the reinforcing 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.

The following sections present PTI equations for 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 of the results of 768 separate analyses which represented full consideration of both center lift and edge lift conditions using a finite element plate-on-elastic-half-space foundation.³⁸ 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 which developed the design equations.^{11,39}

8.5-Data needed for design of reinforced slabs

8.5.1 Soil properties—The designer must have the following information about soil properties:

- Allowable soil bearing pressure, q_{allow} , psf
- Edge moisture variation distance, e_m , ft
- Differential soil movement, Y_m , in.
- Slab-subgrade friction coefficient, μ

8.5.2 Structural data and materials properties—The designer must know the slab length L , spacing S of the stiffening beams, and the beam depth d from top of slab to bottom of beam. The perimeter loading P must also be known. Materials properties required are:

- Specified 28-day compressive strength of concrete, f'_c
- Type, grade, and strength of the prestressing steel
- Type and grade of nonprestressed reinforcement, if needed
- Prestress losses

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

Allowable tensile stress:

$$f_t = 6\sqrt{f'_c}$$

Allowable compressive stress:

$$f_c = 0.45 f'_c$$

Estimated tensile cracking stress:

$$f_{cr} = 7.5 \sqrt{f'_c}$$

Permissible shear stress:

$$v_c = 1.5 \sqrt{f'_c}$$

Design shear stress:

$$v = \frac{VW}{ndb}$$

(use only beam cross-sections in calculating shear resistance)

Terms for shear stress are those given in the PTI method. Permissible shear stress is the limiting unit shear stress allowed in the concrete. The design shear stress is the unit shear stress that would exist in the concrete under application of the (unfactored) design load.

8.6—Design for slabs on expansive soils

The equations presented will determine the moment, deflection, and shear requirements for slabs cast on expansive soils. Eq. (8-1) through (8-4) determine the strength requirements; Eq. (8-5) through (8-9) determine the deflection requirements; and Eq. (8-13), (8-14), and (8-15) determine the shear requirements. Slabs designed by the PTI method must meet these requirements. The designer may select either nonprestressed reinforcement, post-tensioned reinforcement, or a combination of both to meet the strength requirements. Appendix A4 presents a design example.

8.6.1 Moments

Center lift design moment in long direction (strength requirement of the section across the long direction) is given by Eq (8-1):

$$M_l = A_o [B(e_m)^{1.238} + C] \text{ ft-kips/ft} \quad (8-1)$$

where:

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

and for: $0 \leq e_m \leq 5$, $B = 1$ and $C = 0$ and for: $e_m > 5$

$$B = \frac{Y_m - 1}{3} \leq 1.0$$

$$C = \left[8 - \frac{P - 613}{255} \right] \left[\frac{4 - Y_m}{3} \right] \geq 0$$

Center lift design moment in short direction (strength requirement of the section across the short direction) is given by Eq. (8-2):

$$M_s = \left[\frac{58 + e_m}{60} \right] M_l \text{ ft-kips/ft} \quad (8-2)$$

Edge lift design moment in long direction (strength requirement of the section across the long direction) is given by Eq. (8-3):

$$M_l = \frac{S^{0.10} (d e_m)^{0.78} Y_m^{0.66}}{7.2 L^{0.0065} P^{0.064}} \quad (8-3)$$

Eq. (8-4) gives edge lift design moment in short direction (strength requirement of the section across the short direction):

$$M_s = d^{0.35} \left[\frac{19 + e_m}{57.75} \right] M_l \quad (8-4)$$

8.6.2 Differential deflection—Determine allowable and expected deflections from actual section properties. Then calculate the relative stiffness distance β for both long and short direction using Eq. (8-5):

$$\beta = \frac{1}{12} \sqrt[4]{\frac{E_c I}{E_s}} \quad (8-5)$$

Differential deflection distance: Use either L or 6β , whichever is shorter in determining allowable deflections.

Use Eq. (8-6) to obtain allowable differential deflections for center lift, long and short directions:

$$\Delta_{allow} = \frac{12 (L \text{ or } 6 \beta)}{360 * } \quad (8-6)$$

Allowable differential deflection for edge lift, long and short directions is given by Eq. (8-7):

$$\Delta_{allow} = \frac{12 (L \text{ or } 6 \beta)}{360 ** } \quad (8-7)$$

* The design engineer may wish to use 480 or larger, depending on the superstructure resilience and/or requirements.

** The designer may wish to use 480 or larger, depending on the superstructure resilience and/or requirements. Use 800 if the beam stems are unreinforced.

Expected differential deflection without prestressing, center lift, long and short directions, from Eq. (8-8):

$$\Delta_o = \frac{(Y_m L)^{0.205} S^{1.059} P^{0.523} (e_m)^{1.296}}{380 d^{1.214}} \quad (8-8)$$

Expected differential deflection without prestressing, edge lift, long and short directions:

$$\Delta_o = \frac{L^{0.35} S^{0.88} e_m^{0.74} Y_m^{0.76}}{15.90 d^{0.85} P^{0.01}} \quad (8-9)$$

For deflection reduction in center lift condition and addition in edge lift condition due to eccentricity of prestressing, use Eq. (8-10) to get percent of differential deflection reduction or addition:

$$\Delta_c = e \sqrt{\frac{6400}{9L}} \quad (8-10)$$

NOTE: e is a positive number when above the neutral axis and a negative number when below the neutral axis.

Corrected differential deflection considering prestressing:

Center lift:

$$\Delta = \Delta_o \left[\frac{100 - \Delta_c}{100} \right] \quad (8-11)$$

Edge lift:

$$\Delta = \Delta_o \left[\frac{100 + \Delta_c}{100} \right] \quad (8-12)$$

8.6.3 Shear

Expected service shear per foot of structure: Center lift condition, short direction Eq. (8-13):

$$V_s = \frac{1}{1350} L^{0.19} S^{0.45} d^{0.20} P^{0.54} Y_m^{0.04} e_m^{0.97} \quad (8-13)$$

Center lift condition, long direction Eq. (8-14):

$$V_t = \frac{1}{1940} L^{0.09} S^{0.71} d^{0.43} P^{0.44} Y_m^{0.16} e_m^{0.93} \quad (8-14)$$

Edge lift condition, long and short direction Eq. (8-15):

$$V = \frac{L^{0.07} d^{0.40} P^{0.03} e_m^{0.16} Y_m^{0.67}}{3.0 S^{0.015}} \quad (8-15)$$

8.7—Design for slabs on compressible soil

Compressible soils are those soils whose allowable bearing capacity is 1500 psf 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 nonprestressed or prestressed reinforcement, or a combination of both to meet these strength requirements. Eq. (8-16) and (8-17) determine the bending strength requirements; Eq. (8-18) determines the deflection requirement; and Eq. (8-19) and (8-20) determine the shear requirements for design using either prestressed or nonprestressed reinforcement.

8.7.1 Moments

Long direction (ℓ):

$$M_{cs\ell} = \left[\frac{\delta}{\Delta_{ns}} \right]^{0.50} M_{ns} \quad (8-16)$$

where

$$M_{ns} = \frac{D^{1.35} S^{0.36}}{80 L^{0.12} P^{0.10}}$$

and

$$\Delta_{ns} = \frac{L^{1.28} S^{0.80}}{133 d^{0.28} P^{0.62}}$$

Short direction, s :

$$M_{css} = \left[\frac{970 - d}{880} \right] M_{cs\ell} \quad (8-17)$$

8.7.2 Anticipated differential deflection

$$\Delta_{cs} = \delta \exp(Z) \quad (8-18)$$

where

$$Z = 1.78 - 0.013 d - 1.65 \times 10^{-3} P + 3.95 \times 10^{-7} P^2$$

and $\exp(Z)$ = natural base e raised to the power Z
 $= e^Z$

8.7.3 Shear

Long direction:

$$V_{cst} = \left[\frac{\delta}{\Delta_{ns}} \right]^{0.30} V_{ns} \quad (8-19)$$

where

$$V_{ns} = \frac{d^{0.90} (PS)^{0.30}}{550 L^{0.10}}$$

Short direction

$$V_{ns} \left[\frac{116-d}{94} \right] V_{cst} \quad (8-20)$$

8.8-Maximum spacing of post-tensioning tendons in normal weight concrete

Tendon spacing S_{ten} slab-sub-

$$S_{ten} = \frac{\text{effective force, lb per tendon}}{(50 \times 12 t) + \left(12.5 t \times \frac{L}{2} \times \mu \right)} \quad (8-21)$$

PTI recommends⁴⁰ the following coefficients of friction μ for slabs constructed on polyethylene sheeting:

Slabs of uniform thickness: 0.50 - 0.60

Ribbed or stiffened slabs: 0.75

For slabs constructed on a sand base, the recommended coefficient of friction is:

Slabs of uniform thickness: 1.00

Ribbed or stiffened slabs: 1.25

Appendix A4 provides an example of tendon selection, along with the necessary tables.

CHAPTER 9-REDUCING THE EFFECTS OF SLAB SHRINKAGE AND CURLING

9.1-Introduction

This chapter covers the design methods used to reduce the effect of drying shrinkage and curling (warping) in slabs on grade. The material is largely based on Ytterberg's three articles, "Shrinkage and Curling of Slabs on Grade."⁴¹ For additional information on concrete shrinkage, reference should be made to ACI 209R as well as to the 54 references provided by Ytterberg.

To be workable enough to be placed, virtually all concrete is produced with about twice as much water as is needed to hydrate the cement. Because water can only evaporate from the upper surface of slabs on grade, uncombined water creates moisture gradients 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 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. Moisture testing of subgrades and shrinkage testing of concrete must 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.

Curling of slabs on grade has become more prevalent in the past 25 years. This is partly due to the emergence of more finely ground cements and the use of smaller maximum size coarse aggregates, both of which increase the water demand in concrete. The problem may also be compounded by increases in the specified compressive strength because such strength increases are usually achieved by increasing the total volume of water and cement per cubic yard even though the water-cement ratio may be reduced. For the slabs on grade, the commonly specified 28-day compressive strength of 3000 psi in years past has been increased to as much as 5000 psi to permit reduction of calculated slab thickness. The higher strengths can improve durability; however, designers should look at alternatives to high 28-day compressive strength in their quest 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 been increased, nor have well-designed vapor barriers been specified to offset this subgrade moisture increase because 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 grade and thereby increases slab curling.

Designers can take steps to reduce shrinkage cracking and curling through appropriate design and specification provisions regarding relative shrinkage of various concrete mixes, type and location of reinforcement, subgrade friction, smoothness and permeability, slab thickness, shrinkage restraints, location of contraction joints, and by properly designing vapor barriers.

9.2-Drying and thermal shrinkage

All portland cement concrete, along with shrinkage-compensating concrete, shrinks about 0.04 to 0.08 percent due to drying,⁴² but when drying shrinkage is restrained by properly placed reinforcement, the shrinkage strain can be reduced by 10 to 15 percent. For slabs on grade the shrinkage restraint from the subgrade varies with the coefficient of friction 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 much different temperature than from final operating temperature. Thermal contraction can be calculated by using the concrete's coefficient of thermal expansion of 5.5μ in. per deg F. For example, lowering the temperature of a floor slab from 70 F to 0 F will shorten a 100-ft slab by 0.46 in.,

assuming no subgrade drag restraint.

9.3-Curling and warping

Curling of concrete slabs at joints 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 due to creep or 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 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 grade 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. Curling occurs for a distance of 2 to 5 feet from all “free” slab edges. Fig. 9.3.1 and 9.3.2 show the curling effect in exaggerated fashion.

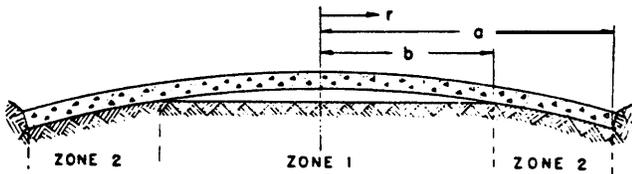


Fig. 9.3.1-Highway slab edges curl downward at edges during the day when the sun warms the top of the slab

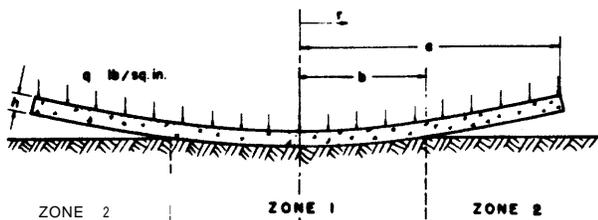


Fig. 9.3.2-Slabs indoors curl upward because of the moisture differential between top and bottom of the slab.

9.4-Factors that affect shrinkage and curling

Drying shrinkage and curling can be reduced by reducing the total water content (not necessarily the water-cement ratio) in concrete. Tremper and Spellman⁴³ found that drying shrinkage is the product, not merely

the summation, of eight individual factors which control the water requirements of concrete (Table 9.4). The table shows the cumulative effect of these eight 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 below.

9.4.1 Effect of maximum size of coarse aggregate- Table 9.4 shows that use of ¾-in. maximum size aggregate (MSA) under conditions where 1½-in. maximum size aggregate could have been used will increase concrete shrinkage about 25 percent because of the greater water demand of ¾-in. MSA as compared with 1½-in. MSA. Besides 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.

