NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT 343

MANUALS FOR THE DESIGN OF BRIDGE FOUNDATIONS

Shallow Foundations
Driven Piles
Retaining Walls and Abutments
Drilled Shafts
Estimating Tolerable Movements
Load Factor Design Specifications and Commentary

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff
Transportation Research Board

This report contains the results of a study that developed recommended load-factor-design specifications for highway-bridge foundations and retaining structures. Comprising a series of engineering-design manuals, the report provides detailed load-factor-design procedures for various foundation types and includes examples showing how the recommended specification can be applied to bridge foundation design. The contents of this report will be of immediate interest and use to bridge design and geotechnical engineers at the federal, state, and local levels, and to specification writing bodies.

Prior to the early 1970s, all highway-bridge design in the United States was performed using the working stress design method. Then, in the mid-1970s, AASHTO adopted load-factor design into the AASHTO *Standard Specifications for Highway Bridges* as an approved design method for portions of the bridge structure above the foundation. Over time, a number of states adopted AASHTO's load-factor-design criteria for bridge-superstructure design. However, many others have not due, in part, to the desire to avoid inconsistency inherent in designing those portions of the structure above the foundation by the load factor method while still designing foundations by working stress.

This inconsistency in design format requires considerable duplication of effort in compiling design forces for the highway structure and its foundation. The development of suitable load-factor-design criteria for bridge foundations would eliminate this inconsistency, saving time and money. Additionally, it would lead to a more uniform margin of safety for all structural components in a highway structure and result in a more consistent and efficient use of materials.

NCHRP Project 24-4 was initiated with the objective of developing load-factor-design provisions which could be considered by AASHTO for inclusion in the *Standard Specifications for Highway Bridges.* Specification provisions and commentary were developed for shallow foundations, driven piles, drilled shafts, and abutments and rigid retaining structures. The specifications employ the same load factors and load combinations that are presently used for AASHTO superstructure design. The recommended specifications and commentary are expected to be considered for adoption by AASHTO in 1992.

In addition to the recommended specifications, five engineering manuals were developed during the course of the study. The manuals describe detailed design procedures for various foundation elements based on the recommended load-factor specifications, and include many examples demonstrating their use. The five engineering manuals cover the design of shallow foundations, driven piles, drilled shafts, retaining walls and abutments, and the estimation of tolerable bridge movements.

This report contains six major sections: the five engineering manuals and the recommended specifications and commentary. The engineering manuals will aid in the understanding of not only the new provisions but of foundation design in general and can be the basis for a future training program.

MANUALS FOR THE DESIGN OF BRIDGE FOUNDATIONS

SUMMARY

Until now, engineers who used AASHTO load factor design procedures for bridge superstructures have had to develop two sets of loadings, one for design of the superstructure and another for design of the foundation. This wasteful duplication of effort was unavoidable because load factor design procedures were not available for foundations. The study described in this report remedies this situation. A recommended accompanying AASHTO design code and commentary for foundations has been developed, based on load factor design procedures. The recommended code and commentary are included in Appendixes B and C.* The superstructure and the foundation can now be designed using the same loads and the same design format.

The recommended AASHTO design code has been made as similar as possible to the existing code. This was done to minimize the difficulties involved in working with the recommended code for engineers who are familiar with the existing code. Changes were made only where necessary to incorporate the load factor design format, to bring the code up to the current state-of-practice, or to remedy omissions in the existing code.

In addition to the draft design code and commentary, five engineering manuals have been developed during the course of this study. The purpose of these manuals is to describe in detail design procedures for foundations, and to give examples showing how the load factor design procedures that form the basis of the recommended code can be applied to foundation design.

These engineering manuals are:

Engineering Manual for Design of Shallow Foundations (Part 1)
Engineering Manual for Design of Driven Piles (Part 2)
Engineering Manual for Design of Retaining Walls and Abutments (Part 3)
Engineering Manual for Design of Drilled Shafts (Part 4)
Engineering Manual for Estimating Tolerable Movements of Bridges (Part 5)

The load factor design procedures described in the recommended AASHTO code and commentary employ the same load factors and load combinations that are used for superstructure design under AASHTO. Developing the load factor design procedures for foundations required extensive studies of margins of safety and reliability of foundations. Through these studies appropriate values were established for the performance (or resistance) factors that are used to modify the nominal capacities of foundations and thereby establish reduced levels of capacity that will result in reliable foundation performance. The details of these studies are presented in Appendix A of this report. A Synopsis, giving a brief account of the conduct of the research, findings, applications, conclusions and recommendations, immediately follows this Summary.

*Appendix B (Specifications) and Appendix C (Commentary) of the agency final report have been published in Part 6, as "Recommended Load Factor Design Specifications and Commentary." Note that these appendixes have been reproduced herein as submitted by the research agency; thus, none of the cross references to them within the published text have been altered in the editorial process so that accuracy of cross references can be retained.

SYNOPSIS OF THE RESEARCH

Problem Statement and Research Objectives

Until now, engineers who used AASHTO load factor design procedures for bridge superstructures have had to work with two sets of loadings, one for design of the superstructure and one for design of the foundation. This wasteful duplication of effort was because there were no load factor design procedures for foundations Under the AASHTO code, foundations could only be designed using the working stress design approach.

The objective of the research study described in this report was to develop recommendations for an AASHTO code for load factor design of foundations, in a form consistent with the AASHTO code for load factor design of superstructures.

The recommended design procedure for bridge foundations is expected to have the following benefits: (1) greater efficiency in the design effort because the same loads can be used for the superstructure and the foundation; (2) more consistent incorporation of margins of safety in the superstructure and foundation because they will be designed using the same loads and consistent design methods; and (3) more efficient use of materials because load factor design procedures afford a more consistent means for setting safety margins.

Because load factor design procedures offer these benefits, it is expected that they will be used widely when engineers become familiar with the method, and learn the advantages of using it.

Scope of Study

As originally proposed, the scope of this study encompassed only (1) development of a recommended AASHTO code for load factor design of bridge foundations and an accompanying commentary, and (2) documentation of the methods used in evaluating load factors and performance (or resistance) factors for design of foundations.

As the study progressed, it became clear that it would be desirable also to develop a more thorough exposition of design methods than would be suitable for the recommended code and its commentary, and to develop a series of examples of the use of the new procedures, so that engineers could understand the new procedures more easily, and more completely. To accomplish this goal, the scope of the study was broadened to include development of a series of five engineering manuals covering the design of foundations and abutments.

Thus, in its final form, the scope of the research study included development of these products: (1) documentation of the methods used in evaluating the load and performance (or resistance) factors used in the recommended code (this procedure, called "calibration" of the code, is described in Appendix A); (2) the recommended AASHTO code for load factor design of foundations, and the accompanying commentary (Appendixes B and C of this report, reproduced here in Part 6); and (3) five engineering manuals: Engineering Manual for Shallow Foundations (Part 1); Engineering Manual for Driven Piles (Part 2); Engineering Manual for Retaining Walls and Abutments (Part 3); Engineering Manual for Drilled Shafts (Part 4); and Engineering Manual for Estimating Tolerable Movements of Bridges (Part 5).

Research Approach

The principal steps involved in the research were: (1) development and distribution of a questionnaire to determine the extent of current use of LFD for highway structures, opinions regarding its use for foundations, and factors that would influence its adoption

by practitioners; (2) review of previous experience with load factor design, both published and unpublished; (3) development of a framework for applying load factor design methods to foundations; (4) review of the state of the art of foundation design and selection of design and selection of design procedures suitable for modern practice; (5) analysis of sources of uncertainty in foundation design and evaluation of load and resistance factors for the recommended load factor design code; (6) development of the engineering manuals, incorporating load factor design concepts and including examples illustrating the use of each of the included design procedures; and (7) development of the recommended code and commentary.

Findings

Survey of Practitioners. A questionnaire on load factor design for highway bridge structures was developed and distributed to the highway departments of the fifty states, the District of Columbia, Puerto Rico, and the U.S. Department of transportation. The return rate was 83 percent.

Of the respondents, 80 percent are currently using load factor design methods for highway structures. Seventy-three percent of the respondents indicated intentions to use load factor design for foundations when the new code becomes available. These responses indicated good potential support for load factor design. The most frequent suggestion from the respondents was that the new code should be simple, and easy to apply. Another frequent suggestion was that the new code should consider all types of foundations, soils, and structures.

The same questionnaire was sent subsequently to 106 consulting engineering design firms. The response rate from these firms was 29 percent, possibly indicating that matters of bridge design codes were of somewhat less vital interest to at least some of the firms that received the questionnaire. The respondents indicated a strong willingness to use load factor design methods if they were adopted by government agencies.

Load Factor Design Format. In the load factor design procedure, margins of safety are incorporated through load factors and performance (or resistance) factors.

Load factors (usually denoted by the symbol γ) account for uncertainties in the magnitudes of the loads that may be imposed on structures and foundations. The loads are modified by multiplying the nominal loads by the load factors, which usually have values larger than unity. The factored loads represent extreme values that have very low probabilities of occurrence.

Performance (or resistance) factors (usually represented by the symbol ϕ) account for uncertainties in the ability of foundations to support loads. The nominal capacity of the foundation is modified by multiplying it by a performance factor, which has a value less than unity. Performance factors account for such things as foundation soils that are weaker than expected, foundations that are not built in precise accordance with plans, and foundation materials (wood, concrete, steel) whose properties may fall short of specifications.

The basic requirement for foundation design in the load factor design format is expressed by the following equation:

$$\phi R_n \geq \Sigma \gamma_i Q_i \qquad (1)$$

where ϕ = performance factor
R_n = nominal resistance or load-carrying capacity of the foundation
γ_i = load factor for load component i
Q_i = load effect due to load component i

In basic terms, this equation expresses the notion that even in the highly unlikely situation where the load-carrying capacity of the foundation is very low, and, at the same time, the loads are very high, the capacity should still be great enough to carry the load.

Limit States. Limit states are limiting conditions of acceptable performance. These can correspond to complete failure, or collapse, or to less severe occurrences such as excessive deflection without failure or collapse. These are usually called "ultimate or strength limit states" and "serviceability limit states."

Strength Limit States correspond to mobilization of the maximum load-carrying capacity of the foundation, which may involve either structural failure of the foundation, or failure of the soil that supports the foundation. Reaching a strength limit state corresponds to complete collapse.

Serviceability Limit States correspond to the threshold of loss of some form of serviceability. For example, if a bridge settles so much that the deck does not drain as designed or there is a bump at the abutment that impairs ride quality or vehicle safety, the bridge will have lost some of its serviceability.

Load Factors. The new draft design code uses the same load factors for foundation design as are used for design of the superstructure. This choice of load factors has the advantage that the same factored loads can be used for both the structure and the foundation, and the design process is therefore more efficient and consistent.

Performance (or Resistance) Factors. Developing suitable values of performance factors for soil-related limit states of foundations was a major effort of this research study. Appendix A of this report documents the methods used in these evaluations.

Where possible, reliability analyses were performed to evaluate performance factors. The objective was to arrive at values of performance factors that correspond to the same level of safety, in terms of probability of failure, as do conventional working stress design procedures.

It was found to be necessary to perform separate studies, and to develop different performance factors, for each combination of foundation type, soil type, soil testing procedure, and method of calculating capacity. Thus, for example, driven piles, deriving support from sand, with capacity estimated using cone penetration test results, have a different value of performance factor than the same foundation with capacity estimated using Standard Penetration Test results. Similarly, piles in clay have different performance factors from piles in sand, and drilled shafts have different performance factors from driven piles.

Comparative analyses were performed to ensure that foundations designed using the new load factor design code will not differ greatly in size or cost from foundations designed using conventional working stress design methods.

Application to Bridge Foundation Design

Recommended AASHTO Design Code. Application of the results of this work to bridge foundation design will be through use of the recommended AASHTO design code. The code and commentary are contained in Part 6—Section 4, Foundations (spread footings, driven piles, and drilled shafts); Section 5, Retaining Walls; Section 7, Substructures (abutments).

As indicated previously, it is anticipated that this code will eliminate costly duplications of effort for engineers who use load factor design procedures for design of bridge superstructures. Gaps and deficiencies have also been addressed, to make the code consistent with the current state of practice in foundation design.

Design Manuals. The design manuals developed through this study contain: the load factors used in the recommended AASHTO code (the same load factors as for superstructure design); the performance factors contained in the recommended AASHTO code; descriptions and explanations of the design methods to which the performance factors apply; and examples illustrating the application of the recommended code and design methods.

These manuals will make it possible for practicing engineers to understand and use the recommended code quickly and efficiently. They represent much more focused and specific design aids than were available previously.

Conclusions and Suggested Research

Conclusions. The load factor design format is suitable for application to design of highway bridge foundations. The recommended code and commentary contained in Part 6 will make this possible, and will eliminate the need for the wasteful duplication of effort that arises when a bridge superstructure is designed by the load factor method and the foundation is designed by working stress design.

The greatest efficiency and consistency can be achieved by using the same load factor values for both structure and foundation. The recommended code uses the superstructure load factors for the foundation, thus making use of the code as simple, consistent, and efficient as possible.

Different values of performance factor are needed for each combination of foundation type, soil type, soil testing procedure, and method of calculating capacity. The recommended code contains values of performance factor for each of the design methods in current use in engineering practice, making it usable for a wide variety of different conditions.

The engineering manuals developed in the course of this study will provide an efficient and effective means for engineers to understand and to use the new code.

Suggested Research. During the course of this study it became evident that the performance of retaining walls and abutments has not been well documented. Design methods for these structures are largely empirical, and it is difficult for design engineers to anticipate performance with a reasonable degree of accuracy. Methods should be developed for estimating vertical movements, horizontal movements, and rotations of retaining walls and abutments, and these methods should be verified by comparison with the behavior of full-scale structures in the field. Research is also needed to develop a better understanding of the behavior of retaining walls and abutments during earthquakes, and to develop improved procedures for earthquake-resistant design of retaining walls and abutments.

APPENDIX A
PROCEDURES FOR EVALUATING PERFORMANCE FACTORS

Appendix A contained in the report as submitted by the research agency is not published herein. The table of contents is reproduced here for the convenience of those interested in the subject area. Qualified researchers may obtain loan copies by written request to the NCHRP, Transportation Research Board Business Office, 2101 Constitution Avenue, N.W., Washington, D.C. 20418.

CONTENTS

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The work was done under the supervision of Professors Barker, Duncan, and Rojiani.

Part 1—Engineering Manual for Shallow Foundations

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CONTENTS

CHAPTER 1

INTRODUCTION

The main purpose of this manual is to present simple guidelines for the analysis and design of shallow foundations in soil and rock. The emphasis is on simple and routine practical procedures, but not on detailed theoretical evaluations.

The design procedures included in this manual are presented using both the conventional working stress design and the recently introduced load and resistance factor design concepts. The two approaches differ in the manner in which uncertainties in design and the provision of safety margin are dealt with. Conventional design is essentially a deterministic approach; while load and resistance factor design is a semiprobabilistic approach. Although the two methods consider safety against failure differently, they treat serviceability considerations in a similar fashion.

Design considerations and various aspects of soil exploration for shallow foundations are discussed in Chapters 2 and 3 respectively. Methods for estimating bearing capacity and deformation of footings in soil are described in Chapters 4 and 5. The design of shallow foundations on rock is discussed in Chapter 6.

CHAPTER 2

DESIGN CONSIDERATIONS FOR SHALLOW FOUNDATIONS

2.1 DESIGN REQUIREMENTS

The function of a footing is to transmit loads from the structure to the supporting soil or rock without failure or excessive movements. If the footing is to fulfill this function successfully, it should be designed to resist all of the loads that it may be subjected to during its lifetime.

2.2 LOADS AND LOAD COMBINATIONS

The types of loads that must be considered in the design of a bridge foundation include: (1) dead loads, (2) live loads consisting primarily of traffic loads, (3) wind loads, (4) lateral earth pressures where applicable, e.g., for footings supporting retaining walls, (5) earthquake forces in seismically active areas, and (6) other environmental loadings, such as current and ice forces.

Most codes specify the types of loads and the load combinations to be considered in foundation design. For example, the recommendations of the American Association Of State Highway and Transportation Officials (AASHTO, 1989) are given in Table 2.1. The types of loads included in each of the load groups are the same for both service load allowable stress design and load factor design procedures. However, the two design approaches use different values of multiplication factors to determine the design loads. Values for service load design procedure are more appropriate for design for movement considerations (e.g., settlement calculation); values for load factor design procedure are applicable for failure considerations (e.g., ultimate bear-

Table 2.1. Load combinations in AASHTO specifications. (After AASHTO, 1989)

	Col. No.	1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14	
			β FACTORS														
	GROUP	γ	D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE	%	
SERVICE LOAD	I	1.0	1	1	0	1	β_E	1	1	0	0	0	0	0	0	100	
	IA	1.0	1	2	0	0	0	0	0	0	0	0	0	0	0	150	
	IB	1.0	1	0	1	1	β_E	1	1	0	0	0	0	0	0	**	
	II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	0	125	
	III	1.0	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125	
	IV	1.0	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125	
	V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	0	140	
	VI	1.0	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140	
	VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	0	133	
	VIII	1.0	1	1	0	1	1	1	1	0	0	0	0	0	1	140	
	IX	1.0	1	0	0	0	1	1	1	1	0	0	0	0	1	150	
	X	1.0	1	1	0	0	β_E	0	0	0	0	0	0	0	0	100	Culvert
LOAD FACTOR DESIGN	I	1.3	β_D	1.67*	0	1.0	β_E	1	1	0	0	0	0	0	0	Not Applicable	
	IA	1.3	β_D	2.20	0	0	0	0	0	0	0	0	0	0	0		
	IB	1.3	β_D	0	1	1.0	β_E	1	1	0	0	0	0	0	0		
	II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0		
	III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0		
	IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0		
	V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0		
	VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0		
	VII	1.3	β_D	0	0	0	β_E	1	1	0	0	0	0	1	0		
	VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1		
	IX	1.20	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1		
	X	1.30	1	1.67	0	0	β_E	0	0	0	0	0	0	0	0		Culvert

$(L + I)_n$ - Live load plus impact for AASHTO Highway H or HS loading
$(L + I)_p$ - Live load plus impact consistent with the overload criteria of the operation agency.

* 1.25 may be used for design of outside roadway beam when combination of sidewalk live load as well as traffic live load plus impact governs the design, but the capacity of the section should not be less than required for highway traffic live load only using a beta factor of 1.67. 1.00 may be used for design of deck slab with combination of loads as described in Article 3.24.2.2.

$$** \text{ Percentage} = \frac{\text{Maximum Unit Stress (Operating Rating)}}{\text{Allowable Basic Unit Stress}} \times 100$$

For Service Load Design

% (Column 14) Percentage of Basic Unit Stress

No increase in allowable unit stresses shall be permitted for members or connections carrying wind loads only.

β_E = 0.70 for vertical loads on Reinforced Concrete Boxes.
β_E = 1.00 for lateral loads on Reinforced Concrete Boxes.
β_E = 1.00 for vertical and lateral loads on all other structures.

For culvert loading specifications, see Article 6.2.

β_E = 1.0 and 0.5 for lateral loads on rigid frames (check both loadings to see which one governs). See Article 3.20.

For Load Factor Design

β_E = 1.3 for lateral earth pressure for retaining walls, reinforced concrete boxes, and rigid frames excluding rigid culverts.
β_E = 0.5 for lateral earth pressure when checking positive moments in either rigid frames or rigid culverts, including reinforced box culverts. This complies with Article 3.20.
β_E = 1.0 for vertical earth pressure
β_D = 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity.... For
β_D = 1.0 when checking member for maximum axial Column load and minimum moment.................. Design
β_D = 1.0 for flexural and tension members
β_E = 1.0 for Rigid Culverts
β_E = 1.5 for Flexible Culverts

For Group X loading (culverts) the β_E factor shall be applied to vertical and horizontal loads.

Determine values of loads to be used in design of footings to support a bridge abutment subjected to the AASHTO Load Group I.

(1) Types of loads in Group I

As can be seen from Table 2.1, the loads in Group I include dead load, live load, centrifugal force, earth pressure, buoyancy and stream flow pressure. These loads have values of β factor greater than zero. Following the AASHTO specifications, impact force is excluded in foundation design. In this example, it is also assumed that there are no loads resulting from buoyancy, centrifugal force or stream flow pressure.

(2) Nominal loads

Dead loads are calculated based on dead weights of the structure, foundation and surcharge materials. Live loads are estimated based on Section 3 of the AASHTO specifications. Earth pressures are estimated using established soil mechanics principle or empirical procedure, as appropriate.

(3) Factored loads

Factored loads are used in the LRFD procedures. Their values are obtained by multiplying the corresponding nominal values computed in Step (2) by the product of the corresponding values of γ and β factors from Table 2.1. The multiplication factors for the three types of loadings considered in this example are computed as follows:

Load Type	Multiplication Factor (= γ x β)	
	Service Load Design	Load Factor Design
Dead Load	1.0 x 1 = 1.0	1.30 x 1 = 1.30
Live Load	1.0 x 1 = 1.0	1.30 x 1.67 = 2.17
Earth Pressure	1.0 x 1 = 1.0	1.30 x 1.30 = 1.69

Figure 2.1. Example 2.1—determination of factored loads.

ing capacity). The use of the load combinations given in Table 2.1 for design is shown in Figure 2.1.

2.3 UNCERTAINTIES IN DESIGN

2.3.1 Sources of Uncertainties

The sources of uncertainties in foundation design can be grouped into four major categories: (1) uncertainties in estimating the loads, (2) uncertainties associated with the variability of the soil conditions at the site, (3) uncertainties in evaluation of the engineering properties of the soils and rocks at the site, and (4) uncertainties regarding the degree to which the analytical model represents the actual behavior of the foundation, structure and the soils and rocks that support it.

The uncertainties associated with the variability of soil conditions, and with evaluation of soil and rock properties, are usually the greatest. This is because the complex geological process involved with the deposition and formation of soil and rock introduces significant inherent variability in these materials.

To some extent, the foregoing uncertainties can be quantified explicitly or implicitly. In contrast, uncertainties associated with human errors or omissions, though they do occur in practice, are seldom quantified in design. They are usually accommodated by quality assurance programs, checking, or independent review.

2.3.2 Safety Margin and Safety Factor

For a safe design, a structure must have adequate capacity (or resistance) to resist the loads to which it is subjected. The reserve capacity, in excess of the required capacity, is the safety margin. An adequate margin is maintained in design by choosing conservative values of load and soil parameters for use in design, and by the use of appropriate safety factors.

The basis for establishing the values of design parameters and safety factors underlines the fundamental differences between the conventional working stress design (WSD) and the load and resistance factor design (LRFD). The WSD procedure is basically a deterministic approach, whereas LRFD is often based on semiprobabilistic concepts.

2.3.3 Working Stress Design

In working stress design both the loads and soil resistances are considered to be deterministic and are characterized in calculations by a single value, called the nominal value. The nominal value used in working stress design is usually either the mean value, or a value that is somewhat more conservative than the mean value. In selecting nominal values, the random nature of the loads and resistances is usually not taken into consideration.

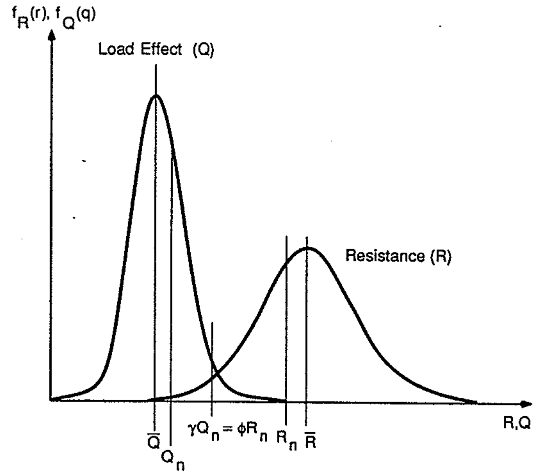
Selection of nominal values for the loads and resistance is an important initial step in design. Dead load can usually be predicted more accurately than live loads whose values are often chosen based on codes, laws, and experience. The selection of design soil parameters, on the other hand, requires careful appraisal of the conditions peculiar to the particular structure and site.

In the WSD procedure, safety is ensured by the use of a single factor of safety, sometimes called the "global" safety factor. An appropriate value of safety factors, which may be defined as the ratio of design resistance to the design load, is chosen based on the uncertainties associated with the design and the consequences of a failure. Typical values of safety factors customarily used in shallow foundations design are given in Table 2.2.

Table 2.2. Safety factors customarily used in foundation design. (After Terzaghi and Peck, 1967)

Failure Type	Failure Mode	Safety Factor*
Shearing	Bearing capacity failure	2.0 - 3.0
	Overturning	2.0 - 2.5
	Overall stability	1.5 - 2.0
	Sliding	1.5 - 2.0
Seepage	Uplift	1.5 - 2.0
	Heave	1.5 - 2.0
	Piping	2.0 - 3.0

*Note: The lower values are used when uncertainty in design is small and consequences of failure are minor; higher values are used when uncertainty in design is large and consequences of failure are major.



Notation:
$\bar{Q}$ = mean of load
$\bar{R}$ = mean of resistance
Q_n = nominal value of load
R_n = nominal value of resistance
$f_R(r)$ = probability density function of random variable R
$f_Q(q)$ = probability density function of random variable Q
γ = load factor
ϕ = performance factor

Figure 2.2. Load and resistance factor design.

2.3.4 Load and Resistance Factor Design

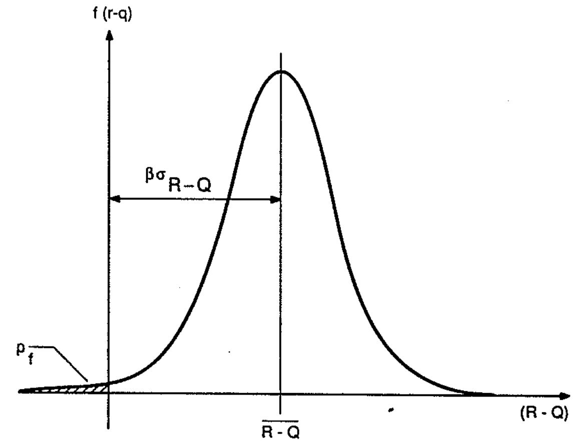
Load and resistance factor design is a recently developed method based on probability or reliability theory. The loads and resistance are treated as random variables and are characterized by probability density functions, as shown in Figure 2.2. Safety is defined in terms of the probability of survival or its complement, the probability of failure. The design is based on some acceptable probability of failure.

For given distributions of load and resistance, the probability of failure can be directly defined. For example, for the combined distribution of resistance minus load shown in Figure 2.3 the probability of failure is defined as the area under the shaded region. In LRFD several partial safety factors are employed to ensure that the probability of failure associated with the design is within the acceptable value. The two partial safety factors are the load and performance factors, as shown in Figure 2.2. The load factors, γ, which often have values larger than unity, account for the uncertainties in loads and their probability of occurrence. The performance (or resistance) factors, ϕ, which are typically less than one, account for soil variabilities and model uncertainties. The design equation for LRFD is as follows:

$$\phi R_n \geq \Sigma \gamma_i Q_i \qquad (2.3.4.1)$$

where ϕ = performance factor, R_n = nominal resistance, Q_i = load effect due to load component, and γ_i = load factor for load component i.

In practice values for the load and performance factors are usually specified in codes and are based on target values of reliability selected to be consistent with current state of practice.



Notation:
R = resistance
Q = load effect
β = safety index
p_f = probability of failure
σ_{R-Q} = standard deviation of random variable, R - Q

Figure 2.3. Definitions of probability of failure and safety index.

Different values of load and performance factors are provided for different limit states, such as ultimate and serviceability limit states.

Ultimate limit states are related to the strength of foundation, and they include bearing capacity failure, horizontal sliding, overturning, and overall stability.

Serviceability limit states are concerned with deformation and durability, and they include considerations of settlement, horizontal movement, tilting, and deterioration of the foundation materials.

Because the values of load and performance factors are intertwined, consistent sets of values must be used in design. For example, the suggested values of performance factor given in Table 2.3 must be used with the values of load factor for LFD given in Table 2.1. It would be inappropriate to use the load factors from Table 2.1 with performance factors taken from an unrelated source or vice versa.

It is important to note that values of load factor for earth pressure given in Table 2.1 can be used directly to amplify the magnitude of active and at-rest earth pressure. The magnitude of passive pressure, which provides a beneficial effect to the foundation system, should be multiplied by the reciprocal of the product of $\gamma \times \beta_E$ given in Table 2.1. It should also be noted that the table does not list recommended values for γ and β for water pressure. If the water pressure is evaluated based on the worst possible position of groundwater table (the highest likely in 100 years), it seems reasonable to use unfactored water pressure in LRFD calculations. Otherwise, the water pressure may be amplified by a load factor of 1.10.

2.4 OTHER DESIGN CONSIDERATIONS

2.4.1 Scour

Scour is the displacement of stream bed materials by the erosive action of stream or tidal currents. It may occur naturally or

Table 2.3. Suggested values of performance factor for ultimate limit states design for shallow foundations.

Type of Limit State	Performance Factor
1. Bearing Capacity	
a. Sand	
- Semi-empirical Procedure using SPT data	0.45
- Semi-empirical Procedure using CPT data	0.55
- Rational Method --	
using ϕ_f estimated from SPT data	0.35
using ϕ_f estimated from CPT data	0.45
b. Clay	
- Semi-empirical Procedure using CPT data	0.50
- Rational Method	
using shear strength measured in lab tests	0.60
using shear strength measured in field vane tests	0.60
using shear strength estimated from CPT data	0.50
c. Rock	
- Semi-empirical procedure (Carter and Kulhawy)	0.60
2. Sliding	
a. Precast concrete placed on sand	
using ϕ_f estimated from SPT data	0.90
using ϕ_f estimated from CPT data	0.90
b. Concrete cast in place on sand	
using ϕ_f estimated from SPT data	0.80
using ϕ_f estimated from CPT data	0.80
c. Clay (where shear strength is less than 0.5 times normal pressure)	
using shear strength measured in lab tests	0.85
using shear strength measured in field tests	0.85
using shear strength estimated from CPT data	0.80
d. Clay (where the strength is greater than 0.5 times normal pressure)	0.85

NOTE:

(1) ϕ_f = frictional angle of sand

(2) Sliding on clay is controlled by the strength of the clay when the clay shear strength is less than 0.5 times the normal stress, and is controlled by the normal stress when the clay shear strength is greater than 0.5 times the normal stress.

it may be the result of channel restriction or changes in flow pattern. Except for unusual circumstances, the greatest scour occurs during the largest flood.

Different materials scour at different rates. Loose granular soils are highly susceptible to scour, while cohesive or cemented soils are more resistant. Typical scour rates of some stream bed materials, expressed in terms of the time taken to reach the maximum scour depth, are listed below (AASHTO, 1970):

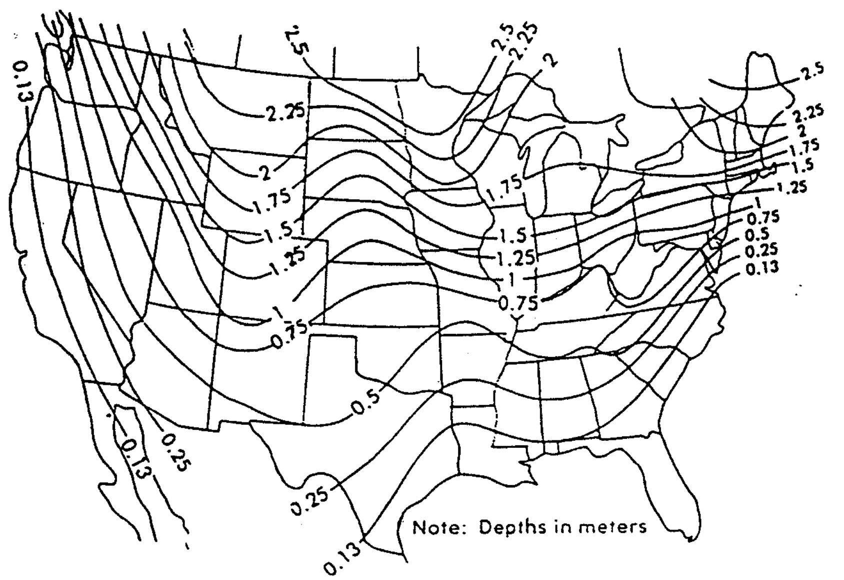


Figure 2.4. Maximum depth of frost penetration in the United States. (After Sowers, 1979)

Material	Time for Maximum Scour
sands & gravels	hours
cohesive soils	days
glacial tills, sandstones & shale	months
limestones	years
dense granites	centuries

For bridges at stream crossings, the potential for scour to undermine bridge foundations should be investigated. Bridge foundations are usually designed to withstand the effects of scour without failure for the worst conditions resulting from the 100-year flood. This usually involves designing the foundations for the after-scour conditions. The analyses are performed on the basis that all the stream bed materials within the scour prism have been removed, and are not available for bearing or lateral support.

The amount of scour depends on many factors, including the hydrological characteristics of the site, the hydraulics of the flow, and the properties of the streambed materials. Detailed discussion of scour can be found in the Technical Advisory on Scour at Bridges published by FHWA (1988), the NCHRP Synthesis of Highway Practice 5 (1970), and an FHWA report by Copp and Johnson (1987).

Scour prediction involves many disciplines of engineering, and prediction models still involve many uncertainties. Collaboration with other branches of engineering, including hydraulics engineering and hydrology, should therefore be sought. In spite of the recent progress made in scour prediction, experience with a given stream is still the best guide for estimating maximum depth of scour.

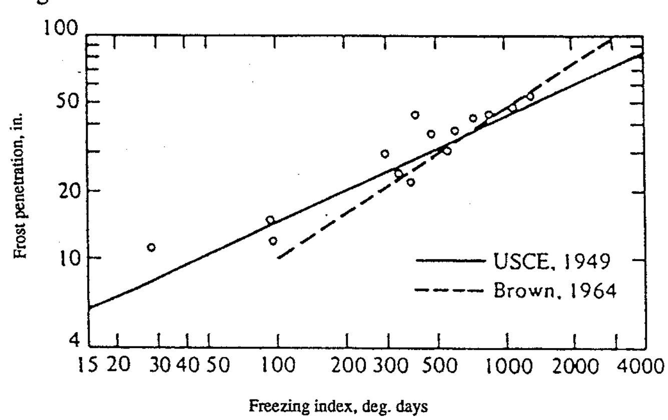
In the absence of detailed scour investigations, Terzaghi and Peck (1967) recommend that the foundation be placed at a depth not less than the elevation of the bottom of low-water channel plus four times the greatest rise of the river level.

Similarly, AASHTO specifications (1989) suggest the following guidelines for placing a foundation in cases where data pertaining to scour are not available: (1) for stream piers and arch abutments, minimum foundation depth > 6 ft below stream bed; (2) for other structures (except culvert), minimum foundation depth > 4 ft below stream bed.

In addition, special precautionary steps must be taken to protect spread footings founded on sand or other highly erodible soils. This may be achieved by paving the stream bed with concrete or by driving sheet piles around the footings.

2.4.2 Frost

In regions where freezing of the ground occurs during the winter months, shallow foundations should be founded below the maximum depth of frost penetration in order to prevent damage from frost heave. The maximum depth of frost penetration is generally estimated from local experience or from maps like the one shown in Figure 2.4. The U.S. Army Corps of Engineers proposed a relationship between frost penetration depth and the freezing index. The freezing index is defined by the equation: freezing index = number of days below 32°F × (32° − average daily temperature). The relationship is shown in Figure 2.5.



Freezing index = No. of days below 32° F X (32° - Average daily temp. in °F)

Figure 2.5. Design curves for maximum frost penetration based on the freezing index. (After U.S. Corps of Engineers, 1949; Brown, 1964)

2.4.3 Expansive and Collapsible Soils

Expansive and collapsible soils may be encountered in many parts of the United States, primarily the arid and semiarid regions of the West and Southwest. Expansive soils, usually highly plastic clays and clay shales, may undergo large volume changes as a result of seasonal changes in water content, and the expansion process may exert enormous swelling pressure on engineered facilities. Collapsible soils are, predominantly, partially saturated silts and lightly cemented sands. They may collapse when wetted.

In areas where these problem soils are found, information from other projects in the area and pertinent site-specific data, including groundwater information and index properties of soils, are usually available. Swelling potential can be estimated by using correlations with index properties, as shown in Figure 2.6. Collapsible soils can be identified by conducting special laboratory consolidation tests on undisturbed test specimens.

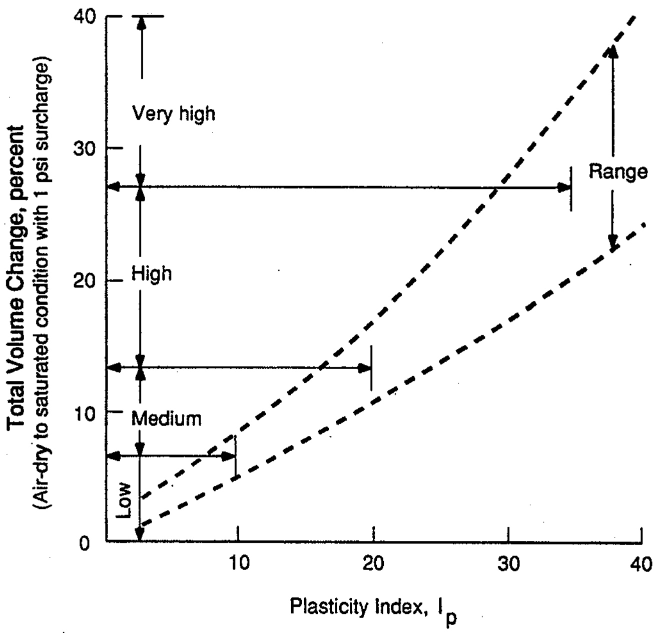


Figure 2.6. Relationship of plasticity index to swell potential of soils. (After Holtz and Gibbs, 1956)

2.4.4 Deterioration

Deterioration of concrete in foundations can be caused by sulfates, organic acids, and other corrosive compounds that are present in the soils or groundwater. The severity of the problem depends on three major factors: the concentrations of the sulfates, organic acids, and corrosive compounds; the level of the groundwater and its movements within the vicinity of the site; and the climatic conditions.

Geotechnical investigation for deterioration studies can be integrated into the subsurface exploration program through sampling and chemical analysis of the groundwater and the soils. Details can be found in the AASHTO *Manual on Subsurface Investigations* (1988).

Once the extent and the severity of the deterioration problem are identified, various measures can be adopted to protect concrete foundations from attack by aggressive agents. These include use of special materials, frequent maintenance, and conservative designs that deliberately disregard portions of the foundation material. The choice depends, among other factors, on the severity of the problem, the decay rates, and the cost.

In areas where sulfates and organic acids are known to be present, special types of cements are often recommended for protecting concrete from the attack of these agents. Detailed information can be found in Tomlinson's text (1986), and also in an American Concrete Institute (ACI) publication known as *ACI's Guide To Durable Concrete* (1982).

At chemical waste sites, appropriate precautionary steps can be taken after the source and nature of the aggressive compounds have been identified. The types, concentrations, and distributions of the deleterious chemicals may vary widely from site to site or even within the site itself, and each case must be evaluated on its own merits. Useful information can be found in the publications of Environmental Protection Agency (EPA).

2.5 CONSTRUCTION ASPECTS

Two aspects of construction are especially important in design. They are: constructability and the effects of construction activities.

The feasibility of constructing the foundation should be evaluated in terms of the difficulties involved. Important considerations include equipment access, storage and handling of excavated materials, feasibility of dewatering, stability of slopes during construction, and maintenance of essential functions during construction. Such factors often govern the design, and should receive thorough consideration early in the project.

Construction activities may alter the properties of soils and may even induce movements or failure. Examples include disturbance of clays due to pile driving, settlement of loose sands due to pile driving, piping or quick conditions resulting from dewatering, and damaging vibrations due to blasting. These effects often determine which construction methods can be used, and it is important to recognize that they may determine how construction can best be done.

CHAPTER 3

SOIL EXPLORATION FOR SHALLOW FOUNDATIONS

3.1 GENERAL

Accurate subsurface information is required for foundation design. Lack of such information may lead to construction disputes and claims, overly conservative designs with extremely high factors of safety, or to unsafe designs.

The field and laboratory investigations used to obtain subsurface information comprise the soil exploration program. It consists of borings and sampling, in situ testing, laboratory testing of soil samples, and is occasionally supplemented with geophysical and other techniques.

A site investigation generally involves three phases: (1) reconnaissance (2) exploratory investigation, and (3) intensive investigation. A reconnaissance study provides information useful for project feasibility, planning, and preliminary design. Foundation design data are obtained during the exploratory and intensive phases of the investigation.

This chapter briefly discusses various aspects of soil exploration, with emphasis on the exploratory and intensive phases of investigations. Detailed information concerning soil investigations can be found in many textbooks (Peck et al., 1974; Sowers, 1979) and in the AASHTO (1988) manual on the subject, and will not be repeated here.

3.2 OBJECTIVES OF SOIL EXPLORATION

The primary objectives of soil exploration are to determine the following: (1) the nature of the deposits, including their geologic origins and other factors that may affect their engineering behavior; (2) the aerial extent, depth, thickness, and elevation of each of the soil strata; (3) the depth to firm soil or rock; (4) the location of groundwater and its fluctuation, and the possible presence of artesian pressures; (5) the engineering properties of soils and rocks that will influence the performance of the foundation; and (6) other pertinent information, such as the chemical properties of soils and groundwater.

Acquisition and interpretation of this information help to define potential problems, to identify important details, and to identify areas where special attention is needed. It also provides the data needed for a design.

3.3 EXPLORATION PROGRAM

Soil exploration programs should be planned to obtain the maximum possible information at minimum cost. In planning an exploration program, it is important to consider the cost of site investigation in comparison with the cost of the foundation. A thorough investigation may result in substantial savings in the cost of a foundation in a particular area. In other cases, no amount of detailed information may change the type, cost, or performance of the foundation.

The planning of a soil investigation program includes both the field and laboratory work. It includes establishing methods for field exploration and in situ testing. It also includes determining the depth and location of borings, test pits, and other sounding techniques, as well as the type and number of laboratory tests. These decisions can only be made effectively after some knowledge of the site conditions is available. Planning is therefore a continuous and progressive process which involves updating or modifying a preliminary plan as work advances and more information is accumulated.

The scope and amount of work in an exploration program are dependent on many site-specific factors, including the type of structure and foundation, the soil conditions, and the project requirements. These factors and their degree of significance can vary so widely from site to site that each exploration program has to be planned individually.

Fortunately, guidelines have been developed over the years by various agencies to assist the planning of exploration programs. Based largely on experience and on some basic principles of soil mechanics, they typically include suggestions on bore hole or sounding spacings, exploration depths, and sampling requirements. For instance, the guidelines developed by the Federal Highway Administration (FHWA, 1985) are given in Table 3.1 and Table 3.2.

3.4 METHODS OF INVESTIGATION

3.4.1 Useful Exploration Techniques

Many techniques are available for exploring subsurface conditions at a site. These techniques differ mainly in the types of tools or equipment that they employ and in the manner used to advance the bore hole. The choice of the procedure or method to be used depends to a large extent on the depth and nature of the soils and the required quality of soil samples. Table 3.3 summarizes the use and limitations of some of the exploratory boring methods. It is intended for use as a quick reference. Details for each of these techniques are described in textbooks (Sowers, 1979; Tomlinson, 1986) and in the AASHTO manual (1988) on soil exploration.

3.4.2 Soil Sampling

Both disturbed and undisturbed samples provide useful information. Disturbed samples are samples that have been distorted and remolded. They are useful for soil identification and index tests, but not for measurement of soil properties. Undisturbed samples are obtained with thin-walled sampling tubes or from test pits. They are useful for all types of soil tests, including measurement of strength, compressibility, and permeability.

As a quick reference, Table 3.4 summarizes the use and limitations of various sampling techniques. For more detailed coverage, the readers are referred to Hvorslev's book (1948).

3.4.3 In Situ Tests

In recent years in situ tests have been used more frequently to determine the strength and deformation characteristics of soils. In many cases these tests provide considerable useful information at reasonable cost.

The use and limitations of several types of in situ tests are summarized in Table 3.5. Depending on whether soil properties are measured directly or are estimated by using empirical correlations, in situ tests may be classified into two major categories: direct tests and indirect tests. For example, the vane shear test is considered a direct test because strength of the soil is related quantitatively to the torque required to turn the vane. On the other hand, the standard penetration test measures the driving resistance of the split-spoon sampler, and is thus an indirect test.

It is important that all in-situ tests be carried out by experienced personnel and in accordance with the standardized or generally accepted procedures. Relevant standards are indicated in Table 3.5.

3.4.4 Groundwater

Reliable information on groundwater is essential for foundation design. In most cases the location of the groundwater level

Table 3.1. Guidelines for "minimum" boring programs. (After Federal Highway Administration, 1985)

Geotechnical Feature	Minimum Number Of Borings	Minimum Depth Of Borings
Structure Foundation	1 per substructure unit under 100 ft in width 2 per substructure unit over 100 ft in width	Advance Boring: (1) through unsuitable foundation soils (such as peats, highly organic clays, soft clays, etc.) into competent materials of suitable bearing capacity and : (2) to depth where added stresses due to estimated foundation load is less than 10% of the existing effective soil overburden stress or (3) minimum of 10 ft into bedrock, if encountered at shallower depth.
Retaining Walls	Borings spaced every 100 ft to 200 ft. Some borings should be in front and in back of wall	Extend borings to a depth of 2 times wallheight or minimum of 10 ft into bedrock.
Bridge Approach Embankments Over Soft Ground	When approach embankments are to be placed over soft ground, at least one boring should be made at each embankment to determine the problems associated with stability and settlement of the embankment. Typically, test borings taken for the approach embankments are located at proposed abutment locations to serve a dual function.	Same as established above for bridge foundation. Additional shallow exploration (hand auger holes) taken at approach embankment locations is an economical way to determine depth of unsuitable surface soils or topsoil.
Cuts and Embankments	Borings typically spaced every 200 ft (erratic conditions) to 500 ft (uniform conditions) with at least one boring taken in each separate landform. For high cuts and fills, should have a minimum of 2 borings along a straight line perpendicular to the centerline or planned slope face to establish geological cross-section for analysis.	Cuts: 1) In stable materials extend borings minimum 10 to 15 ft below grade. 2) In weak soils, extend borings below grade to: firm materials, or to the depth of cut below grade whichever occurs first. Embankments: Extend borings to firm material or to depth of twice the embankment height
Landslides	Minimum 2 borings along a straight line perpendicular to the centerline or planned slope surface to establish geological section for analysis. Number of sections depends on extent of stability problems. For active slide, place at least one boring above and below sliding area.	Extend borings to an elevation below active or potential failure surface and into hard stratum, or to a depth for which failure is unlikely because of geometry of cross-sections.
Materials Sites (Borrow Pits)	Borings spaced every 100 to 200 ft.	Extend exploration to base of deposit or to depth required to provided needed quantity.

Table 3.2. Guidelines for sampling and testing criteria. (After Federal Highway Administration, 1985)

Item	Recommendations
Sand- Gravel Soils	SPT (split-spoon) samples should be taken at 5-ft intervals or at significant changes in soil strata. Continuous SPT samples are recommended in the top 15 ft of borings made at locations where spread footings may be placed in natural soils. SPT jar or bag samples should be sent to lab for classification testing and verification of field visual soil identification.
Silty- Clay Soils	SPT and 'undisturbed' thin wall tube samples should be taken at 5-ft intervals or at significant changes of strata. Take alternate SPT and tube samples in same boring or take tube samples in separate undisturbed boring. Tube samples should be sent to lab for consolidation testing (for settlement analysis) and strength testing (for slope stability and bearing capacity analysis). Field vane shear testing is also recommended to obtain in situ shear strength of soft clays, silts and well rotted peats.
Rock	Continuous cores should be obtained in rock or shale using double or triple tube core barrels. In structural foundation investigations, core a minimum of 10 ft into rock to ensure that it is a bedrock and not a boulder. Core samples should be sent to the lab for possible strength testing (unconfined compression) for foundation investigation. Percent core recovery and RQD value should be determined in field or lab for each core run and recorded on boring log. However, it would be easier in field to distinguish breaks caused by drilling operations.
Ground Water	Water level encountered during drilling, at completion of boring, and at 24 hours after completion of boring should be recorded on boring log. In low permeability soils, such as silts and clays, a false indication of the water level may be obtained when water is used as drilling fluid and adequate time is not permitted after hole completion for the water level to stabilize (more than one week may be required). In such soils a plastic pipe water observation well should be installed to allow monitoring of the water level over a period of time. Seasonal fluctuation of water table should be determined where fluctuation will have significant impact on design or construction. Artesian pressure and seepage zones, if encountered, should also be noted on the boring log. The top foot or so of the annular space between water observation well pipes and borehole wall should be backfilled with grout, bentonite, or sand-cement mixture to prevent surface water inflow which can cause erroneous groundwater level readings.

Table 3.3. Methods for exploratory borings. (After Sowers, 1979)

Method	Procedure	Use	Limitations
Auger Boring (ASTM D-1452)	Hand or power auger with removal of material at regular short intervals	Identify changes in soil texture above water table. Locate ground water	Grinds soft particles-stopped by rock, etc.
Test Boring (ASTM D-1586)	Drill hole sample at intervals with 1.4 in ID and 2 in OD split barrel sampler driven 18 inches in three 6-in intervals by 140-lb hammer falling 30 in. Below water maintain hydrostatic balance with fluid.	Identify texture and structure, estimate density or consistency in soil or soft rock	Gravel, hard seams.
Continuous core: soil (ASTM D-2113)	Force or drill tube into soil until resistance prevents further movement. Remove cuttings with air or water.	Identify soil texture and structure continuously in cohesive soils.	Gravel, hard seams, sands. Misleading squeeze in some clays.
Borehole camera, TV	View inside of borehole	Examine stratification in place.	Textural changes indistinct.
Continuous core: rock (ASTM D-2113)	Rotate tube with diamond-studded bit to cut annular hole. Cuttings removed by circulating water. Core retained in tube by cylindrical wedge. Best with stationary inner tube to protect core.	Identify rock strata and structural defects continuously.	No data on soft seams, etc.
Pits, trenches	Excavate pit or trench by hand, larger auger and by excavator.	Visual examination of structure and stratification above water table.	Caving of walls Ground water.
*Wash boring; rotary wet drilling; rotary air drilling	Chop with chisel bit or rotate toothed cutter. Cuttings washed to surface by circulating water or drilling mud through bit.	Identify coarser fraction from cuttings, hardness from drilling rate.	Misleading if appreciable fines present
*Churn or cable drilling	Pound and churn soil boulders and rock to slurry by dropping heavy chisel bit in wet hole. Bail water and cuttings at intervals.	Drill and identify broken rock, etc., from cuttings	Strata difficult to define. Quick condition formed in sands.
*Percussion drilling	Impact-drill with jack hammer; remove cuttings with compressed air.	Identify rock from cuttings, hardness from rate.	Plugged by wet soil.

*Note: These methods are frequently used to advance the hole in test boring and core drilling; for quick but crude exploration; they are occasionally used independently.

Table 3.4. Use and limitation of soil sampling techniques. (Modified after Sowers, 1979)

Method	Sample Condition and Use	Limitations
Auger (ASTM D-1452)	Disturbed for soil identification; water content above water table.	Structure destroyed. Soil mixed with water below water table.
Split barrel (ASTM D-1586)	Intact but disturbed. Soil identification, structure, water content; density of very wide ranges of soils.	Sample distorted-disturbance too great for strength, consolidation tests.
Thin-walled tube (ASTM D-1587)	Relatively undisturbed sample for shear, consolidation,density etc., of most soils.	Sample lost in very soft clay or loose sand below water table.
	Stiff clays only (samples driven with hammer)	Slight disturbance.
Thin-walled tube, fixed piston.	Relatively undisturbed sample of very soft silts, clays; loose sand if hole filled with heavy drilling fluid.	Sample sometimes lost in soft clay, loose sand.
Swedish foil	Relatively undisturbed continuous (to 40 ft long) sample of soft clay, for shear, consolidation, etc.	Gravel, coarse sand, or hard strata strain damage sampler.
Rotary core: soil	Relatively undisturbed sample of firm to stiff cohesive soils, soft rock; continuous.	Torsion failure in soft soils and sometimes in sands.
Rotary core: rock	Continuous core in hard rock; nearly continuous in soft or fractured rock with M-type double tube.	Fractured or very soft rock not recovered.

Table 3.5. Use and limitations of in situ tests. (Modified after Sowers, 1979; Canadian Foundation Engineering Manual, 1985)

Method	Best Suited To	Not Applicable To	Properties Measured	Limitations
A. Direct Tests				
Vane shear (ASTM D-2573)	Clay	Silt, sand, and gravel.	Shear strength	Progressive failure in sensitive soils.
Load test (ASTM D-1194,	Soft rock, sand, and stiff clay	Soft clay	Ultimate bearing, short-term deflection.	Interpretation in terms of prototype difficult.
Well test	All soils	--	Effective horizontal permeability of mass.	Questionable above water table; not effective for vertical permeability.
Borehole shear	Sand, soft clay.	Stiff clay	Shear strength	Uncertainty in drainage.
B. Indirect Tests				
Standard Penetration Test (ASTM D-1586)	Sand, clay	Gravel	Estimate strength and/or density. Estimate bearing capacity.	Changes density of loose sands-sensitive to to procedure changes.
Static cone penetration test (ASTM D-3441)	Sand, clay	--	Estimate strength and/or density Estimate bearing capacity	Interpretation varies with soil; identi-fication of soils questionable; sensitive to procedure changes.
Pressuremeter Test (ASTM D-4719)	Soft rock, sand, clay	--	Estimate ultimate bearing capacity and compressibility	Interpretation difficult. Requires highly skilled personnel.
Dilatometer test	Sand, clay	--	Horizontal stress, soil stiffness	Introduced in U.S.A. in 1981.

Table 3.6. Use of routine laboratory soil tests. (Modified after Sowers, 1979)

Test	Soil Type	Sample Type	Use of Data
Specific gravity	All	Disturbed	Void ratio, minerals
Grain size	Sands, gravels	Disturbed	Classification. Estimate permea-bility, shear strength, frost action and compaction.
Grain shape	Sands, gravels	Disturbed	Classification. Estimate shear strength.
Liquid and plastic limits	silts, clays	Disturbed	Classification. Estimate compress-ibility and compaction.
Water content	clays	Disturbed	Correlate with strength, compressibility, and compaction.
Void ratio	clays	Disturbed	Estimate strength and compressi-bility.
Unconfined compression	clays sands	Undisturbed compacted	Estimate shear strength.
Triaxial compression	clays sands	Undisturbed Compacted	Estimate shear strength.
Direct shear	clays sands	Undisturbed Compacted	Estimate shear strength.
Consolidation	clays	Undisturbed	Estimate compressi-bility.

is measured during and at the end of the drilling. These observations may not provide useful information if drilling mud is used, or if the site has perched water table or artesian water pressure conditions. In such instances, observation wells or piezometers may be required.

3.4.5 Laboratory Tests

Laboratory tests are commonly performed to classify soils and to assess their engineering properties. Some of the laboratory soil tests used for foundation design are summarized in Table 3.6. Procedures for these tests are given in the standards published by AASHTO (1986) and ASTM (1990).

3.5 USEFUL CORRELATIONS

Many useful correlations have been established between the engineering properties of soils and various indirect and classification properties. For small projects or preliminary studies, such correlations are often used extensively. In other cases these correlations serve as alternative sources of design information.

Various types of correlations have been summarized by Sowers (1979), as shown in Table 3.7. An extensive collection of strength correlations has been compiled by Duncan et al. (1989). Some of the more widely used correlations are included in Figures 3.1 through 3.7 and Tables 3.7 through 3.12.

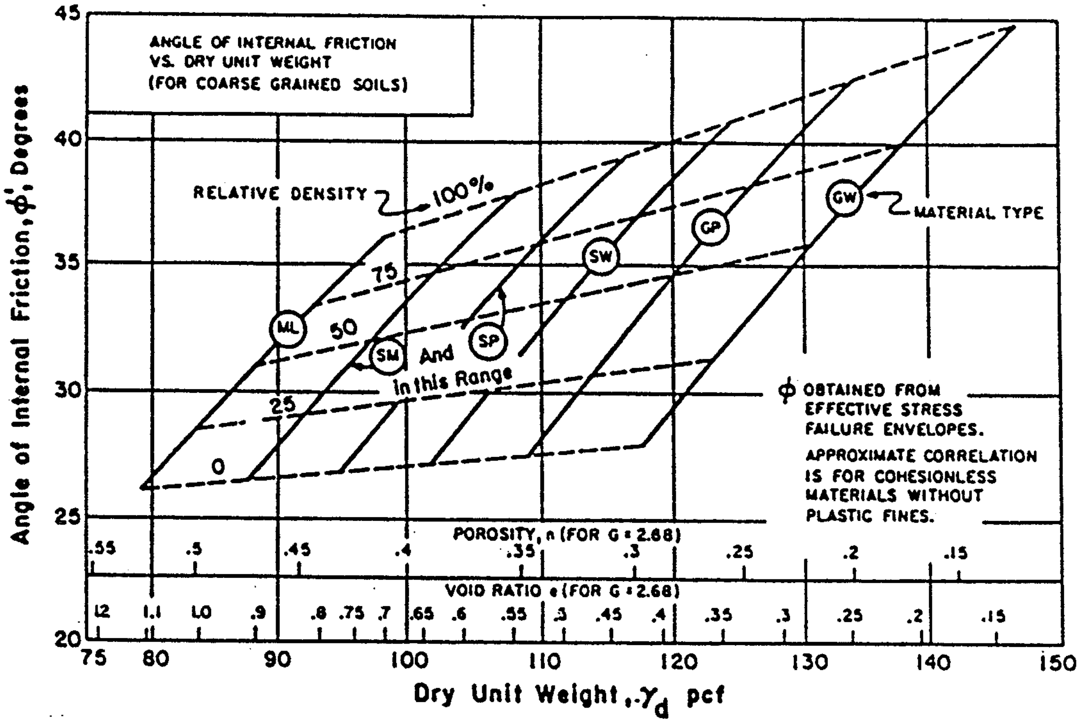


Figure 3.1. Approximate relationship between the angle of internal friction and the dry unit weight for granular soils. (After NAVFAC, 1982)

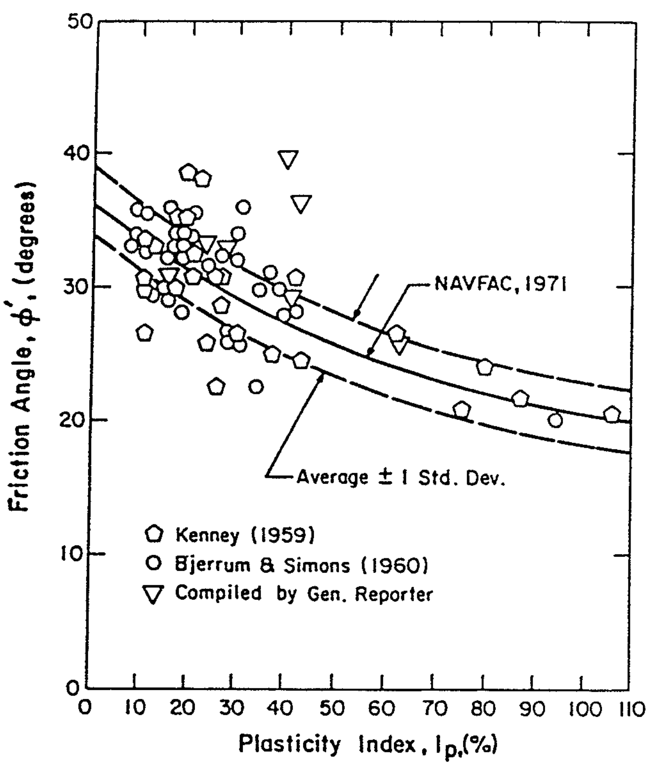


Figure 3.2. Correlation between peak effective friction angle and plasticity index for clays. (After Duncan et al., 1989)

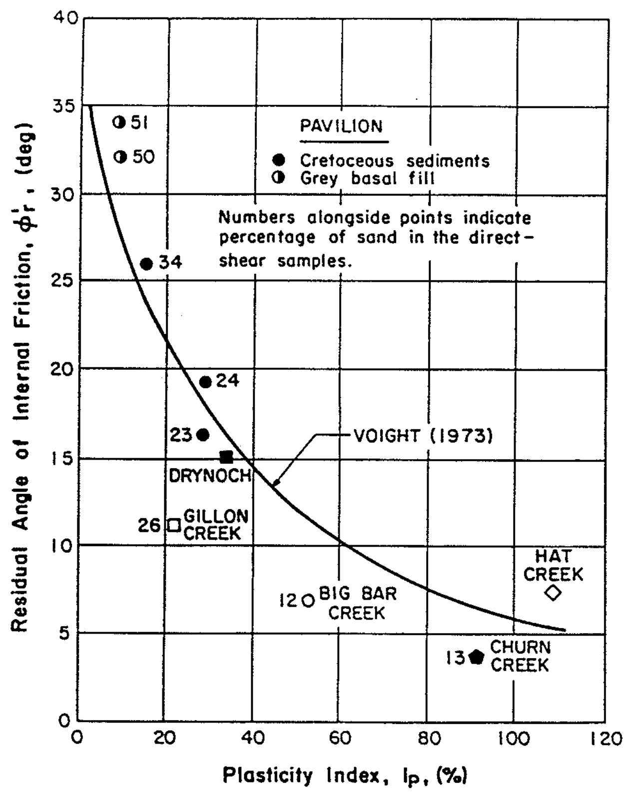


Figure 3.3. Relationship between residual angle of internal friction and plasticity index for clays. (After Duncan et al., 1989)

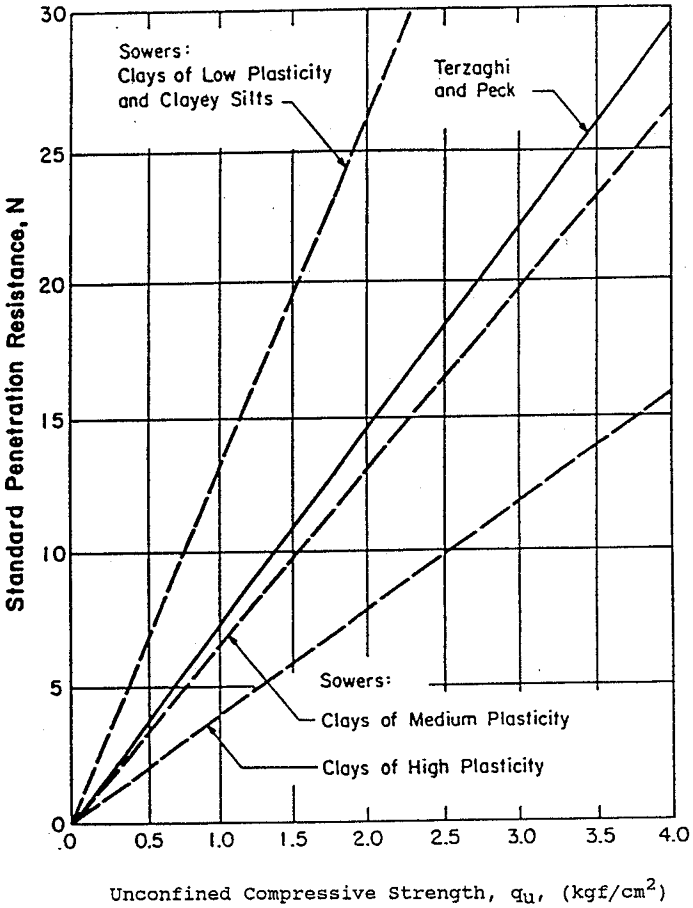


Figure 3.4. Relationship between standard penetration resistance, N, and unconfined compressive strength, q_u. (After NAVFAC, 1982)

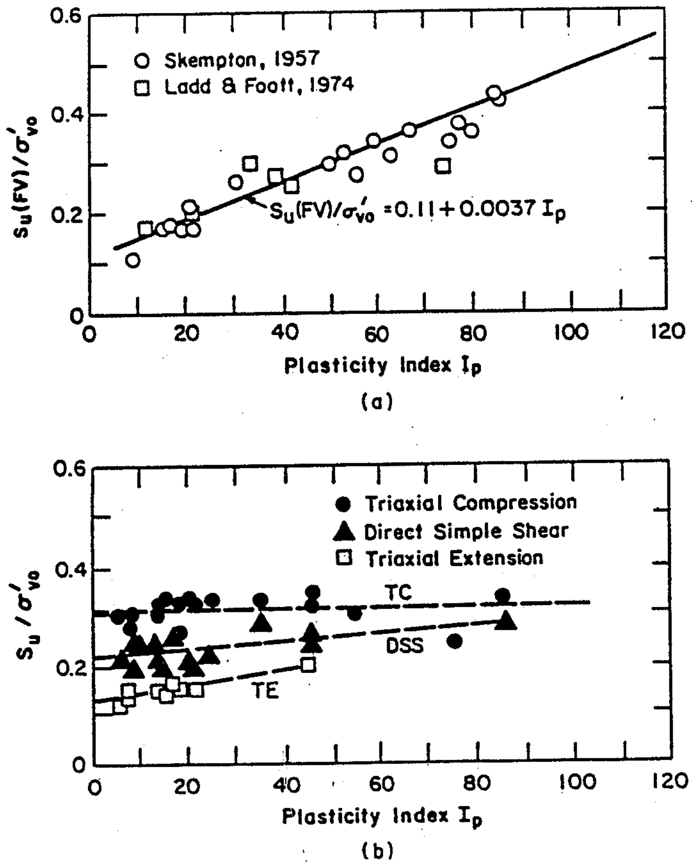


Figure 3.5. Variation of s_u/σ_{vo}' with plasticity index for normally consolidated clays. (After Robertson and Campanella, 1984, and Jamiolkowski, et al., 1985)

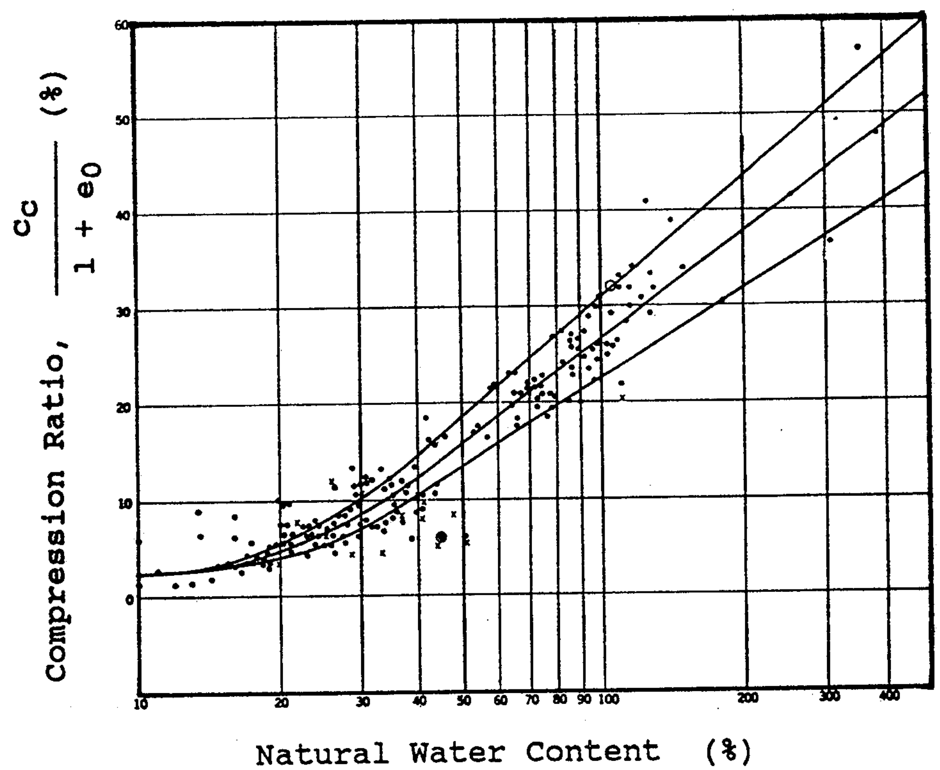


Figure 3.6. Relation between compression ratio and natural water content. (After Lambe and Whitman, 1969)

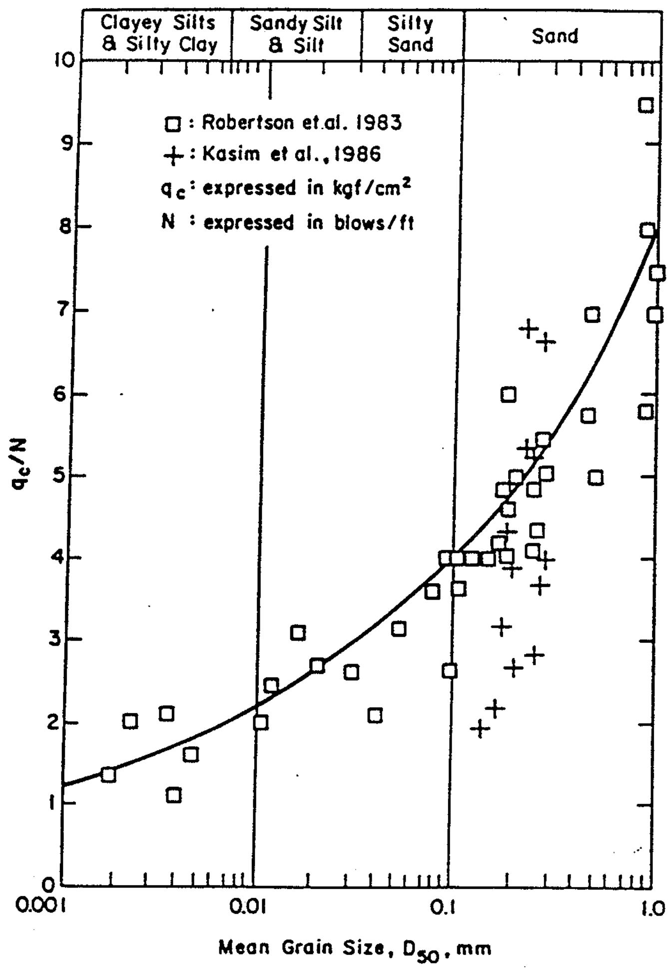


Figure 3.7. Variation of q_c/N with mean grain size. (After Robertson and Campanella, 1984, and Kasim et al., 1986)

Table 3.7. Correlation of test data. (After Sowers, 1979)

Simple Test	Possible Correlation
Water content	Shear strength of clay. Compression index of clay.
Grain size (D_{10}, D_{15}, C_u)	Permeability, strength, and drainability of cohesionless soils.
Liquid limit	Compressibility.
Plastic index	Swell-shrink, drained angle of shearing resistance of clay.
Void ratio, unit weight	Compressibility and shear strength.
Relative density (D_r)	Strength, compressibility of cohesionless soil.
Seismic velocity, (v)	Modulus of elasticity; strength of soil, rock.
Electrical resistivity	Water, clay, organic and salt content.
Penetration resistance, static and dynamic	Shear strength, relative density, modulus of compressibility.