Table 9.4-Cumulative effect of adverse factors on concrete shrinkage⁴³

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, whereas with reasonable precautions temperatures of 60 F could have been maintained	8	1.00 x 1.08 = 1.08
Used in 6 to 7-in. slump where 3-4 in. could have been used	10	1.08 x 1.10 = 1.19
Excessive haul in transit mixer, too long a waiting period at job site, or too many revolutions at mixing speed	10	1.19 x 1.10 = 1.64
Use of ¾ in. maximum size aggregate under conditions where 1½ in. could have been used	25	1.31 x 1.25 = 1.64
Use of cement having relatively high shrinkage characteristics	25	1.64 x 1.25 = 2.05
Excessive “dirt” in aggregate due to insufficient washing or contamination during handling	25	2.05 x 1.25 = 2.56
Use of aggregates of poor inherent quality with respect to shrinkage	50	2.56 x 1.50 = 3.84
Use of admixture that produces high shrinkage	30	3.84 x 1.30 = 5.00
Total Increase	Summation 183%	Cumulative 400%

In actual practice, the dry-rodded volume of the coarse aggregate is about ½ to ⅔ of the concrete volume if 1/2-in. MSA is used, but can be as high as ¾ if 1½-in. MSA is used (ACI 211.1, Table 5.3.6). Use of large size

coarse aggregates may be more expensive than smaller size aggregates, but it can save on cement content. Designers must specify the nominal top size coarse aggregate if a larger size is desired.

9.4.2 Influence of cement-Table 9.4 shows the possibility of a 25 percent 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. Designers rarely specify the type of cement to be used for slabs on grade. Type I and III cements, however, can cause higher concrete shrinkage than Type II cement because of their higher water demands. Thus, specifying minimum concrete compressive strength without regard to either cement type or relative cement mortar shrinkage can contribute to slab shrinkage and curling.

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

9.4.3 Influence of slump-Instead of expecting slump to control shrinkage, designers should effect real shrinkage reduction by specifying low shrinkage, stony concrete mixes with large maximum size coarse aggregate.

Table 9.4 shows that a 6- to 7-in. slump concrete will have only 10 percent more shrinkage than a 3- to 4-in. slump concrete. 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 have to be controlled to have a satisfactory slab with regard to shrinkage in the hardened state.

9.4.4 Influence of water reducing admixtures-Water reductions of approximately 7 percent may be achieved with ASTM C 494 Type A water reducing admixtures, but their effect on shrinkage and curling is minimal. However, Tremper and Spellman and others have found that chloride-based admixtures of this type definitely increase shortening of the concrete.

Some water reducing admixtures increase concrete shrinkage, even at reduced mixing water contents, as shown by numerous investigators cited in Reference 41. It cannot be stated that a reduction in mixing water content permitted by use of water reducers will always decrease shrinkage proportionally. In many cases, shrinkage is not changed much by the introduction of a water-reducing or high-range water-reducing admixture (Types A and F, ASTM C 494) or by a nominal change of slump from 5 in. down to 3 in. Designers should note that ASTM C 494 allows concrete made with admixtures to have 35 percent greater shrinkage than the same concrete without the admixture.

9.5-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 an increase in shrinkage, because more cement and more water per cubic yard (not necessarily a higher water-cement ratio), 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 assure that slab thickness is sufficient to transmit loads to the subgrade. A 60-day, 90-day, or longer strength, rather than a 28-day strength, should be considered for designing slab thickness. This assumes that the design loads will 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. As only one example, quadrupling the slab contact area of base plates beneath post loads (8 x 8-in. plates instead of 4 x 4-in. plates) could decrease the required slab thickness by more than 1 in.

9.6-Compressive strength and abrasion resistance

Abrasion resistance is a function of the water-cement ratio (compressive strength) at the top surface of the concrete. The 6 x 12-in. cylinder tested to measure compressive strength is not a measure of this slab surface strength.

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. Reference 44 reports 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 wear and abuse on the floor, 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 water-cement ratio can be used for best abrasion resistance.

Slab shrinkage can be substantially reduced by vacuum dewatering. An additional benefit of vacuum dewatering is substantially increased abrasion resistance.⁴⁴ Vacuum dewatered concrete has its lowest water-cement ratio in the upper half rather than in the lower half, as is the case for conventional floor construction.

9.7-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 the floor must be jointed no more than 10 to 15 ft from the wall so that the remainder of the floor is free to shrink and move.

In most slabs on grade, it is desirable to try 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. of subgrade 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 columns and column footings, the slab should be isolated from guard posts 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 around all restraints to allow the slabs to shrink and move relative to the fixed items. Electrical conduit and storm drain lines must be buried in the subgrade so that they do not either reduce the slab thickness or restrain drying shrinkage.

9.8-Subgrade and vapor barriers

A permeable subgrade, with a smooth, low-friction surface helps reduce shrinkage cracking because it allows the slab to shrink with minimal restraint. It also allows some of the water from the bottom of the slab to leave before the concrete sets. Vapor barriers in direct contact with the slab are discouraged because they increase slab curling as explained below.

Vapor barrier design should receive the same attention as the design of a roof membrane. The barrier should be covered with at least 3 in. of fine granular material to provide a permeable, absorptive base directly under the slab. However, using 6 in. or more of this material over the barrier will improve constructibility and minimize damage. Nicholson⁴⁵⁵ showed that serious shrinkage cracking and curling can occur when concrete slabs are cast on an impervious base. If the subgrade is kept moist by groundwater or if the slab is placed on a wet subgrade then this will increase the moisture gradient in the slab and will increase upward curl. If crushed stone is used as a subgrade material, the upper surface of the crushed stone should be choked off with sand or a smaller crushed stone material to provide a smooth surface that will allow the slab on grade to shrink with minimum restraint.

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

9.9-Distributed reinforcement to reduce curling and number of joints

Since 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

Table 9.9-PCA's suggested spacing (in feet) of contraction joints for plain slabs on grade (from Reference 8)

Slab thickness, in.	Slump 4 to 6 inches		Slump less than 4 in.
	Aggregate < ¾ in.	Aggregate > ¾ in.	
5 in.	10	13	15
6 in.	12	15	18
7 in.	14	18	21
8 in.	16	20	24
9 in.	18	23	27
10 in.	20	24	30

shrinkage of the concrete. 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. In order to avoid being pushed down by the feet of construction workers, reinforcing wire or bars should preferably be spaced a minimum of 14 in. in each direction. The deformed wire or bar should have a minimum diameter of ¾ in. to provide sufficient stiffness to prevent bending during concreting.

If greater joint spacings than those recommended by PCA (Table 9.9) for plain slabs are desirable, it is worthwhile to pay the extra cost and specify distributed steel as a means to reduce the number of joints in slabs on grade. One should bear in mind that the specified steel must be stiff enough and have a great 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.

Since joints are a maintenance problem, it may be desirable to reduce the number of joints by limiting the shrinkage cracking to acceptable crack widths through the use of distributed reinforcement in the slab. The primary purpose of reinforcement in slabs on grade is to hold tightly closed any cracks that may occur between joints. The subgrade drag formula (Chapter 6) can be used to calculate the distance between free joints that can be tolerated with various percentages of steel.

9.10-Thickened edges to reduce curling

Curling is greatest at corners of slabs, and corner curling is reduced as slab thickness increases. For example, corner curling vertical deflections of 0.05 and 0.11 in. were measured for 8- and 6- in.-thick slabs, respectively, after 15 days of surface drying.

Edge curling can be reduced by thickening slab edges at floor construction joints. 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. It is recommended that free slab edges at construction joints be thickened 50 percent with a gradual 1 in 20 slope. Providing the subgrade is smooth with a low coefficient of friction as detailed in Sec. 9.8, then thickened edges should not be a crack-producing restraint.

9.11-Relation between curing and curling

Since curling and drying shrinkage are both a function of potentially free water in the concrete at the time of set of the concrete, curing methods which retain water in the concrete will delay the shrinkage and curling of enclosed slabs on grade.

Childs and Kapernick⁴⁶ 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. It was found that 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.

It is important for the designer to note that all curing methods (water, paper, or sealers) 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.

9.12-Warping stresses in relation to joint spacing

Several sources^{47,48} 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 from the corner diagonally inward. Critical lengths in feet are shown below for slabs 4 to 10 in. thick and temperature gradients T of 20, 30, and 40 degrees F.

Slab thickness	T=20	T=30	T=40
4 in.	21	--	--
6 in.	26	27	--
8 in.	--	34	35
10 in.	--	38	40

A modulus of subgrade reaction k of 100 and $E_c = 3 \times 10^6$ were used in determining these values.

Computer studies indicate that these lengths increase mostly with slab thickness and with the moisture gradient and only slightly with changes in modulus of elasticity and modulus of subgrade reaction. Fig. 9.12⁴⁹ shows both deformation and warping stress curves for three highway slabs with lengths less than, equal to, and greater than the critical slab length. Note that warping stress does not increase as slab length increases beyond the critical length because vertical deformation does not increase.

PCA8 states that there will be a marked loss of effectiveness of aggregate interlock at contraction joints if the joints are too far apart. This statement refers to Type A slabs (plain) and does not mention curling. Table 9.9 gives PCA recommendations for spacings of contraction joints in plain slabs. The Type A or B slab on grade may be more economical if contraction joint spacing is in-

Deformation: upper side warmer

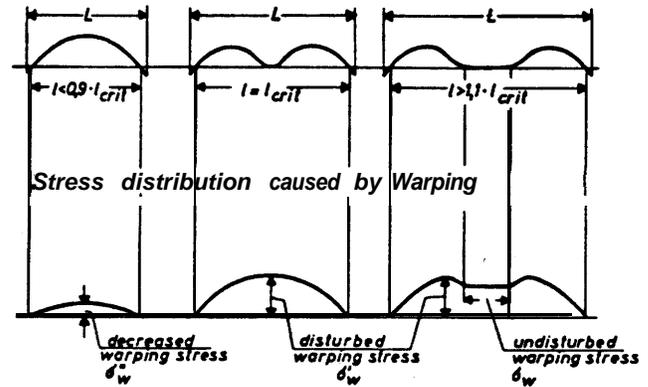


Fig. 9.12-Effect of slab length on warping and warping stress in an exposed highway slab (from Reference 49)

creased beyond lengths recommended by PCA by using distributed reinforcement as computed from the subgrade drag formula (Chapter 6), but not less than 0.15 percent of the cross-section area. Lowest floor and fork lift truck maintenance cost may well be achieved with the least number and length of joints. Increased joint spacings larger than the critical slab length will not increase warping stresses.

9.13-Warping stresses and deformation

Using the concept of a subgrade reaction modulus, Westergaard⁵⁰ provided equations for warping stress and edge deflections caused by temperature gradients in slabs on ground. Although his paper does not refer to moisture gradients, it is equally applicable to either temperature or moisture gradients across the thickness of a slab on grade. The only shortcoming is the assumption that slabs on ground would everywhere be 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 everywhere supported by the subgrade, and unsupported edges suffer higher stresses than if they were supported.

In 1938, Bradbury⁵¹ extended Westergaard's work with a working stress formula referred to as the Westergaard-Bradbury formula. Ironically, this formula is still in use today and is included in such a respected publication as Reference 52. Kelley⁴⁷ in 1939 used the Westergaard-Bradbury formula to calculate the warping stresses shown in Fig. 9.13.1 for 6-in. and 9-in. slabs on grade. Note that Kelley calculated a maximum stress of about 390 psi for a 9-in. slab with a length of 24 ft.

In 1959, Leonards and Hari⁴⁸ calculated the warping stresses shown in Fig. 9.13.2, presented here for general understanding. The upper center set of curves in Fig. 9.13.2 shows a maximum warping stress of about 560 psi for almost the same assumptions made by Kelley when he computed a stress of 390 psi. The only significant difference is that Kelley used a 27-deg F change in tem-

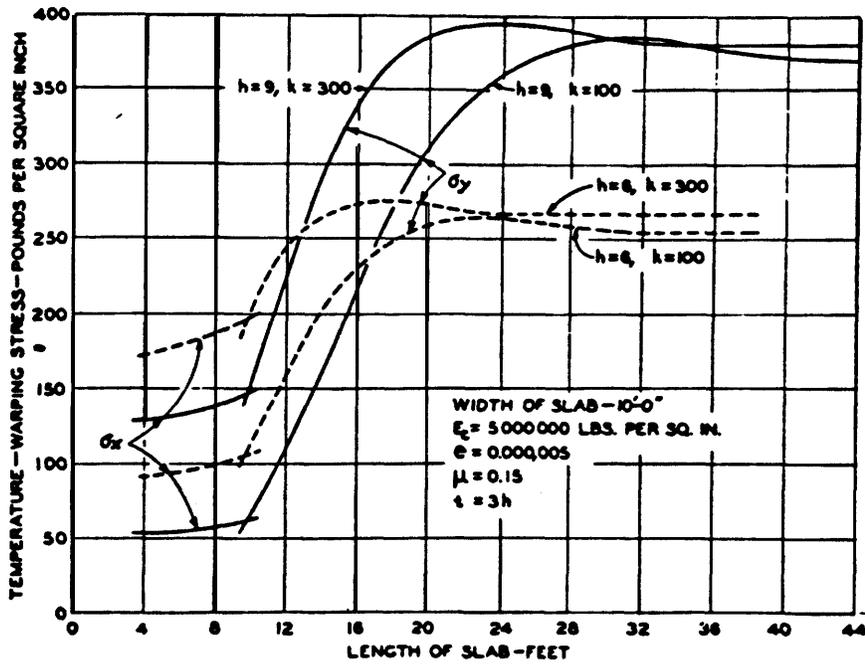


Fig. 9.13.1-Slab length increases beyond a certain amount do not increase warping stress in the slab interior (from Reference 47)

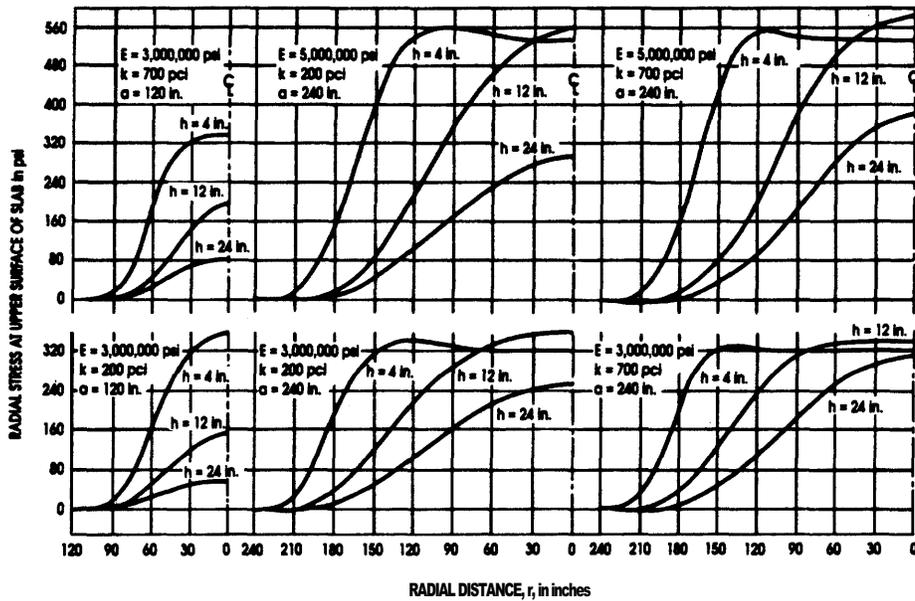


Fig. 9.13.2-Representative radial stresses for an effective temperature difference of 30 F between top and bottom (Reference 48)

temperature across the slab while Leonards and Harr used a 30-deg F temperature difference across the slab thickness. Adjusting for this gradient difference, Kelley's stress would be 433 psi instead of 390 psi ($390 \times 30/27 = 433$), but Leonards and Harr's 560-psi stress is still 29 percent greater than the stress Kelley calculated.

Leonards and Harr 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. Fig. 9.13.3 shows their vertical deflection

curves for the same six cases of slabs whose warping stress were shown in Fig. 9.13.2. The upward slab edge lift and downward slab center deflection shown in Fig. 9.13.3 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-deg F per inch of slab thickness temperature gradient for such slabs under roof. Leonards and Harr assumed a 30-deg gradient across all the slabs shown in Fig. 9.13.2 and 9.13.3 no matter what the thickness. They

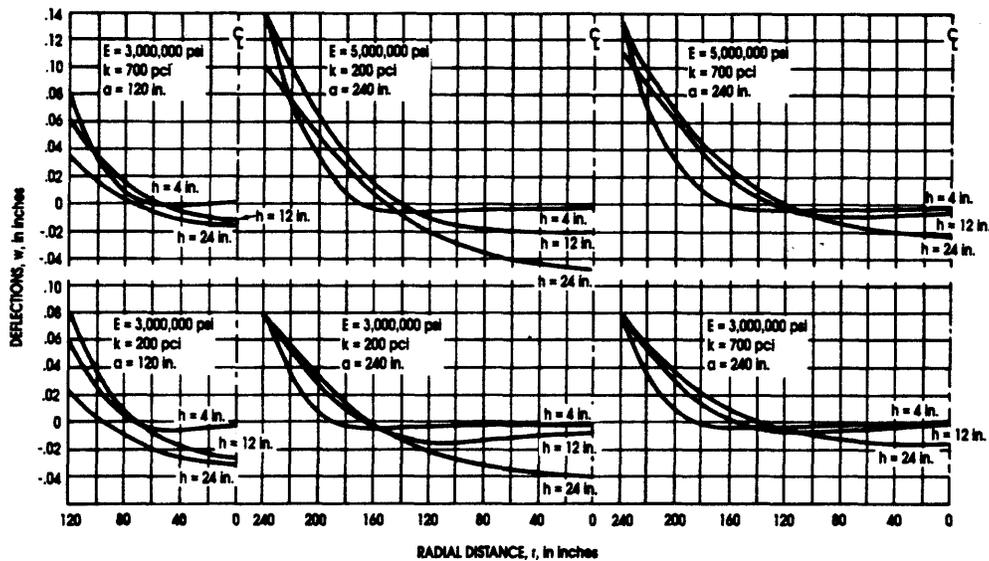


Fig. 9.13.3-Representative curling deflection curves for 20- and 40-ft slabs with an effective temperature difference of 30 F between top and bottom (from Reference 48)

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 unsupported slab centers is documented in Ytterberg's 1987 paper.⁴¹ Since 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 of slab panels on grade should probably 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 k increases, when the fact is 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.

ACI 325 recommends that highway slabs on grade be designed for a 3 deg F per in. daytime positive gradient (downward curl) and a 1 deg F per in. nighttime negative gradient (upward curl). Enclosed slabs on grade should be designed for a negative gradient (upward curl) of 3 to 6 deg F per in. according to Leonards and Harr.

The Westergaard/Bradbury formula⁵² concluded that warping stress in slabs is proportional to the modulus of elasticity of concrete, and mostly 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 like granite and especially

traprock.

9.14-Effect of eliminating 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 contraction joints. The construction joints surrounding 10,000 to 12,000 sq ft 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 contraction joints within the latter slabs will take up most of the shrinkage. Post-tensioned and shrinkage-compensated slabs do not have intermediate contraction joints. Where vehicle traffic will cross construction joints in post-tensioned or shrinkage-compensated slabs *on* ground, the top edges of the construction joints should be protected with back-to-back steel angles or by other equally durable material.

9.15-Summary and conclusions

Designers of enclosed slabs on grade 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 subgrade. Measure subgrade moisture content.
- If there is a high water table or wet subgrade, carefully design the vapor barrier and any protective cover over the vapor barrier.

DESIGN DETAILS

- Calculate the slab thickness, and consider thickening

of slab edges in terms of load-carrying ability and slab restraint.

- Specify distributed reinforcement in the upper half of the slab to minimize contraction joints. Shrinkage reinforcement is not needed in the bottom half of slabs on grade.
- In selecting reinforcement, select practical spacings and diameters of wires and bars, considering at least 14-in. spacing and 3/8-in. diameter.
- Eliminate as many slab restraints as possible. 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.
- Consider vacuum de-watering, shrinkage-compensating concrete, or post-tensioning as design options.

CONTROL OF THE CONCRETE MIX

- Specify workable concrete with the largest practical maximum size of coarse aggregate.
- 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 use of concrete with lower shrinkage than could be obtained with the same compressive strength at 28 days.
- Prior to slab installation, consider shrinkage testing of various cements (mortars) and concrete mixes.
- Specify the cement type and brand. During slab installation, consider daily shrinkage testing of concrete to assure consistency.
- Consider a daily check of aggregate gradation to assure uniform water demand and shrinkage of concrete.

CHAPTER 10-REFERENCES

10.1-Recommended references

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

American Concrete Institute

116R Cement and Concrete Terminology
 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](#)
 318 Building Code Requirements for Reinforced Concrete
 318.1 Building Code Requirements for Structural Plain Concrete
 325.1R Design of Concrete Overlays for Pavements
 325.3R Guide for Design of Foundations and

Shoulders for Concrete Pavements
 330R Guide for Design and Construction of Concrete Parking Lots
 332R Guide to Residential Cast-in-Place Concrete Construction
 336.2R Suggested Design Procedure for Combined Footings and Mats
 544.4R Design Considerations for Steel Fiber Reinforced Concrete

American Society for Testing and Materials

A 421 Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete
 A 497 Standard Specification for Steel Welded Wire Fabric, Deformed, for Concrete Reinforcement
 A 615 Standard Specification for Deformed and Plain Billet Steel Bars for Concrete Reinforcement
 C 127 Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate
 C 150 Standard Specification for Portland Cement
 C 157 Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete
 C 494 Standard Specification for Chemical Admixtures for Concrete
 C 595 Standard Specification for Blended Hydraulic Cements
 C 845 Standard Specification for Expansive Hydraulic Cement
 C 878 Standard Test Method for Restrained Expansion of Shrinkage-Compensating Concrete
 D421 Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants
 D 422 Standard Method for Particle Size Analysis of Soils
 D 427 Standard Test Method for Shrinkage Factors of Soils
 D 698 Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-lb. Rammer and 12-in. Drop
 D 854 Standard Test Method for Specific Gravity of Soils
 D 1196 Standard Method for Nonrepetitive Static Plate Load Test of Soils and Flexible Pavement Components for Use in Design and Evaluation of Airport Pavements
 D 1556 Standard Test Method for Density of Soil in Place by the Sand Cone Method
 D 1557 Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-lb. Rammer and 18-in. Drop
 D 1621 Standard Test Method for Compressive Prop-

- erties of Rigid Cellular Plastics
- D 1751 Standard Specification for Preformed Expansion Joint Fillers for Concrete Paving and Structural Construction (Nonextruding and Resilient Types)
- D 1752 Standard Specification for Preformed Sponge Rubber and Cork Expansion Joint Fillers for Concrete Paving and Structural Construction
- D 2216 Standard Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures
- D 2487 Classification of Soils for Engineering Purposes
- D 2922 Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)
- D 3575 Standard Test Methods for Flexible Cellular Materials Made from Olefin Plastic
- D 4318 Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

The above publications may be obtained from the following organizations:

American Concrete Institute
P.O. Box 19150
Detroit, MI 48219

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

10.2-Cited references

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APPENDIX

CHAPTER A1-DESIGN EXAMPLES USING THE PCA METHOD

A1.1-Introduction

The following two examples show the determination of thickness for a slab-on-grade using design charts published by The Portland Cement Association in References 8 and 14. Both examples select the thickness based on limiting the tension on the bottom of the slab.

A1.2-PCA thickness design for a single axle load

This procedure selects the thickness of a concrete slab for a single axle loading with single wheels at each end. Use of the design chart, [Fig. A1.2.1](#), is illustrated by assuming the following:

Loading: Axle load = 22.4 kips

Effective contact area of one wheel = 25 sq. in.
 Wheel spacing = 40 in.
 Subgrade modulus $k = 200$ pci

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 = $335/22.4$
 $= 4.96 = 15$

Solution: Thickness = 7 3/4 in., as determined from Fig. A1.2.1.

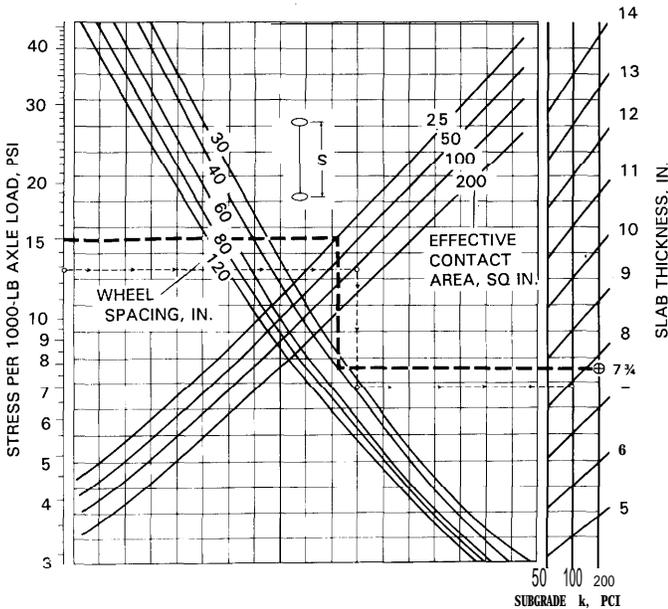


Fig. A.1.2.1-PCA design chart for axles with single wheels

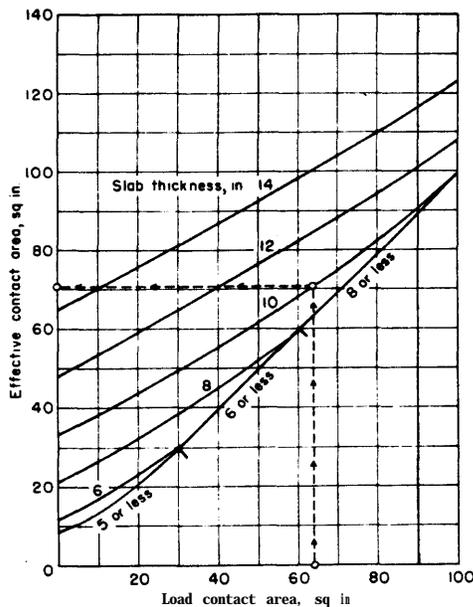


Fig. A1.2.2-Relationship between load contact area and effective load contact area

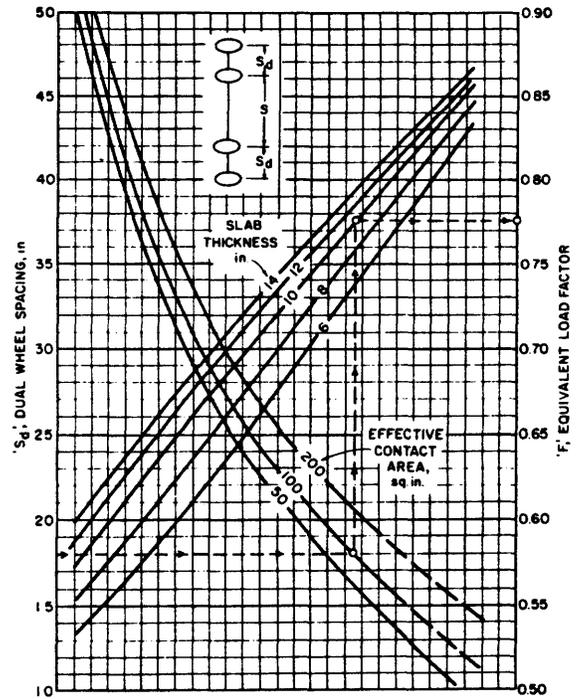


Fig. A1.2.3-PCA design chart for axles with dual wheels

Fig. A1.2.2 and A.1.2.3 are also included for determining the effective load contact 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. A1.3.1, such as from rack storage supports. The use of the design chart, Fig. A1.3.2, is illustrated by assuming the following:

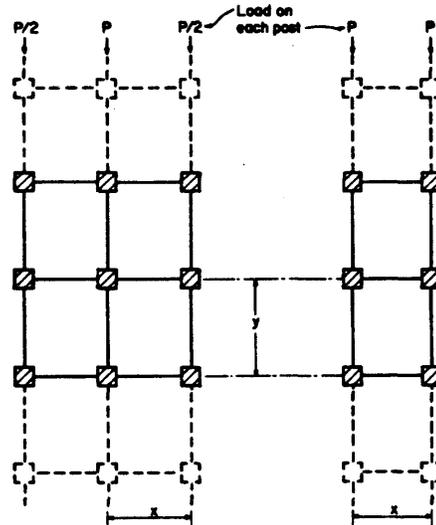


Fig. A1.3.1-Post configurations and loads

Loading: Post load = 15.5 kips
 Plate contact area for each post = 36 sq. in.
 Long spacing $y = 100$ in.
 Short spacing $x = 40$ in.
 Material: Concrete
 Compressive strength = 4000 psi
 Modulus of rupture = 570 psi
 $k = 100 \text{ pci}$

Table A1.4.1-Allowable distributed loads for unjointed aisle with nonuniform loading and variable layout (from Reference 14)

Slab thickness, in.	Subgrade k , ⁽¹⁾ pci	Allowable load, psf ⁽²⁾			
		Concrete flexural strength, psi			
		550	600	650	700
5	50	535	585	635	685
	100	760	830	900	965
	200	1,075	1,175	1,270	1,370
6	50	585	640	695	750
	100	830	905	980	1,055
	200	1,175	1,280	1,390	1,495
8	50	680	740	800	865
	100	960	1,045	1,135	1,220
	200	1,355	1,480	1,603	1,725
10	50	760	830	895	965
	100	1,070	1,170	1,265	1,365
	200	1,515	1,655	1,790	1,930
12	50	830	905	980	1,055
	100	1,175	1,280	1,390	1,495
	200	1,660	1,810	1,965	2,115
14	50	895	980	1,060	1,140
	100	1,270	1,385	1,500	1,615
	200	1,795	1,960	2,120	2,285

¹ k of subgrade; disregard increase in k due to subbase.
² For allowable stress equal to $\frac{1}{4}$ flexural strength.

Based on aisle and load widths giving maximum stress.

407/15.5

8 1/4 in. as determined from Fig.

A1.3.2.

Fig. A1.3.3 and A1.3.4 are also included for rack and post loads with subgrade modulus values of $k=50 \text{ pci}$ and 200 pci, respectively.

A1.4-Other PCA design information

Tables A1.4.1 and A1.4.2 are also included for uniform load applications. Examples of their used may be found in References 8 and 14. PCA joint spacing recommendations can be found in Chapter 9, Table 9.9.

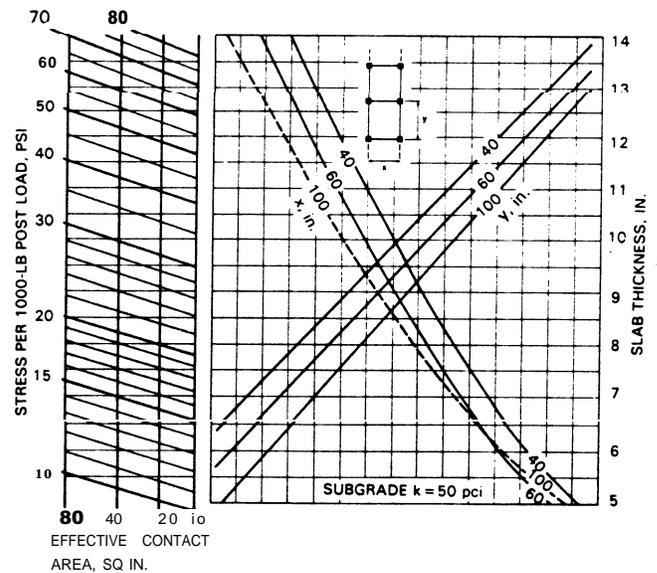


Fig. A1.3.3-PCA design chart for post loads where subgrade modulus is 50 pci

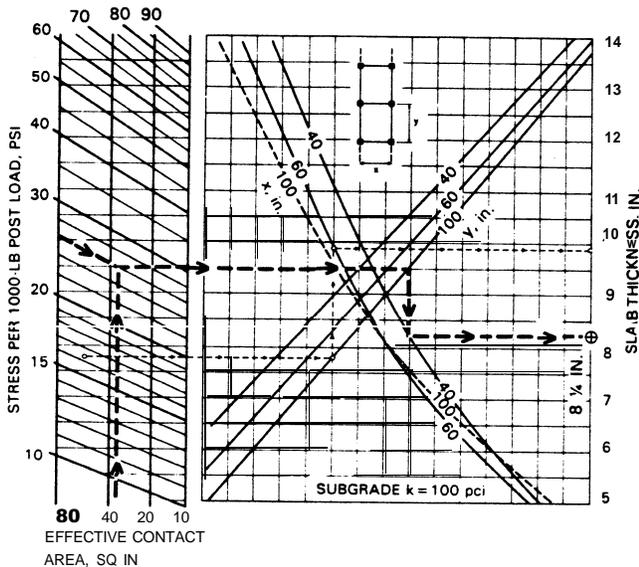


Fig. A1.3.2-PCA design chart for post loads where subgrade modulus is 100 pci

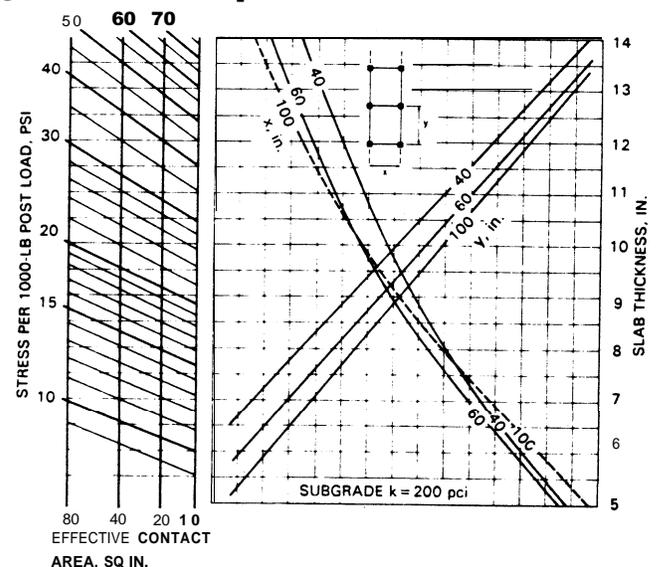


Fig. A1.3.4-PCA design chart for post loads where subgrade modulus is 200 pci

Table A 1.4.2-Allowable distribution loads, unjointed aisles, uniform loading and variable layout, PCA method

Notes: (1) *k* of subgrade; disregard increase in *k* due to subbase. (2) Critical aisle width equals 2.209 times the radius of relative stiffness.

Assumed load width = 300 in.; allowable load varies only slightly for other load widths. Allowable stress = 1/2 flexural strength

Slab thickness, in.	Working stress, psi	Critical aisle width, ft.(2)	Allowable load, psi					
			At critical aisle width	At other aisle widths				
				6-ft. aisle	8-ft. aisle	10-ft. aisle	12-ft. aisle	14-ft. aisle
Subgrade <i>k</i> = 50 pci⁽¹⁾								
5	300	5.6	610	615	670	815	1,050	1,215
	350	5.6	710	715	785	950	1,225	1,420
	400	5.6	815	820	895	1,085	1,400	1,620
6	300	6.4	670	675	695	780	945	1,175
	350	6.4	785	785	810	910	1,100	1,370
	400	6.4	895	895	925	1,040	1,260	1,570
8	300	8.0	770	800	770	800	880	1,010
	350	8.0	900	935	900	935	1,025	1,180
	400	8.0	1,025	1,070	1,025	1,065	1,175	1,350
10	300	9.4	845	930	865	850	885	960
	350	9.4	985	1,085	1,000	990	1,035	1,120
	400	9.4	1,130	1,240	1,145	1,135	1,185	1,285
12	300	10.8	915	1,065	965	915	925	965
	350	10.8	1,065	1,240	1,115	1,070	1,080	1,125
	400	10.8	1,220	1,420	1,270	1,220	1,230	1,290
14	300	12.1	980	1,225	1,070	1,000	980	995
	350	12.1	1,145	1,430	1,245	1,170	1,145	1,160
	400	12.1	1,310	1,630	1,425	1,335	1,310	1,330
Subgrade <i>k</i> = 100 pci⁽¹⁾								
5	300	4.7	065	900	1,090	1,470	1,745	1,810
	350	4.7	1,010	1,050	1,270	1,715	2,035	2,115
	400	4.7	1,155	1,200	1,455	1,955	2,325	2,415
6	300	5.4	950	955	1,065	1,320	1,700	1,925
	350	5.4	1,105	1,115	1,245	1,540	1,985	2,245
	400	5.4	1,265	1,275	1,420	1,760	2,270	2,565
8	300	6.7	1,095	1,105	1,120	1,240	1,465	1,815
	350	6.7	1,260	1,285	1,305	1,445	1,705	2,120
	400	6.7	1,460	1,470	1,495	1,650	1,950	2,420
10	300	7.9	1,215	1,265	1,215	1,270	1,395	1,610
	350	7.9	1,420	1,475	1,420	1,480	1,630	1,880
	400	7.9	1,625	1,645	1,625	1,690	1,860	2,150
12	300	9.1	1,320	1,425	1,325	1,330	1,400	1,535
	350	9.1	1,540	1,665	1,545	1,560	1,635	1,795
	400	9.1	1,755	1,900	1,770	1,770	1,865	2,050
14	300	10.2	1,405	1,590	1,445	1,405	1,435	1,525
	350	10.2	1,640	1,855	1,685	1,640	1,675	1,775
	400	10.2	1,075	2,120	1,925	1,875	1,915	2,030
Subgrade <i>k</i> = 200 pci⁽¹⁾								
5	300	4.0	1,225	1,400	1,930	2,450	2,565	2,520
	350	4.0	1,425	1,630	2,255	2,860	2,990	2,940
	400	4.0	1,630	1,665	2,575	3,270	3,420	3,360
6	300	4.5	1,340	1,415	1,755	2,395	2,740	2,810
	350	4.5	1,565	1,650	2,050	2,800	3,200	3,275
	400	4.5	1,785	1,890	2,345	3,190	3,655	3,745
8	300	5.6	1,550	1,550	1,695	2,045	2,635	3,070
	350	5.6	1,810	1,810	1,980	2,385	3,075	3,580
	400	5.6	2,065	2,070	2,615	2,730	3,515	4,095
10	350	6.6	1,730	1,745	1,775	1,965	2,330	2,895
	400	6.6	2,020	2,035	2,070	2,290	2,715	3,300
	400	6.6	2,310	2,325	2,365	2,620	3,105	3,860
12	300	7.6	1,890	1,945	1,895	1,995	2,230	2,610
	350	7.6	2,205	2,270	2,210	2,330	2,600	3,045
	400	7.6	2,520	2,595	2,525	2,660	2,972	3,480
14	300	8.6	2,025	2,150	2,030	2,065	2,310	2,460
	350	8.6	2,360	2,510	2,365	2,405	2,580	2,890
	400	8.6	2,700	2,870	2,705	2,750	2,950	3,305

**CHAPTER A2-SLAB THICKNESS
DESIGN BY WRI METHOD**

$E = 3000 \text{ ksi}$
 Thickness = 8 in. (trial value)
 Subgrade modulus $k = 400 \text{ pci}$

A2.1-Introduction

The following two examples show the determination of thickness for a slab on grade intended to have mild steel reinforcement for shrinkage and temperature stresses. The amount of steel is commonly selected using the subgrade drag theory presented in Chapter 6 and discussed in Reference 53.