Table 3.8. Relationship among relative density, penetration resistance, and angle of internal friction of cohesionless soils. (After Meyerhof, 1956)

State of Packing	Relative Density	Standard Penetration Resistance N	Static Cone Resistance q_c	Angle of Internal Friction ϕ'
	Percent	blows/ft	tsf or kgf/cm²	degrees
Very loose	< 20	< 4	< 20	< 30
Loose	20 - 40	4 - 10	20 - 40	30 - 35
Compact	40 - 60	10 - 30	40 - 120	35 - 40
Dense	60 - 80	30 - 50	120 - 200	40 - 45
Very Dense	> 80	> 50	> 200	> 45

Note: N = 15 + (N' - 15)/2 for N' > 15 in saturated very fine or silty sand, where N' = measured blow count, and N = blow count corrected for dynamic pore pressure effects during the SPT. The values of ϕ' are for clean sand.

Reduce ϕ' by 5 degrees for clayey sand.

Increase ϕ' by 5 degrees for gravelly sand.

Table 3.9. Approximate relation between undrained strength ratio, s_u/σ_{vo}', and overconsolidation ratio, OCR. (After Schmertmann, 1978)

S_u/σ_{vo}'	Approximate OCR	Remarks
0 - 0.1	less than 1	still consolidating
0.10 - 0.25	1	normally consolidated
0.26 - 0.5	1 to 1.5 (assume 1)	normally consolidated
0.51 - 1.0	3	overconsolidated
1 - 4	6	overconsolidated
over 4	greater than 6	overconsolidated

Note: Aging (or secondary compression), cementation and other processes may result in a higher overconsolidation ratio (apparent overconsolidation) although the clay remains normally consolidated. An aged clay is seldom found to have an OCR greater than about 1.5.

Table 3.10. Equations for stress-strain modulus, E_s, by several test methods. (Modified after Bowles, 1988)

Soil	SPT*	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $E_s = (15000 \text{ to } 22000)\ln N$	$E_s = 2 \text{ to } 4\ q_c$ $E_s = (1+D_r^2)q_c$
Sand (over-consolidated)	$E_s = 18000 + 750N$ $E_{sOCR} = E_{sNC}\ (OCR)^{0.5}$	$E_s = 6 \text{ to } 30\ q_c$
Gravelly sand and gravel	$E_s = 600(N + 6)$ $N \leq 15$ $E_s = 600(N + 6) + 2000$ $N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = 3 \text{ to } 6\ q_c$
Silty sand	$E_s = 300(N + 6)$	$E_s = 1 \text{ to } 2\ q_c$
Soft clay	---	$E_s = 3 \text{ to } 8\ q_c$
Clay	expressed in terms undrained shear strength, S_u; and in same pressure units as S_u. $I_p > 30$ or organic $E_s = 100 \text{ to } 500\ Su$ $I_p < 30$ or stiff $E_s = 500 \text{ to } 1500\ Su$ $E_{sOCR} = E_{sNC}\ (OCR)^{0.5}$	

Note:* Units of E_s is in kPa (1 tsf ≈ 100 kPa).

Table 3.11. Approximate correlation between coefficient of consolidation, c_v, and liquid limit. (After Terzaghi and Peck, 1967)

Liquid Limit (%)	Range of C_v (ft^2/yr)
30	20 to 360
40	12 to 230
50	8 to 150
60	5 to 90
70	3 to 53
80	2 to 35
90	1 to 23

Table 3.12. Correlation between cone resistance, q_c, and standard penetration resistance, N. (After Schmertmann, 1970)

Soil Type	q_c/N
Silts, sandy silts, slightly cohesive silt-sand mixtures	2.0
Clean, fine to medium sands, and slightly silty sands	3.5
Coarse sands and sands with little gravel	5.0
Sandy gravel and gravel	6.0

Note: Units of q_c are kgf/cm^2 or tsf;
Units of N are blows/ft.
N: Uncorrected.

CHAPTER 4

BEARING CAPACITY OF SHALLOW FOUNDATIONS IN SOIL

4.1 GENERAL

A basic requirement for any foundation is that it can safely support the load that it carries. The foundation itself must not suffer structural failure, and the soil beneath it must not be loaded so heavily that its supporting capacity is exceeded.

Structural failure in a foundation can be avoided by assuring that the foundation has sufficient shear and moment capacity to distribute the load it carries into the soil on which it rests. Failure of the soil beneath a foundation can be avoided by making the foundation large enough so that the stresses induced in the supporting soils are less than their shear strengths.

This chapter is concerned with methods of evaluating the ultimate bearing capacities of soil, and with the determination of bearing loads that provide an adequate margin of safety with respect to soil failure.

4.2 PRESUMPTIVE BEARING PRESSURES

The quickest (and least accurate) method of determining allowable bearing loads for foundations is the use of "presumptive bearing pressures." As may be seen from Table 4.1, these relate the allowable bearing pressure to the type of soil and a qualitative description of its condition, termed "consistency in place."

The procedure for designing a footage using presumptive bearing pressures is direct and simple. Having determined the allowable bearing pressure from information like that in Table 4.1, the designer can determine the required size of the footing by dividing the load supported by the footing by the allowable bearing pressure. Any footing with this or larger area is acceptable.

Although the use of presumptive bearing pressures is simple, it is not very accurate. It is not possible to determine the allowable bearing pressure accurately based only on the type of soil and a qualitative description of its consistency. Foundation designs based on presumptive bearing pressures are excessively conservative and wasteful in some cases, and unsafe in others. Presumptive bearing pressures are best used for preliminary estimates of foundation size, and for a rough check on bearing pressures derived using more reliable methods.

4.3 BEARING PRESSURES FROM STANDARD PENETRATION TESTS (SPT)

The standard penetration test is widely used for soil exploration. It provides a disturbed, but representative, sample for classification and a measure of soil strength and density through the number of blows, N, required to drive the sampler.

SPT blow counts can be used to estimate soil properties such as the undrained shear strength, s_u, of clays, and the angle of internal friction, ϕ, of sands. These can be used with bearing capacity theories to estimate ultimate bearing capacities for soils. Bearing capacity theories are discussed in a subsequent section.

SPT blow counts can also be used to estimate bearing pressures directly, through empirical correlations. Meyerhof (1956) proposed the following formula for estimating ultimate bearing pressure using SPT blow counts:

$$q_{ult} = \frac{\overline{N}B}{10}\left(C_{w1} + C_{w2}\frac{D_f}{B}\right) \tag{4.3.1}$$

where q_{ult} = ultimate bearing capacity (ultimate corresponds to failure of the soil beneath the footing), t/ft^2; $\overline{N}$ = average blow count, corrected for submergence effect; B = footing width (least dimension), ft; D_f = depth from ground surface to base of footing, ft; and C_{w1}, C_{w2} = correction factors whose values depend on the position of the water table:

Table 4.1. Presumptive allowable bearing pressures for spread footing foundations. (Modified after U.S. Department of the Navy, 1982)

Type of Bearing Material	Consistency in Place	Allowable Bearing Pressure (tsf) Ordinary Range	Allowable Bearing Pressure (tsf) Recommended Value for Use
Massive crystalline igneous and metamorphic rock: graphite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	60 to 100	80
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	30 to 40	35
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	15 to 25	20
Weathered or broken bedrock of any kind exept highly argillaceous rock (shale)	Medium hard rock	8 to 12	10
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	8 to 12	10
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC,SC)	Very dense	8 to 12	10
Gravel, gravel-sand mixtures, boulder-gravel mixtures (GW,GP, SW, SP)	Very dense Medium dense to dense Loose	6 to 10 4 to 7 2 to 6	7 5 3
Coarse to medium sand, sand with little gravel. (SW,SP)	Very dense Medium dense to dense Loose	4 to 6 2 to 4 1 to 3	4 3 1.5
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	3 to 5 2 to 4 1 to 2	3 2.5 1.5
Fine sand, silty or clayey medium to fine sand (SP,SM,SC)	Very dense medium dense to dense Loose	3 to 5 2 to 4 1 to 2	3 2.5 1.5
Homogeneous inorganic clay, sandy or silty clay (CL,CH)	Very stiff to hard Medium stiff to stiff Soft	3 to 6 1 to 3 0.5 to 1	4 2 0.5
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML,MH)	Very stiff to hard Medium stiff to stiff Soft	2 to 4 1 to 3 0.5 to 1	3 1.5 0.5

Note: These values should be used with service (or unfactored) loads.

for $D_w \geq D_f + 1.5B$, $C_{w1} = C_{w2} = 1.0$

for $D_w = D_f$, $C_{w1} = 0.5$ and $C_{w2} = 1.0$

for $D_w = 0$, $C_{w1} = 0.5$ and $C_{w2} = 0.5$

Values for other positions of the water table (between $D_w = 0$ and $D_w = D_f + 1.5B$) can be determined by interpolation.

Equation 4.3.1 applies only for vertically loaded footings. To account for the effect of load inclination, Eq. 4.3.1 may be written as:

$$q_{ult} = \frac{\overline{N}B}{10}\left(C_{w1} + C_{w2}\frac{D_f}{B}\right)R_I \qquad (4.3.2)$$

where R_I = load inclination factor from Table 4.2 (dimensionless).

The value of $\overline{N}$ is determined in two steps. First, in saturated very fine or silty sand, the measured SPT blow count is corrected for submergence effect as follows:

$$N = 15 + 1/2\,(N' - 15) \text{ for } N' > 15 \qquad (4.3.3)$$

where N′ = measured SPT blow count.

The average value of N is determined within the range of depth from the bottom of footing to a depth of 1.5B below the footing (Meyerhof, 1956). This average value used in Eq. 4.3.1 or 4.3.2 is also used to calculate footing settlement.

An example of the use of SPT data to estimate bearing capacity of footings on sand is given in Figure 4.1.

Table 4.2. Load inclination factor, R_I, for use with empirical procedures.

(i) For Square Footings

H/V	Load Inclination Factor, R_I		
	$D_f/B = 0$	$D_f/B = 1$	$D_f/B = 5$
0.10	0.75	0.80	0.85
0.15	0.65	0.75	0.80
0.20	0.55	0.65	0.70
0.25	0.50	0.55	0.65
0.30	0.40	0.50	0.55
0.35	0.35	0.45	0.50
0.40	0.30	0.35	0.45
0.45	0.25	0.30	0.40
0.50	0.20	0.25	0.30
0.55	0.15	0.20	0.25
0.60	0.10	0.15	0.20

(ii) For Rectangle Footings

H/V	Load Inclination Factor, R_I					
	Load Inclined in Width Direction			Load Inclined in Length direction		
	$D_f/B = 0$	$D_f/B = 1$	$D_f/B = 5$	$D_f/B = 0$	$D_f/B = 1$	$D_f/B = 5$
0.10	0.70	0.75	0.80	0.80	0.85	0.90
0.15	0.60	0.65	0.70	0.70	0.80	0.85
0.20	0.50	0.60	0.65	0.65	0.70	0.75
0.25	0.40	0.50	0.55	0.55	0.65	0.70
0.30	0.35	0.40	0.50	0.50	0.60	0.65
0.35	0.30	0.35	0.40	0.40	0.55	0.60
0.40	0.25	0.30	0.35	0.35	0.50	0.55
0.45	0.20	0.25	0.30	0.30	0.45	0.50
0.50	0.15	0.20	0.25	0.25	0.35	0.45
0.55	0.10	0.15	0.20	0.20	0.30	0.40
0.60	0.05	0.10	0.15	0.15	0.25	0.35

For the conditions shown in Figure 4.1a, estimate the bearing capacity of a 15 ft square footing using the results from SPT. The footing will be 5 ft below the ground surface, and a firm stratum is encountered at depth 45 ft below the bottom of the footing.

(1) Determine the minimum average value of SPT blow count within 1.5 B (i.e. to a depth of 23 ft) below the bearing level.

Depth (ft)	Boring 1			Boring 2			Boring 3		
	N	N'	$\bar{N}$	N	N'	$\bar{N}$	N	N'	$\bar{N}$
6	17	17	17	9	9	9	16	16	16
9	12	12	15	8	8	9	35	35	26
12	16	16	15	9	9	9	26	26	26
15	18	18	16	14	14	10	24	24	25
18	17	17	16	15	15	11	20	20	24
21	15	15	16	12	12	11	21	21	24
24	16	16	16	13	13	11	17	17	23
27	12	12	15	11	11	11	20	20	22

N' = Corrected N values for submergence effect; applicable only to silty sands encountered below the water table. Thus $\bar{N}$ = 11 (the minimum average value)

Figure 4.1. Example 4.1—estimating bearing capacity of a footing using standard penetration test result.

(2) Estimate ultimate bearing capacity, q_{ult}.

$$q_{ult} = \frac{\bar{N}B}{10}\left(C_{w1} + C_{w2}\frac{D_f}{B}\right)$$

Consider conditions where the water table is at depth 20 ft below the ground surface (i.e., highest position of groundwater recorded in the three borings), values of C_{w1} and C_{w2} are determined as follows:

$$C_{w1} = 0.5 + 15/22.5 \times 0.5 = 0.83$$

$$C_{w2} = 1.0$$

$$q_{ult} = \frac{11 \times 15}{10}\left(0.83 + 1 \times \frac{5}{15}\right) = 19.2 \text{ t/ft}^2$$

with a safety factor of 3.0, the allowable bearing capacity of the footing is:

$$q_a = \frac{q_{ult}}{3.0} = \frac{19.1}{3.0} = 6.4 \text{ t/ft}^2$$

Allowance for safety margin can also be provided through the use of load and resistance factors concept. The procedure for evaluating bearing capacity of soil using LRFD concept is discussed in Section 4.11.

(Note that settlement of the footing should be checked using the procedures given in Chapter 5 to ensure that its magnitude does not exceed the tolerance).

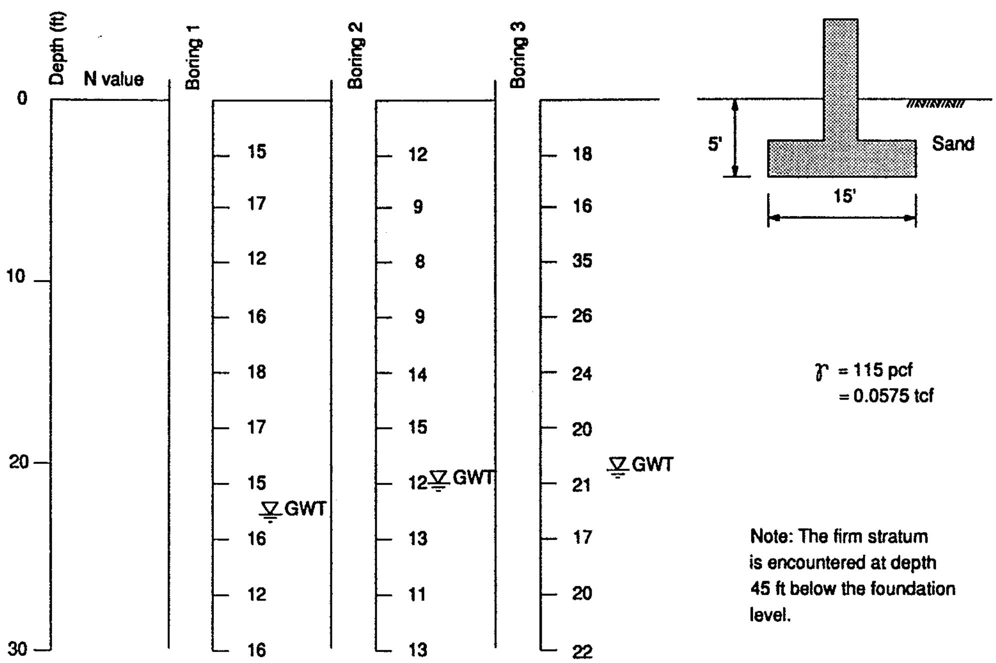


Figure 4.1a. Standard penetration test data for a housing developing site. (After Garga and Quin, 1974)

4.4 BEARING PRESSURES FROM CONE PENETRATION TESTS (CPT)

The cone penetration test has gained widespread acceptance for soil exploration in the United States. It provides a continuous record of resistance to penetration by a 10 cm^2 penetrometer and a measure of shaft friction on the cone shaft. Used in conjunction with conventional methods for drilling and sampling, it has proven to be a valuable in situ test.

Cone penetration resistance is the tip bearing pressure required to cause continuous penetration of the cone through the soil at a speed of 2 cm/sec. The tip resistance, q_c, is usually reported in kg/cm^2, which is essentially the same value when converted to tons/ft^2.

Values of q_c can be used to estimate soil properties such as the undrained shear strength, s_u, of clays and the angle of internal friction, ϕ, of sands. These can be used in the rational bearing capacity theories described subsequently in Section 4.6.

Cone penetration resistance can also be used to estimate bearing pressures directly, through empirical correlations.

Meyerhof (1956) proposed a relationship between ultimate bearing capacity and cone penetration resistance in sands. His equation can be modified to include the effect of load inclination, as given below:

$$q_{ult} = \frac{q_c}{40} B \left(C_{w1} + C_{w2} \frac{D_f}{B} \right) R_I \qquad (4.4.1)$$

where q_c = average value of cone penetration resistance measured within the range of depth from footing base to 1.5B below the footing; B = footing width, in ft; C_{w1} and C_{w2} are the water table correction factors discussed in connection with the standard penetration test, and R_I = load inclination factor from Table 4.2.

As reported by Schmertmann (1978), Awkati (1970) has correlated values of ultimate bearing capacity to cone penetration resistance in clays. Recommended values based on the chart he presented are summarized below:

	Value of q_{ult} (t/ft^2)	
q_c (kg/cm^2 or t/ft^2)	Strip Footing	Square Footing
10	5	9
20	8	12
30	11	16
40	13	19
50	15	22

These values are intended for use with footings that are below the ground surface.

It is not necessary to correct for the position of the groundwater table for footings on clay.

Procedures for estimating bearing capacity using cone penetration test data (CPT) are shown in Figure 4.2.

4.5 BEARING PRESSURES FROM PRESSUREMETER TESTS (PMT)

Pressuremeter tests are performed by inflating a membrane within a drilled hole and measuring the volume of expansion as a function of the applied pressure. Although not widely used in the United States at present (1990), it is gaining wider acceptance. The test is usually performed by specialty contractors who also provide recommendations regarding interpretation of the results. An excellent reference on the subject is the recent book by Briaud (1990).

Menard (1965), Baguelin et al. (1978), and Briaud (1986) suggested the following empirical relationship between ultimate bearing capacity and the limit pressure measured in the pressuremeter test:

$$q_{ult} = r_o + k\,(p_\ell - p_o) \qquad (4.5.1)$$

where r_o = initial total vertical pressure at foundation level, in pressure units; k = empirical bearing capacity coefficient from Figure 4.3, dimensionless; p_ℓ = limit pressure measured in the pressuremeter test, in pressure units; and p_o = total horizontal pressure at the depth where the pressuremeter test is performed, in pressure units.

Any consistent pressure units can be used in the calculations. An average value of limit pressure over the range of depth from 1.5B above to 1.5B below foundation level is commonly used in design. For cases where values of p_ℓ vary significantly within a depth B above or below the bearing level, Menard (1965) and Baguelin et al. (1978) recommended special averaging techniques that are based on experience.

4.6 BEARING CAPACITY THEORY

Saturated clays have undrained friction angles, ϕ_u, equal to zero. For these materials the ultimate bearing capacity is related to the undrained shear strength, s_u = c, by the following equation:

$$q_{ult} = c\,N_{cm} + \gamma\,D_f\,N_{qm} \qquad (4.6.1)$$

where c = s_u = undrained shear strength, in pressure units; N_{cm}, N_{qm} = modified bearing capacity factors which are functions of footing shape, embedment depth and load inclination, dimensionless; γ = total unit weight of clay, in weight per unit volume; and D_f = footing depth, in length units.

Brinch Hansen (1957) suggested the following expression for N_{cm} for footings with $D_f/B \leq 2.5$, $B/L \leq 1$ and $H/V \leq 0.4$:

$$N_{cm} = 5\left(1 + 0.2\frac{D_f}{B}\right)\left(1 + 0.2\frac{B}{L}\right)\left(1 - 1.3\frac{H}{V}\right) \qquad (4.6.2)$$

where L = length of footing, in length units; H = horizontal component of resultant load, in force units, V = vertical component of resultant load, same force units as H; and the other terms are as defined previously. For values of $D_f/B > 2.5$, the following expression is used:

$$N_{cm} = 7.5\left(1 + 0.2\frac{B}{L}\right)\left(1 - 1.3\frac{H}{V}\right) \qquad (4.6.3)$$

Equations 4.6.2 and 4.6.3 are valid for values of $H/V \leq 0.4$. For saturated clay $\phi_u = 0$, $N_{qm} = 1.0$.

Sands and gravels (cohesionless soils) have no cohesion (c =

The result of the cone penetration test performed adjacent to Boring 3 of Example 4.1 is shown in figure 4.2a. Based on this result, estimate the bearing capacity of the 15 ft square footing described in Example 4.1.

(1) Determine value of cone resistance for use in design.

Since there is only one test result, the value of q_c used in design will be the average of cone resistance measured within the range of depth from the bottom of the footing to a depth of 1.5B below the footing base. Thus, from Figure 4.2(a),

$$q_c \approx 120 \text{ kg/cm}^2 = 120 \text{ t/ft}^2$$

(2) Estimate ultimate bearing capacity.

$$q_{ult} = \frac{q_c B}{40}\left(C_{w1} + C_{w2}\frac{D_f}{B}\right)$$

Values of C_{w1} and C_{w2} are calculated as follows:

$$C_{w1} = 0.5 + \frac{20 - 5}{1.5 \times 15} \times 0.5 = 0.83$$

$$C_{w2} = 1.0$$

$$q_{ult} = \frac{120 \times 15}{40}\left(0.83 + 1.0 \times \frac{5}{15}\right) = 52.3 \text{ t/ft}^2$$

Using a safety factor of 3, the allowable bearing pressure from failure criterion is:

$$q_a = \frac{q_{ult}}{3.0} = \frac{52.3}{3} = 17.5 \text{ t/ft}^2$$

Note that the allowable bearing pressure determined using CPT result is about three times the value estimated using SPT results given in Example 4.1. This is not surprising because: (1) the soil conditions adjacent to Boring 3, where the CPT was performed, are generally better than those in Borings 1 and 2 (see Figure 4.1a), and (2) the allowable bearing pressure estimated using SPT data is based on minimum average value of SPT blow count which is much smaller than the average SPT blow count from Boring 3.

Figure 4.2. Example 4.2—estimating bearing capacity of a footing using cone penetration test data.

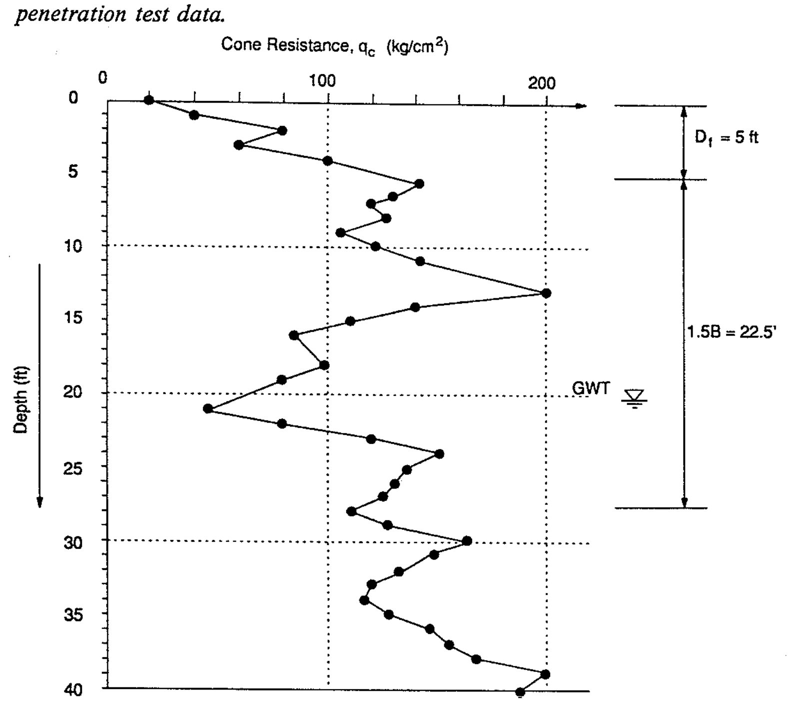
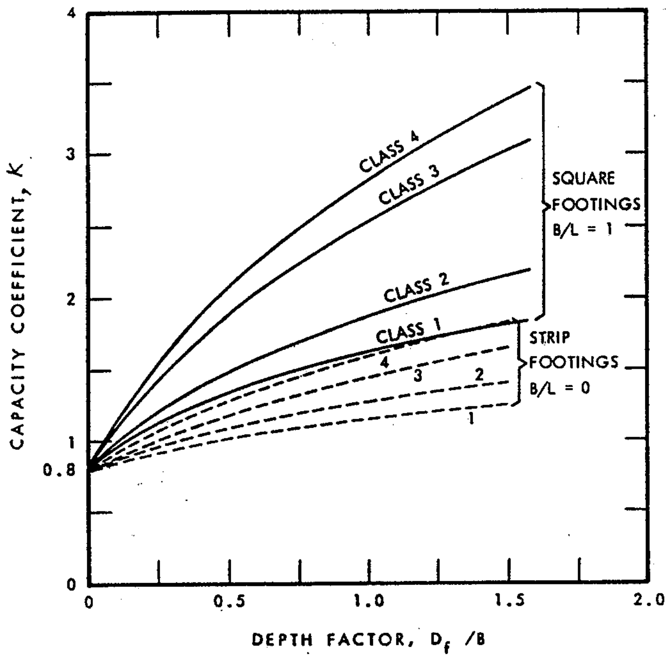


Figure 4.2a. Result of cone penetration test performed adjacent to boring 3 in Figure 4.1a. (After Garga and Quin, 1974)



Soil Type	Consistency or Density	$(P_\ell - P_o)(t/ft^2)$	Class
Clay	Soft to Very Firm	< 12	1
	Stiff	8-40	2
Sand and Gravel	Loose	4-8	2
	Medium to Dense	10-20	3
	Very Dense	30-60	4
Silt	Loose to Medium	< 7	1
	Dense	12-30	2
Rock	Very Low Strength	10-30	2
	Low Strength	30-60	3
	Medium to High Strength	$60\text{-}100^+$	4

Figure 4.3. Values of empirical capacity coefficient, k. (After Canadian Foundation Engineering Manual, 1985)

0). For these materials and conditions where the water table is deep ($D_w \geq D_f + 1.5B$), the ultimate bearing capacity can be expressed in the form:

$$q_{ult} = \frac{1}{2}\gamma B\, N_{\gamma m} + \gamma D_f\, N_{qm} \qquad (4.6.4)$$

where γ = total (moist) unit weight of soil; $N_{\gamma m}$ = dimensionless modified bearing capacity factor which is a function of ϕ, footing shape, soil compressibility, embedment depth, and load inclination; and N_{qm} = another dimensionless modified bearing capacity factor, which is also a function of ϕ, footing shape, soil compressibility, embedment depth, and load inclination.

Values of $N_{\gamma m}$ may be calculated using the expression

$$N_{\gamma m} = N_\gamma\, s_\gamma\, c_\gamma\, i_\gamma\, d_\gamma \qquad (4.6.5)$$

where N_γ is the dimensionless bearing capacity factor from Table 4.3; s_γ, c_γ, i_γ, and d_γ are correction factors that account for the effect of footing shape, soil compressibility, load inclination, and footing embedment. Values of s_γ, c_γ, and i_γ are given in Tables 4.4 through 4.6, and d_γ = 1.0 for all conditions.

Similarly, values of N_{qm} may be determined using the expression:

$$N_{qm} = N_q\, s_q\, c_q\, i_q\, d_q \qquad (4.6.6)$$

where N_q = another dimensionless bearing capacity factor whose value is also given in Table 4.3; s_q, c_q, i_q, d_q are correction factors accounting for the effects of footing shape, soil compressibility, load inclination, and embedment depth. Their values are given in Tables 4.4 through 4.7.

In deriving the values for c_c and c_q, the compressibility index as defined by Vesic (1973) was estimated using the empirical correlation:

$$I_r = 2D_r\sqrt{\frac{p_a}{\sigma_v'}} \qquad (4.6.7)$$

where I_r = compressibility index, dimensionless; D_r = relative density, in percent; p_a = atmospheric pressure, in pressure units; and σ_v' = effective overburden pressure, same pressure unit as p_a.

The empirical correlation for compressibility index given in Eq. 4.6.7 was established using a hyperbolic relationship to model the stress-strain behavior of the soil. Typical soil properties were obtained from Duncan et al. (1980).

The step-by-step procedure for estimating bearing capacity using rational theory is illustrated by an example in Figure 4.4.

4.7 EFFECT OF GROUNDWATER TABLE

Equation 4.6.4 applies to conditions where the depth to the groundwater table, D_w, is greater than $D_f + 1.5B$ below the ground surface. For cases where the groundwater is shallower, the following expression can be used:

$$q_{ult} = C_{wl}\,\frac{1}{2}\,\gamma\, BN_{\gamma m} + C_{w2}\gamma D_f N_{qm} \qquad (4.7.1)$$

where the values of C_{wl} and C_{w2} vary with D_w, as discussed in Section 4.3.

4.8 EFFECT OF LOAD ECCENTRICITY

If the load does not act at the centroid of the footing, a reduced effective footing area should be used. As shown in Figure 4.5, the width and length of the effective area are

$$B' = 2y \qquad (4.8.1a)$$

$$L' = 2x \qquad (4.8.1b)$$

where B' = reduced effective footing width, length units; L' = reduced effective footing length, length units; x = minimum distance from edge of footing to point of load application, length units; and y = least distance from edge of footing to point of load application, measured perpendicular to x, length units.

Table 4.3. Bearing capacity factors N_γ and N_q for sands and gravels. (After Vesic, 1973)

Friction Angle, ϕ (degree)	N_γ	N_q
28	17	15
30	22	18
32	30	23
34	41	29
36	56	38
38	78	49
40	110	64
42	155	85
44	225	115
46	330	160
48	495	220
50	760	320

Table 4.4. Shape factors s_γ and s_q for sands and gravels. (After Kulhawy et al., 1983)

Friction Angle (ϕ)	s_q			
	L/B = 1	L/B = 2	L/B = 5	L/B = 10
28	1.53	1.27	1.11	1.05
30	1.58	1.29	1.11	1.06
32	1.62	1.31	1.12	1.06
34	1.67	1.34	1.13	1.07
36	1.73	1.36	1.14	1.07
38	1.78	1.39	1.16	1.08
40	1.84	1.42	1.17	1.08
42	1.90	1.45	1.18	1.09
44	1.90	1.45	1.19	1.10
46	2.03	1.52	1.21	1.10
48	2.11	1.56	1.22	1.11
50	2.19	1.60	1.24	1.12

L/B	s_γ
1	0.60
2	0.80
5	0.92
10	0.96

Table 4.5. Compressibility factors C_γ and C_q for sands and gravels.