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 selecting 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. A2.2.1, A2.2.2, and A2.2.3. The procedure starts with Fig. A2.2.1 where a concrete modulus of elasticity E and slab thickness H , and modulus of subgrade reaction k are assumed or known. For example, taking

Fig. A.2.2.1 gives the relative stiffness parameter $D/k = 3.4 \times 10^5 \text{ in.}^4$. The procedure then uses Fig. A2.2.2. Wheel Contact Area = 28 sq in.

$$\begin{aligned} \text{Diameter of equivalent circle} &= \sqrt{[28 \times 4] / \pi} \\ &= 6 \text{ in.} \end{aligned}$$

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. A2.2.2. The smaller chart in the figure gives the additional moment due to the other wheel as 16 in.-lb per in. of width per kip of wheel load. Moment = 265 + 16 = 281 in.-lb/in&p (Note that in.-lb/in. = ft-lb/ft)

Axle Load = 14.6 kips
 Wheel Load = 7.3 kips

$$\text{Design Moment} = 281 \times 7.3 = 2051 \text{ ft-lb/ft}$$

Then from Fig. A2.2.3:
 Allowable tensile stress = 190 psi

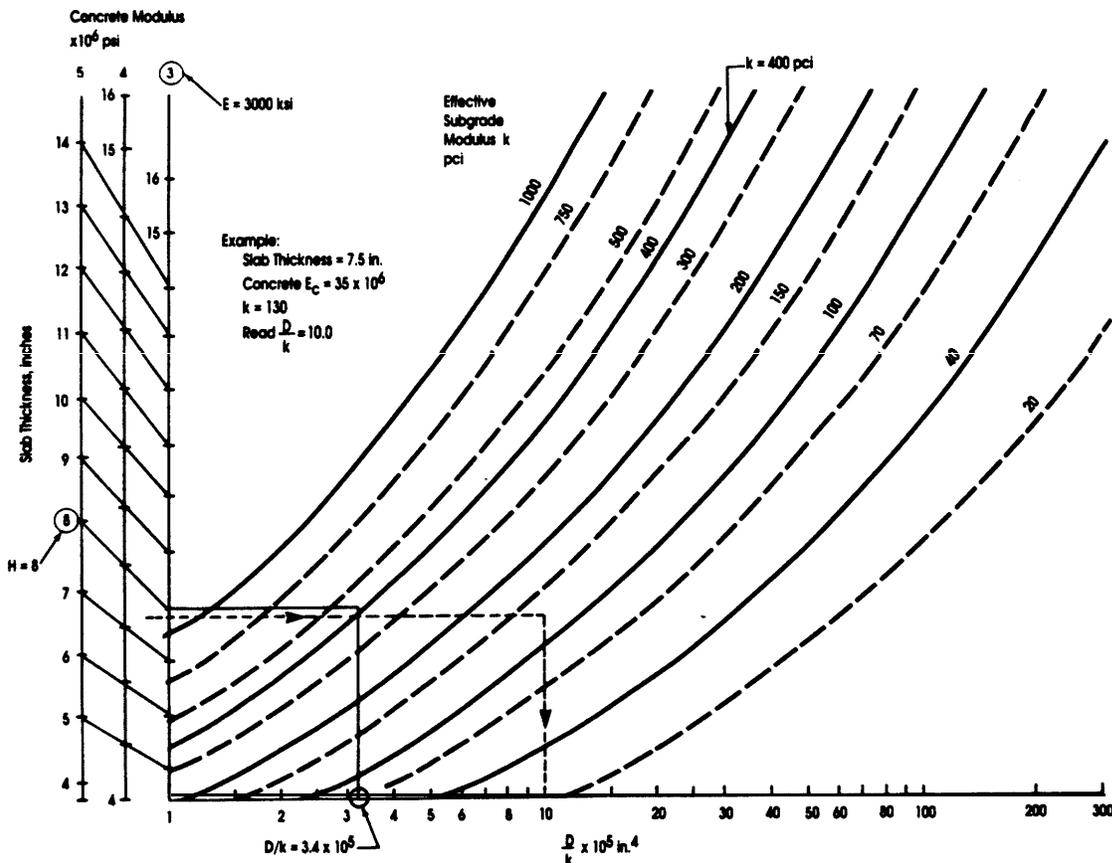


Fig. A2.2.1-Subgrade and slab stiffness relationship, used with WRI design procedure

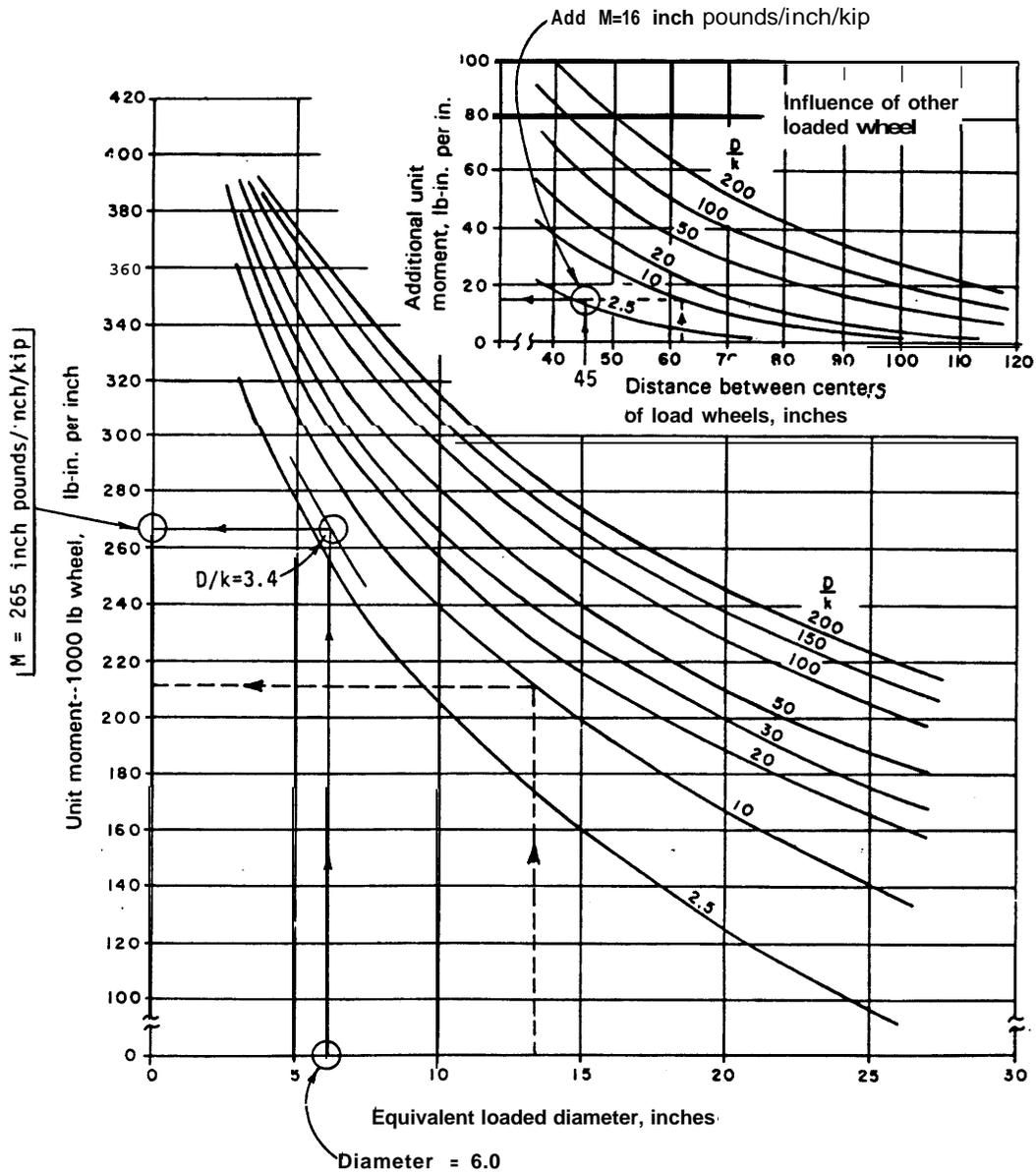


Fig. A2.2.2--Wheel loading design chart used with WRI procedure

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. A2.2.1 and A2.3. Note that Fig. A2.2.3 is a part of Fig. A2.3., separated here for clarity of procedure.

The procedure starts as before with determination of the term $D/k = 3.4 \times 10^5 \text{ in.}^4$. It then goes to Fig. A2.3 as follows:

Aisle width = 10 ft = 120 in.

Uniform load = 2500 psf = 2.5 ksf

Allowable tension = $MOR/SF = 190 \text{ psi}$

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.

A2.4-Comments

These procedures assume the use of conventional steel reinforcement in the concrete slab. The applied moments from the loads are not used in selecting the steel reinforcement except in the case of a Type F structurally reinforced slab.

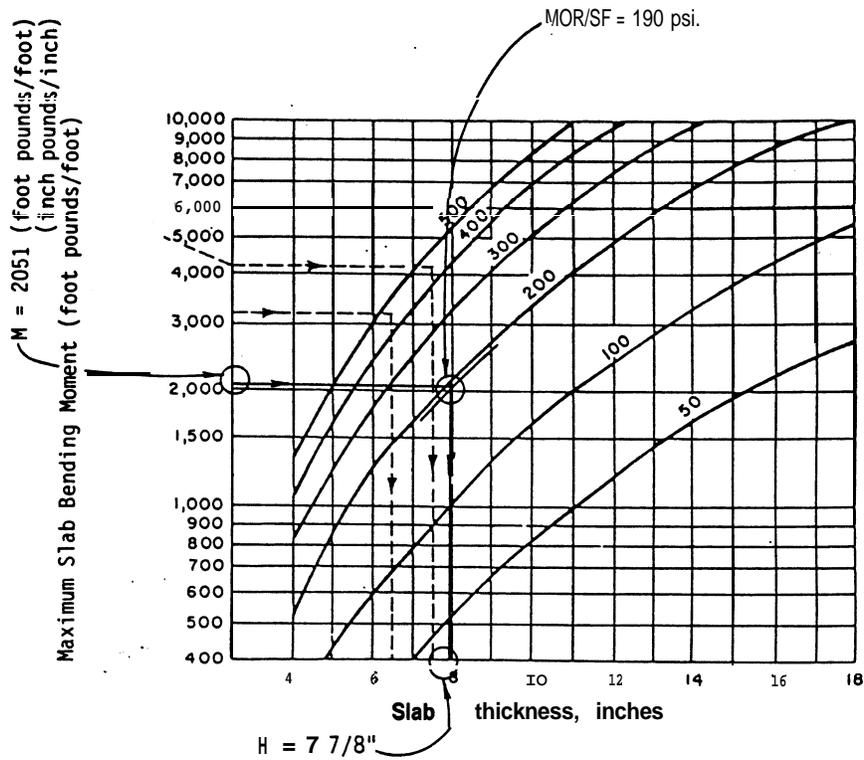


Fig. A2.2.3-Slab tensile stress charts used with WRI design procedure

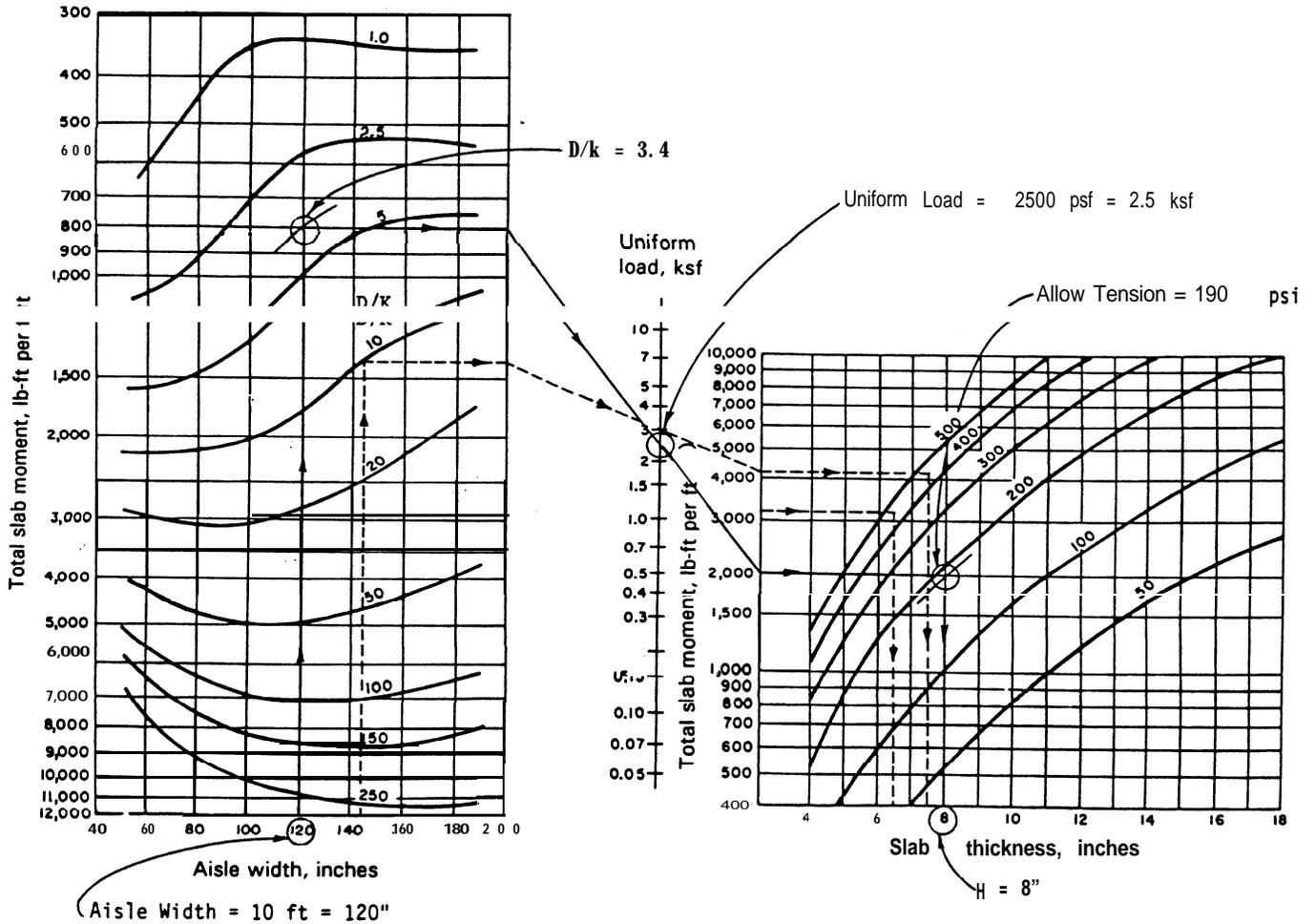


Fig. A2.3-Uniform load design and slab tensile stress charts used with WRI design procedure

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	10000	16000	20000	52000
Design axle load, lb	10000	15000	25000	36000	43000	120000
No. of tires	4	4	6	6	6	6
Type of tire	Solid	Solid	Pneumatic	Pneumatic	Pneumatic	Pneumatic
Tire contact area, sq in.	27.0	36.1	62.5	100	119	316
Effective 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

CHAPTER A3-DESIGN EXAMPLES USING COE CHARTS

A3.1-Introduction

The following examples show the determination of thickness for a slab on grade using the procedures published by the U.S. Army Corps of Engineers. The procedure appears in References 10, 18, and 54.

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 A.3.1). The procedure uses an impact factor of 25 percent, 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. A.3.1).

The six categories shown in Table A3.1 are those most commonly used. Fig. A3.1 shows a total of ten categories. 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. 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 $k = 100$ pci

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

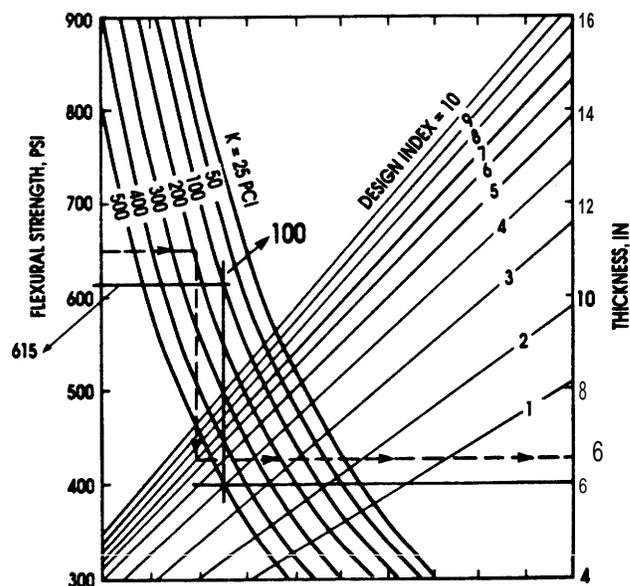


Fig. A3.1-COE curves for determining concrete floor thickness by design index

Concrete flexural strength: 500 psi

Modulus of subgrade reaction $k = 300$ pci

Use the design chart Fig. A3.3, entering with the flexural strength of 500 psi on the left, proceeding to an intersection with the curve for $k = 300$, then down to the line representing axle load, and across to the curve for the number of vehicle passes. The solution is a slab thickness of 5¼ in.

CHAPTER A4-SLAB DESIGN USING POST-TENSIONING

This chapter uses as a design example a 3-story apartment house in Houston, Texas 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 Chapter 8, in three sections:

- Design data including calculated soil values as well as given information

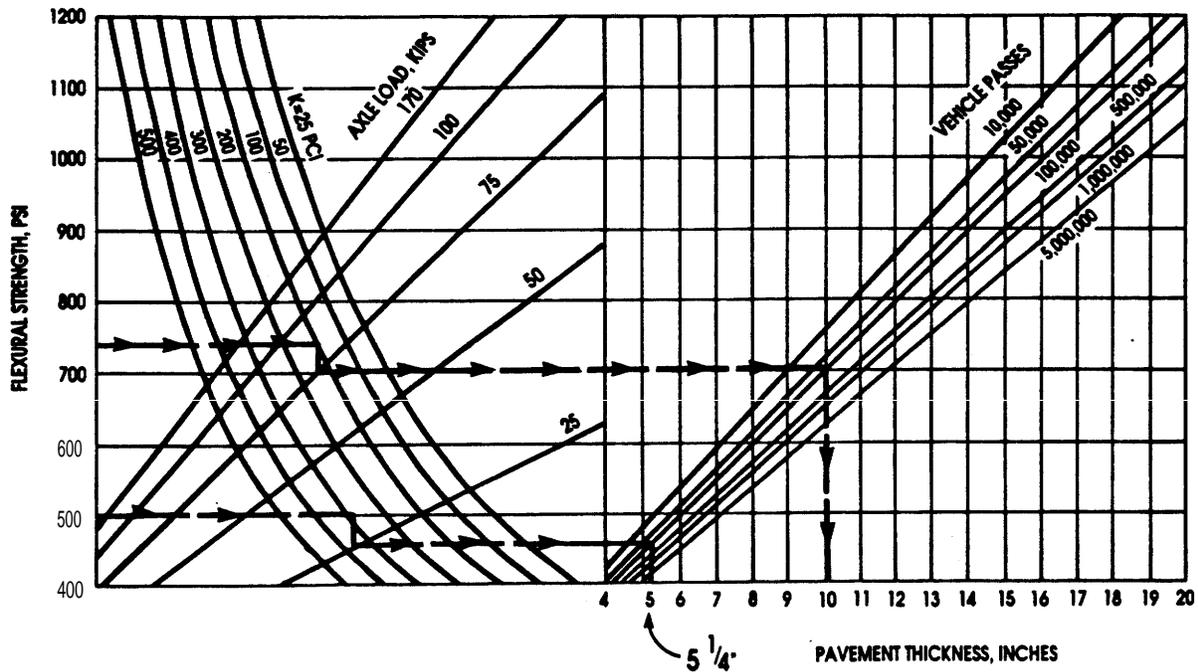


Fig. A3.3--COE design curves for concrete floor slabs with heavy forklift traffic

- Design for the edge lift condition
- Design for center lift condition

The necessary Post-Tensioning Institute design tables are included in this appendix as Tables A4.1 through A4.4. All of the figures and tables are taken from Reference 11.

A4.1-Design data including design soil values

- A. Loading
 - 1) Perimeter loading = 2280 lb/ft.
 - 2) Live load = 40 psf
- 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" 7-wire strand
- C. Soils Investigation
 - 1) Atterberg Limits:
 - $PL = 30$
 - $LL = 70$
 - $PI = 40$
 - 2) Percent clay: 65 percent
 - 3) Unconfined compressive strength, $q_u = 3400$ psf
 - 4) Soil modulus of elasticity: $E_s = 1000$ psi
 - 5) Depth to constant suction: $Z = 5$ ft
- D. Design soil values
 - 1) Thornthwaite moisture index: From Fig. A4.1.1, $I_m = +18$
 - 2) Edge moisture variation distance from Fig. A4.1.2, $e_m = 4.0$ ft. (Center lift)

- 3) Cation exchange activity: $e_m = 5.0$ ft. (Edge lift)
 $C.E.C. = (P.L.)^{1.17}$

$$CEAc = \frac{C.E.C.}{\% \text{ Clay}} = \frac{(30)^{1.17}}{65.0} = 0.82$$

- 4) Clay activity ratio:

$$Ac = \frac{PI}{\% \text{ Clay}} = \frac{40}{65} = 0.62$$

- 5) Principal clay mineral: From Fig. A4.1.3, principal clay mineral is montmorillonite.
- 6) Constant suction value: From Fig. A4.1.4, $pF = 3.3$
- 7) Estimated velocity of moisture flow:

$$Vel. = 0.5 \frac{I_m}{12}$$

$$Vel. = 0.5 \frac{18}{12} = 0.75 \text{ in./month}$$

0.75 > 0.70 Use 0.70 in/month maximum

- 8) Estimated differential swell: for $z = 5$ ft and $vel = 0.7$ in./month

Center Lift: interpolating in Tables A4.1 and A4.2 (in this appendix) for 65 percent clay, $pF = 3.3$, and $e_m = 4.0$ ft, $Y_m = 0.384$ in.

Edge Lift: interpolating in Tables A4.3 and A4.4 for 65 percent clay, $pF = 3.3$, and e_m

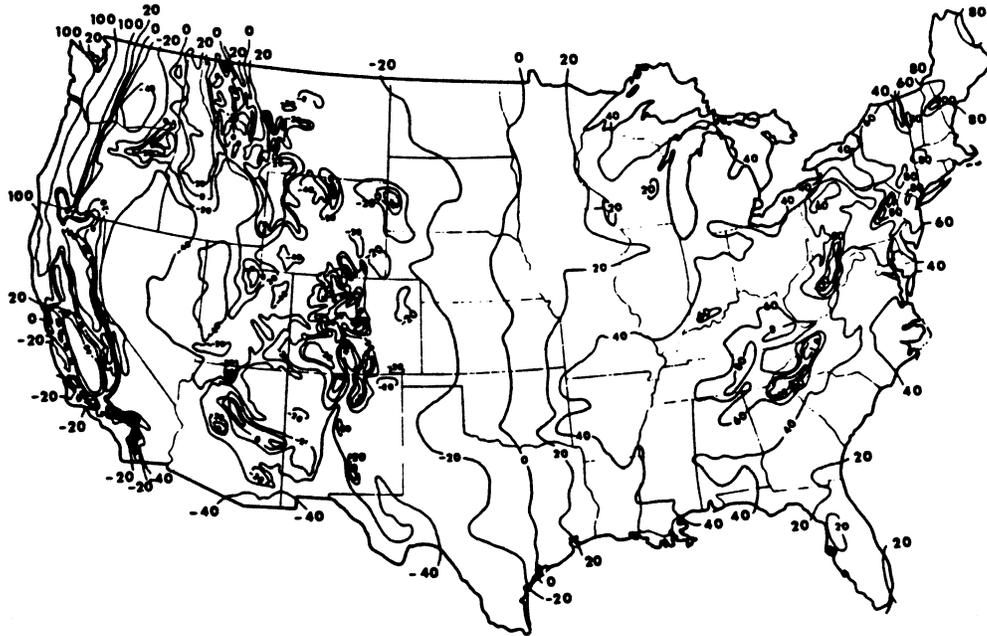


Fig. A4.1.1—Thornthwaite moisture index, in. per year

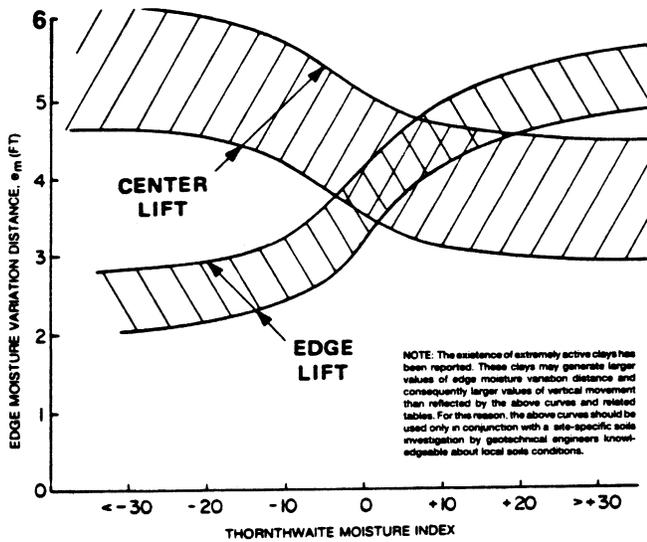


Fig. A4.1.2—Thornthwaite index versus edge moisture variation distance

$$= 5.0 \text{ ft}, Y_m = 0.338 \text{ in.}$$

E. Assume spacing of stiffening beams as sketched in Fig. A4.1.5.

A4.2—Design for edge lift

A. Calculate approximate depth of stiffening beams where:

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

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

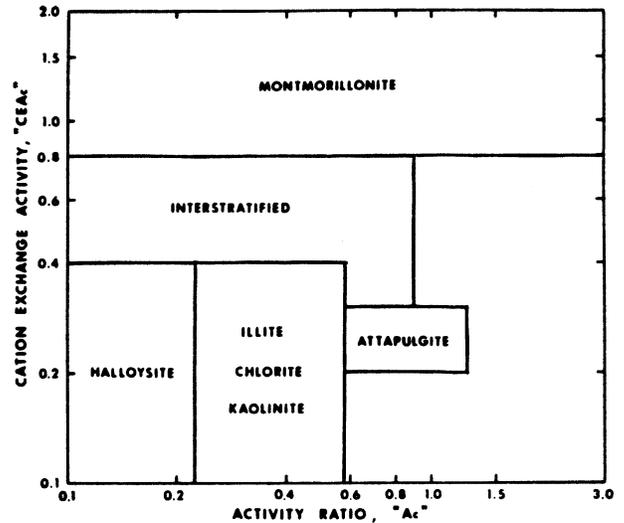


Fig. A4.1.3—Clay type classification

1) Long Direction: $L = 120$ ft; assume $\beta = 10$ ft, $6\beta = 60$ ft. Governs

$$\Delta_{allow} = \frac{12 \times 60}{1700} = 0.424 \text{ in.}$$

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

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

NOTE For practical applications, the maximum value of pF will seldom exceed a magnitude of 3.6.