(i) Square footings

Relative Density (%)	Assumed Friction Angle (degree)	$c_\gamma = c_q$			
		q = 0.25 t/ft^2	q = 0.5 t/ft^2	q = 1 t/ft^2	q = 2 t/ft^2
0	0	1.00	1.00	1.00	1.00
20	30	1.00	1.00	0.92	0.89
30	32	1.00	1.00	0.85	0.77
40	35	1.00	0.97	0.82	0.75
50	37	1.00	0.96	0.81	0.73
60	40	1.00	0.86	0.72	0.65
70	42	0.96	0.80	0.66	0.60
80	45	0.79	0.66	0.54	0.48
100	50	0.52	0.42	0.35	0.31

(ii) Strip footings

Relative Density (%)	Assumed Friction Angle (degree)	$c_\gamma = c_q$			
		q = 0.25 t/ft^2	q = 0.5 t/ft^2	q = 1 t/ft^2	q = 2 t/ft^2
0	0	1.00	1.00	1.00	1.00
20	30	0.85	0.75	0.65	0.60
30	32	0.80	0.68	0.58	0.53
40	35	0.76	0.64	0.54	0.49
50	37	0.73	0.61	0.52	0.47
60	40	0.62	0.52	0.43	0.39
70	42	0.56	0.47	0.39	0.35
80	45	0.44	0.36	0.30	0.27
100	50	0.25	0.21	0.17	0.15

Table 4.6. Load inclination factors i_γ and i_q for sands and gravels. (After Kulhawy et al., 1983)

(i) Load Inclined in Direction of Footing Width

H/V	i_γ Strip	i_γ L/B = 2	i_γ Square	i_q Strip	i_q L/B = 2	i_q Square
0.10	0.73	0.76	0.77	0.81	0.84	0.85
0.15	0.61	0.65	0.67	0.72	0.76	0.78
0.20	0.51	0.55	0.57	0.64	0.69	0.72
0.25	0.42	0.46	0.49	0.56	0.62	0.65
0.30	0.34	0.39	0.41	0.49	0.55	0.59
0.35	0.27	0.32	0.34	0.42	0.49	0.52
0.40	0.22	0.26	0.28	0.36	0.43	0.46
0.45	0.17	0.20	0.22	0.30	0.37	0.41
0.50	0.13	0.16	0.18	0.25	0.31	0.35
0.55	0.09	0.12	0.14	0.20	0.26	0.30
0.60	0.06	0.09	0.10	0.16	0.22	0.25
0.65	0.04	0.06	0.07	0.12	0.17	0.21
0.70	0.03	0.04	0.05	0.09	0.13	0.16

(ii) Load Inclined in Direction of Footing Length

H/V	i_γ Strip	i_γ L/B = 2	i_γ Square	i_q Strip	i_q L/B = 2	i_q Square
0.10	0.81	0.78	0.77	0.90	0.87	0.85
0.15	0.72	0.68	0.67	0.85	0.81	0.78
0.20	0.64	0.59	0.57	0.80	0.74	0.72
0.25	0.56	0.51	0.49	0.75	0.68	0.65
0.30	0.49	0.44	0.41	0.70	0.62	0.59
0.35	0.42	0.37	0.34	0.65	0.56	0.52
0.40	0.36	0.30	0.28	0.60	0.51	0.46
0.45	0.30	0.25	0.22	0.55	0.45	0.41
0.50	0.25	0.20	0.18	0.50	0.40	0.35
0.55	0.20	0.16	0.14	0.45	0.34	0.30
0.60	0.16	0.12	0.10	0.40	0.29	0.25
0.65	0.12	0.09	0.07	0.35	0.25	0.21
0.70	0.09	0.06	0.05	0.30	0.20	0.16

Table 4.7. Depth factor, d_q, for sands and gravels. (After Brinch Hansen, 1970)

Friction Angle ϕ	D_f/B	d_q*
32°	1	1.20
	2	1.30
	4	1.35
	8	1.40
37°	1	1.20
	2	1.25
	4	1.30
	8	1.35
42°	1	1.15
	2	1.20
	4	1.25
	8	1.30

*Note: Values of d_q given in this table are applicable if the soils above the footing level are as competent as the soils beneath the footing level. If they are weaker, use d_q = 1.00.

Based on the information given in Figure 4.1a, estimate the bearing capacity of a 15' square footing using rational theory. The footing is founded 5' below the ground surface, and is subjected to vertical loadings only.

Based on the soil information, $\overline{N} = 11$ (see Example 4.1), indicating that $\phi \approx 35°$ (from Table 3.8).

From Table 4.3, values of N_γ and N_q are interpolated below:

$$N_\gamma = \frac{41 + 56}{2} = 49$$

$$N_q = \frac{29 + 38}{2} = 34$$

From Table 4.4, for Square footing, $S_\gamma = 0.60$ and

$$S_q = \frac{1.67 + 1.73}{2} = 1.70$$

Effective overburden pressure at depth $\frac{B}{2}$ below the footing base is:

$$\sigma'_v = (0.0575)\left(5 + \frac{15}{2}\right) = 0.72 \text{ t/ft}^2$$

From Table 4.5, values of C_γ and C_q are interpolated as:

$$C_\gamma = C_q = 0.97 + \left(\frac{0.82 - 0.97}{1.0 - 0.5}\right) \times (0.72 - 0.05) = 0.90$$

Since the soil above the footing level is less competent than the soil beneath the footing level, and it is also likely to be disturbed during excavation, use $d_q = 1.0$.

For footings subjected to vertical loadings only, $i_\gamma = i_q = 1.0$.

$$N_{\gamma m} = N_\gamma \cdot S_\gamma \cdot C_\gamma \cdot i_\gamma \cdot d_\gamma$$

$$N_{\gamma m} = (49)(0.60)(0.90)(1.0)(1.0) = 26.5$$

$$N_{qm} = N_q \cdot S_q \cdot C_q \cdot i_q \cdot d_q$$

$$N_{qm} = (34)(1.70)(0.90)(1.0)(1.0) = 52.0$$

The ultimate bearing capacity, q_{ult}, is then

$$q_{ult} = \frac{1}{2}\gamma B\, C_{w1}\, N_{\gamma m} + \gamma D_f C_{w2} N_{qm}$$

Values of Cw1 and Cw2 are interpolated as:

$$C_{w1} = 0.5 + \frac{20 - 5}{1.5 \times 15} \times (1.0 - 0.5) = 0.83$$

$$C_{w2} = 1.0$$

$$q_{ult} = \frac{1}{2} \times 0.0575 \times 15 \times 0.83 \times 26.5 + 0.0575 \times 5 \times 1.0 \times 52.0$$

$$q_{ult} = 24.4 \text{ t/ft}^2$$

With a safety factor of 3, the allowable bearing pressure is

$$q_a = \frac{q_{ult}}{3} = \frac{24.4}{3} = 8.1 \text{ t/ft}^2$$

Figure 4.4. Example 4.3—estimating bearing capacity of a footing on sand based on rational theory.

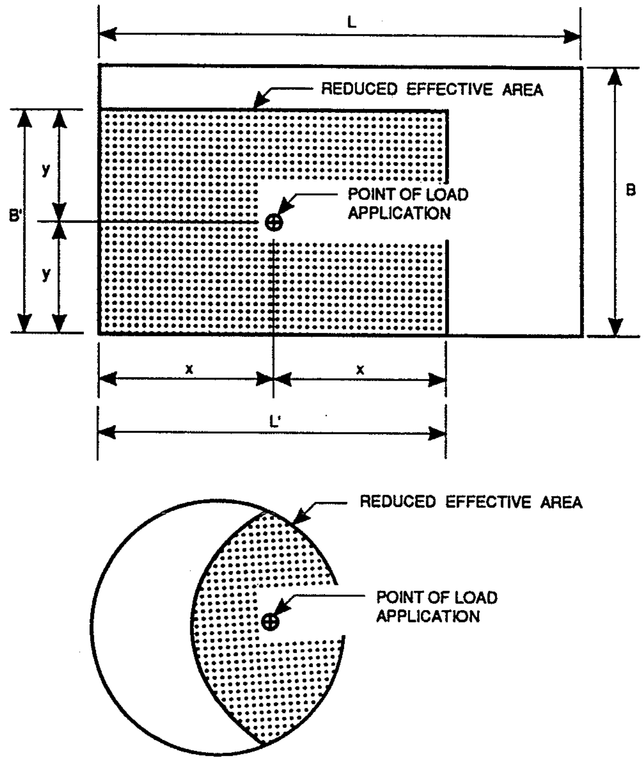


Figure 4.5. Reduced effective areas for eccentrically loaded footings.

For footings that are not rectangular, such as the circular footing at the bottom of Figure 4.5, the effective area can be estimated using simple approximations and judgment. The reduced effective area is always concentrically loaded, so the bearing pressure on the reduced effective area is always uniform.

An example of estimating bearing capacity of a footing subject to eccentric and inclined loads is given in Figure 4.6.

4.9 OTHER CONDITIONS

Theories have been developed to calculate bearing capacities for footings with various types of special conditions. Among these are foundations on slopes or adjacent to slopes (Meyerhof, 1953), footings on layered soils (NAVFAC-DM7, 1982; Meyerhof, 1957), and foundations with inclined bases (Kulhawy, et al., 1983).

4.10 SLIDING RESISTANCE

Footings that support inclined loads and footings on slopes may slide on the underlying soil. The safety of such footings against sliding should be evaluated. Sliding will not occur if the shear strength at any point on the assumed slip surface (usually the interface between the foundation and soil) exceeds the shear stress at the same slip surface under service loading conditions.

If the soil beneath the footing is sand, the sliding capacity can be estimated using the following expression:

$$Q_\tau = V \tan\delta \qquad (4.10.1)$$

where Q_τ = sliding capacity of the footing, in force units; V = resultant vertical force on the slip surface under service loading conditions, in force units; and δ = angle of friction between soil and foundation material along the slip surface.

If the soil beneath the foundation is sand and the base of the footing is rough (the usual case for concrete cast against soil), sliding is resisted by the full shear strength of the soil, and $\tan\delta = \tan\phi$, where ϕ is the angle of internal friction. For precast concrete footings, which may be smoother, $\tan\delta = 0.8 \tan\phi$ should be used for design (Potyondy, 1961).

If the soil beneath the footing is clay, consideration should be given to the possibility of sliding by shear within the clay as well as sliding on the interface between the footing and the clay. The maximum sliding resistance should be taken as one-half the

Based on the information given in Figure 4.1a, estimate bearing capacity of a 15 ft square footing which is subjected to a load inclined at 10° from the vertical and with an eccentricity of 3 ft (See the sketch below). The footing is founded 5' below the ground surface.

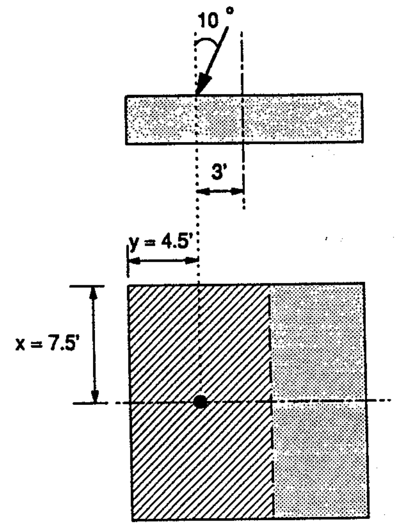
Effective dimension of the footing

$$B' = 2y = (2)\ (4.5) = 9'$$

$$L' = 2x = (2)\ (7.5) = 15'$$

Load inclination angle, $\alpha = 10°$

$$H/V = \tan \alpha = \tan 10° = 0.18$$



Based on the soil information from Figure 4.1a, $\overline{N} = 11$.

Vertical component of ultimate bearing capacity is,

$$q_{ult_{vert}} = \frac{\overline{N}B'}{10}\left(C_{w1} + C_{w2}\frac{D_f}{B'}\right)$$

Since the water table is located at depth greater than 1.5B' (i.e., 13.5ft) below the footing base, $C_{w1} = C_{w2} = 1.0$

$$q_{ult_{vert}} = \frac{11 \times 9}{10}\left(1 + 1 \times \frac{5}{9}\right) = 15.4\ t/ft^2$$

Reduction in bearing capacity due to the effect of load inclination from Table 4.2 (ii):

	Value of Reduction Factor, R_I		
H/V	$D_f/B' = 0.0$	$D_f/B' = 0.55$	$D_f/B' = 1.0$
0.15	0.60	0.63	0.65
0.20	0.50	0.56	0.60

Value of R_I for H/V = 0.18 and Df/B' = 0.55 is interpolated as:

$$R_I = 0.63 + \left(\frac{0.56 - 0.63}{0.20 - 0.15}\right)(0.18 - 0.15) = 0.59$$

$$q_{ult} = R_I\ q_{ult_{vert}} = (0.59)\ (15.4) = 9.1\ t/ft^2$$

with a safety factor of 3, the vertical component of allowable bearing pressure from strength consideration is,

$$q_a = q_{ult}/3.0 = \frac{9.1}{3.0} = 3.0\ t/ft^2$$

and the allowable vertical load is,

$$Q_a = q_a \times B' \times L' = (3.0)\ (9)\ (15) = 405 \text{ tons}$$

Figure 4.6. Example 4.4—estimating bearing capacity of a footing subjected to eccentric and inclined load using SPT data.

normal stress on the interface between the concrete and the clay, or the adhesion of the clay, whichever is smaller. For cast in place concrete the adhesion may be taken as the full undrained shear strength of the clay. The adhesion may be reduced to 0.5 to 0.7 times the undrained strength if the concrete is wet.

Procedures for evaluating safety of a footing against sliding are illustrated by an example in Figure 4.7.

4.11 SAFETY FACTORS, LOAD FACTORS, AND RESISTANCE FACTORS

Good design requires a margin of safety between the actual loading conditions and the loading conditions that would cause failure. A margin of safety can be achieved using a safety factor, which is defined as follows:

$$F = \frac{q_{ult}}{p} \tag{4.11.1}$$

where F = safety factor, dimensionless; q_{ult} = ultimate bearing capacity, in pressure units; and p = bearing pressure under nominal load conditions, in pressure units.

For footings supporting bridges and buildings, the factor of safety should be 2.5 to 3.0, or higher.

The factor of safety against sliding can be expressed as

$$F = \frac{s}{\tau} \tag{4.11.2}$$

where s = shear strength of the interface between footing and soil, or soil and soil, in pressure units; and τ = shear stress on the interface under service load condition, in pressure units. Conventional design practice requires the factor of safety against sliding to be 1.5 or higher.

An alternate method of providing a margin of safety is through the use of load factors (commonly called γ by structural engineers) and performance factors (commonly called ϕ by structural engineers). The requirement of good design can be expressed as

$$\phi\, q_{ult} \geq \gamma\, p \tag{4.11.3}$$

where ϕ = performance factor, dimensionless; γ = load factor, dimensionless; and p = bearing pressure under nominal load conditions, in pressure units.

Load factors reflect the uncertainties in the magnitudes of the loads to which the footings are subjected. Values of γ are on the order of 1.1 to 1.3 for dead loads, and are on the order of 2.0 for live loads. Values of load factors recommended by AASHTO (1989) can be found in Table 2.1. Performance factors reflect the uncertainties in soil strength values. Typical values of ϕ range from 0.35 to 0.90 for soils, depending on soil type and method of strength determination. Suggested values of performance factors for shallow foundation design are provided in Table 2.3.

Designs based on safety factors are equivalent to designs based on load and performance factors if the following relationship is satisfied:

$$F = \frac{\gamma}{\phi} \tag{4.11.4}$$

Calculate the safety against sliding for the footing described in Example 4.4. The resultant load on the footing has a vertical component of 120 tons and a horizontal component of 21 tons.

From the soil information given in Figure 4.1a, $\overline{N} = 11$.

For $\overline{N} = 11$, $\phi_f \approx 35°$

The concrete footing will be cast in situ, thus

$\delta = \phi_f = 35°$

The sliding resistance is

$Q_\tau = V \tan\delta = (120)(\tan 35°) = 84$ tons

The horizontal load on the footing is

H = 21 tons

Factor of safety against sliding is

$F = \frac{Q_\tau}{H} = \frac{84}{21} = 4.0 > 1.5$ (O.K.)

Alternatively, the safety margin may be provided through the use of load and resistance factors. Procedures for the LRFD approach are shown in Figure 4.9: Example 4.7.

Figure 4.7. Example 4.5—checking the safety of a footing on sand against sliding.

The use of separate load and performance factors is logical because loads and resistance have separate and unrelated sources of uncertainty. Using separate load and performance factors is a convenient and rational way of accounting for the sources of uncertainty in design.

In Figure 4.8 the load carrying capacity of the footing discussed in Figure 4.2 is recalculated using the load and resistance factor design concept. The LRFD procedures for checking slid-

Using load and resistance factor concepts, determine whether a 15 ft square footing carrying a service dead load of 250 tons and a service live load of 50 tons has adequate soil bearing capacity. The footing is embedded 5 ft below ground surface, and the soil information is given in Figure 4.1a.

(1) Calculate the factored load.

Load factors, $\gamma_D = 1.3$, $\gamma_L = 2.17$

Factored load $= \gamma_D P_D + \gamma_L P_L = (1.3)(250) + (2.17)(50)$

$= 435$ tons

(2) Estimate ultimate bearing capacity.

The ultimate bearing capacity may be determined using any method based on SPT results. If Meyerhof's method is used, as in Example 4.1,

$q_{ult} = 19.2$ t/ft^2

(3) Calculate factored load carrying capacity

Using $\phi = 0.45$, the factored bearing pressure is

$q_{fult} = \phi\, q_{ult} = (0.45)(19.2) = 8.6$ t/ft^2.

Magnitude of factored load carrying capacity is

$Q_{fu} = q_{fult} \cdot A = (8.6)(15)(15) = 1935$ tons.

Since the factored load carrying capacity (Q_{fu} = 1935 tons) is considerably greater than the factored load (P_f = 435 tons) the footing has adequate capacity with regard to soil failure. However, it is likely that the design would be governed by settlement considerations. This can be evaluated using the procedures discussed in Chapter 5.

Figure 4.8. Example 4.6—load and resistance factor design procedure for estimating load carrying capacity of a footing on sand.

ing resistance of a footing are illustrated by an example given in Figure 4.9.

Example 4.7 - For the footing given in Example 4.5, determine whether it has sufficient sliding capacity using load and resistance factors concept.

The magnitude of the factored lateral load is

$H_f = \gamma_D H_D = (1.3)(21) = 27.3$ tons

Assuming the concrete footing will be cast in situ,

$\delta \cong \phi = 35^\circ$, for $\bar{N} = 11$. (see Figure 4.1)

The factored sliding capacity is

$Q_\tau = V_f \tan\delta = (1.3)(120)(\tan 35^\circ) = 109.2$ tons

This assumes that both the vertical and horizontal components of the applied load are due to dead load.

For a performance factor, $\phi = 0.80$ (from Table 2.3), the factored sliding capacity is

$Q_{f\tau} = \phi\, Q_\tau = (0.80)(109.2) = 87.4$ tons

Since $Q_{f\tau} > H_f$, the footing is safe against sliding failure.

Figure 4.9. Example 4.7—load and resistance factor design procedure for estimating sliding capacity of a footing subjected to inclined load.

CHAPTER 5

SETTLEMENTS OF FOOTINGS

5.1 GENERAL

The settlements of footings must be small enough so that the structures they support are not damaged by movements of the footings and their functions are not impaired.

Design of footings to prevent excessive movements involves two considerations. One is determining the amount of movement that the supported structure can tolerate. The second is estimating the amount of movement that will be caused by the loads carried by the footings.

This chapter is concerned with methods of estimating settlements. The chapter also deals with procedures for ensuring that uncertainties in the estimated movements are accounted for in the design. Methods for establishing tolerable settlements of footings are given in Part 5.

5.2 SETTLEMENTS OF FOOTINGS ON SAND FROM STANDARD PENETRATION TESTS

5.2.1 Terzaghi and Peck Method

Terzaghi and Peck (1948, 1967) developed a procedure which uses SPT results to estimate settlements of footings on sand. Their method, which has been widely used, is considered by some engineers to be quite conservative, in the sense that it often overestimates settlements.

Recent studies (Tan and Duncan, 1991) have shown that about 85 percent of the time, settlements estimated using the Terzaghi and Peck method are larger than the actual settlements. Thus, while it is true that the method is not highly precise, it is also true that the method is reliable, in the sense that it seldom underestimates settlements. Because settlements of footings on sand are highly erratic, it is inevitable that methods that rarely underestimate settlements will often overestimate settlements by a fairly wide margin. The method proposed by Terzaghi and Peck appears to provide a suitable compromise between accuracy and reliability. Although it is one of the simplest methods, it is as accurate as any other method with the same degree of reliability.

The Terzaghi and Peck method is presented in the form of the chart shown in Figure 5.1. This chart is based on the notion that the tolerable amount of settlement is 1 in. For a given combination of footing width and SPT blow count, N, the chart is used to determine the bearing pressure that will result in 1 in. of settlement. The chart can be used to estimate the bearing pressure corresponding to settlements other than 1 in., by considering that settlements are very nearly proportional to bearing pressure. Thus, for 2 in. of settlement the bearing pressure would be twice as great as shown in Figure 5.1; and for half an inch of settlement, the pressure would be half of the values from Figure 5.1.

The value of N used in estimating the bearing pressure is determined by averaging the values measured in all borings, within the range of depth from the bottom of the footings to depth B below the bottom of the footings. B is the footing width, which can be estimated roughly for the purpose of averaging N values. Values of N are averaged separately for each boring, and the lowest average value is used for design.

Figure 5.1 can be used directly when the water table is at a depth below the bottom of the footings that is at least twice the footing width. For higher positions of water table, the bearing pressures determined from Figure 5.1 should be reduced. If the water table is at the ground surface, the bearing pressure should be reduced as follows: (1) for shallow foundations, with D_f/B less than one-half, pressures from Figure 5.1 should be reduced by 50 percent; (2) for deeper foundations, with D_f/B closer to unity, pressures from Figure 5.1 need only be reduced by 30 percent.

If the position of the water table is between depth 2B and the ground surface, the bearing pressure should be reduced by an amount between zero and 50 percent for shallow footings ($D_f/B < 0.5$); for deeper foundations ($D_f/B \simeq 1$), the bearing pressure should be reduced by an amount between zero and 30 percent.

As shown by Bazaraa (1967) the relationships among bearing pressure, settlement, footing width and N values shown in Figure 5.1 can be expressed in terms of the following equation

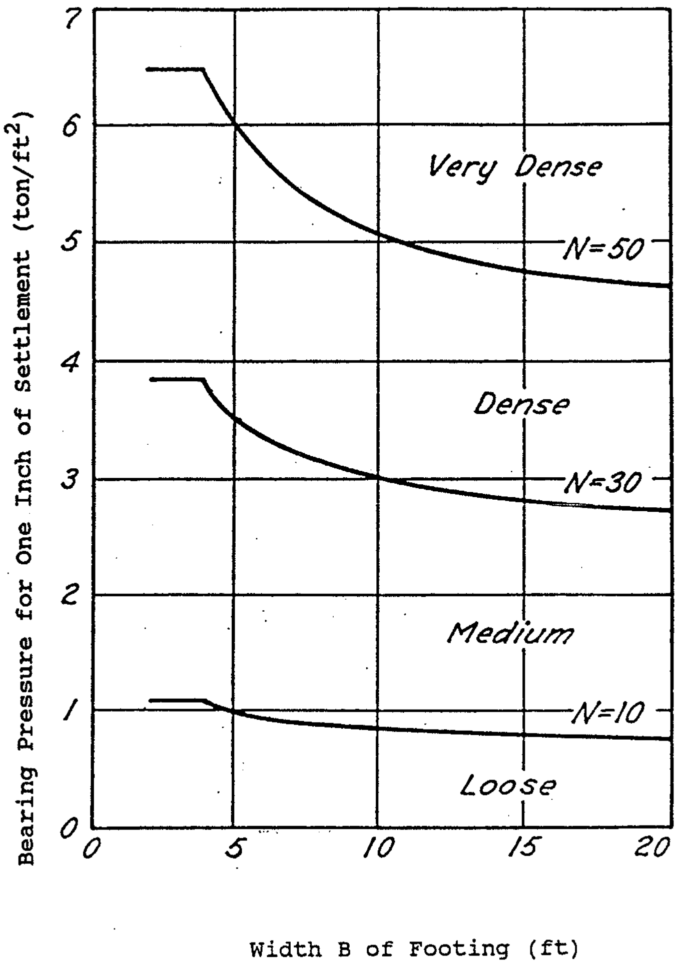


Figure 5.1. Bearing pressure for one inch of settlement of footings on sands. (After Terzaghi and Peck, 1967)

Example 5.1

Estimate the settlement of a 15 ft square footing using the soil information given in Figure 4.1a. The footing is designed to carry a service dead load of 250 tons and a service live load of 50 tons, and is placed 5 ft below the ground surface.

Following Terzaghi and Peck's recommendation, minimum average value of SPT blow counts within the range of depth from the bottom of footing to depth B below the footing base is used in settlement calculation. From soil information given in Figure 4.1,

From Boring 1, $\overline{N} = 16$

From Boring 2, $\overline{N} = 11$

From Boring 3, $\overline{N} = 24$

Use $\overline{N} = 11$ for settlement calculation.

From Figure 5.1 the bearing pressure corresponding to one inch settlement is,

$q_{1"} = 0.8$ t/ft^2 for B = 15' and $\overline{N} = 11$

Since the water table is at depth approximately B below the footing base, correction for effect of water table is required.

$$\text{Reduction} = 0.5 + \left(\frac{20-5}{2 \times 15}\right) + (1.0 - 0.5) = 0.75$$

Thus $q_{1"} = (0.75)(0.8) = 0.6$ t/ft^2

Design pressure is

$$p = \frac{250 + 50}{15 \times 15} = 1.33 \text{ t/ft}^2$$

$$\text{Estimated settlement of footing} = \frac{1.33}{0.6} = 2.2 \text{ inches}$$

Figure 5.2. Example 5.1—estimating settlement of a footing on sand using Terzaghi and Peck procedure.

$$p = \rho \frac{\overline{N}}{3} \left[\frac{B+1}{2B}\right]^2 \tag{5.2.1}$$

where p = bearing pressure corresponding to a given magnitude of settlement, ρ, in t/ft^2; ρ = settlement in inches; $\overline{N}$ = average blow count from the standard penetration test; and B = footing width in feet.

As in the case of Figure 5.1, bearing pressures calculated using Eq. 5.2.1 should be reduced if the depth to the water table is less than 2B below the bottom of the footing.

The use of this procedure can be illustrated by reference to a simple example. Suppose that the minimum average N for a site has been determined to be 15. For footings with width B = 10 ft, the bearing pressure causing 1.0 in. of settlement would be 1.5 t/ft^2 if the water table was at a depth greater than twice the footing width. If the water table is at the ground surface and the footing is shallow ($D_f < 5$ ft), the bearing pressure causing 1.0 in. of settlement would be 0.8 t/ft^2 (50 percent less). The bearing pressure causing 2 in. of settlement would be about 3.0 t/ft^2 if the water table was deep, and about 1.5 t/ft^2 if the water table was at the ground surface.

An example illustrating the procedures for estimating settlement using the Terzaghi and Peck method is given in Figure 5.2.

5.2.2 D'Appolonia et al. (1970) Method

The method developed by D'Appolonia et al. (1970) is based on elastic theory. It uses SPT blow count as the basis for estimating in situ soil compressibility. The following expression is used for calculating settlements of footings on sand:

$$\rho = \mu_o \, \mu_1 \frac{pB}{M} \tag{5.2.2}$$

where ρ = settlement of footing (same length units as B); μ_o, μ_1 = settlement influence factors dependent on footing geometry, depth of embedment, and depth to the relatively incompressible layer, dimensionless; p = average applied pressure under service loading condition, in pressure units; B = footing width, in length units; and M = modulus of compressibility, same pressure units as p.

Values of μ_o and μ_1 are given in Figure 5.3, and the correlation

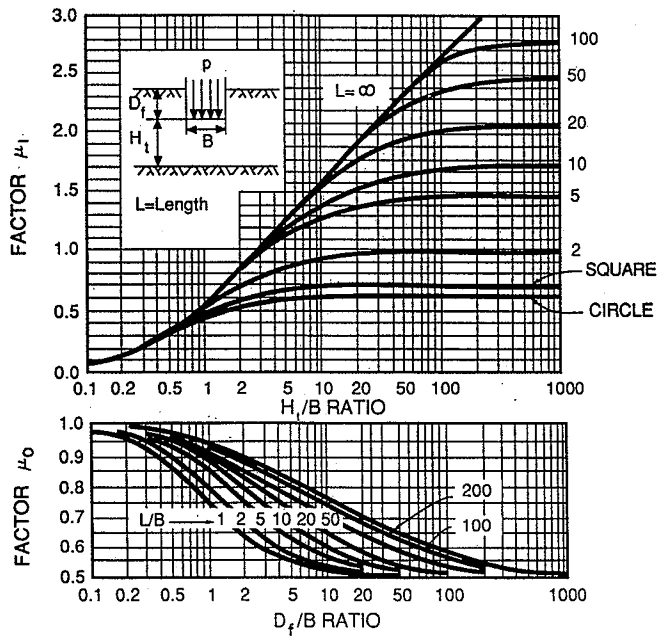


Figure 5.3. Settlement influence factors μ_o and μ_1 for the D'Appolonia et al. procedure. (After D'Appolonia, et al., 1970)

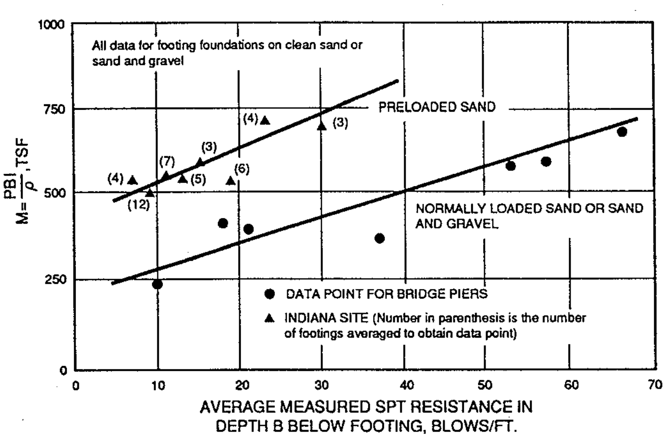


Figure 5.4. Correlation between modulus of compressibility and average value SPT blow count. (After D'Appolonia, et al., 1970)

between modulus of compressibility, M, and average SPT blow count is given in Figure 5.4. The value of SPT blow count used to estimate M is the average value within the range of depth from the bottom of the footing to depth B below the footing base. D'Appolonia et al. agreed with Meyerhof's conclusion that the effect of groundwater on soil modulus is reflected in the measured SPT, and that no correction for groundwater effect is required. The use of this method is shown in Figure 5.5.

As is the case for Terzaghi and Peck's method, the D'Appolonia et al. method can be rearranged to provide the following relationship between bearing pressure and settlement of footings on sand:

$$p = \frac{1}{\mu_o \mu_1} \frac{\rho M}{B} \tag{5.2.3}$$

where p = bearing pressure corresponding to a given magnitude of settlement, ρ, in pressure units; ρ = settlement of footing, same length units as B; μ_o, μ_1 = settlement influence factors, dimensionless; B = footing width, in length units; and M = modulus of compressibility, same pressure units as p.

Equation 5.2.3 represents a convenient form for calculating bearing pressure corresponding to a given magnitude of settlement.

Studies by Tan and Duncan (1991) show that the D'Appolonia et al. method appears to be one of the most accurate methods for estimating settlements of footings on sand, in the sense that the average settlements estimated using this method are about equal to the average value of actual settlements for a large number of actual footings. However, it is important to note that their method underestimates settlements about half the time. Values of settlement estimated using the method can be adjusted easily to provide the same level of reliability as the Terzaghi and Peck method. The adjustment process is discussed in Section 5.10.

Example 5.2

Estimate settlement of the footing given in Example 5.1 using D'Appolonia, et al.'s procedure. The sand is normally consolidated, and the firm stratum is encountered at depth 45' below the footing base.

(1) Determine minimum average value of $\overline{N}$.

Use the minimum average value of measured SPT below counts within the range of depth B below the footing base. From soil information given in Figure 4.1a,

$\overline{N}_1$ = 16 for Boring 1

$\overline{N}_2$ = 11 for Boring 2

$\overline{N}_3$ = 24 for Boring 3

(2) Determine value of modulus of compressibility

From Figure 5.4, for $\overline{N}$ = 11 and normally loaded sand,

$M \approx 260 \text{ t/ft}^2$

(3) Determine values of settlement influence factors, μ_0 and μ_1.

$$D_f/B = \frac{5}{15} = 0.33, \qquad \frac{H_t}{B} = \frac{45}{15} = 3.0$$

From Figure 5.3, $\mu_0 = 0.92$ and $\mu_1 = 0.65$

(4) Estimate settlement of footing.

Estimated settlement of the footing is

$$\rho = \mu_0 \mu_1 \frac{pB}{M} = (0.92)\,(0.65)\,\left\{\frac{\left(\frac{250+50}{15 \times 15}\right)}{260}\,(15)\right\}$$

ρ = 0.048 ft or 0.57 inch

Figure 5.5. Example 5.2—estimating settlement of a footing on sand using D'Appolonia et al. procedure.

5.3 SETTLEMENTS OF FOOTINGS ON SAND FROM CONE PENETRATION TESTS

Schmertmann (1970, 1978) developed a procedure using cone penetration test results to estimate settlements of footings on sand. His method has a rational basis, and uses cone penetration resistance, q_c, as a measure of in situ soil compressibility.

The expression proposed by Schmertmann (1978) for calculating settlements of footings on sand is as follows:

$$\rho = C_p\, C_t\, \Delta_p\, \Sigma \left(\frac{I_z}{E_s}\, \Delta Z\right) \qquad (5.3.1)$$

where ρ = estimated settlement, the same length units as ΔZ; C_p = pressure change correction factor for initial overburden pressure from Table 5.1, dimensionless; C_t = time rate factor or (creep correlation factor), from Table 5.2, dimensionless; Δ_p = net increase in bearing pressure at foundation level, the same pressure unit as q_c; I_Z = settlement influence factor, which varies with depth and L/B ratio, as defined in Figure 5.6 and Table 5.3, dimensionless; E_s = in situ soil modulus, which can be related to the value of cone resistance, q_c. in the same pressure unit as q_c; and ΔZ = thickness of sublayer, in length units.

The variation of settlement influence factor with depth is shown in Figure 5.6. The values of the quantities that define the dimensions of the settlement influence diagram are given in Table 5.3.

Values of in situ soil modulus for square footings can be estimated using the following expression:

$$E_S = 2.5\, q_c \qquad (5.3.2)$$

where E_S = in situ soil modulus, in same pressure units as q_c; and q_c = cone penetration resistance, in pressure units.

For footings with a length/width ratio greater than or equal to 10, the soil deforms in a condition closer to plane strain, and the soil is stiffer because of the increased confinement. For these long footings, the expression for estimating the in situ soil modulus is:

$$E_S = 3.5\, q_c \qquad (5.3.3)$$

For footings with a length/width ratio between 1 and 10, the E_s/q_c ratio can be estimated by interpolation.

Schmertmann's method provides a logical means for accounting the variations in sand density and compressibility with depth.

Table 5.1. Pressure change correction factor, C_p.

σ_{vo}'/Δ_p	C_p
0.0	1.0
0.2	0.9
0.4	0.8
0.6	0.7
0.8	0.6
≥1.0	0.5

Note: σ_{vo}' = initial vertical pressure at level of bottom of footing (in pressure units)
Δ_p = average net bearing pressure at foundation level (same pressure units as σ_{vo}')

Table 5.2. Time rate factor, C_t, for settlements of cohesionless soils.