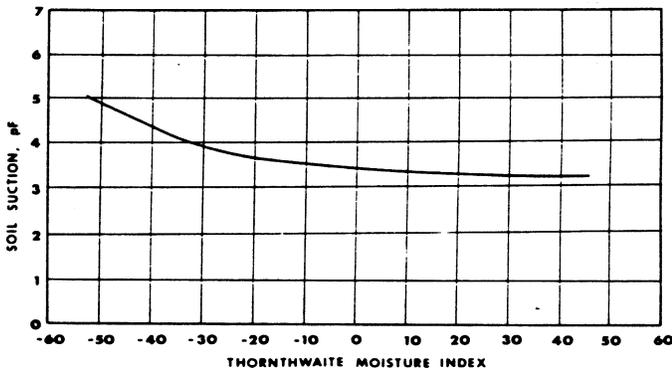


Fig. A4.1.4—Variation of constant soil suction with Thornthwaite index

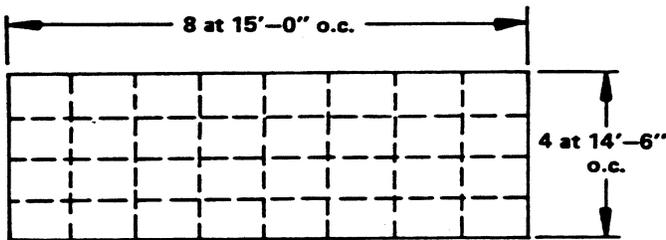


Fig. A 4.1.5—Beam layout for apartment house example

- 2) Short direction; assume $\beta = 10$ ft
 $L = 58$ ft. $< 6\beta$. Therefore 58 ft governs.
 $\Delta_{allow} = [12 \times 58]/1700 = 0.409$ 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
 $q_{allow} = 3,400$ psf = 3.40 ksf
 - 2) Applied loading
 Slab = $120 \times 58 \times 0.333 \times 0.150 = 347.65$ kips
 Beams = $9 \times 58 \times 1.0 \times 1.833 \times 0.150 = 143.52$
 Beams = $5 \times 111 \times 1.0 \times 1.833 \times 0.150 = 152.59$
 Perimeter = $2.280 \times 35 = 811.68$
 Live Load = $0.040 \times 58 \times 120 = 278.40$
- Total 1733.84 kips

For 1.0-ft-wide beams, the assumed spacing on Fig. A4.1.5 provides 1077 sq ft of bearing area. The soil bearing pressure is then:

$$w = 1733.84/1077 = 1.610 \text{ kips/sq ft}$$

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	4,104	8,136
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 re-quired

- 1) Number of tendons required for minimum average prestress of 50 psi. Stress in tendons immediately after anchoring:
 $f_{ps} = 0.7 f_{pu} = (0.7) (270) = 189$ ksi

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

$$N_T = \frac{(\text{min. prestress}) \times (\text{area slab})}{1000 \times (\text{eff. tend. stress}) \times (\text{area tendon})}$$

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

= 8.44

$$N_T(\text{Short}) = \frac{(50 \text{ psi}) (8136 \text{ in.}^2)}{(1000) (159 \text{ ksi}) (0.153 \text{ in.}^2)}$$

= 16.72

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

Weight of beams and slab = 643.76 kips

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

Table A4.1-PTI table for differential swell occurring at the perimeter of a slab for center lift. Swelling condition in primarily montmorillonite clay soil 60 percent clay

Percent Clay (%)	Depth of Constant Suction (FT)	Constant Suction (pF)	Velocity of Moisture Flow (inches/month)	DIFFERENTIAL SWELL (INCHES)								
				EDGE DISTANCE PENETRATION (FT)								
				1 FT	2 FT	3 FT	4 FT	5 FT	6 FT	7 FT	8 FT	
60	3	3.2	0.1	0.003	0.006	0.009	0.013	0.016	0.019	0.022	0.026	
			0.3	0.009	0.019	0.028	0.038	0.048	0.059	0.069	0.080	
			0.5	0.015	0.031	0.048	0.065	0.082	0.101	0.120	0.140	
				0.7	0.022	0.044	0.068	0.092	0.119	0.147	0.176	0.209
		3.4		0.1	0.006	0.013	0.019	0.026	9.033	0.040	0.047	0.054
	0.3			0.019	0.039	0.060	0.082	0.105	0.130	0.157	0.186	
	0.5			0.031	0.065	0.102	0.143	0.189	0.240	0.300	0.372	
				0.7	0.044	0.093	0.147	0.211	0.286	0.380	0.503	0.653
		3.6		0.1	0.015	0.031	0.048	0.065	0.083	0.102	0.122	0.144
	0.3			0.044	0.095	0.152	0.219	0.300	0.403	0.543	0.758	
	0.5			0.073	0.161	0.272	0.420	0.645	1.056	2.037	4.865	
				0.7	0.101	0.229	0.407	0.689	1.246	2.689	6.912	
	5	3.2	0.1	0.008	0.015	0.023	0.030	0.038	0.045	0.053	0.060	
0.3			0.022	0.044	0.067	0.090	0.113	0.138	0.162	0.187		
0.5			9.937	0.074	0.113	0.153	0.194	0.237	0.282	D.329		
			0.7	0.050	0.103	0.158	0.217	0.278	0.343	0.412	0.487	
	3.4		0.1	0.015	0.030	0.043	0.061	0.077	0.093	0.109	0.126	
0.3			0.045	0.091	0.149	0.191	0.245	0.302	0.363	0.426		
0.5			0.073	0.152	0.237	0.331	0.435	0.551	0.686	0.846		
			0.7	0.102	0.214	0.341	0.485	0.655	0.865	1.140	1.541	
	3.6		0.1	0.035	0.071	0.109	0.149	0.190	0.233	0.279	0.326	
0.3			0.103	0.219	0.349	0.499	0.678	0.901	1.200	1.652		
0.5			0.169	0.370	0.618	0.945	1.425	2.280	4.284	9.923		
			0.7	0.234	0.526	0.922	1.528	2.686	5.574			
	7	3.2	0.1	0.013	0.027	0.041	0.055	0.069	0.083	0.097	0.111	
D.3			0.041	0.082	0.125	0.168	0.212	0.256	0.302	0.349		
0.5			0.068	0.137	0.209	0.283	0.360	0.441	0.524	0.612		
			0.7	0.093	0.191	0.294	0.402	0.516	0.637	0.767	0.907	
	3.4		0.1	0.027	0.055	0.083	0.112	0.142	0.171	0.201	0.232	
0.3			0.082	0.167	0.256	0.351	0.449	0.555	0.666	0.786		
0.5			0.135	0.280	0.438	0.609	0.799	1.013	1.260	1.553		
			0.7	0.158	0.395	0.627	0.892	1.204	1.587	2.092	2.840	
	3.56		0.1	0.053	0.107	0.163	0.221	0.281	0.342	0.407	0.474	
0.3			0.156	0.326	0.512	0.720	0.957	1.232	1.566	1.994		
0.5			0.256	0.549	0.895	1.320	1.879	2.702	4.182	8.216		
			0.7	0.354	0.779	1.317	2.059	3.247	5.677			

Table A4.2-PTI table for differential swell occurring at the perimeter of a slab for center lift. Swelling condition in primarily montmorillonite clay soil 70 percent clay

Percent Clay (%)	Depth to Constant Suction (FT)	Constant Suction (pF)	Velocity of Moisture Flow (inches/month)	DIFFERENTIAL SWELL (INCHES)								
				EDGE DISTANCE PENETRATION (FT)								
				1 FT	2 FT	3 FT	4 FT	5 FT	6 FT	7 FT	8 FT	
70	3	3.2	0.1	0.004	0.007	0.011	0.015	0.019	0.023	0.026	0.030	
			0.3	0.011	0.022	0.034	0.046	0.057	0.070	0.082	0.095	
			0.5	0.018	0.037	0.057	0.077	0.098	0.120	0.143	0.167	
				0.7	0.026	0.052	0.082	0.110	0.141	0.174	0.210	0.248
		3.4		0.1	0.007	0.015	0.023	0.031	0.039	0.048	0.056	0.065
	0.3			0.023	0.047	0.072	0.098	0.126	0.156	0.188	0.222	
	0.5			0.038	0.073	0.122	0.171	0.225	0.287	0.358	0.443	
				0.7	0.052	0.110	0.176	0.251	0.341	0.452	0.600	0.814
		3.6		0.1	0.018	0.037	0.056	0.077	0.098	0.121	0.145	0.171
	0.3			0.054	0.114	0.182	0.262	0.359	0.481	0.648	0.903	
	0.5			0.088	0.192	0.324	0.502	0.769	1.258	2.428	5.7%	
				0.7	0.120	0.273	0.485	0.821	1.485	3.203	8.234	
	5	3.2	0.1	0.009	0.017	0.026	0.035	0.044	0.053	0.062	0.071	
0.3			0.026	0.053	0.080	0.107	0.135	0.164	0.193	0.223		
0.5			0.044	0.088	0.134	0.182	0.231	0.283	0.336	0.392		
			0.7	0.060	0.123	0.189	0.258	0.331	0.409	0.491	0.580	
	3.4		0.1	0.018	0.036	0.055	0.073	0.092	0.111	0.131	0.151	
0.3			0.053	0.109	0.167	0.228	0.292	0.360	0.433	0.511		
0.5			0.088	0.182	0.284	0.395	0.519	0.658	0.818	1.008		
			0.7	0.121	0.256	0.406	0.578	0.781	1.031	1.358	1.837	
	3.6		0.1	0.042	0.086	0.131	0.178	0.227	0.278	0.332	0.389	
0.3			0.123	0.260	0.415	0.594	0.807	1.073	1.429	1.968		
0.5			0.202	0.441	0.737	1.126	1.6%	2.717	5.104	11.822		
			0.7	0.278	0.627	1.098	1.820	3.199	6.640			
	7	3.2	0.1	0.016	0.032	0.049	0.065	0.082	0.098	0.115	0.132	
0.3			0.048	0.097	0.148	0.199	0.251	0.305	0.359	0.415		
0.5			0.081	0.163	0.249	0.338	0.429	0.525	0.624	0.729		
			0.7	0.112	0.229	0.351	0.480	0.616	0.759	0.915	1.081	
	3.4		0.1	0.032	0.066	0.099	0.134	0.168	0.204	0.240	0.276	
0.3			0.098	0.199	0.306	0.418	0.536	0.661	0.794	0.937		
0.5			0.162	0.334	0.522	0.727	0.952	1.207	1.501	1.851		
			0.7	0.224	0.470	0.747	1.063	1.435	1.891	2.492	3.383	
	3.56		0.1	0.063	0.128	0.194	0.263	0.334	0.407	0.484	0.563	
0.3			0.185	0.387	0.609	0.857	1.139	1.468	1.865	2.376		
0.5			0.305	0.655	1.067	1.573	2.239	3.219	4.983	9.162		
			0.7	0.421	0.928	1.569	2.453	3.868	6.763			

Table A4.3-PTI table for differential swell occurring at the perimeter of a slab for edge lift. Swelling condition in primarily montmorillonite clay soil 60 percent clay

Percent Clay (%)	Depth to Constant Suction (FT)	Constant Suction (pF)	Velocity of Moisture Flow (inches/month)	DIFFERENTIAL SWELL (INCHES)							
				EDGE DISTANCE PENETRATION (FT)							
				1 FT	2 FT	3 FT	4 FT	5 FT	6 FT	7 FT	8 FT
60	3	3.2	0.1	0.003	0.006	0.008	0.011	0.014	0.016	0.019	0.022
				0.008	0.016	0.025	0.033	0.040	0.048	0.056	0.064
				0.014	0.027	0.041	0.054	0.066	0.079	0.091	0.104
		3.4	0.3	0.019	0.038	0.056	0.074	0.091	0.109	0.125	0.142
				0.006	0.012	0.017	0.023	0.029	0.035	0.040	0.046
				0.018	0.035	0.051	0.068	0.084	0.099	0.115	0.130
	3.6	0.5	0.029	0.057	0.084	0.110	0.135	0.160	0.183	0.206	
			0.041	0.079	0.116	0.151	0.184	0.216	0.247	0.277	
			0.071	0.136	0.195	0.251	0.303	0.352	0.399	0.433	
	3.0	0.7	0.098	0.185	0.264	0.336	0.402	0.463	0.521	0.575	
			0.035	0.069	0.102	0.133	0.163	0.191	0.219	0.246	
			0.104	0.195	0.277	0.352	0.421	0.484	0.544	0.599	
	5	3.2	0.1	0.169	0.309	0.428	0.533	0.627	0.712	0.790	0.863
				0.233	0.413	0.562	0.690	0.802	0.903	0.994	1.077
				0.006	0.012	0.018	0.024	0.030	0.036	0.042	0.048
		3.4	0.3	0.018	0.036	0.054	0.071	0.089	0.106	0.123	0.140
				0.030	0.060	0.090	0.118	0.146	0.174	0.202	0.229
				0.042	0.083	0.124	0.163	0.202	0.240	0.278	0.314
	3.6	0.5	0.013	0.026	0.039	0.051	0.064	0.076	0.089	0.101	
			0.039	0.077	0.114	0.151	0.187	0.222	0.256	0.291	
			0.065	0.127	0.190	0.246	0.303	0.359	0.413	0.465	
	3.0	0.7	0.090	0.177	0.259	0.339	0.415	0.488	0.559	0.628	
			0.032	0.064	0.095	0.126	0.156	0.186	0.215	0.244	
			0.096	0.188	0.276	0.360	0.440	0.518	0.593	0.665	
7	3.2	0.1	0.160	0.308	0.446	0.575	0.697	0.812	0.921	1.025	
			0.224	0.425	0.607	0.775	0.931	1.077	1.214	1.343	
			0.080	0.156	0.230	0.301	0.369	0.436	0.500	0.562	
	3.4	0.3	0.238	0.450	0.642	0.817	0.980	1.132	1.274	1.407	
			0.395	0.724	1.009	1.261	1.488	1.695	1.886	2.063	
			0.551	0.984	1.345	1.657	1.933	2.181	2.407	2.614	
3.6	0.5	0.010	0.021	0.031	0.042	0.052	0.062	0.072	0.083		
		0.031	0.062	0.093	0.124	0.154	0.184	0.214	0.243		
		0.052	0.104	0.155	0.205	0.254	0.303	0.351	0.398		
3.0	0.7	0.073	0.145	0.215	0.284	0.352	0.419	0.484	0.548		
		0.023	0.045	0.067	0.090	0.112	0.134	0.155	0.177		
		0.068	0.135	0.200	0.264	0.328	0.390	0.451	0.511		
3.6	0.1	0.113	0.223	0.330	0.434	0.535	0.633	0.729	0.823		
		0.159	0.311	0.458	0.598	0.734	0.865	0.992	1.115		
		0.057	0.113	0.168	0.222	0.275	0.328	0.380	0.431		
3.8	0.3	0.171	0.334	0.490	0.640	0.785	0.924	1.058	1.188		
		0.286	0.551	0.799	1.031	1.251	1.459	1.658	1.847		
		0.402	0.764	1.095	1.400	1.684	1.950	2.200	2.437		

Table A4.4-PTI table for differential swell occurring at the perimeter of a slab for edge lift. Swelling condition in primarily montmorillonite clay soil 70 percent clay

Percent Clay (%)	Depth to Constant Suction (FT)	Constant Suction (pF)	Velocity of Moisture Flow (inches/month)	DIFFERENTIAL SWELL (INCHES)							
				EDGE DISTANCE PENETRATION (FT)							
				1 FT	2 FT	3 FT	4 FT	5 FT	6 FT	7 FT	8 FT
70	3	3.2	0.1	0.003	0.007	0.010	0.013	0.016	0.020	0.023	0.026
				0.010	0.020	0.029	0.039	0.048	0.058	0.067	0.076
				0.016	0.032	0.048	0.064	0.079	0.094	0.109	0.123
		3.4	0.3	0.023	0.045	0.067	0.088	0.109	0.129	0.149	0.169
				0.007	0.014	0.021	0.028	0.034	0.041	0.048	0.054
				0.021	0.041	0.061	0.081	0.100	0.118	0.137	0.155
	3.6	0.5	0.035	0.068	0.100	0.131	0.161	0.190	0.219	0.246	
			0.048	0.094	0.138	0.179	0.219	0.258	0.294	0.330	
			0.071	0.136	0.195	0.251	0.303	0.352	0.399	0.433	
	3.0	0.7	0.098	0.185	0.264	0.336	0.402	0.463	0.521	0.575	
			0.042	0.082	0.121	0.158	0.194	0.228	0.261	0.293	
			0.124	0.233	0.330	0.419	0.501	0.577	0.648	0.714	
	5	3.2	0.1	0.169	0.309	0.428	0.533	0.627	0.712	0.790	0.863
				0.233	0.413	0.562	0.690	0.802	0.903	0.994	1.077
				0.007	0.014	0.022	0.029	0.036	0.043	0.050	0.057
		3.4	0.3	0.022	0.043	0.064	0.085	0.106	0.126	0.147	0.167
				0.036	0.071	0.106	0.140	0.174	0.207	0.240	0.273
				0.050	0.099	0.147	0.195	0.241	0.286	0.331	0.374
	3.6	0.5	0.015	0.031	0.046	0.061	0.076	0.091	0.106	0.121	
			0.046	0.092	0.136	0.179	0.222	0.264	0.306	0.346	
			0.077	0.151	0.223	0.293	0.361	0.427	0.492	0.554	
	3.0	0.7	0.108	0.210	0.309	0.403	0.494	0.582	0.666	0.748	
			0.038	0.076	0.113	0.150	0.186	0.221	0.256	0.290	
			0.115	0.224	0.329	0.429	0.525	0.617	0.706	0.792	
7	3.2	0.1	0.191	0.367	0.531	0.685	0.830	0.967	1.097	1.221	
			0.267	0.506	0.724	0.924	1.110	1.283	1.446	1.600	
			0.095	0.186	0.274	0.358	0.440	0.519	0.595	0.669	
	3.4	0.3	0.203	0.536	0.764	0.974	1.168	1.340	1.517	1.677	
			0.470	0.862	1.262	1.502	1.773	2.020	2.247	2.458	
			0.656	1.172	1.603	1.974	2.303	2.582	2.867	3.114	
3.6	0.5	0.012	0.025	0.037	0.050	0.062	0.074	0.086	0.098		
		0.037	0.074	0.111	0.147	0.183	0.219	0.255	0.290		
		0.062	0.124	0.184	0.244	0.303	0.361	0.418	0.475		
3.0	0.7	0.087	0.173	0.256	0.339	0.419	0.499	0.577	0.653		
		0.027	0.054	0.080	0.107	0.133	0.159	0.185	0.211		
		0.081	0.160	0.238	0.315	0.390	0.464	0.537	0.609		
3.6	0.1	0.135	0.266	0.393	0.517	0.637	0.754	0.869	0.980		
		0.189	0.371	0.545	0.713	0.875	1.031	1.182	1.329		
		0.068	0.134	0.200	0.264	0.328	0.391	0.453	0.514		
3.8	0.3	0.204	0.398	0.584	0.763	0.935	1.101	1.261	1.415		
		0.341	0.656	0.951	1.229	1.490	1.739	1.975	2.200		
		0.479	0.910	1.304	1.668	2.006	2.323	2.621	2.903		

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

- 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
 Since 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.

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

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

E. Calculate design moments

- 1) Long direction

$$M_\ell = \frac{(S)^{0.10} (d e_m)^{0.78} (Y_m)^{0.66}}{7.2(L)^{0.0065} (P)^{0.04}}$$

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

$$M_\ell = 2.81 \text{ ft. kips/ft.}$$

- 2) Short direction

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

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

A4.2—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

$$(12 \times 58) {}_p M_t = S_B [(P_r/A) + f_i] - P_r e$$

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

$${}_p M_t = 2710 \text{ in.-kips}/(12 \times 58) = 3.89 \text{ ft-kips/ft.}$$

$$3.89 > 2.81 \text{ OK}$$

- b) Compression in top fiber

$$(12 \times 58) {}_p M_c = S_T [f'_c - P_r/A] - P_r e$$

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

$${}_p M_c = 39,776 \text{ in.-kips}/(12 \times 58) = 57.15 \text{ ft-kips/ft}$$

$$57.15 > 2.81 \text{ OK}$$

- 2) Short Direction

- a) Tension in Bottom Fiber

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

$${}_p M_t = 5371 \text{ in.-kips}/(12 \times 120) = 3.73 \text{ ft-kips/ft}$$

$$3.73 > 3.65 \text{ OK}$$

- b) Compression in Top Fiber

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

$${}_p M_c = 82,377 \text{ in.-kips}/(12 \times 120) = 57.21 \text{ ft-kips/ft}$$

$$57.21 > 3.65 \text{ OK}$$

G. Deflection Calculations

- 1) Long Direction

- a) Allowable Differential Deflection

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

$$6\beta = 66.48 \text{ ft} < 120 \text{ ft, so } 6\beta \text{ governs}$$

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

- b) Expected Differential Deflection

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

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

$$\Delta = 0.296 \text{ in. } 0.296 < 0.997 \text{ OK}$$

- 2) Short Direction

- a) Allowable Differential Deflection

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

$$6\Delta = 77.64 > 58 \text{ ft, so } 58 \text{ ft governs}$$

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

- b) Expected Differential Deflection

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

$$\Delta = 0.236 \text{ in. } 0.236 < 0.870 \text{ in. OK}$$

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

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

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

$$V_\ell = 1.300 \text{ kips/ft.}$$

- b) Permissible shear stress

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

- 9 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.

A4.3-Design for center lift

- A. Calculate Design Moments

- 1) Long Direction

$$M_\ell = A_o [B(e_m)^{1.238} + C]$$

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

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

$$A_o = 1.612$$

$$0 \leq e_m \leq 5$$

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

$$M_\ell = (1.612)(4.0)^{1.238} = 8.97 \text{ ft-kips/ft.}$$

- 2) Short Direction

$$M_s = [(58 + e_m)/60] M_\ell$$

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

- B. Calculate allowable moments and compare with design moments

- 1) Long Direction

- a) Tension in Top Fiber

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

$$(12 \times 58) {}_n M_t = (33,702) [(461.7/4104) + 0.3291 + (461.7)(4.18)]$$

$${}_n M_t = \frac{16,809 \text{ in. -kips}}{12 \times 58} = 24.15 \text{ ft-kips/ft}$$

$$24.15 > 8.97 \text{ OK}$$

- b) Compression in bottom fiber

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

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

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

$$21.46 > 8.97 \text{ OK}$$

- 2) Short direction

- a) Tension in top fiber

$$(12 \times 120) {}_n M_t = 66,861 [(656.1/8136) + 0.3291 + (656.1)(3.80)]$$

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

$$20.75 > 9.27 \text{ ft-kips/ft}$$

- b) Compression in bottom fiber

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

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

$$18.65 > 9.27 \text{ OK}$$

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 calculation of each, see Reference 11.

- 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 1/4 in. on center beginning 1 ft from each edge. 5 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. 9 beams, 12 in. wide, 26 in. deep evenly spaced at 14 ft 10 1/2 in. on centers.

CHAPTER A5-EXAMPLES USING SHRINKAGE-COMPENSATING CONCRETE

A5.1-Introduction

The following examples show two approaches for the design of a slab on grade using shrinkage-compensating concrete. 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 Chapter A1, A2, or A3. 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 mix and the reinforcing steel to compensate for subsequent drying shrinkage. Since 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. The 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 may be one of two procedures. One is the determination of the required prism expansion to ensure shrinkage compensation, which leads to the design for properties of the concrete mix. This is shown in Sec. A5.2. The alternate procedure is used

when the properties of the mix are known (expansion percentage) and the selection of the proper amount of steel is desired. This is shown in Sec. A5.3. Expansion of the length of the slab is also de-termined in both cases.

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 sq in./ft, which is 0.5 percent.

Joint spacing = 80 ft

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

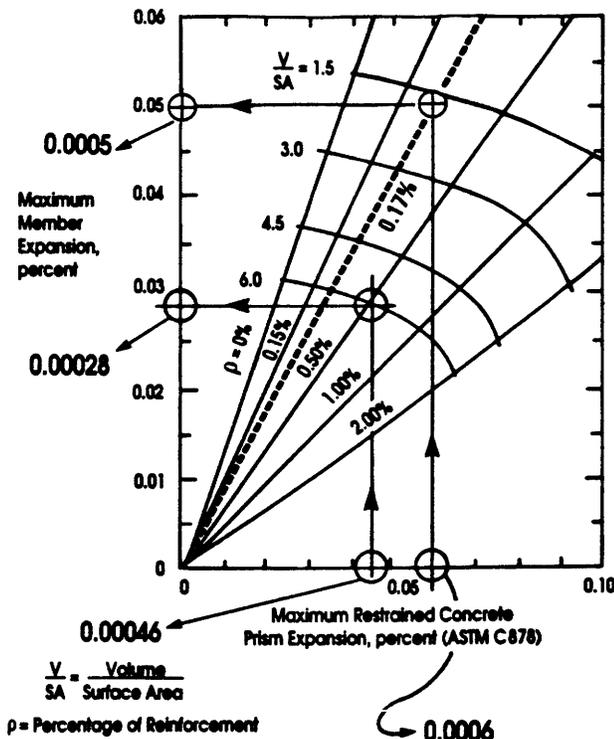


Fig. A5.2-Prediction of member expansion from prism data (from Reference 31)

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 percent
= 0.00046 in./in

This expansion, as determined by ASTM C 878 from tests of the concrete mix, is used with Fig. A5.2 to determine member expansion.

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

This is to be combined with the joint spacing to

determine the total slab motion as well as 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 x 80 x 12 = 0.269 in.

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

Filler thickness = 2 x 0.27
= 0.54 in.

ACI 223 discusses these features in greater detail.

A5.3-Design example selecting reinforcement where thickness, joint spacing, and prism expansion are known

The following data are known:

Thickness = 6 in.

Joint spacing = 100 ft

Prism expansion = 0.0006 in./in.

(ASTM C 878)

In this example, the amount of reinforcement will be selected by the subgrade drag equation (Chapter 6) using an assumed friction coefficient of 1.5 and an assumed steel stress of 45 ksi.

$$A_s = (F L w) / 2 f_s$$

$$= (1.5 \times 100 \times 75) / (2 \times 45,000)$$

$$= 0.125 \text{ sq in. per ft}$$

This is 0.1736 percent (0.001736).

From Fig. A5.2, the member expansion can be obtained:

Member expansion = 0.050 percent
= 0.00050 in./in.

With a joint spacing of 100 ft, the slab motion (expansion) is then

0.00050 X 100 X 12 = 0.60 in.

Use a joint material which will compress at least twice this amount:

2 x 0.60 = 1.2 in.

or a material that will compress at least 0.6 in. at each end of the 100-ft length.

If reinforcement (say wire) spacing is to be 12 in. on center, then use

0.001736 x 6 x 12 = 0.125 sq in./ft. This would be D14/D14 wire. Other options could be D18/D18 at 16 x 16 in., or D20/D20 at 18 x 18-in. spacing.

CONVERSION FACTORS

LENGTH

1 inch = 2.54 cm	1 cm = 0.39 inch
1 foot = 0.305 m	1 m = 3.28 feet
1 mile = 1.61 km	1 km = 0.62 miles

inches to centimeters	multiply by 2.5	ounces to grams	multiply by 28.3
centimeters to inches	multiply by 0.4	grams to ounces	multiply by 0.035
feet to meters	multiply by 2.5	pounds to kilograms	multiply by 0.45
meters to feet	multiply by 3.3	kilograms to pounds	multiply by 2.2

VOLUME

1 fluid ounce = 29.57 ml	10 ml = 0.34 fl oz
1 quart (32 fl oz) = 946.35 ml	1 l = 1.06 US qt
1 gal(128 fl oz) = 3.79 l	3.79 l = 1 US gal

ounces to milliliters	multiply by 30
milliliters to ounces	multiply by 0.03
quarts to liters	multiply by 0.95
liters to quarts	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

TEMPERATURE

$$^{\circ}\text{C} = \frac{^{\circ}\text{F} - 32}{1.8} \quad \quad \quad ^{\circ}\text{F} = (1.8 \times ^{\circ}\text{C}) + 32$$

$$1 \text{ deg F/in.} = 0.22 \text{ deg C/cm}$$

SPECIFIC WEIGHT.

1 lb water = 27.7 inch ³ = 0.1198 gal
1 ft ³ water = 62.43 lb
1 gallon water = 8.345 lb

WATER-CEMENT RATIO

Multiply W/C by 11.3 to obtain gallons per bag

AREA

$$1 \text{ in.}^2 = 6.452 \text{ cm}^2$$

This report was submitted to letter ballot of the committee and approved according to Institute balloting procedures.