Time	C_t
1 month	1.0
4 months	1.1
1 year	1.2
3 years	1.3
10 years	1.4
30 years	1.5

Table 5.3. Coefficients to define the dimensions of Schmertmann's improved settlement influence factor diagram in Figure 5.6. (After Schmertmann, et al., 1978)

L/B	Max. Depth of Influence Z_{max}/B	Depth to Peak Value Z_p/B	Value of I_Z at top I_{Zt}	Peak Value of Stress Influence Factor I_{Zp}: $\Delta p/\sigma_{vp}' = 1$	$\Delta p/\sigma_{vp}' = 2$	$\Delta p/\sigma_{vp}' = 4$	$\Delta p/\sigma_{vp}' = 10$
1	2.00	0.50	0.10	0.60	0.64	0.70	0.82
2	2.20	0.55	0.11	0.60	0.64	0.70	0.82
4	2.65	0.65	0.13	0.60	0.64	0.70	0.82
8	3.55	0.90	0.18	0.60	0.64	0.70	0.82
≥ 10	4.00	1.00	0.20	0.60	0.64	0.70	0.82

Note: B = footing width
L = footing length
$\Delta p = \sigma_{vf}' - \sigma_{vo}'$ = net bearing pressure
σ_{vf}' = final vertical pressure at level of bottom of footing
σ_{vo}' = initial vertical pressure at level of bottom of footing
σ_{vp}' = initial vertical pressure at depth of peak influence

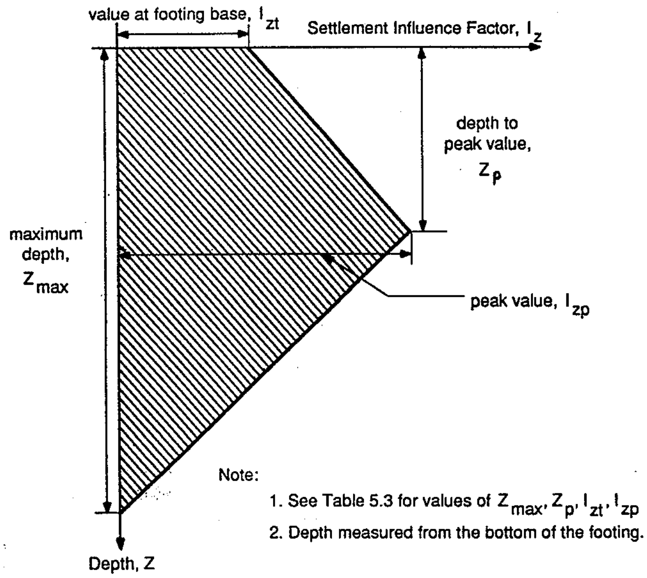


Figure 5.6. Variation of Schmertmann's improved settlement influence factors with depth. (After Schmertmann, et al., 1978)

In cases where there is pronounced layering, persistent from boring to boring, the soil beneath the foundation level is divided into a number of sublayers for purposes of calculating settlement. The sublayers are chosen so that the values of cone resistance within each sublayer are essentially constant. The value of the settlement influence factor for each sublayer is evaluated at the mid-height of the sublayer. Because the value of settlement influence factor peaks at some depth below the foundation level, Z_p, it is necessary that a sublayer boundary occurs at Z_p.

The computation process involves a rather lengthy procedure, and is warranted only when the site is characterized by pronounced and persistent layering.

For sand layers of limited thickness, overlying firm soil or rock, the influence diagram is terminated at the top of the firm layer. The method thus provides a logical means for accounting for the influence of the finite thickness of sand layer on settlement.

If the soil at the site can be represented as being homogeneous, the process is simplified. In this case, it is not necessary to divide the soil beneath the footing level into sublayers. For homogeneous conditions, the settlement can be calculated using the following expression:

$$\rho = I' \, C_p \, C_t \left(\frac{\Delta_p}{q_c}\right) B \qquad (5.3.4)$$

where I′ = equivalent settlement influence factor from Table 5.4. Other terms are as previously defined.

Schmertmann's method can also be used to estimate the pressure required to cause a given amount of settlement by rearranging Eq. 5.3.4 in the following form:

$$\Delta_p = \rho \frac{q_c}{B\, I'} \frac{1}{C_p \, C_t} \qquad (5.3.5)$$

This form is convenient for determining values of bearing pressures when the tolerable settlement for a particular structure has been established.

The use of Schmertmann's method is shown by Example 3 in Figures 5.7, 5.7a and 5.7b.

Example 5.3

Estimate the settlement of a 15 ft square footing using the cone penetration test data given in Figure 4.2a. The footing is designed to carry a service dead load of 250 tons and a service live load of 50 tons. The embedment depth of the footing is 5 ft.

(1) Determine dimension of settlement influence diagram.

For a square footing, $\frac{L}{B} = 1.0$

From Table 5.3, $\frac{Z_{max}}{B} = 2.00$, $\frac{Z_p}{B} = 0.50$, and $I_{zt} = 0.10$

Assume weight of the footing is about the same as weight of the soil excavated so that

$$\Delta_p = \frac{250 + 50}{15 \times 15} = 1.33 \text{ t/ft}^2$$

Effective overburden pressure at Z_p is

$$\sigma'_{vp} = (0.0575)\,(5 + 0.5 \times 15) = 0.72 \text{ t/ft}^2$$

$$\frac{\Delta_p}{\sigma'_{vp}} = \frac{1.33}{0.72} = 1.85$$

From Table 5.3, value of I_{zp} is interpolated as:

$$I_{zp} = 0.60 + \left(\frac{0.64 - 0.60}{2 - 1}\right)(1.85 - 1) = 0.63$$

Based on the values of Z_{max}, Z_p, I_{zt} and I_{zp}, the settlement influence diagram is drawn, as shown in Figure 5.7a.

Figure 5.7. Example 5.3—estimating settlement of a footing on sand using cone penetration test data.

Table 5.4. Values of equivalent settlement influence factor.

L/B	$\frac{\Delta p}{\sigma_{vp}'} = 1$	Equivalent Settlement Influence Factor, I′: $\frac{\Delta p}{\sigma_{vp}'} = 2$	$\frac{\Delta p}{\sigma_{vp}'} = 4$	$\frac{\Delta p}{\sigma_{vp}'} = 10$
1	0.25	0.27	0.29	0.34
2	0.27	0.28	0.31	0.36
4	0.30	0.32	0.35	0.40
8	0.35	0.37	0.40	0.46
≥ 10	0.37	0.39	0.43	0.50

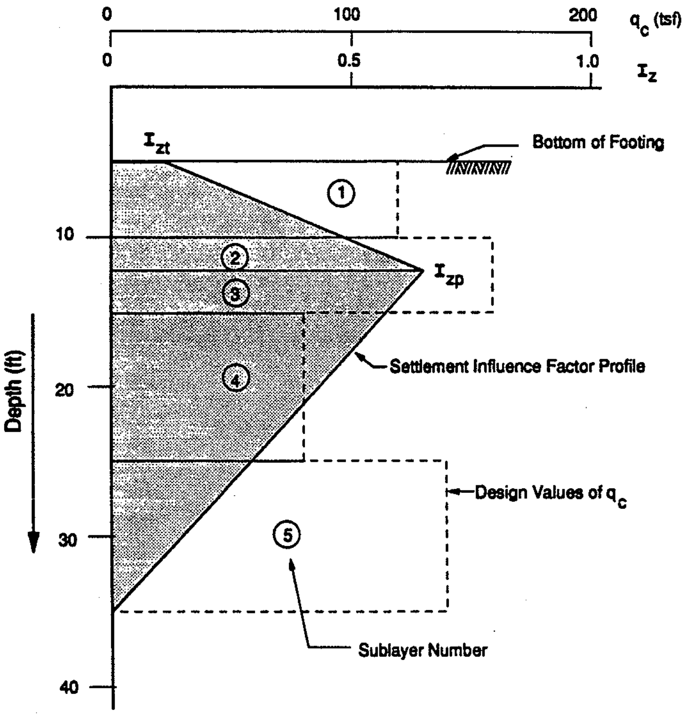


Figure 5.7a. Settlement influence diagram and design values of cone resistance for the 15-ft square footing.

5.4 SETTLEMENTS OF FOOTINGS ON SANDS, SILTS AND CLAYS BY JANBU'S TANGENT MODULUS METHOD

Janbu (1963, 1967) developed a unified approach for estimating settlements of footings on soil using tangent modulus values to characterize soil compressibility.

In Janbu's method the soil beneath the footing is divided into a number of sublayers, each characterized by a value of constrained tangent modulus, M_t. The settlement of the footing is estimated by summing the reductions in thickness of each of the sublayers beneath the footing, as shown by the expression:

$$\rho = \Sigma \left[\frac{\Delta\sigma_v'}{M_t} \cdot \Delta Z\right] \tag{5.4.1}$$

where ρ = settlement, any length units; $\Delta\sigma_v'$ = increase in effective stress within the sublayer due to the load on the footing, stress units—the same units as M_t (this increase in stress due to the footing load can be estimated using elastic stress distribution theory); M_t = tangent value of constrained modulus of the soil, stress units—the same units as $\Delta\sigma_v'$; and ΔZ = sublayer thickness, length units—the same units as ρ.

Values of constrained tangent modulus vary with the type of soil, its density, whether it is normally consolidated or overconsolidated, and the stresses acting on it before and after the load is applied to the footing.

Values of M_t can be measured using conventional laboratory consolidation tests (oedometer, or one-dimensional compression tests). In the case of natural soils the tests should be performed on good quality undisturbed specimens. In the case of compacted fills, the specimens should be prepared at the same dry density

(2) Calculate settlement of the footing.

Based on data from Figure 4.2a, design value of q_c is determined as shown in Figure 5.7a. For settlement computation, the soil beneath the footing is divided into five sublayers.

Sublayer Number	Depth (ft)	Thickness Δz (ft)	Average q_c (tsf)	I_z	E_s=2.5 q_c (tsf)	$\frac{I_z \Delta z}{E_s}$ (ft^3/t)
1	5-10	5.0	120	0.28	300	0.0047
2	10-12.5	2.5	160	0.55	400	0.0034
3	12.5-15	2.5	160	0.60	400	0.0038
4	15-25	10.0	80	0.42	200	0.0210
5	25-35	10.0	140	0.14	350	0.0040
					Σ	0.0369

The initial effective overburden pressure at foundation level is,

σ_{vo}' = (0.0575) (5) = 0.29 tsf.

$\frac{\sigma_{vo}'}{\Delta_p} = \frac{0.29}{1.33} = 0.22$

From Table 5.1, C_p = 0.89

Immediate settlement of the footing, ρ_i, is estimated as follows:

$$\rho_i = C_p\, C_t\, \Delta_p\, \Sigma \frac{I_z \Delta z}{E_s}$$

ρ_i = (0.89) (1.0) (1.33) (0.0369) = 0.044 ft or 0.52 inch

Estimated settlement of the footing over ten years is

$\rho_t = C_t\, \rho_i$

From Table 5.2, C_t = 1.4

ρ_t = (1.4) (0.52) = 0.73 inch

As noted in Figure 4.2a, the CPT was performed adjacent to Boring 3. The SPT blow counts obtained from Boring 3 are consistently higher than those from Borings 1 and 2. It is not surprising that settlement computed using CPT data, as shown in this example, is smaller than that computed using the Terzaghi and Peck method which uses minimum average SPT N-value for settlement calculations.

Figure 5.7b. Estimating settlement of a footing on sand using cone penetration test data.

and compaction water content as the fill in the field.

The pressures used in the laboratory tests should cover the range from the initial pressure on the sublayer (before the load is applied) to the final pressure on the sublayer (after the load is applied). As shown in Figure 5.8, M_t is determined by dividing the stress increment in the sublayer ($\Delta\sigma_v' = \sigma_{vf}' - \sigma_{vo}'$) by the corresponding strain, $\Delta\epsilon_v$.

When laboratory tests are not available for evaluation of M_t. values can be estimated using the information given in Table 5.5, together with the following equations:

For sands and silts, values of M_t can be estimated using the expression:

$$M_t = m\, p_a \left(\frac{\sigma_{va}'}{p_a}\right)^{0.5} \tag{5.4.2}$$

where m = dimensionless modulus number shown in Table 5.5 σ_{va}' = average vertical stress = 1/2 ($\sigma_{vo}' + \sigma_{vf}'$) expressed in in the same pressure units as M_t and p_a; p_a = atmospheric pressure, expressed in the same pressure units as M_t and σ_{va}';

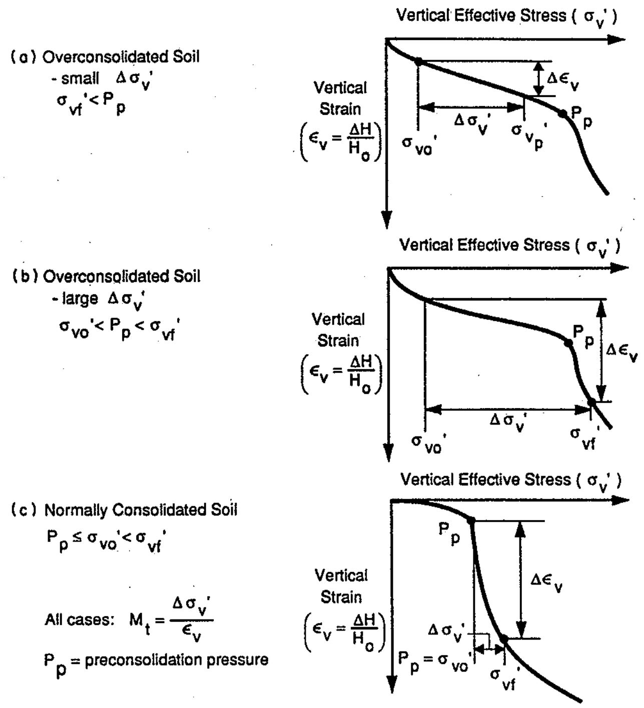


Figure 5.8. Determination of M_t from results of laboratory consolidation tests.

For normally consolidated clays, values of M_t can be estimated using this expression:

$$M_t = m \cdot \sigma_{va}' \tag{5.4.3}$$

where m = dimensionless modulus number shown in Table 5.5, and σ_{va}' = average vertical stress = $1/2\ (\sigma_{vo}' + \sigma_{vf}')$ expressed in the same pressure units as M_t.

As shown in Figure 5.8 and Table 5.5, values of M_t are higher for overconsolidated soils ($p_p > \sigma_{vo}'$). When the final pressures do not exceed the preconsolidation pressure ($\sigma_{vf}' < p_p$) the soil is being reloaded, and the value of M_t is higher than for cases where σ_{vf}' exceeds p_p.

The procedure for estimating settlement of a footing using Janbu's tangent modulus method is shown in Figure 5.9.

5.5 SETTLEMENTS OF FOOTINGS ON SANDS AND CLAYS FROM PRESSUREMETER TESTS

Menard and Rousseau (1962) proposed a procedure for estimating settlements of footings using pressuremeter test results. Their procedure uses pressuremeter modulus as a measure of in situ soil compressibility.

The pressuremeter modulus, which is derived from the slope of a straight portion of the pressure-expansion curve, is sensitive to soil disturbance. In dense sands and stiff clays, soil disturbance due to boring and probe installation is usually not excessive, and the method is reliable (Baguelin et al., 1978; Robertson, 1986).

Table 5.5. Values of modulus number, m, for sands, silts, and clays. (After Janbu, 1985)

Sand	Relative Density (Dr)	Value of m: $\sigma_{vo}' = P_p$	$\sigma_{vo}' < P_p < \sigma_{vf}'$	$\sigma_{vf}' < P_p$
	30% (Loose)	80 - 160	120 - 300	240 - 500
	50% (Medium)	120 - 240	200 - 400	350 - 700
	70% (Dense)	200 - 400	300 - 700	600 - 1200
Silt	**Porisity (N)**	**Values of m: $\sigma_{vo}' = P_p$**	**$\sigma_{vo}' < P_p < \sigma_{vf}'$**	**$\sigma_{vf}' < P_p$**
	50%	25 - 50	40 - 200	120 - 240
	40%	60 - 120	80 - 400	300 - 600
	30%	100 - 200	150 - 700	500 - 1000
Clay	**In Situ Water Content**	**Value of m: $\sigma_{vo}' = P_p$**	**$\sigma_{vo}' < P_p < \sigma_{vf}'$**	**$\sigma_{vf}' < P_p$**
	70%	6 - 12	10 - 80	60 - 120
	50%	9 - 18	15 - 120	90 - 180
	30%	15 - 35	25 - 200	150 - 350

Pp = preconsolidation pressure = highest pressure to which the soil has been subjected in the past.

Using Janbu's tangent modulus method, estimate the settlement of a 15 ft square footing which is designed to support a dead load of 250 tons and a live load of 50 tons. The soil information is the same as that given in Figure 4.2a, and the footing will be 5 ft below the ground surface.

(1) Calculate the initial and final vertical pressures.

The initial and final overburden pressures beneath the footing, to a depth of one footing width below its base, are tabulated below. The increase in pressure, $\Delta\sigma'_v$, due to the applied load was estimated using Westergard's theory. Variations of initial, final and average vertical pressures with depth are plotted in Figure 5.9a.

Sublayer Depth (ft)			Initial Effective Vertical Pressure σ'_{vo}	Pressure Change $\Delta\sigma'_v$	Final Effective Vertical Pressure σ'_{vf}	Average Vertical Pressure σ'_{va}	Preconsolidation Pressure Pp
Top	Bottom	Average	(psf)	(psf)	(psf)	(psf)	(psf)
5	10	7.5	862	1658	2520	1691	Assume
10	15	12.5	1438	972	2410	1924	normally consolidated
15	20	17.5	2012	598	2610	2311	($\rho_p = \sigma'_{vo}$)

Figure 5.9. Example 5.4—estimating settlement of a footing on sand using Janbu's tangent modulus method.

(2) Determine values of tangent modulus.

To account for the erratic nature of sand deposits, use minimum average value of N as suggested by Terzaghi and Peck for settlement computation, i.e., $\overline{N}_1 = 11$. This implies the sand is loose to medium.

The target modulus values are estimated using Equation 5.4.2 and are tabulated below:

Sublayer Number	Average Depth (ft)	Average Vertical Pressure σ'_{va} (psf)	Modulus Number, m	Tangent Modulus, Mt (psf)
1	7.5	1691	120	2.22×10^5
2	12.5	1924	120	2.36×10^5
3	17.5	2311	120	2.59×10^5

(3) Calculate settlement of the footing

Sublayer Number	Thickness, Δz (ft)	Pressure change, $\Delta\sigma'_v$ (psf)	Change in Thickness, ΔH (ft)
1	5	1658	0.037
2	5	972	0.021
3	5	598	0.011
Thus, settlement of the footing is,		Total	0.069

$\rho = 0.069$ ft or 0.83 inch

If the sand is assumed to be preloaded, then m = 350, and

For sublayer 1, $M_t = 6.48 \times 10^5$ psf

For sublayer 2, $M_t = 6.88 \times 10^5$ psf

For sublayer 3, $M_t = 7.55 \times 10^5$ psf

The estimted settlement is then,

$$\rho = \Sigma \left[\frac{\Delta\sigma'_v}{M_t} . \Delta z \right]$$

$$\rho = \frac{1658}{6.48 \times 10^5} \times 5 + \frac{972}{6.88 \times 10^5} \times 5 + \frac{598}{7.55 \times 10^5} \times 5$$

$\rho = 0.024$ ft or 0.29 inch

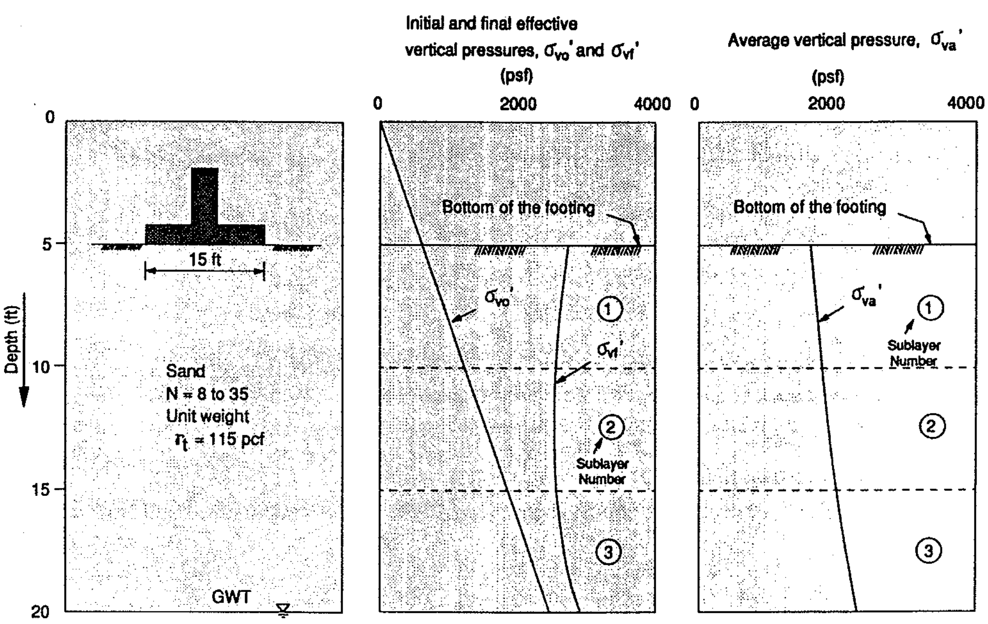


Figure 5.9a. Example 5.4 (continued)

For soft clays, loose sands and silts, disturbance is likely to affect the test results to a greater extent, and use of the pressuremeter method has sometimes led to unsatisfactory results (Baguelin et al., 1978). For these oils the pressuremeter method is not recommended.

For pressuremeter tests the settlements of embedded footings (depth of embedment, D_f. equal to or greater than the footing width, B) can be estimated using the following expression:

$$\rho = \frac{0.22p}{E_p} B_o \left(\lambda_d \frac{B}{B_o}\right)^{\alpha} + \frac{0.11p}{E_p} \alpha \lambda_c B \qquad (5.5.1)$$

where ρ = estimated footing settlement, in same length units as B; p = average bearing pressure at foundation level, in same pressure units as E_p; E_p = pressuremeter modulus, in pressure units; B_o = reference width, taken as 2 ft or 0.6 m; λ_d. λ_c = dimensionless shape factors, from Table 5.6; and α = empirical settlement coefficient defined by Menard (1972), from Table 5.7, dimensionless.

If the footing is not embedded ($D_f = 0$), Baguelin et al. (1978) indicated that the settlement should be estimated to be 1.2 times the value calculated using Eq. 5.5.1. Settlements for partially embedded footings ($0 < D_f < B$) can be estimated by interpolation.

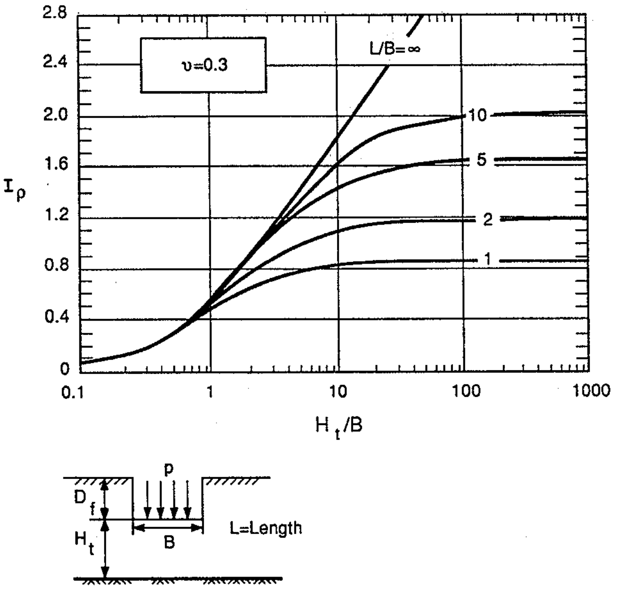


Figure 5.10. Settlement influence factor from Steinbrenner's approximation for m = 0.3. (After Taylor and Matyas, 1983)

5.6 SETTLEMENTS OF FOOTINGS ON SOIL BY ELASTIC METHODS

Settlements of footings on sands and clays may be estimated using elastic theory by means of the following equation which was suggested by Steinbrenner (1934) and Giroud (1972):

$$\rho_f = \frac{pB}{E_s} I_\rho \qquad (5.6.1)$$

where ρ_f = final settlement, in same length units as B; p = average bearing pressure at the base of footing in pressure units; E_s = in situ soil modulus, in same pressure units as p; I_ρ = dimensionless settlement influence factor, whose value depends on footing geometry, thickness of the compressible layer, and the value of Poisson's ratio; and B = width of footing, in length units.

For estimating settlements of sands and clays for drained conditions, the value of Poisson's ratio, ν, may be taken as 0.3. Values of I_ρ for $\nu = 0.3$ (developed by Steinbrenner, 1934) are given in Figure 5.10.

The accuracy of the settlement calculation depends on the accuracy with which the soil modulus can be estimated. Because of sample disturbance, values of soil modulus obtained from laboratory tests are unreliable. It is preferable that values be estimated from empirical correlations, such as the one shown in Figure 5.11, which relates soil modulus to SPT blow counts.

If settlement observations on similar soils are available, values of soil modulus can be back calculated from these data. This is probably the most reliable method of estimating modulus values.

Elastic theory is also often used to estimate immediate (or elastic) settlements of foundations on clay. The equation for calculating immediate settlements, as discussed by Steinbrenner (1934) and Christian and Carrier (1978), has a form similar to Eq. 5.6.1:

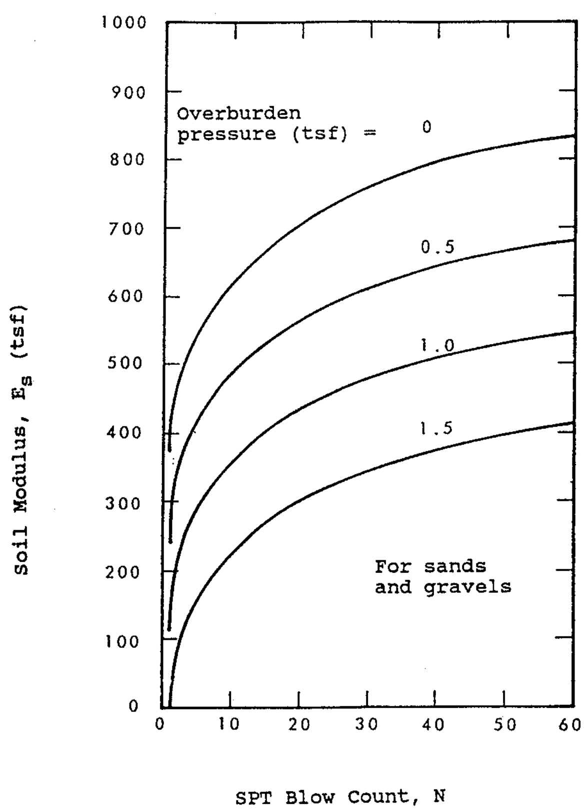


Figure 5.11. Relationship between soil modulus, E_s, and SPT blow count, N. (After Schultze and Melzer, 1965)

$$\rho_i = \frac{pB}{E_u} \mu_o \mu_1 \qquad (5.6.2)$$

Table 5.6. Values of shape factors for settlement computations using Menard pressuremeter test results. (After Baquelin et al., 1978)

L/B	λ_d	λ_c
1 (circle)	1.00	1.00
1 (square)	1.12	1.10
2	1.53	1.20
3	1.78	1.30
5	2.14	1.40
20	2.65	1.50

Table 5.7. Values of empirical settlement coefficient, α, for typical soils.

Soil Type	Condition	E_p/p_l	α
Clay	overconsolidated	> 16	1
	normally consolidated	9 - 16	2/3
	weathered or remolded	7 - 9	1/2
Silt	overconsolidated	> 14	2/3
	normally consolidated	8 - 14	1/2
	weathered or remolded	-	1/2
Sand	overconsolidated	> 12	1/2
	normally consolidated	7 - 12	1/3
	weathered or remolded	-	1/3
Sand and Gravel	overconsolidated	> 10	1/3
	normally consolidated	6 - 10	1/4
	weathered or remolded	-	1/4

Note: p_l = limiting pressure determined from pressuremeter test result (in pressure units)

where ρ_i = immediate settlement, in same length units as B; p = average bearing pressure at the base of the footing, in pressure units; E_u = undrained soil modulus, in same pressure units as p; μ_o = dimensionless settlement influence factor which describes the effect of footing embedment, whose value is given in Figure 5.12; μ_1 = dimensionless settlement influence factor which describes the effect of finite thickness for the compressible layer (values of μ_1 are given in Figure 5.12); and B = width of foundation, in length units.

As for the determination of soil modulus for drained conditions, it is preferable that undrained soil modulus be estimated using empirical correlations of the type shown in Figure 5.13. In this figure, values of undrained soil modulus are related to the plasticity index (PI), the undrained shear strength, s_u. and stress history of the clay, expressed in terms of overconsolidation ratio (OCR). The overconsolidation ratio is the maximum effective pressure to which the soil has been subjected in the past, p_p, divided by the present effective overburden pressure, σ_{vo}', i.e., OCR = p_p/σ_{vo}'.

The use of elastic theory to estimate settlement of a footing on sand is shown in Figure 5.14.

5.7 SETTLEMENTS OF FOOTINGS DUE TO CONSOLIDATION OF CLAYS

Settlements resulting from consolidation of normally consolidated and lightly overconsolidated clays can be very large. Where such soils are loaded by fills, settlements as large as several feet may occur. Methods for estimating the magnitudes of consolidation settlements, and the rates at which they occur, are discussed in many soil mechanics textbooks and engineering manuals (Sowers, 1979; Terzaghi and Peck, 1967; NAVFAC, 1982).

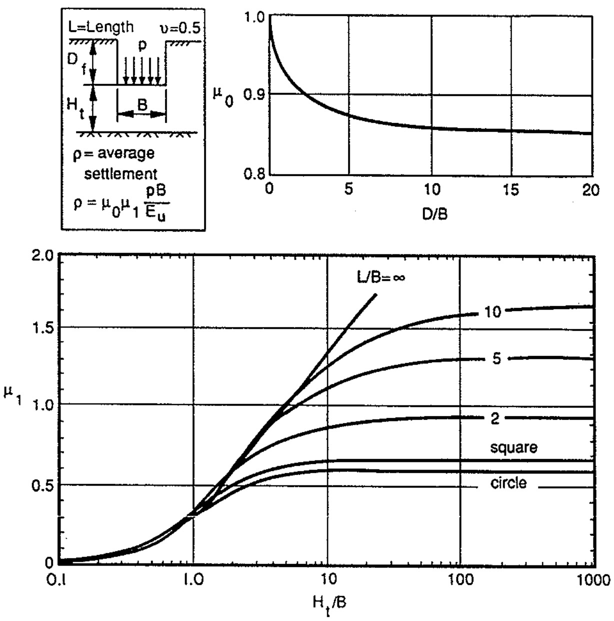


Figure 5.12. Settlement influence factors μ_o and μ_1. (After Christian and Carrier, 1978)

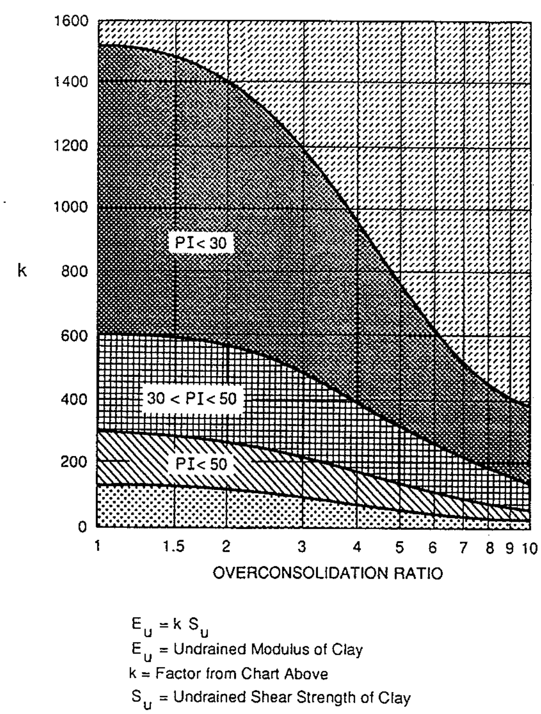


Figure 5.13. Chart for estimating undrained modulus of clay. (After Duncan and Buchignani, 1976)

Use elastic theory to estimate the settlement of a 15 ft square footing which carries a service dead load of 250 tons and a service live load of 50 tons. The footing is embedded 5 ft below the ground surface. The soil information is the same as that given in Example 4.1. The firm stratum is encountered at 45' below the bottom of the footing.

(1) Determine the initial effective overburden pressure.

The initial effective overburden pressure at depth $\frac{B}{2}$ below the bottom of the footing is

$$\sigma'_{vo} = (0.0575) \times (5 + \tfrac{15}{2}) = 0.72 \text{ t/ft}^2$$

(2) Estimate soil modulus.

From Figure 5.8, for $\overline{N}_1 = 11$ and $\sigma'_{vo} = 0.72$ t/ft²

$$E_s = 410 \text{ t/ft}^2$$

(3) Estimate settlement of the footing.

Using elastic theory, settlement of the footing is

$$\rho = \frac{pB}{E_s} I_p$$

From Figure 5.7, for $\frac{H_t}{B} = \frac{45}{15} = 3$, $I_p = 0.7$

$$\rho = \frac{\frac{(250 + 50)}{15 \times 15}(15)}{410}(0.7) = 0.034 \text{ ft}$$

or $\rho = 0.4$ inch

Figure 5.14. Example 5.5—estimating settlement of a footing on sand using elastic theory.

Highly compressible clays are rarely capable of supporting footings for bridges, buildings, or other structures where the tolerable amount of settlement is limited to a small fraction of a foot. For this reason, detailed consideration of consolidation settlements is not included in this manual.

Settlements of footings founded on less compressible, overconsolidated clays can be estimated using Janbu's procedure, Baguelin's pressuremeter procedure, or elastic theory. Settlements of footings on heavily overconsolidated clays usually occur fairly rapidly, and it is reasonable to assume that they take place as rapidly as the loads are applied.

5.8 TIME-DEPENDENT SETTLEMENTS OF FOOTINGS ON SANDS

Footings on sands continue to settle at a slow and continually decreasing rate after the amount by which the settlements increase with time can be estimated by multiplying the calculated value of immediate settlement by a time rate factor, C_t.

The time rate factor is included in Schmertmann's method of settlement calculation, as may be seen by reference to Eqs. 5.3.1 and 5.3.4. The same allowance for increased settlements with time can be used with other methods. This can be done through the following equation:

$$\rho_t = \rho_i \cdot C_t \tag{5.8.1}$$

where ρ_t = settlement at time t; ρ_i = settlement calculated by Terzaghi and Peck's method, Janbu's method, Baguelin's pressuremeter method, or using elastic theory; and C_t = time rate factor from Table 5.2.

5.9 SETTLEMENTS DUE TO SECONDARY COMPRESSION OF CLAYS

Settlements of footings on clays continue at a slow and continually decreasing rate after the clay undergoes its initial compression. This process is called "secondary compression."

Secondary compression settlements can be estimated using the expression:

$$\rho_{sc} = C_\alpha H_t \log \frac{t_{sc}}{t_p} \tag{5.9.1}$$

where ρ_{sc} = settlement due to secondary compression, in the same length units as H; C_α = coefficient of secondary compression, dimensionless—values of C_α are given in Table 5.8; H_t = total thickness of layer undergoing secondary compression, in length unit; t_{sc} = time for which secondary compression is to be calculated, in years; and t_p = time for primary compression, in years—t_p should be taken as not less than 1 year.

5.10 DESIGN FOR TOLERABLE SETTLEMENT

Design of shallow foundations is often limited by considerations of tolerable settlement. This fact is illustrated by Examples 4.1 and 5.1 (see Figures 4.1 and 5.2). In Example 4.1 it was determined that a bearing pressure of 6.4 t/ft² would provide a factor of safety of about 3.0. In Example 5.1 it was determined that the bearing pressure would have to be limited to 0.6 t/ft² to limit the settlement to 1.0 in. Thus, if the tolerable settlement of the footing was 1.0 in., its design would be controlled by consideration of settlement rather than consideration of ultimate bearing capacity. This is usually the case with spread footings on sand.

Design of footings to satisfy settlement criteria involves these steps: (1) establish the upper limit of tolerable settlement for the footing; (2) select a footing size that is safe against bearing capacity failure and estimate its settlement; and (3) compare the settlement estimated in step (2) with the tolerable value. If the estimated value of settlement exceeds the tolerable value, the footing is redesigned so that the settlement is reduced.

With some methods of estimating settlement, it is possible to determine fairly directly what bearing pressure will satisfy the movement criteria. For example, Bazarra (1967) showed that Terzaghi and Peck's method corresponds to the following rela-

Table 5.8. Values of c_α for clays.

Natural Water Content of Clay	Value of C_α For OCR = 1	Value of C_α For OCR ≥ 5
10%	0.001	0.0003 to 0.0005
20%	0.002	0.0006 to 0.0010
40%	0.004	0.0012 to 0.0020
80%	0.008	0.0025 to 0.0040

tionship between bearing pressure and settlement of footings on sand above the water table:

$$p_\rho = \rho_{tol}\frac{\overline{N}}{3}\left(\frac{B+1}{2B}\right)^2 \qquad (5.10.1)$$

where p_ρ = bearing pressure corresponding to the tolerable movement, tsf; ρ_{tol} = tolerable movement, in.; $\overline{N}$ = minimum average SPT-N values, blows/ft; and B = footing width, ft.

The adequacy of a design for settlement criteria depends to a large extent on the accuracy with which the settlement can be estimated. Since the densities and compressibilities of sand deposits are inherently variable, it is not possible to estimate settlement of footings on sand with high accuracy. If a number of footings of the same size were constructed on the same sand deposit, each would settle a different amount when subjected to the same load. Terzaghi and Peck (1967) indicated that in this circumstance, the footing that settled the most would settle about 4 times as much as the footing that settled the least. Thus, the actual settlements would cover a wide range, and no method of calculation could ever give exactly the "right answer" for all of the footings.

It is important to realize that various methods of estimating settlements of footings on sand, which lead to different estimates of the settlement, would compare differently to the range of actual settlements of identical footings. Recent studies have compared measured settlements with settlements calculated using various procedures (Tan and Duncan, 1991). These studies show that methods which result in settlements close to the average of measured settlements are likely to underestimate settlements half the time and overestimate them half the time. Methods that are more conservative (notably Terzaghi and Peck's method) tend to overestimate settlements more than half the time and to underestimate them relatively infrequently.

The studies indicate that there exists a trade-off between accuracy and reliability. A relatively accurate method is one that would result in estimated settlement about equal to the average settlement for a group of footings. A reliable method is one that predicts settlements that are greater than or equal to the actual settlement most of the time. The studies show that any method for estimating settlements of footings on sand can be modified, by multiplying the estimated settlements by an adjustment factor to yield about the same combination of accuracy and reliability as any other method. For instance, the D'Appolonia, et al., method predicts settlements that are about equal to the average value of actual settlements, and it underestimates settlements about half the time. To ensure that the settlements calculated using the D'Appolonia, et al., method equal or exceed the measured settlement about 90 percent of the time, the settlements computed using the procedure should be multiplied by a factor of two. This adjustment would increase the "reliability" of the method from about 50 percent to about 90 percent.

Adjustment factors for 50 percent and 90 percent reliability in calculated values of displacement are given in Table 5.9 for Terzaghi and Peck, D'Appolonia et al., and Schmertmann methods.

Table 5.9. Values of adjustment factor for 50 percent and 90 percent reliability in displacement estimates.

Method	Soil Type	Adjustment Factor	
		For 50% Reliability	For 90% Reliability
Terzaghi and Peck	Sand	0.45	1.05
Schmertmann	Sand	0.60	1.25
D'Appolonia, et al.	Sand	1.00	2.00

CHAPTER 6

SHALLOW FOUNDATIONS ON ROCK

6.1 GENERAL

Footings on rock must satisfy the same design criteria as footings on soil. They must be able to support the loads that they carry safely, without excessive movements that may damage or impair the functions of the supported structure.

Most continuous sound rocks have relatively high strengths and low compressibilities. In comparison with footings in soil, design of footings to be supported by these materials is rather simple and straightforward, and is often governed by structural considerations. For instance, if the allowable concrete strength is less than the rock strength, the determination of allowable "rock" bearing pressures may be unnecessary, and concrete strength will determine the required footing size.

Continuous and sound rock masses are, however, rarely encountered at shallow depth. Most near-surface rock masses are

broken by one or more sets of joints or fractures that divide the mass into blocks. Design of footings in these discontinuous rocks is usually controlled by geotechnical considerations, particularly by the characteristics of rock defect such as joints, seams, faults, and bedding planes.

This chapter presents a simple overview of current design procedures for estimating bearing capacity and settlement of footings on discontinuous or jointed rocks. The emphasis is on practical procedures that do not require detailed analyses.

6.2 BEARING CAPACITY OF FOUNDATIONS ON ROCK

6.2.1 Load Test

Full scale load tests are the most reliable method for determining bearing capacity of foundations on rock. However, load tests are relatively expensive, and are only warranted when very high loads are anticipated, for example, on piers for high-rise buildings or abutments for arch bridges.

6.2.2 Presumptive Bearing Values

Many codes provide presumptive design bearing values for foundations in rock. As given in Table 6.1, these values provide allowable bearing pressures based on descriptions of rock type and quality. The recommended values, however, do not take into consideration the type and function of the structures, the loading conditions, tolerable movement criteria, or the strength and deformation characteristics of the rock masses. In addition, even for the same type of rock, there are considerable differences among the values recommended by various codes.

Presumptive values often tend to be quite conservative. However, these values may provide reasonable estimates for bearing capacity of foundations of simple structures on good quality rock masses. In these cases the structural strength of the foundation usually governs the foundation design. For structures imposing large loadings, the use of presumptive design values as a basis of design is not recommended. For such heavy structures, use of the presumptive values may lead to overly expensive foundations.

6.2.3 Empirical Design Procedure for Reasonably Sound Rock

Peck et al. (1974) suggested an empirical correlation for estimating allowable bearing pressures of foundations on jointed rock based on an index of rock mass quality known as rock quality designation (RQD). The correlation, as given in Table 6.2, is intended for rock masses with tight joints "not wider than a fraction of an inch." The authors also indicated that, for footings designed with the allowable values given in Table 6.2, their settlements would be less than 0.5 in.

Value of RQD is computed as the percent of modified core recovery, as follows:

$$\text{RQD} = \frac{\begin{array}{c}\text{Sum of lengths of ``sound''} \\ \text{core pieces} > 4 \text{ in.}\end{array}}{\text{Total core run length}} \times 100 \qquad (6.2.3.1)$$

To meet the soundness requirement, Deere and Deere (1988) recommended that rocks of grades IV (highly weathered), V (completely weathered), and VI (residual soil), as defined by the International Society of Rock Mechanics (1978), be discounted for the determination of RQD, even though their lengths are greater than 4 in. Fractures caused by drilling operation must also be excluded. Determination of RQD is illustrated in Figure 6.1. Values of RQD reflect the relative intensity of jointing and, hence, the compressibility of a rock mass. The relationship between rock quality and RQD is given in Table 6.3. An RQD of 100 percent would represent an excellent quality rock mass whose engineering properties are similar to those of an intact rock specimen; an RQD less than 25 percent, on the other hand, would represent a very poor quality rock mass whose engineering properties are similar to those of soil.

The value of RQD for use in Table 6.2 may be taken as the average value of RQD within a depth equal to one footing width below the bottom of the foundation, provided the RQD is reasonably uniform within these depths. In most cases, however, values of RQD tend to increase with depth. For these cases, Peck et al. recommended that an average value of RQD within a depth equal to one-fourth of the footing width from the bottom of the foundation be used instead.

Peck et al. further recommended that the allowable bearing pressures from Table 6.2 should not exceed the unconfined compressive strength of the intact rock core sample and the allowable stress of the foundation material. No increase in bearing pressure is allowed for footing embedment because the design values given in Table 6.2 are based on settlement limitation rather than rock strength.

An example of the use of this procedure is given in Figure 6.2.

6.2.4 Empirical Design Procedure for Less Competent Jointed Rock

Carter and Kulhawy (1988) developed an empirical procedure for estimating ultimate bearing capacity of jointed or broken rock. The procedure is based on unconfined compressive strength of the intact rock core sample. Depending on rock mass quality, ultimate bearing capacity of the rock mass varies from a small fraction to six times the unconfined strength of the rock core sample. The authors further indicated that the rock mass quality should preferably be determined using the Geomechanics Rock Mass Rating (RMR) System (Bieniawski, 1988), or the Norwegian Geotechnical Institute (NGI) Rock Mass Classification System (Barton et al., 1974).

6.2.5 Rational Methods

Depending on the relative spacing of joints and rock layering, bearing capacity failures for foundations in rock may take several different forms, as shown in Figure 6.3. Except for the case of a rock mass with closed joints, the failure modes are different from those in soil. Procedures for estimating the bearing capacity have been developed for each of the failure modes shown in Figure 6.3. Details of these procedures can be found in Kulhawy and Goodman (1987), Goodman (1989), and Sowers (1979).

Table 6.1. Presumptive bearing pressures (tsf) for foundations on rock. (After Putnam, 1981)

Code	Year [1]	Bedrock[2]	Sound Foliated Rock	Sound Sedimentary Rock	Soft Rock[3]	Soft Shale	Broken Shale
Baltimore	1962	100	35	---	10	---	(4)
BOCA	1970	100	40	25	10	4	1.5
Boston	1970	100	50	10	10	---	(4)
Chicago	1970	100	100	---	---	---	---
Cleveland	1951/1969	---	---	25	---	---	---
Dallas	1968	$.2q_u$(5)	$2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
Detroit	1956	100	100	9600	12	12	---
Indiana	1967	$.2q_u$	$2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
Kansas City	1961/1969	$.2q_u$	$2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
Los Angeles	1970	10	4	3	1	1	1
New York City	1970	60	60	60	8	---	---
New York State	---	100	40	15	---	---	---
Ohio	1970	100	40	15	10	4	---
Philadelphia	1969	50	15	10-15	8	---	---
Pittsburgh	1959/1969	25	25	25	8	8	---
Richmond	1968	100	40	25	10	4	1.5
St. Louis	1960/1970	100	40	25	10	1.5	1.5
San Francisco	1969	3-5	3-5	3-5	---	---	---
Uniform Building Code	1970	$.2q_u$	$2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
NBC Canada	1970	---	---	100	---	---	---
New South Wales, Australia	1974	---	---	33	13	4.5	---

Note: 1 - Year of code or original year and date of revision
2 - Massive crystalline bedrock
3 - Soft and broken rock, not including shale
4 - Allowable bearing pressure to be determined by appropriate city official
5 - q_u = unconfined compressive strength

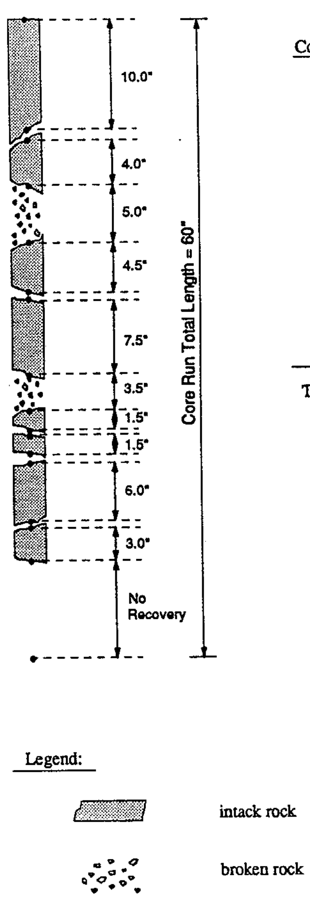
Table 6.2. Allowable bearing pressures of jointed rock. (After Peck, Hanson and Thornburn, 1974)

RQD	Allowable Bearing Pressure* (tsf)
100	300
90	200
75	120
50	65
25	30
0	10

*Note: If the recommended value of allowable bearing pressure exceeds the unconfined compressive strength of the rock or allowable stress of concrete, the allowable bearing pressure should be taken as the unconfined compressive strength, or the allowable stress of concrete, whichever is less.

Table 6.3. RQD as an index of rock quality. (After Deere, 1963)

RQD (%)	Rock Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very poor



Core Recovery	Modified Core Recovery
10.0"	10.0"
4.0"	4.0"
broken rocks	—
4.5"	4.5"
7.5"	7.5"
3.5"	—
1.5"	—
1.5"	—
6.0"	6.0"
3.0"	—
Total = 41.5"	Total = 32.0"

$$\text{Core Recovery} = \frac{41.5}{60} \times 100\,\% = 69\,\%$$

$$\text{RQD} = \frac{32.0}{60} \times 100\,\% = 53\,\%$$

Figure 6.1. Determination of RQD (modified core recovery).

The site conditions at a bridge abutment, near Pennsylvania Turnpike (Lehigh county) are given in Figure 6.2a. Based on this information, estimate the allowable bearing pressure for the 29.75 ft x 14.0 ft footing for the bridge abutment. The footing was placed at an elevation of 380 ft.

(1) Determine design value for RQD.

Borehole #79

Core Run 1 (El. 377.5 ~ 382.5 ft) -- RQD = 25%

Core Run 2 (El. 372.5 ~377.5 ft) -- RQD = 29%

Borehole #80

Core Run 1 (El. 375.6 ~ 380.6 ft) -- RQD = 15%

Core Run 2 (El. 370.6 ~ 375.6 ft) --RQD = 70%

Use RQD = 15% for the estimation of allowable bearing pressure, q_a.

(2) Estimate the allowable bearing pressure.

From Table 6.2, the allowable bearing pressure can be interpolated as follows:

$$q_a = 10 + \left(\frac{15 - 0}{25}\right)(30 - 10) = 22 \text{ tsf}$$

According to Peck, et al., settlement of the footing would be less than 0.5 inch. In Fig. 6.4 - Example 6.2, the settlement is estimated using elastic theory.

Figure 6.2. Example 6.1—estimating allowable bearing pressure of a footing on rock using the Peck et al. procedure.

6.3 SETTLEMENTS OF FOUNDATIONS ON ROCK

For most ordinary structures, where the imposed loadings are not exceptionally large, settlements of footings supported on rock are not large enough to cause problems. As noted earlier, if footings for these structures are designed on the basis of the procedure proposed by Peck et al. (1974), the settlements will usually be smaller than 0.5 in. However, in some cases such as piers for high-rise structures or abutments for arch bridges, where the foundations may be subjected to very large loadings, and where settlement tolerance may be small, estimation of settlement may be an important design consideration.

The characteristics of the discontinuities in a rock mass have a dominant influence on its compressibility. In rock masses containing seams of soft material, in porous limestone and in clayshale, consolidation and secondary settlements may occur. In these cases, the procedures described in Chapter 5 may be used to estimate settlements. For most other rock masses, the settlement occurs immediately upon application of the load, and its magnitude may be estimated by using elastic theory.

According to elastic theory, the settlement of a footing is related to footing size and load as follows:

$$\rho_m = \frac{P\,(1 - \nu_m^{\,2})}{\beta_z E_m A^{0.5}} \tag{6.3.1}$$

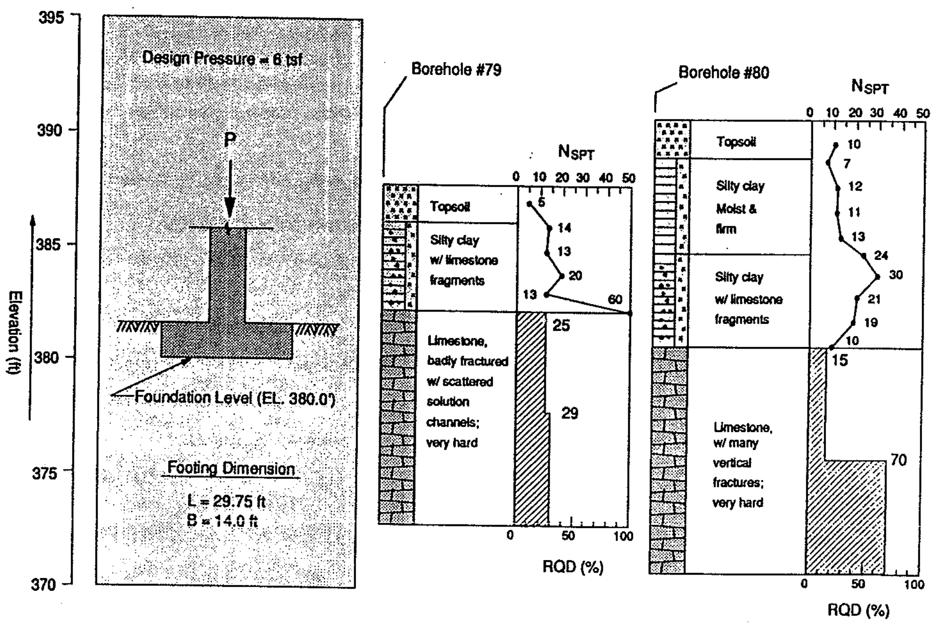


Figure 6.2a. Geological conditions for a bridge abutment footing near Pennsylvania Turnpike (Lehigh County).

in which ρ_m = settlement of rock mass, in length units; P = applied load, in force units; ν_m = Poisson's ratio for rock mass, dimensionless; β_z = shape and rigidity factor, dimensionless; E_m = Young's modulus for rock mass, in pressure units (this value must represent the properties of the rock mass, including joints, and not simply the intact material between joints); and A = footing area, in length^2 units.

Using typical values of ν_m and β_z, Kulhawy (1978) suggested that for circular, square, and rectangular footings (L/B $\leq$ 3) Eq. 6.3.1 may be written as:

$$\rho_m \simeq \frac{0.9P}{E_m A^{0.5}} \tag{6.3.2}$$

When L/B $\geq$ 10, Eq. 6.3.1 may be approximately expressed as:

$$\rho_m \simeq \frac{0.7P}{E_m A^{0.5}} \tag{6.3.3}$$

For rectangular footings with L/B ratio ranging between 3 and 10, the settlement may be determined by interpolating the results from Eqs. 6.3.2 and 6.3.3.

The accuracy with which settlement can be estimated by using elastic theory is dependent on the accuracy with which the value of rock mass modulus can be estimated. In some cases the value of E_m can be estimated through empirical correlations with the value of Young's modulus for the intact rock between joints. For unusual or poor rock mass condition, it may be necessary to determine the modulus from in situ tests, such as plate loading and pressuremeter tests.

Because of the presence of rock fractures, the rock mass modulus is smaller than the modulus of the intact rock between joints. The difference in modulus is related to the discontinuity spacing, which in turn can be correlated with RQD, as indicated in Table 6.4 (Kulhawy, 1978). To use Table 6.4, values of E_r (Young modulus of rock core sample) and K_n (normal stiffness of discon-

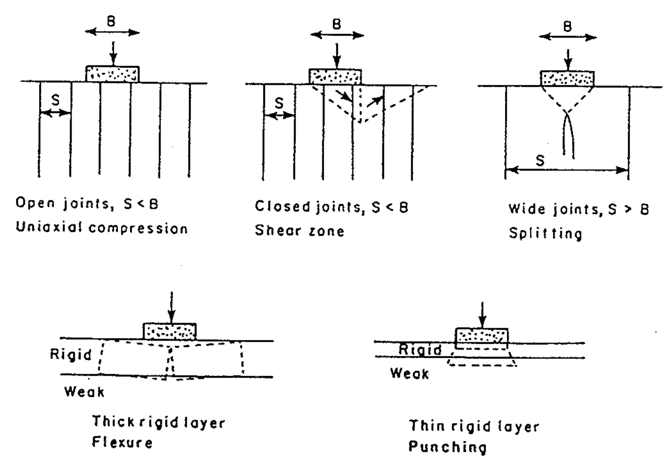


Figure 6.3. Bearing capacity failure modes for foundations in rock. (After Sowers, 1979)

tinuities) can be determined from laboratory testing. In the absence of site-specific data, Kulhawy (1978) suggested that typical values of E_r/K_n ranging from 0.2 m to 4.2 m, with an average value of about 1.2 m, may be used for preliminary analyses.

The use of Kulhawy's procedure to estimate settlement of a foundation on jointed rock is shown in Figure 6.4.

6.4 DESIGN OF SHALLOW FOUNDATIONS IN ROCK USING LOAD AND RESISTANCE FACTOR DESIGN APPROACH

As discussed in previous chapters, an alternative method of providing safety margin for foundations is through the use of load and resistance factors. In principle, the procedures de-

Table 6.4. Values of modulus reduction factor, $\alpha_E = E_m/E_r$. (After Kulhawy, 1978).

RQD (%)	Value of α_E for E_r/K_n*		
	0.1m	0.5m	1.0m
<10	0.22	0.06	0.03
20	0.35	0.10	0.05
30	0.40	0.13	0.06
40	0.44	0.15	0.08
50	0.46	0.16	0.09
60	0.50	0.18	0.11
70	0.53	0.20	0.12
80	0.56	0.22	0.14
90	0.70	0.30	0.18
100	0.92	0.75	0.60

*Note: E_r/K_n is in metres; and
- E_r = Young's modulus for rock core sample
- K_n = Normal stiffness for discontinuities
- E_m = Young's modulus for rock mass
- For $E_r/K_n \geq 10$, use equivalent soil modulus for analysis.

scribed in Section 4.13 may be extended directly for use in design of foundations on rock.

In LRFD the safety against ultimate bearing capacity failure (an ultimate limit state) is ensured if:

$$\phi(q_{ult}A) \geq \Sigma\gamma_i Q_i \qquad (6.4.1)$$

in which q_{ult} = ultimate bearing capacity of the rock mass; A = footing area, in length^2 units; Q_i = load effect due to load component i, in force units; ϕ = performance factor, dimensionless; and γ = load factor, dimensionless. Values of load factors are given in Table 2.1.

For foundations on good quality rock, the design may be controlled by structural considerations. The safety against structural failure can be checked using the following expression:

$$\phi_a P_n \geq \Sigma\gamma_i Q_i \qquad (6.4.2)$$

in which P_n = nominal structural axial load capacity of the footing; ϕ_a = performance factor for axial loading, and other terms are as defined previously (for concrete footings, ϕ_a is usually taken as 0.7). The use of LRFD concepts in design of a footing on jointed rock masses is illustrated in Figure 6.5.

6.5 SPECIAL GEOLOGICAL PROBLEMS

Some special geological features in rock may present difficulties in foundation design. These include: weathering of rock, solution cavities, swelling of rock, creep, and mining subsidence. These special problems may call for special design considerations or foundations treatments. In some instances, the presence of sink holes in limestone may make the use of footing foundation impractical. An excellent discussion on these special geological problems is presented by Peck (1976), citing case study examples.

Estimate the settlement of the 29.75 ft x 14.0 footing described in Example 6.1. The footing is 3 ft thick, and the elastic modulus of the intact limestone, E_r, is estimated to be 40 x 10^3 tsf. For the purpose of this example, the design pressure (unfactored) for the footing is assumed to be 6 tsf.

Since normal stiffness, K_n, for the discontinuity is not known,

$E_r/K_n \approx 0.5$m for a first approximation.

From Table 6.4, for RQD = 15%, the modulus reduction factor, α_E, is

$$\alpha_E = 0.08$$

Elastic modulus for the rock mass can be estimated as

$$E_m = \alpha_E\, E_r = (0.08)\,(40 \times 10^3) = 3.2 \times 10^3 \text{ tsf}$$

The unfactored design load is

$$P = pA = (6)\,(29.75)\,(14.0) = 2500 \text{ tons}$$

For $L/B = \frac{29.75}{14.0} = 2.1$, settlement of the rock mass can be estimated using the expression:

$$\rho_m = \frac{0.9P}{E_m A^{0.5}} = \frac{(0.9)\,(2500)}{(3.2 \times 10^3)\,(29.75 \times 14.0)^{0.5}}$$

$$\rho_m = 0.034 \text{ ft or } 0.41 \text{ inch}$$

which is smaller than the tolerable value of one inch of settlement frequently adopted in practice.

If the design were to be based on the allowable bearing pressure of 22 t/ft^2, as calculated using Peck, et al. procedure in Example 6.1, a smaller footing would have been used.

Area of the smaller footing, $A' = \frac{2500}{22} = 114 \text{ ft}^2$

Estimated settlement of the rock mass would then be

$$\rho'_m = \frac{(0.9)\,(2500)}{(3.2 \times 10^3)\,(114)^{0.5}} = 0.066 \text{ ft}$$

or $\rho'_m = 0.8$ inch

The calculated value agrees fairly well with the value of 0.5 inch suggested with Peck, et al., provided the assumed value of $E_r/K_n \approx 0.5$ m is representative of the site condition.

Figure 6.4. Example 6.2—estimating settlement of a footing on jointed rock.

6.6 CONSTRUCTION CONSIDERATIONS

According to Sowers (1979), two major practical concerns for footings on rock are as follows: (1) Good contact between rock mass and foundation—To ensure proper performance of foundation, good contact between rock mass and the foundation is necessary. The presence of local defects may create contact problems that require special treatment. Figure 6.6 presents several typical contact problems and the suggested solutions. These include filling up a narrow soft seam with "dental" concrete, anchoring footings on dipping rock surface with dowel bars or rock bolts, and avoiding the so-called shelf hazard by placing the foundation on the stiffer rock layer. (2) Effect of excavation on rock quality—Excavation by blasting often results in overbreak and fractures or opening of joints in the rock. To avoid potential

settlement problems, the excavated rock surface should be properly cleaned, and the fractured rock below the foundation level should be replaced by lean concrete or well-compacted gravel.

Using LRFD concepts, determine whether the 29.75 x 14.0 ft footing described in Example 6.1 has adequate capacity against soil failure beneath the footing. For the purpose of this example, the footing was assumed to be founded at an elevation of 375.0 ft, and that the intact limestone has an unconfined compressive strength of 1500 tsf.

(1) Calculate magnitude of factored loads.

Total unfactored load = (6) (29.75) (14.0) = 2500 tons

The proportion of dead and live loads are not known. For this example it will be assumed that the dead load is 1875 tons (75% of the total) and that the live load is 625 tons (25%).

Factored dead load = $\gamma_D P_D$ = (1.3) (1875) = 2438 tons

Factored live load = $\gamma_L P_L$ = (2.17) (624) = 1354 tons

Total factored load = 2438 + 1354 ≈ 3790 tons

(2) Calculate magnitude of factored bearing capacity.

Using the procedure described in Section 6.2.4, ultimate bearing capacity (q_{ult}) of the rock mass was estimated as,

q_{ult} = 22.5 tsf

Ultimate bearing load, $Q_{ult} = q_{ult} A$

Q_{ult} = (22.5) (29.75) (14.0) = 9370 tons

With a performance factor (ϕ) of 0.6, the factored bearing load is,

$Q_{fu} = \phi\, Q_{ult}$ = (0.6) (9370) = 5620 tons

which is greater than the total factored load of 3790 tons. Thus, the footing has adequate capacity against bearing capacity failure.

Figure 6.5. Example 6.3—design of a footing on rock using LRFD procedure (ultimate limit state).

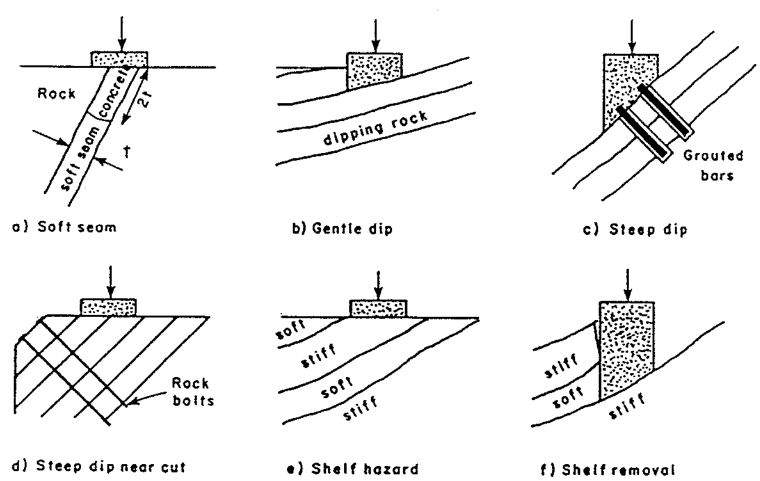


Figure 6.6. Rock foundation contact problems. (After Sowers, 1979)

References

American Association of State Highway and Transportation Officials (1989), Standard Specifications for Highway Bridges, Fourteen Edition, Washington, D.C.

American Association of State Highway and Transportation Officials (1970), NCHRP Synthesis of Highway Practice No. 5 - Scour At Bridge Waterways, Highway Research Board, National Research Council, 37 pp.

American Association of State Highway and Transportation Officials (1988), Manual on Subsurface Investigations, Washington, D.C., 391 pp.

American Association of State Highway and Transportation Officials (1986), AASHTO Materials, Part II - Test, 13th Edition, Washington, D. C., pp. 273 - 1177.

American Concrete Institute (1982), ACI's Guide to Durable Concrete, ACI Guide No. 201.2R-77, Detroit, 37 pp.

American Society of Testing and Materials (1990), Annual Book of Standards, Vol. 04.08: Soil and Rock, Building Stone, Geotextiles, ASTM, Philadelphia, 1189 pp.

Baguelin, F., Jezequel, J. F., and Shields, D. H. (1978), The Pressuremeter and Foundation Engineering, Trans Tech Publications, Clausthal, 617 pp.

Barton, N. R., Lien, R., and Linde, J. (1974), "Engineering Classification of Rock Masses and Its Application in Tunnelling", Rock Mechanics, Vol. 6, No. 4, pp. 189-236.

Bazaraa, A. R. S. S. (1967), "Use of Standard Penetration Test for Estimating Settlements of Shallow Foundations on Sand", Ph. D. Dissertation Submitted to Department of Civil Engineering, University of Illinois, 380 pp.

Bieniawski, Z. T. (1988), "The Rock Mass Rating (RMR) System (Geomechanics Classification) in Engineering Practice", Proc., Rock Mechanics Classification System for Engineering Purposes, ASTM STP 984, ASTM, Philadelphia, pp. 17-34.

Bjerrum, L. (1963), "Allowable Settlement of Structures", Proc., 3rd European Conf. on Soil Mechanics and Foundation Engineering, Wiesbaden, Vol. 2, pp. 73-96.

Bowles, J. E. (1988), Foundation Analysis and Design, Fourth Edition, McGraw-Hill Publication Co., p. 255.

Briaud, J. L. (1986), "Pressuremeter and Foundation Design", Proc. Conference on Use of In Situ Tests in Geotechnical Engineering, ASCE Geotechnical Publication No. 6, Virginia Polytechnic Institute and State University, pp. 74-116.

Briaud, J. L. (1990), The Pressuremeter, The Netherlands, A. A. Balkeema, in press, 314pp.

Brown, W. G. (1964), "Difficulties Associated with Predicting Depth of Freeze or Thaw", Canadian Geotechnical Journal, Vol. 1, No. 4, pp. 215-226.

Canadian Geotechnical Society (1985), Canadian Foundation Engineering Manual, Second Edition, 456 pp.

Carter, J. P. and Kulhawy, F. H. (1988), Analysis and Design of Drilled Shaft Foundations Socketed into Rock, Report No. EL-5918, Empire State Electric Engineering Research Corporation and Electric Power Research Institute, 158 pp.

Christian, J. T. and Carrier, W. D. (1978), "Janbu, Bjerrum and Kjaernsli's Chart Reinterpreted", Canadian Geotechnical Journal, Vol. 15, pp. 123-128.

Copp, H. D. and Johnson, J. P. (1987), Riverbed Scour At Bridge Piers, US Department of Transportation, Federal Highway Administration, 73pp.

D'Appolonia, D. J., D'Appolonia, E., and Brisette, R. F. (1970), "Settlement of Spread Footings on Sand", (closure), Proc. Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM2, pp. 754-761.

Deere, D. U. (1963), "Technical Description of Rock Cores For Engineering Purposes", Rock Mechanics and Engineering Geology, Vol. 1, No. 1, pp. 16-22.

Deere, D. U. and Deere, D. W. (1988), "The Rock Quality Designation (RQD) Index in Practice", Proc., Rock Classification System for Engineering Purposes, ASTM STP 984, ASTM, Philadelphia, pp. 91-101.

Duncan, J. M. and Buchignani, A. L. (1976), An Engineering Manual for Settlement Studies, Department of Civil Engineering, University of California at Berkeley, 94 pp.

Duncan, J. M., Horz, R. C., and Yang, T. L. (1989), Shear Strength Correlations for Geotechnical Engineering, Charles E. Via Jr. Department of Civil Engineering, Virginia Polytechnic Institute and State University, 100 pp.

Duncan, J. M. and Tan (1991), Engineering Manual for Estimating Tolerable Movements of Bridges, Final Draft for NCHRP 24-4, Charles E. Via Jr. Department of Civil Engineering, Virginia Polytechnic and State University.

Federal Highway Administration (1985), Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications, US Department of Transportation, 33 pp.

Federal Highway Administration (1988), "Technical Advisory -- Interim Procedures for Evaluating Scour at Bridges", US Department of Transportation, Office of Engineering, Bridge Division, 62 pp.

Garga, V. K. and Quin, J. T. (1974), "An Investigation on Settlement of Direct Foundations on Sand", Proc., Conference on Settlements of Structures, Cambridge, England, Pentech Press, pp 22-36.

Goodman, R. E. (1989), Introduction to Rock Mechanics, Second Edition, John Wiley and Sons, New York, 562 pp.

Hansen, J. B. and Hansen, B. (1957), "Foundations of Structures --(a) General Subjects and Foundations other than piled Foundations", General Report, 4th ICSMFE, London, Vol II, pp 441-447.

Hansen, J. B. (1970), "A Revised and Extended Formula for Bearing Capacity", Bulletin No. 28, Danish Geotechnical Institute, Copenhagen, pp 5-11.

Holtz, W. G. and Gibbs, H. J. (1956), "Engineering Properties of Expansive Clays", Transactions, ASCE, Vol. 120.

Hvorslev, M. J. (1948), Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes, Waterway Experiment Station, Vicksburg, Miss., 465 pp.

International Society for Rock Mechanics (1978), "Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses", prepared by Commission on Standardization on Laboratory and Field Tests, pp 319-368. (Reprinted in ISRM Suggested Methods: Rock Characterization, Testing and Monitoring, E. T. Brown, Ed., Pergamon Press, Oxford, England, 211 pp.)

Janbu, N. (1963), "Soil Compressibility As Determined By Oedometer and Triaxial Tests", Proc., European Conference of Soil Mechanics and Foundation Engineering, Vol. I, Wiesbaden, .

Janbu, N. (1967), Settlement Calculations Based on Tangent Modulus Concept, Bulletin No. 2, Soil Mechanics and Foundation Engineering Series, The Technical University of Norway, Trondheim, 57 pp.

Janbu, N. (1985), "Soil Models in Offshore Engineering", 25th Rankine Lecture, Geotechnique, Vol 35, No. 3, p.241.

Kasim, A. G., Chu, M. Y., and Jensen, C. N. (1986), "Field Correlation of Cone and Standard Penetration Tests", Proc., Journal of the Geotechnical Engineering Division, ASCE, Vol. 112, No. GT3, pp. 368-372.

Kulhawy, F. H. (1978), "Geomechanical Model for Rock Foundation Settlement", Proc., Journal of Geotechnical Engineering Division, ASCE, Vol. 104, No. GT2, pp. 211-227.

Kulhawy, F. H. and Goodman, R. E. (1987), "Foundations in Rock", Chapter 55 in Ground Engineering Reference Manual, Edited by F. G. Bell, Butterworths Publishing Co.

Kulhawy, F. H., Trautmann, C.H., Beech, J. F., O'Rourke, T. D., and McGuire, W. (1983), "Bearing Capacity of Shallow Foundations in Soil", Chapter 7 of Transmission Line Structure Foundations for Uplift-Compression Loading, EPRI Report EL-2870, Electric Power Research Institute, Palo Alto, California, 23 pp.

Lambe, T. W. and Whitman, R. V. (1969), Soil Mechanics, John Wiley and Sons, New York, 553 pp.

Menard, L. (1965), "Regle pour le calcul de la force portante et du tassement des fondations en fonction des resultats pressiometriques", Proc., 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, Vol. 2, pp. 295-299.

Menard, L. and Rouseau, J. (1962), "L' Evaluation des Tossements Tendances Nouvelles", Sols-Soils, Vol. 1., No. 1.

Mesri, G. (1973), "Coefficient of Secondary Compression", Proc., Journal of Soil Mechanics and Foundation Engineering, ASCE, Vol. 99, No. SM1, pp. 123-137.

Meyerhof, G. G. (1953), "The Ultimate Bearing Capacity of Foundations Under Eccentric and Inclined Loads", Proc., 3rd International Conference on Soil Mechanics and Foundation Engineering, Zurich, Vol 1, pp. 440-445.

Meyerhof, G. G. (1956), "Penetration Tests and Bearing Capacity of Cohesionless Soils", Proc., Journal of Soil Mechanics and Foundation Engineering, ASCE, Vol. 82, No. SM1, pp. 1-11.

Meyerhof, G. G. (1957), "The Ultimate Bearing Capacity of Foundations On Slopes", Proc., 4th International Conference on Soil Mechanics and Foundation Engineering, London, Vol. 1, pp. 384-386.

Meyerhof, G. G. (1970), "Safety Factors in Soil Mechanics", Canadian Geotechnical Journal, Vol. 7, pp. 349-355.

Peck, R. B. (1976), "Rock Foundations for Structures", Proc., Conference on Rock Engineering for Foundations and Slopes, Vol. II, Boulder, CO., pp. 1-21.

Peck, R. B., Hanson, W. E., and Thornburn, T. H. (1974), Foundation Engineering, Second Edition, John Wiley and Sons, New York, 514 pp.

Putnam, J. B. (1981), "Analysis and Design of Foundations on Continuous Rock", M. Sc. Thesis, Submitted to Syracuse University.

Robertson, P. K. (1986), "In Situ Testing and Its Application to Foundation Engineering", Canadian Geotechnical Journal, Vol.23, No., pp. 573-594.

Robertson, P. K. and Campanella, R. G. (1984), Guidelines for Use and Interpretation of the Electronic Cone Penetration Test, Soil Mechanics Series No. 69, Department of Civil Engineering, The University of British Columbia, Vancouver.

Schmertmann, J. H. (1970), "Static Cone to Compute Static Settlement Over Sand", Proc., Journal of Soil Mechanics and Foundation Engineering, ASCE, Vol. 96, NO. 3, pp. 1011-1043.

Schmertmann, J. H. (1977), Guidelines for Cone Penetration Test - Performance and Design, US Department of Transportation, Federal Highway Administration, pp. 54-55.

Schmertmann, J. H., Hartman, J. P., and Brown, P. R. (1978), "Improved Strain Influence Factor Diagram", Proc., Journal of Geotechnical Engineering Division, ASCE, Vol. 104, No. GT8, pp. 1131-1135.

Schultz, E. and Melzer, K. J. (1965), "The Determination of the Density and the Modulus of Compressibility of Non-cohesive Soils by Soundings", Proc., 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, Vol. 1, pp 354-358.

Sowers, G. F. (1979), Introductory Soil Mechanics and Foundations: Geotechnical Engineering, MacMillan Publishing Co., New York, 621 pp.

Steinbrenner, W. (1934), " Tafeln zur Setzungberechnung", Die Strasse, pp 121-124.

Tan, C. K. and Duncan, J. M. (1991), "Settlements of Footings on Sand - Accuracy and Reliability", to be published in the Proc., Geotechnical Engineering Congress, Boulder, CO.

Taylor, B. B. and Matyas, E. L. (1983), "Influence Factor for Settlement Estimates of Footings on Finite Layers", Canadian Geotechnical Journal, Vol. 20, No. , pp 832-835.

Terzaghi, K. and Peck, R.B. (1967), Soil Mechanics in Engineering Practice, Second Edition, John Wiley and Sons, New York, 729 pp.

Tomlinson, M. J. (1986), Foundation Design and Construction, Fifth Edition, Longman Scientific and Technical, London, England, 842 pp.

US Corps of Engineers (1949), Addendum No. 1, 1945-47, Report on Frost Penetration, 1944-45, Corps of Engineers, US Army, New England Division, Boston.

US Department of Navy (1982), NAVFAC DM- 7.1-- Soil Mechanics, Naval Facilities Engineering Command, VA, 348 pp.

Vesic, A. S. (1973), "Analysis of Ultimate Loads on Shallow Foundations", Proc., Journal of Soil Mechanics and Foundation Engineering, ASCE, Vol. 99, No. SM1, pp 45-73.

Notations and Symbols

<u>SYMBOLS</u>

A	= area of footing base
B	= footing width
B'	= reduced effective footing width
C_c	= compression index
C_p	= correction factor for initial overburden pressure at foundation level
C_t	= time rate (or creep correction) factor for settlement of cohesionless soils
C_u	= coefficient of uniformity
C_w	= correction factor for water table
C_α	= coefficient of secondary compression
c	= cohesion
c_γ, c_q	= compressibility factors
c_v	= coefficient of consolidation
D_{10}	= size of soil particle corresponding to 10% passing by weight
D_{15}	= size of soil particle corresponding to 15% passing by weight
D_f	= embedment depth for foundations
D_r	= relative density of soils
D_w	= depth to groundwater table
d_q	= depth correction factor
E	= elastic modulus
E_m	= elastic modulus for rock masses
Ep	= pressuremeter modulus
E_r	= elastic modulus for intact rock core sample
E_s	= in situ modulus of soils
E_u	= undrained modulus of soils
e	= void ratio
e_o	= initial void ratio
e_x	= eccentricity of vertical load
F	= safety factor
H	= horizontal load (unfactored)
H_f	= factored horizontal load
H_t	= thickness of compressible layer
I'	= equivalent Schmertmann's improved settlement influence factor
I_h	= horizontal movement influence factor used in elastic theory
I_p	= plasticity index
I_r	= compressibility index
I_z	= Schmertmann's improved settlement influence factor
I_{zp}	= Schmertmann's peak settlement influence factor
I_{zt}	= Schmertmann's improved settlement influence factor evaluated at bottom of footing
I_ρ	= settlement influence factor used in elastic theory
I_θ	= rotation influence factor used in elastic theory
i_γ, i_c, i_q	= load inclination factors used in bearing capacity theory
K_n	= normal stiffness for rock discontinuities
k	= bearing capacity factor used with pressuremeter data
L	= footing length
L'	= reduced effective footing length
m	= Janbu's modulus number
M_t	= Janbu's tangent modulus
N	= Standard Penetration Test (SPT) blow count
$\bar{N}$	= average SPT blow count, corrected for submergence effect
N_c, N_γ, N_q	= bearing capacity factors in rational theory
$N_{\gamma m}, N_{cm}, N_{qm}$	= modified bearing capacity factors in rational theory
P	= applied load (unfactored)

p_D = dead load (unfactored)

p_L = live load (unfactored)

p_n = nominal axial structural load (unfactored)

p = average bearing pressure

p_a = atmospheric pressure

p_o = total horizontal pressure at depths where pressuremeter tests are performed

p_l = limiting pressure measured in pressuremeter tests

p_p = preconsolidation pressure

Q = load effect (unfactored)

Q_τ = sliding resistance of a footing

q_a = allowable bearing pressure

q_c = cone resistance (tip)

q_u = unconfined compressive strength

q_{ult} = ultimate bearing capacity (or pressure) of foundation soils or rock masses

$q_{ult\ vert}$ = vertical component of ultimate bearing capacity of footings subjected to inclined load

R_I = reduction factor for load inclination used in empirical procedures

R_n = nominal resistance

r_o = initial total vertical pressure at foundation level

s = shear strength of the interface between footing and soil

s_c, s_q = shape factors

s_u = undrained shear strength of soils

t = time

t_p = time for primary compression

t_{sc} = time for which secondary settlement is to be calculated

V = vertical load (unfactored)

V_f = factored vertical load

v = seismic velocity

y = moment arm for horizontal load

z = depth

z_{max} = maximum depth defined in Schmertmann's improved settlement influence diagram

z_p = depth to Schmertmann's peak settlement influence factor

<u>Greek</u>

α = empirical settlement coefficient used with pressuremeter test data

α_E = modulus reduction factor

β = load factor coefficient for a load component, when used in Section 2.2

β = safety index, when used in Section 2.3.4

γ = load factor used in load and resistance factor design (LRFD) procedures

γ = unit weight of soils

γ_d = dry unit weight of soils

δ = frictional angle of the interface between foundation and soil

Δz = thickness of a sublayer

$\Delta\sigma_v'$ = increase in effective vertical stress

Δh = horizontal movement

Δp = net bearing pressure increase at foundation level

ϵ_v = vertical strain

θ = rotation

λ_d, λ_c = shape factors used with pressuremeter test data

μ_o = influence factors for immediate settlement, accounting for effect of footing embedment

μ_1 = influence factors for immediate settlement, accounting for effect of finite thickness of a compressible layer

ν = Poisson's ratio

ρ = footing settlement

ρ_f = final settlement

ρ_i = initial settlement

ρ_m = settlement of rock masses

ρ_{sc} = secondary settlement

ρ_{tol} = tolerable settlement

ρ_t = settlement at time, t

σ = standard deviation

σ_v' = effective overburden pressure

σ_{va}' = average effective vertical pressure

σ_{vo}' = initial effective vertical pressure (before construction) at foundation level when used in Section 5.3

σ_{vo}' = initial effective vertical pressure (before the footing load is applied) at mid-height of a sublayer when used in Section 5.4

σ_{vf}' = final effective vertical pressure

σ_{vp}' = effective vertical pressure evaluated at depth of Schmertmann's peak settlement influence factor

τ = shear stress of the interface under service load condition

ϕ = performance factor used in LRFD procedure

ϕ = frictional angle of soils or rocks

ϕ_a = performance factor for axial load

Part 2—Engineering Manual for Driven Piles

P.S.K. Ooi, J.M. Duncan, K.B. Rojiani
R.M. Barker

CONTENTS

CHAPTER 1

INTRODUCTION

Piles are used to support many bridges, buildings, and other structures. The primary function of these foundations is to transmit loads to the ground safely and to avoid excessive settlements or lateral movements. Piles are especially useful where underlying layers include weak or compressible strata.

The purpose of this manual is to draw together practical procedures for the design of pile foundations. The theoretical and empirical procedures described provide methods suitable for design of single piles and pile groups that are subjected to vertical and horizontal loads.

The design procedures presented in this manual incorporate the concepts of load factor design, or LFD. The LFD approach provides a logical method of dealing with uncertainties of component loads, strength and behavior, and for incorporating suitable margins of safety. LFD and other procedures similar in format are being used with increasing frequency in civil engineering. Load factor design has been incorporated in the American Association of State Highway and Transportation Officials (AASHTO) specifications for design of bridge superstructures since the mid-1970s. Bridge engineers who use LFD for the superstructure must develop two sets of loads—one for the design of the superstructure and another for the design of the foundations (Barker et al., 1988). Development of load factor design procedures for bridge foundations will make this duplication of effort unnecessary.

In the sections that follow, a brief description of various types of deep foundations and piles is given in Chapter 2. Chapter 3 discusses the design requirements and the factors influencing the safety of pile foundations in bridges. Chapter 4 considers axial loading of piles, and Chapter 5 presents a new approach for the design of laterally loaded piles. Design examples are presented in the concluding sections of the design procedures.

CHAPTER 2

CLASSIFICATION OF DEEP FOUNDATIONS AND PILES

2.1 TYPES OF DEEP FOUNDATIONS

Deep foundations can be described as columnar elements in the soil which transfer the loads from a superstructure (such as a bridge or a building) into the soil or rock. Deep foundations must be able to support axial, horizontal, and uplift loads effectively.

Deep foundations can be divided into two classes: (1) piles that are installed by driving and (2) drilled shafts that are installed by placing concrete in drilled holes.

Driven piles can be subdivided into two categories: (1) displacement piles, which have solid sections or hollow sections with a closed end (a relatively large volume of soil is displaced by the pile during penetration); and (2) nondisplacement piles, which have relatively small cross-sectional areas, such as H piles and open-ended pipe piles that do not plug.

This manual discusses the design aspects of displacement and nondisplacement piles. The design of drilled shafts is dealt with separately in Part 4.

2.2 TYPES OF PILES

Figure 2.1 (Carson, 1965) shows typical maximum lengths and loadings frequently used in design for various types of piles. The advantages and disadvantages of each type of pile are discussed in the following sections.

2.2.1 Timber Piles

Timber piles are straight and slender sections of tree trunks with their branches removed. The lumber should be straight-grained with no defects, and the taper should be uniform.

Timber piles projecting above the groundwater must be treated with preservatives to retard deterioration. The bark should be removed because it reduces the depth of impregnation of the preservative.

Advantages: (1) They are light and therefore easy to handle. (2) They have a high strength to weight ratio. (3) They are resistant to decay when placed below the groundwater table.

Disadvantages: (1) They have relatively low structural capacities. (2) They are vulnerable to damage during driving through hard soil. (3) They are vulnerable to decay when placed above the groundwater table or in a splash zone. (4) They are difficult to splice.

2.2.2 Precast Concrete Piles (Including Prestressed Piles)

Precast concrete piles are long and slender units of reinforced concrete with square, circular, or octagonal cross sections. Prestress can be applied to precast concrete piles to achieve higher strength to weight ratio.

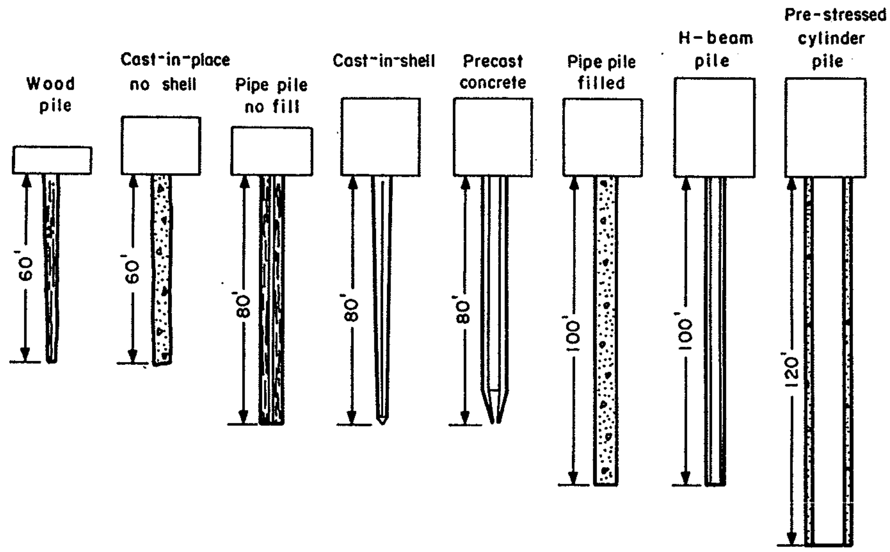


Figure 2.1. Summary of approximate maximum lengths of piles and unfactored loadings frequently used in design. (After Carson, 1965)

Advantages: (1) They have relatively large axial capacities. (2) The concrete mix can be designed for chemically aggressive ground or marine environments. (3) They can withstand hard driving. (4) Concrete piles can be prestressed. This results in a pile section with a higher strength to weight ratio. Prestressing offers an additional advantage in that it closes up cracks that are caused during driving and handling.

Disadvantages: (1) They are susceptible to damage during handling and driving. (2) It is difficult and costly to cut off excess length or splice more length after driving.

2.2.3 Steel-H Piles

H piles are made of steel, rolled into the shape of an H. They have two flanges connected by a web. The flange width is usually at least 85 percent of the depth of the pile section so that the pile is strong along its weak axis (Teng, 1962).

Advantages: (1) Steel-H piles are robust and light. (2) They can be easily spliced. (3) They are available in a variety of sizes. (4) They have high axial capacities and good resistance to buckling. (5) They can withstand hard driving and are useful for penetrating hard layers and even soft rock. (6) Only a small volume of soil is displaced during driving of H piles. Therefore, they are preferred in groups where the piles are closely spaced, and where undesirable ground heave and lateral displacements of the soil are anticipated.

Disadvantages: (1) They are susceptible to corrosion if unprotected. (2) End bearing resistance of unplugged H piles is relatively small because of its small cross-sectional area. (3) Steel-H piles are easily deflected by hard sloping layers and by underground obstructions.

2.2.4 Steel Pipe Piles

Steel pipe piles may be driven with either open or closed ends. They may be unfilled or filled with concrete. Unfilled open end pipe piles can be used instead of closed end ones if greater penetration depths are desired because the soil inside can be removed during driving.

Advantages: (1) They are light and not easily damaged during handling. (2) They can be spliced easily. (3) Pipe sections are available in a variety of sizes. (4) They have relatively high axial capacities and high resistance to buckling. (5) Closed end pipe piles can be easily inspected for deviations from the intended alignment. (6) The quality of steel and wall thickness can be strategically varied with depth according to the severity of the loads and bending moments anticipated along the pile.

Disadvantage: (1) They are vulnerable to corrosion if unprotected.

2.2.5 Other Pile Types

Two other types of piles commonly used include cast-in-place concrete piles and composite piles.

Cast-in-place concrete piles are constructed by first driving a steel shell into the ground. Driving with the aid of a mandrel inserted in the shell is optional. A reinforcing cage is then lowered into the shell, after which concrete is poured. The shell is withdrawn as the concrete is poured in the case of an open ended shell, or it may be left in the ground.

Steel shells are either uniform or tapered in cross section. Tapered shells provide a higher shaft resistance for piles in clay (Teng, 1962). Shells that are withdrawn can be reused. Another advantage of cast-in-place piles is that the alignment of the shell can be inspected before the concrete is poured.

Composite piles are combinations of different types of piles or drilled shafts; for example, a steel-H pile "stinger" placed on the end of a prestressed concrete pile. They are used to circumvent difficulties arising due to the site or ground conditions. The structural capacity of the pile is governed by the weakest material used. Good quality joints of two different pile materials must be ensured during construction.

2.3 FACTORS GOVERNING THE CHOICE OF PILES

The advantages and disadvantages of the various types of piles listed in the preceding sections merit consideration during pile selection. The following eight factors govern the choice of the

pile type: (1) structural strength of the pile, (2) durability, (3) ease of handling, (4) ease of splicing, (5) penetrability into hard strata, (6) ground displacement during driving, (7) availability, and (8) cost.

CHAPTER 3

DESIGN REQUIREMENTS FOR PILE FOUNDATIONS

The simplest and most economical type of foundation is the spread footing. However, spread footings are not always suitable. For instance, when a structure is underlain by soft clay or loose sand or is subject to scour, pile foundations may be a better alternative.

Pile foundations must be capable of transmitting the loads to the soil without reaching a "limit state". A limit state is reached when the structure no longer fulfills its design requirements. There are two types of limit states: (1) An *ultimate limit state* corresponds to the maximum load carrying capacity of the foundation. This may be reached through either structural or soil failure. An ultimate limit state corresponds to complete collapse. (2) A *serviceability limit state* corresponds to loss of serviceability, and occurs before collapse. A serviceability limit state involves unacceptable deformations or undesirable damage levels. This may be reached through excessive differential or total settlements, excessive lateral displacements, or structural deterioration of the piles.

3.1 LOAD FACTOR DESIGN CONCEPT

In load factor design (LFD), it is recognized that loads and resistances are probabilistic and not deterministic in nature. Different types and magnitudes of loads have varying probabilities of occurrence. In order to account for their differing probabilities of occurrence, each load component is amplified by a load factor, the value of which depends on the level of uncertainty of the load component.

The factored loads are compared to the design strengths or resistances. The design resistances are obtained by multiplying nominal values of resistance by performance factors, usually denoted as ϕ. The objective of design is to ensure that the design resistance is greater than or equal to the sum of the factored loads, i.e.,

$$\phi R \geq \Sigma \gamma_i Q_i \tag{3.1.1}$$

where ϕ = performance factor, R = resistance corresponding to the limit state considered, Q_i = load effect due to load component i, and γ_i = load factor for load component i.

Various combinations of loads are considered in design to ensure that the structure and foundation will have sufficient capacity to resist all of the types of loading to which it may be subjected during its life. This manual uses the load factors and load combinations described in the 1989 AASHTO specifications for the design of bridges.

3.2 LOAD FACTORS AND LOAD COMBINATIONS

Loads acting on bridge superstructures include one or more of the following: dead load, live and impact loads, thrust due to earth pressures, buoyancy, wind load, longitudinal and centrifugal forces caused by moving vehicles, earthquake loads, stream and ice flow forces, and forces induced by changes in the dimensions of the structure, such as shrinkage and temperature effects.

One difference between the loads acting on the superstructure and those that act on the foundation is that impact loads are usually assumed to be fully dissipated before reaching the foundation (exceptions are pile bent piers and integral abutments where the foundation should be designed to carry the impact loads). The load combinations and load factors for the design of the superstructure, as given in the 1989 AASHTO specifications, can be used for the design of foundations as follows:

$$\begin{aligned}\text{Total Load} = \gamma[&\beta_D D + \beta_L L + \beta_C CF + \beta_E E + \\ &\beta_B B + \beta_{SF} SF + \beta_W W + \beta_{WL} WL \\ &+ \beta_{LF} LF + \beta_R (R + S + T) + \\ &\beta_{EQ} EQ + \beta_{ICE} ICE]\end{aligned} \tag{3.2.1}$$

where γ = load factor (see Tables 3.1 and 3.2); β = coefficient (see Tables 3.1 and 3.2); D = dead load; L = live load; E = earth pressure; B = buoyancy; W = wind load; WL = wind load on live load, 100 pounds per linear ft; LF = longitudinal force from live load; CF = centrifugal force; R = rib shortening; S = shrinkage; T = temperature; EQ = earthquake; SF = stream flow pressure; and ICE = ice pressure.

The load combinations considered by AASHTO are given in Table 3.1. Each line in the table, designated by loading group numbers I through IX, gives the values of the load factors, γ, and the coefficients β that govern the contributions to the total load. For example, in group (load combination) I, total load = $1.3(D + 1.67L_n + CF + \beta_E E + B + SF)$.

Loading groups I, II, and III usually apply to the design of the superstructures and substructures; groups IV, V, and VI apply usually to the design of arches and frames; and groups VII, VIII, and IX apply usually to the design of substructures (Heins and Firmage, 1979). Column 14 of Table 3.2 gives the percentage increase in allowable stresses permitted in the load combinations, and is mainly used in working stress design. The increase in allowable stresses accounts for the fact that the probability of the load components reaching their maximum values simultaneously varies from one load combination to another.

Table 3.1. Table of coefficients of γ and β for ultimate limit states. (After AASHTO, 1989)

Col.No.	1	2	3	3A	4	5	6	7	8	9	10	11	12	13
		β-FACTORS												
GROUP	γ	D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE
I	1.3	β_D	1.67	0	1	β_E	1	1	0	0	0	0	0	0
IA	1.3	β_D	2.2	0	0	0	0	0	0	0	0	0	0	0
IB	1.3	β_D	0	1	1	β_E	1	1	0	0	0	0	0	0
II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0
III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0
IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0
V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0
VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0
VII	1.3	β_D	0	0	0	β_E	1	1	0	0	0	0	1	0
VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1
IX	1.2	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1

$(L+I)_n$ - Live load plus impact for AASHTO Highway H or HS loading
$(L+I)_p$ - Live load plus impact consistent with the overload criteria of the operation agency.

β_E = 1.3 for lateral earth pressure for retaining walls and rigid frames.
β_E = 0.5 for lateral earth pressure when checking positive moments in rigid frames.
β_E = 1.0 for vertical earth pressure
β_D = 1.0 for flexural and tension members

For Column Design

β_D = 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity
β_D = 1.0 when checking member for maximum axial load and minimum moment

Table 3.2. Table of coefficients of γ and β for serviceability limit states. (After AASHTO, 1989)

Col.No.	1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14
		β-FACTORS													
GROUP	γ	D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE	%
I	1	1	1	0	1	β_E	1	1	0	0	0	0	0	0	100
IA	1	1	2	0	0	0	0	0	0	0	0	0	0	0	150
IB	1	1	0	1	1	β_E	1	1	0	0	0	0	0	0	**
II	1	1	0	0	0	1	1	1	1	0	0	0	0	0	125
III	1	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125
IV	1	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125
V	1	1	0	0	0	1	1	1	1	0	0	1	0	0	140
VI	1	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140
VII	1	1	0	0	0	1	1	1	0	0	0	0	1	0	133
VIII	1	1	1	0	1	1	1	1	0	0	0	0	0	1	140
IX	1	1	0	0	0	1	1	1	1	0	0	0	0	1	150

$(L+I)_n$ - Live load plus impact for AASHTO Highway H or HS loading
$(L+I)_p$ - Live load plus impact consistent with the overload criteria of the operation agency.

** Percentage = $\dfrac{\text{Maximum Unit Stress (Operating Rating)}}{\text{Allowable Basic Unit Stress}} \times 100$

% in Column 14 is the maximum permissible percentage of basic unit stress for load group indicated

No increase in allowable unit stresses shall be permitted for members or connections carrying wind loads only.

β_E = 1.0 for vertical and lateral loads on all structures except reinforced concrete boxes.
β_E = 1.0 and 0.5 for lateral loads on rigid frames (check both loadings to see which one governs)

3.3 DESIGN REQUIREMENTS FOR PILES

Piles should be designed for both axial and lateral loading conditions. The two principal design considerations for piles under axial loads are ultimate load capacity and settlement. The ultimate load capacity of a pile may be governed either by the structural capacity of the pile or the bearing capacity of the soil. Piles that are subjected to lateral loads must also be safe against ultimate failure of the soil or the pile, and excessive lateral deflections.

3.3.1 Structural Capacity

Axially loaded piles may fail in compression or by buckling. Buckling may occur in long and slender piles that extend for a portion of their lengths through water or air. Scour of the soil around the piles could expose portions of their lengths and increase the likelihood of buckling.

A pile will fail in compression when the loads exceed the structural or soil capacity. The structural capacity of the pile is usually greater than the ultimate soil capacity except when the pile bears on sound rock. Nevertheless, the adequacy of the pile against structural failure must always be checked. The tensile capacity of piles should also be checked when the piles are subject to uplift loads.

Laterally loaded piles will fail in flexure if the induced bending moment exceeds the moment capacity of the pile. The structural capacity of the pile is dependent on both the moment and axial load. The structural adequacy of piles is checked using load-moment interaction diagrams. These are envelopes of the combinations of moment and axial load that would cause pile failure.

The amount of lateral reinforcement (spiral or ties) required for prestressed concrete piles is less than the amount required for columns (see PCI, 1985; or Issa and Yuan, 1989).

3.3.2 Soil Capacity

The ultimate bearing capacity of a pile is the sum of its tip and shaft capacities. During failure, the shear stress at the interface of the pile and soil reaches a limiting value. Piles are usually driven in groups, and the most important consideration is the capacity of the pile group. This is usually taken as the minimum of two values: (1) the sum of the individual pile capacities or (2) the ultimate capacity of the pile group and the soil between the piles, acting as a unit.

The ultimate lateral capacity of piles is usually not a controlling factor in the design of piles to resist lateral loads. The governing criterion in the lateral load design is usually either the maximum tolerable deflection or the structural capacity of the piles.

3.3.3 Movement

Horizontal movements in buildings are caused by wind loads, earth pressures, and earthquakes. Horizontal movements occur at bridge abutments and piers because of lateral forces from earth pressure, wind loads, stream flow forces, braking forces of

vehicles, and earthquakes. Lateral movements of buildings must be limited to prevent architectural and structural damage. Lateral movements of abutments and piers must be limited to prevent damage to bearings and expansion joints (functional and structural damage), and poor ride quality.

Excessive movements of pile foundations supporting bridges may lead to discontinuities in the slope of the riding surface, damage to the bridge superstructure, jamming of bearings and expansion joints, or even collapse. It is necessary in bridge design to estimate the maximum settlement and lateral movement anticipated in the foundations and to ensure that they fall within tolerable limits.

Load tests on instrumented piles have shown that the movement required to mobilize skin friction in piles is smaller than that required to mobilize end-bearing. The shaft capacity of a pile is fully mobilized when the settlement is between 0.1 in. and 0.4 in. (Circeo, 1986). The tip capacity, however, is mobilized after the pile settles about 8 percent of its diameter (Kulhawy et al., 1983). This is an important design consideration when the working load acting on the pile exceeds the shaft resistance. In this case, larger settlements are required to mobilize the portion of the tip resistance that supports the load not carried by skin friction.

Horizontal displacements occur at bridge abutments and piers because of lateral forces from earth pressure, wind loads, stream flow forces, braking forces of vehicles, and earthquakes. Lateral movements of abutments and piers must be limited to prevent damage to bearings and expansion joints. If both vertical and horizontal displacements are possible, the horizontal displacement of bridge foundations should be limited to 1 in. If vertical displacements are small, the horizontal displacements should be limited to 1.5 in. (Moulton et al., 1985).

3.3.4 Pile Driving and Installation

Piles can be damaged when stresses induced during pile driving exceed the structural capacity of the pile. The impact of the hammer during driving sends a compressive stress wave down the pile. If a pile is driven through soil of high resistance into a soil of low resistance, the stress wave is reflected at the pile point, causing tension to develop near the pile tip, and these stresses can damage concrete piles. On the other hand, if the pile is driven onto a hard rock, reflection of the stress wave at the pile-rock interface induces a compression stress at the toe that is twice that at the head (Tomlinson, 1987).

Driving stresses can be estimated using wave equation analyses, which were first developed by Smith (1960). Finite difference algorithms that model the pile and the soil by masses, springs, dashpots, and plastic resisting elements are used to calculate the penetration of the pile induced by the hammer blow, and the stresses in the pile. However, two major uncertainties are involved in wave equation analyses (Lawton et al., undated): (1) the uncertainty in the actual energy that is imparted by the hammer, and (2) the uncertainty in the distribution of the soil resistance along the pile.

The pile driving analyzer (PDA) was developed by Goble to overcome these shortcomings. Using measured force and accelerations at the pile head, the energy of the hammer imparted to the pile can be accurately determined. The soil resistance distribution can be calculated more accurately through iterative procedures using a wave equation solver called CAPWAP (Holloway, 1978). The PDA requires skilled personnel.

Wave equation analyses combined with PDA measurements provide an effective means of assessing stresses induced in piles during driving. They can be used effectively in the field as a means of checking the ultimate capacities of piles estimated using static methods, later described in this manual.

3.4 PILE SPACING

Piles are usually driven at spacings of 2.5 to 4 pile diameters. Close spacings minimize the cost of the pile cap. However, driving piles at close spacings in dense sands and saturated plastic soils can cause heave or lateral ground displacements that may damage or cause misalignment of previously driven piles. Close spacings may be advantageous with loose sands because they become compacted after driving (Teng, 1962).

3.5 OTHER DESIGN CONSIDERATIONS

3.5.1 Scour

Scour around bridge foundations can create a severe safety hazard. Therefore, bridge foundations should be designed to survive the effects of possible scour. Geotechnical analyses of bridge foundations should be performed assuming that the soil above the estimated scour line has been removed and is not available to provide bearing or lateral support (FHWA, 1988).

Three possible effects of scour should be considered in design (FHWA, 1988):

1. *Aggradation and Degradation*—aggradation is the deposition of stream bed material eroded from other portions of a stream; whereas, degradation is the removal of stream bed material thereby lowering the bed elevation. Aggradation and degradation are long-term effects caused by natural or man-made conditions.
2. *General Scour and Contraction Scour*—general scour and contraction scour are characterized by the removal of stream bed material across the entire width of the stream because of increasing flow velocities. Flow velocities increase as a result of contraction of the flow channel or change in the downstream water surface elevation. One instance when contraction scour may occur is when the approach embankment of a bridge encroaches into the stream.
3. *Local Scour*—local scour occurs when bed material is removed from a small portion of the width of the stream. Obstructions to flow, such as bridge piers and abutments, induce acceleration of the flow, causing vortices that wash away the bed material.

Scour is usually evaluated for a flood with a return period of about 100 years. The FHWA recommends that the top of the pile cap should be located below the depth of contraction scour to reduce obstruction to flow and to minimize local scour. Also a few long piles should be used rather than many short piles. This results in higher safety against pile failure due to scour.

3.5.2 Deterioration

Most piles are made of concrete, steel, or timber. Concrete piles may be attacked by deleterious substances in the ground such as organic materials, acids, sulfates, salt, and so on. Abrasion of concrete piles can occur if the piles are exposed to soils being moved by currents and waves, floating debris and ice. High quality concrete and ample cover for the reinforcement provide protection against abrasion and corrosion. In an environment rich in sulfates, sulfate resisting cement can be used in the concrete mix.

Steel piles that are exposed along portions of their lengths are subjected to corrosion when placed in hostile chemical environments, while embedded steel piles corrode at an insignificant rate in the absence of oxygen. The following precautions can be used to reduce the rate of corrosion in piles that are exposed along portions of their lengths: (1) Provide additional sacrificial steel thickness. (2) Remove or treat the corrosive soil. (3) Provide cathodic protection to the piles, i.e., introduce an electric current towards the piles so that there is no electron loss (corrosion) from the steel piles. (4) Provide a protective coating.

Untreated timber piles projecting above the groundwater table are subjected to decay caused by alternate cycles of wetting and drying, and attacks by chemicals, fungi, and insects. The rate of deterioration can be retarded by using piles treated with creosote and other chemical solutions.

3.6 DESIGN PROCEDURE FOR PILE FOUNDATIONS

The design of pile foundations involves the following steps:

1. Develop a soil profile based on soil borings for the site. Include details of strength profiles, compressibility characteristics, stress history and geology of the soils, and identify the favorable and unfavorable zones in the subsoil.
2. Estimate the loads for the ultimate and the serviceability limit states. Loads due to negative skin friction should be included.
3. For stream crossings, determine the water profiles for the site and the expected depth of scour during flood.
4. Select candidate pile types and pile lengths. Consider the factors described in Section 2 and eliminate all unsatisfactory alternatives.
5. Make a general economic comparison of the candidate piles and design with the most cost-effective one(s) according to the steps below.
6. Estimate the axial pile capacity considering both soil and structural capacity.
7. Determine the required number of piles and their spacing and locations.
8. Estimate the capacity of the pile group. If the group capacity is not sufficient, increase the number of piles or the pile spacing.
9. Check for possible punching of the pile group into any weak stratum that may be present beneath the bearing stratum.
10. Determine the tolerable settlement of the pile group and estimate its settlement. If the settlement is greater than the tolerable settlement, increase the length of the piles or the pile spacing (see Section 4.2.2).
11. Check the uplift capacity of the pile group, if it will be subject to uplift loads.
12. Check the structural capacity of the piles under lateral loading.
13. Determine the tolerable lateral displacement of the pile group and calculate the lateral displacement. If the lateral displacement is greater than the tolerable lateral displacement, increase the number of piles or the pile spacing (see Section 5.2.1.2).
14. Determine whether pile load tests are required to verify the design.

A summary of the ultimate and serviceability limit states that should be considered during the design stage is given in Table 3.3.

Table 3.3. Summary of ultimate and serviceability limit states that must be considered in the design of pile foundations.

DESIGN CONSIDERATION	ULTIMATE LIMIT STATE	SERVICEABILITY LIMIT STATE
Structural capacity of single piles	X	
Bearing capacity of single piles	X	
Bearing capacity of pile groups	X	
Punching into lower weak stratum	X	
Settlement of pile groups		X
Tensile capacity of piles during uplift	X	
Uplift capacity of single piles	X	
Structural capacity of piles under lateral loading	X	
Lateral movement of pile groups when subjected to lateral loads		X

CHAPTER 4

DESIGN OF PILES FOR AXIAL LOADING

Significant advances have been made in recent years in developing improved understanding of the behavior of axially loaded piles. Three limit states may be reached in piles subjected to axial loads. These are: (1) structural failure of the pile, (2) bearing capacity failure of the soil, and (3) excessive settlement. Failure of the pile or the soil is called an "ultimate limit state" (ULS). Excessive settlement, a less drastic occurrence, is called a "serviceability limit state" (SLS).

Both ultimate and serviceability limit states are addressed in this section. The structural capacity of piles is discussed first, followed by the bearing capacity of single piles and pile groups. Settlement of pile groups is considered last.

4.1 SINGLE PILES

4.1.1 Structural Capacity

Axially loaded piles can fail structurally either in compression or by buckling. Buckling usually does not take place in piles of "normal dimensions driven through soft soils" (Poulos, 1980). However, buckling analyses are warranted in long and slender piles that extend for a portion of their lengths through water or air. Scour around piles increases the likelihood that they may fail by buckling, and the maximum possible depth of scour must be considered in design.

4.1.1.1 Axial Compression

The axial load in a pile should not exceed the factored axial structural capacity. The following criterion expresses this fact:

$$r\phi_a P_n \geq \gamma_D P_D + \gamma_L P_L \qquad (4.1.1.1)$$

where r = eccentricity factor (Table 4.1), ϕ_a = performance factor for the nominal structural capacity (Table 4.1), P_n = nominal structural capacity of the pile, P_D and P_L are the axial loads due to dead and live loads respectively and γ_D and γ_L are the dead and live load factors. In general, for conditions where other types of loads may act on piles, the design criterion may be expressed as:

$$\phi_a P_n \geq \Sigma \gamma_i P_i \qquad (4.1.1.2)$$

where γ_i = load factor for the load i and P_i = axial load due to load i.

Expressions for the nominal axial pile capacity can be found in Table A2.1 (in Appendix 2) for steel, timber, prestressed and precast concrete piles. Values of the performance factor, ϕ_a, are given in Table 4.1.

The pile carrying the maximum load in an eccentrically loaded pile group must be checked for structural failure. In Figure 4.1, the factored total vertical load acting on a group of piles, denoted as P_g, acts at a distance e_x and e_y from the centroid in the x and y directions. The factored axial load on any pile, $P_{x,y}$ may be calculated from the following expression (Scott, 1980):

$$P_{x,y} = P_g\left[\frac{1}{N_{pile}} + \frac{e_x x}{\Sigma x^2} + \frac{e_y y}{\Sigma y^2}\right] \qquad (4.1.1.3)$$

where x and y are the distances of the pile from the centroid in the x and y directions respectively, and N_{pile} is the number of piles in the group.

Table 4.1. Performance factors for the nominal axial structural capacity of piles.

PILE TYPE	PERFORMANCE FACTOR, ϕ_a	ECCENTRICITY FACTOR, r
Prestressed Concrete Piles	0.75 for spiral columns 0.70 for tied columns	0.85 for spiral columns 0.80 for tied columns
Precast Concrete Piles	0.75 for spiral columns 0.70 for tied columns	0.85 for spiral columns 0.80 for tied columns
Steel-H Piles	0.85	0.78
Steel Pipe Piles	0.85	0.87
Timber Piles	1.20*	0.82

* Davisson et al. (1983) stated that the minimum factor of safety for the structural capacity of timber piles in compression is 1.25. The performance factor is greater than unity since the average load factor for vertical loads (dead and live loads) is greater than the factor of safety itself.

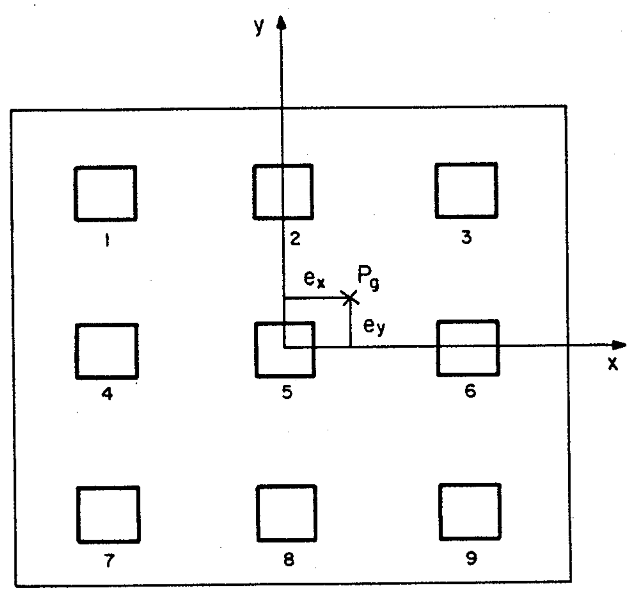


Figure 4.1. Eccentric loading on a pile group.

For example, consider the pile group shown in Figure 4.1. If $P_g = 500$ tons, $e_x = 2$ ft, $e_y = 1$ ft, and the pile spacing is 3 ft center-to-center, the maximum pile load can be determined as follows: $\Sigma x^2 = 6\ (3)^2 = 54$; $\Sigma y^2 = 6\ (3)^2 = 54$.

The most heavily loaded pile is pile 3, in the first quadrant: $P_3 = 500\ [1/9 + (2)(3)/54 + (1)(3)/54] = 139$ tons.

Pile number 7 carries a tensile force: $P_7 = 500\ [1/9 - (2)(3)/54 - (1)(3)/54] = -27.8$ tons.

Design against tensile failure is considered in Section 4.4.

4.1.1.2 Buckling of Partially Embedded Piles

Piles that extend above the ground through air or water may buckle when subjected to axial loads, and the possibility of buckling failure may control their structural capacity. In order to evaluate the buckling capacity of partially embedded piles, it is necessary to determine at what depth below the ground surface should the pile be assumed to be fixed. Davisson and Robinson (1965) have developed a method for estimating this depth to fixity.

Davisson and Robinson's Procedure. Davisson and Robinson (1965) presented solutions for the buckling loads of partially embedded piles in terms of an equivalent free standing length. The equivalent free standing length is the sum of the unsupported pile length above ground, and an additional length to the depth to fixity below ground. This depth to fixity is a function of the flexural stiffness of the pile (E_pI_p) and the soil stiffness. The soil stiffness can be expressed in terms of a soil modulus (E_s, force/length^2). The soil modulus is usually considered to remain constant with depth in clays, and to vary linearly with depth in granular soils.

For long piles, the equivalent free standing length, L_{eq}, can be written as follows:

Modulus constant with depth (clays)

$$L_{eq} = L_u + 1.4R \qquad (4.1.2.1)$$

Modulus increasing linearly with depth (sands)

$$L_{eq} = L_u + 1.8T \qquad (4.1.2.2)$$

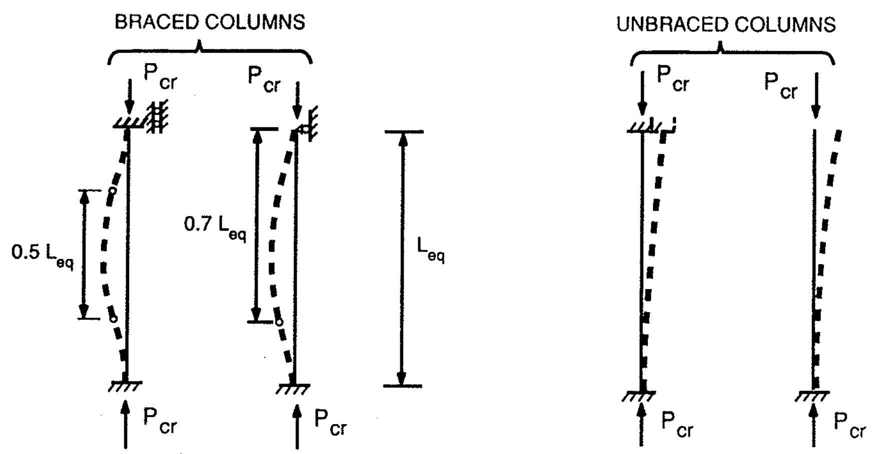
where L_u = unsupported length of pile extending above ground; $R = [E_pI_p/E_s]^{0.25}$ in units of length; E_p = Young's modulus of pile, force/length^2; I_p = moment of inertia of pile, length^4; E_s = soil modulus, force/length^2; $E_s = 67S_u$ for clays (Davisson and Robinson, 1965); S_u = undrained shear strength of clays, force/length^2; $T = [E_pI_p/n_h]^{0.2}$; n_h = rate of increase of soil modulus with depth, force/length^3; $n_h = E_s/z$; and z = depth.

Davisson and Robinson's (1965) procedure applies to different boundary conditions at the top of the pile; the bottom boundary condition is assumed to be fixed against rotation and translation at the depth of fixity. Selection of appropriate boundary conditions at the top of the pile depends on the type of structure, the fixity of bearings, and the number of rows of piles along the length and width of the pile group. Figure 4.2 shows four possible boundary conditions at the top of the pile where it connects with the structure, and expressions for the critical buckling load in ideal columns for each case.

Based on lateral load tests of piles in sand, Alizadeh and Davisson (1970) found that n_h is strongly dependent on deflection when the lateral deflection is less than 3 percent of the pile width. At larger deflections, the value of n_h becomes almost independent of the lateral deflection.

Terzaghi (1955) recommended values of n_h that are appropriate for lateral deflections that are about 5 percent of the pile width (Table 4.2). Reese et al. (1974) recommended using values of n_h that are between 3 and 4 times larger than Terzaghi's recommended values for constructing the initial slope of p-y curves.

For analysis of pile buckling, values of n_h corresponding to smaller deflections, on the order of 0.5 percent of the pile width or less, appear to be most appropriate. Evans (1982) showed that for lateral deflections of this magnitude, it is reasonable to use values of n_h about 3 times as high as the values recommended by Terzaghi. The two right hand columns of Table 4.2 contain



$$P_{cr} = \frac{4\pi^2 E_p I_p}{L_{eq}^2} \qquad P_{cr} = \frac{2\pi^2 E_p I_p}{L_{eq}^2} \qquad P_{cr} = \frac{\pi^2 E_p I_p}{L_{eq}^2} \qquad P_{cr} = \frac{\pi^2 E_p I_p}{4L_{eq}^2}$$

Figure 4.2. Critical buckling loads for centrally loaded columns with various end conditions.

Table 4.2. Coefficient of horizontal subgrade reaction (n_h) in lb/in.^{3}

	Terzaghi (1955)		Reese et al. (1974)	Recommended	
	Dry or Moist Sand	Submerged Sand	Submerged Sand	Dry or Moist Sand	Submerged Sand
Loose	8	5	20	30	15
Medium	24	16	60	80	40
Dense	65	39	125	200	100

values of n_h appropriate for lateral deflections on the order of 0.5 percent of the pile width.

Group Effects on Buckling Loads. The effect of pile spacing on the soil modulus has been studied by Prakash (Prakash and Sharma, 1990). He found that, at pile spacings greater than 8 times the pile width, neighboring piles have no effect on the soil modulus or buckling capacity. However, at a pile spacing of 3 times the pile width, the effective soil modulus is reduced to 25 percent of the value applicable to a single pile. For intermediate spacings, the modulus values can be estimated by interpolation.

Design Procedure. Buckling loads for partially embedded free standing piles can be calculated using the following steps:

1. Estimate the value of n_h (for sands) or E_s (for clays) and calculate the value of T (for sands) or R (for clays). For pile groups, the soil modulus should be reduced to account for the effects of neighboring piles as described earlier.
2. Calculate the equivalent length of the pile, L_{eq}, using Eqs. 4.1.2.1 or 4.1.2.2, whichever is appropriate.
3. Use the appropriate expression from Figure 4.2 to calculate the buckling load, P_{cr}. Four equations are given in Figure 4.2 for four different restraint conditions at the top of the pile.

After the ideal buckling load has been determined, the safe design load for the column, considering the effects of end moments and eccentricity of loading, can be determined using normal design procedures for columns and beam columns.

4.1.2 Presumptive Bearing Capacities of Soils and Rocks

In the absence of sufficient soil strength data to estimate pile capacities rationally, bearing capacities of piles may be estimated using presumptive bearing capacities. These values should be used only as a rough guide to possible capacities. When used in design, presumptive bearing capacities must be substantiated by pile load tests or rational methods of analysis based on soil data from the site.

Presumptive bearing capacities that have been published previously are "allowable" values intended for use in working stress design.

4.1.3 Rational Methods of Estimating Pile Bearing Capacities

The ultimate bearing capacity of a pile is the sum of the skin and point resistances, minus the weight of the pile:

$$Q_{ult} = Q_s + O_p - W \tag{4.1.3.1}$$

where Q_{ult} = total ultimate bearing capacity of a pile; Q_s = ultimate load carried by pile shaft = $A_s q_s$; Q_p = ultimate load carried by pile point = $A_p q_p$; A_s = surface area of pile shaft; A_p = area of pile point; q_s = ultimate unit skin resistance of pile; q_p = ultimate unit point resistance of pile; and W = weight of the pile.

In most cases (with the exception of large concrete piles in bent piers), the weight of the pile is small compared to the other terms, and is usually disregarded.

The load factor design criterion may be expressed as:

$$\phi_q Q_{ult} \geq \gamma_D P_D + \gamma_L P_L \tag{4.1.3.2}$$

where ϕ_q = the performance factor for the ultimate bearing capacity of a pile, or in general,

$$\phi_q Q_{ult} \geq \Sigma \gamma_i P_i \tag{4.1.3.3}$$

where γ_i is the load factor for load i and P_i is the axial load due to load i.

One rational method of estimating the bearing capacity of piles in compression is called the "static" approach. Static formulas are based either on classical soil mechanics theories or empirical correlations. These include the α, β, and λ methods, and methods based on in situ tests such as the cone penetration test (CPT) or the standard penetration test (SPT). The α, β, and λ methods are more suited for piles in cohesive soils, while the SPT and CPT correlations are better suited for piles in cohesionless soils.

4.1.3.1 Rational Methods to Estimate Skin Friction

When piles are driven into saturated clays, the soil around the pile is severely disturbed. Installation induces high pore pressures in the soil, which dissipate with time. In some cases, after complete consolidation, the shear strength of the clay at the pile interface may be greater than that of the soil prior to driving. For sensitive clays or stiff overconsolidated clays, the final shear strength is considerably less than that of the undisturbed soil (Meyerhof, 1976).

1. The α method relates the adhesion between the pile and the clay to the undrained shear strength of the clay. The ultimate unit skin friction, q_s, can be expressed by:

$$q_s = \alpha S_u \tag{4.1.3.4}$$

where S_u = mean undrained shear strength; α = adhesion factor applied to S_u.

Tomlinson (1987) found that the adhesion factor, α, varies with the value of the undrained shear strength, S_u as shown in Figure 4.3. Although not shown in the figure, there is considerable scatter around the curves because factors such as pile length, overconsolidation ratio, and coefficient of lateral earth pressure were neglected; all of these factors affect the pile capacity. Uncertainty in the undrained shear strength also contributes to the scatter. However, the α-method is used frequently in practice because it is simple, and also because no method is available that fully reflects pile installation and all of the factors involved in the reconsolidation processes.

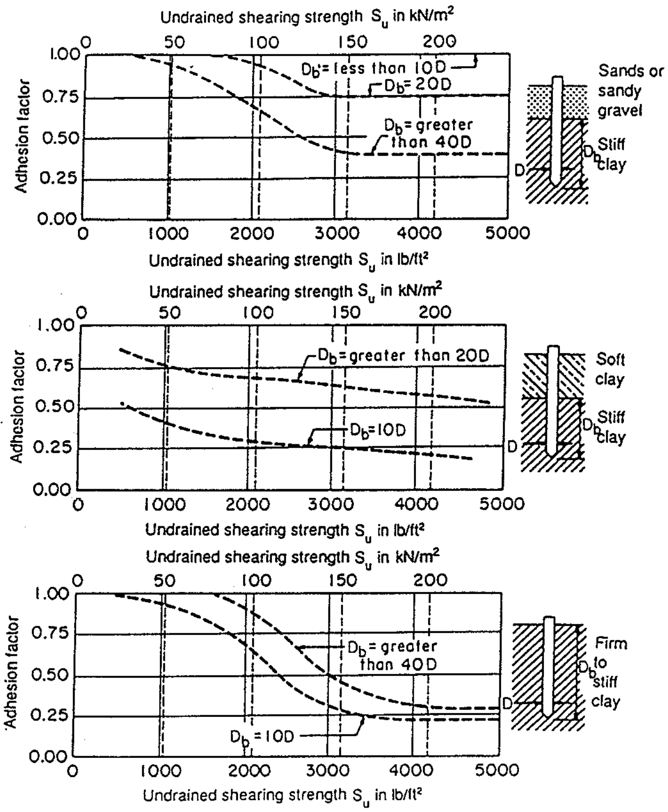


Figure 4.3. Design curves for adhesion factors for piles driven into clay soils. (After Tomlinson, 1987)

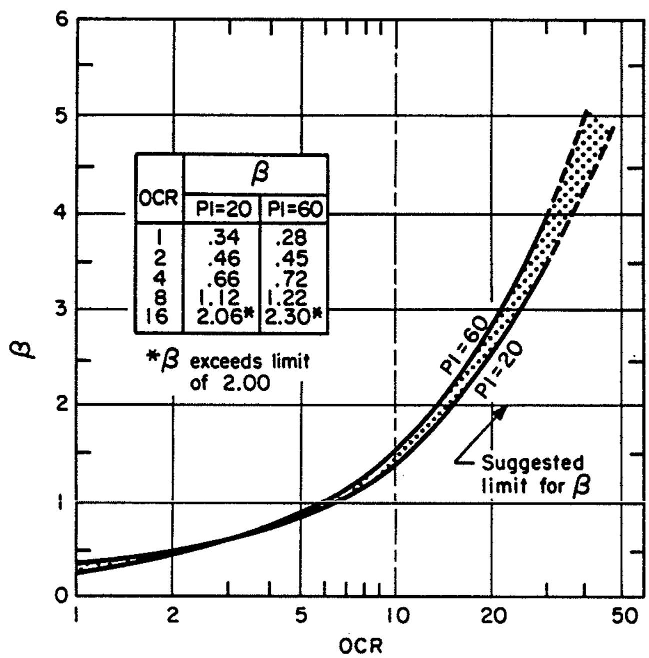


Figure 4.4. β versus OCR for full displacement piles. (After Esrig and Kirby, 1979)

The adhesion factor also depends on the type of soil above the cohesive bearing stratum (Figure 4.3). Soil from the upper layers may be carried down with the pile into the clay bearing stratum. Bringing down soft clay will tend to reduce adhesion, while dragdown of cohesionless soil will increase adhesion in the lower cohesive stratum.

2. The β-method is an effective stress method for predicting skin friction of piles. The ultimate unit skin friction, q_s, is related to the effective stresses in the ground as follows:

$$q_s = \sigma_h' \tan\delta = K \tan\delta\ \sigma_v' = \beta\ \sigma_v' \qquad (4.1.3.5)$$

where σ_h' and σ_v' are the horizontal and vertical effective stresses respectively, δ is the angle of shearing resistance between the soil and the pile, K is the coefficient of lateral earth pressure, and β equals K$\tan\delta$.

The value of the parameter K is very important. Kulhawy et al. (1983) noted that "the coefficient, K, is a function of the original in situ horizontal stresses and the stress changes caused in response to construction, loading and time." When a pile is first driven into the ground, the displaced soil exerts horizontal stresses on the pile. Excess pore pressures are generated and, thus, σ_v' is low, giving a high initial K value. As pore pressure dissipates, K changes with time. Depending on the overconsolidation ratio (OCR), the value of K may be higher or lower than the at-rest coefficient of lateral earth pressure, K_0. Esrig and Kirby (1979) developed the relationship between β and OCR that is shown in Figure 4.4.

The β-method has been found to work best for piles in normally consolidated and lightly overconsolidated clays. The method tends to overpredict skin friction of piles in heavily overconsolidated soils. Esrig and Kirby suggested that for heavily overconsolidated clays, the value of β should not exceed 2.

3. Vijayvergiya and Focht (1972) recognized that the passive lateral earth pressure ($\sigma_h' = \sigma_v' + 2S_u$) and the ultimate unit skin friction of a pile are related. They proposed the following relationship:

$$q_s = \lambda(\sigma_v' + 2S_u) \qquad (4.1.3.6)$$

where λ is an empirical coefficient shown in Figure 4.5. The value of λ decreases with pile length and was found empirically by examining the results of load tests on steel pipe piles.

4.1.3.2 Rational Methods to Estimate Tip (or Toe) Resistance

The following expression for the ultimate bearing capacity of a strip footing on the ground surface has been derived using the concepts of plasticity theory (Kulhawy et al., 1983):

$$q_p = cN_c + 0.5\gamma' DN_\gamma + \sigma_v' N_q \qquad (4.1.3.7)$$

where c = cohesion of soil below the base of the footing; σ_v' = vertical effective stress at the base of the footing; γ' = effective unit weight of soil below the base of the footing; D = width of the footing; and N_c, N_q, N_γ = bearing capacity factors which are related to the friction angle of the soil.

The tip resistance of a pile point can also be treated as a bearing capacity problem. Equation 4.1.3.7 can be modified to

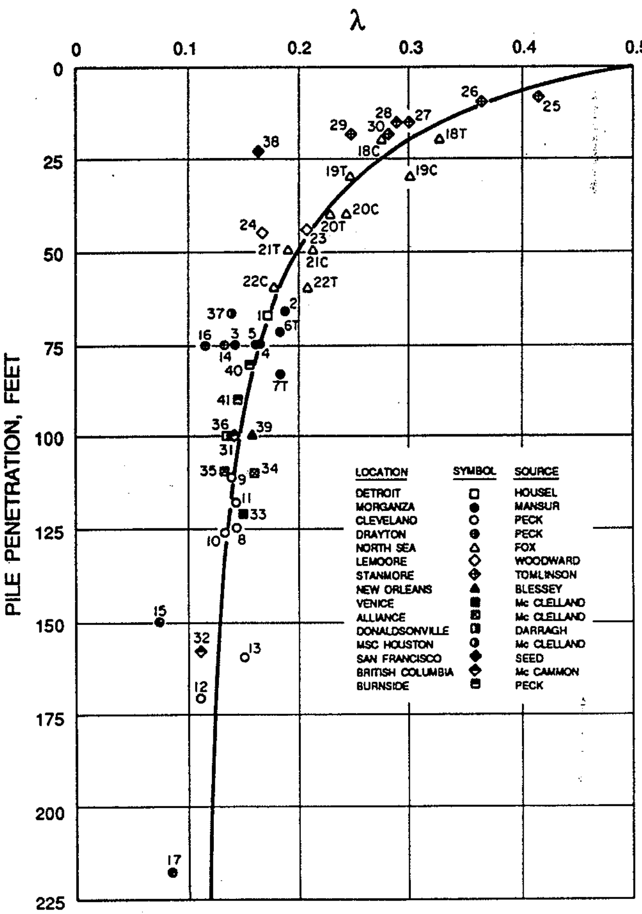


Figure 4.5. λ coefficient for driven pipe piles. (After Vijayvergiya and Focht, 1972)

include shape and depth effects of the pile and rigidity of the soil (Kulhawy et al., 1983):

$$q_p = cN_c s_c d_c r_c + 0.5\gamma' D N_\gamma s_\gamma d_\gamma r_\gamma + \sigma_v' N_q s_q d_q r_q \quad (4.1.3.8)$$

where s_c, s_γ, and s_q are shape factors; d_c, d_γ, and d_q are depth factors; and r_c, r_γ and r_q are factors that take into account the rigidity of the soil.

1. For piles in saturated clay with a zero friction angle and $c = S_u$, $N_\gamma = 0$, $N_q s_q d_q r_q = 1$, and $N_c s_c d_c r_c = 9$ for piles with depth to width ratios greater than 4 (Skempton, 1951). Equation 4.1.3.8, for the unit tip capacity thus reduces to:

$$q_p = 9S_u + \sigma_v \quad (4.1.3.9)$$

or alternatively, the ultimate tip capacity can be expressed as:

$$Q_p = 9S_u A_p + \sigma_v A_p \quad (4.1.3.10)$$

The quantity $\sigma_v A_p$ is comparable in magnitude to the weight of the pile, W. Therefore, the weight of the pile, W, may be neglected in Eq. 4.1.3.1 if the ultimate tip capacity is written as:

$$Q_p = 9S_u A_p \quad (4.1.3.11)$$

where S_u is the undrained shear strength of the clay near the pile base.

2. In coarse-grained, cohesionless soils such as sands, c = 0. The friction angle of sands can be correlated to the standard penetration test blow-count and the cone penetration resistance, as described in Appendix 3. For piles with large depth to width ratios, the second term of Eq. 4.1.3.8 is small compared to the third term. For instance, where the depth to width ratio is between 4 and 5, the second term is less than 10 percent of the third term (Kulhawy et al., 1983). Thus, the drained ultimate tip resistance may be approximated as follows:

$$q_p = \sigma_v' N_q s_q d_q r_q \quad (4.1.3.12)$$

where $N_q s_q d_q r_q$ = bearing capacity factor obtained from Figure 4.6 (Kulhawy et al., 1983). The rigidity index, a term which accounts for soil deformability and the variation of the bearing capacity factor with depth, is defined by Vesic (1975) as follows:

$$I_r = \frac{E_s}{2(1+\mu)\sigma_v' \tan\phi'} \quad (4.1.3.13)$$

where E_s = Young's modulus of the soil, μ = Poisson's ratio of the soil, σ_v' = vertical effective stress measured at a depth of

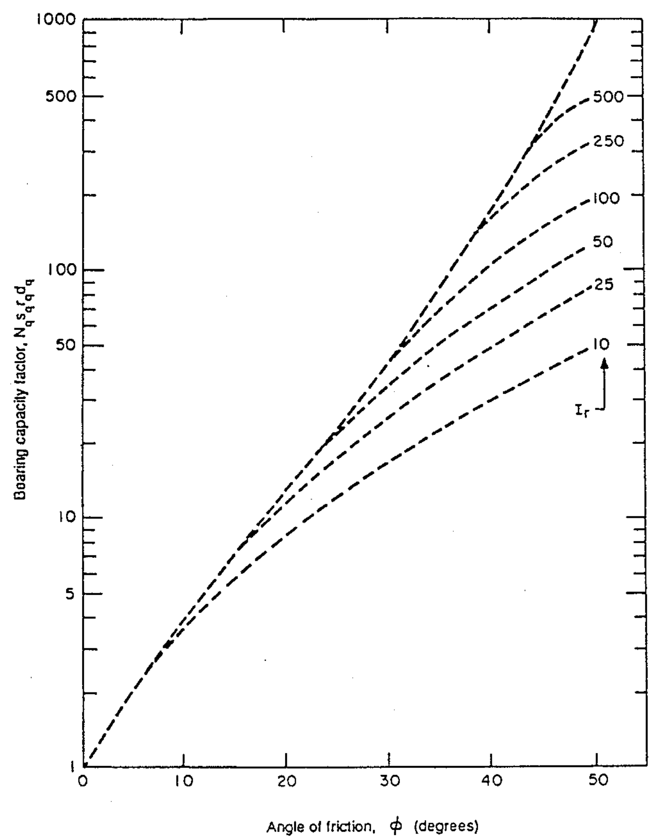


Figure 4.6. Modified N_q bearing capacity factor for deep foundations. (After Kulhawy et al., 1983)

$D/2$ below the base of the pile, and ϕ' = effective friction angle of the soil. Kulhawy et al. (1983) approximated the rigidity index for sands as follows:

For loose sands $$I_r = \frac{30}{(\sigma_v')^{0.5}\tan\phi'} \quad (4.1.3.14)$$

For dense sands $$I_r = \frac{110}{(\sigma_v')^{0.5}\tan\phi'} \quad (4.1.3.15)$$

where σ_v' is in tsf.

4.1.3.3 Tip Resistance of Piles Driven to Rock

The ultimate unit end-bearing capacity, q_p, of piles driven to rock may be estimated from the uniaxial compression strength as follows (Canadian Foundation Engineering Manual, 1985):

$$q_p = 3\sigma_c K_{sp} d \quad (4.1.3.16)$$

where σ_c = average uniaxial compression strength of the rock core; K_{sp} = dimensionless bearing capacity coefficient (Figure 4.7) and

$$K_{sp} = \frac{3 + s_d/D}{10[1 + 300t_d/s_d]^{0.5}} \quad (4.1.3.17)$$

d = dimensionless depth factor $= 1 + 0.4H_s/D_s \le 3.4$; s_d = spacing of discontinuities; t_d = width of discontinuities; D = pile width; H_s = depth of embedment of pile socketed into rock = 0 for piles resting on top of bedrock; and D_s = diameter of socket.

This method is not applicable to soft stratified rocks, such as shale or limestone. When this method is applicable, the rocks are usually so sound that the structural capacity will govern the design (Fellenius et al., 1989). This method is applicable only if $s_d > 1$ ft, $t_d < 0.25$ in. for unfilled discontinuities or $t_d < 1$ in. for discontinuities filled with soil or rock debris, and $D > 1$ ft.

4.1.3.4 In Situ Test Methods

In situ tests are widely used in cohesionless soils because obtaining good quality samples of cohesionless soils is very difficult. In situ test parameters may be used to estimate the tip resistance and skin friction of piles. There are two frequently used in situ test methods for predicting pile capacity. These are the standard penetration test (SPT) method and the cone penetration test (CPT) method:

1. *SPT method*—Meyerhof (1976) correlated the tip capacity and shaft resistance of piles with the SPT blow count. This method applies only to sands and nonplastic silts.

The ultimate unit tip resistance for piles, q_p (in tons per square foot) driven to a depth D_b into a cohesionless soil stratum can be approximated by:

$$q_p = \frac{0.4N_{corr}D_b}{D} \le q_l \quad (4.1.3.18)$$

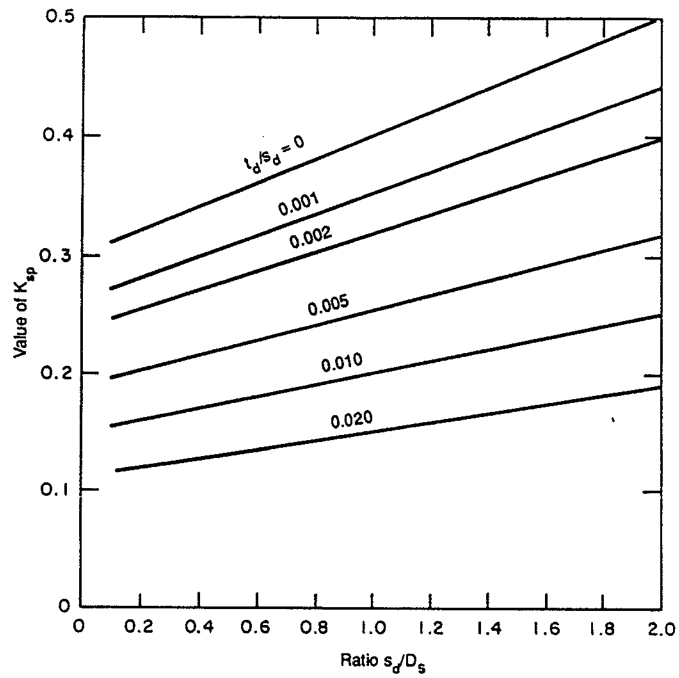


Figure 4.7. Bearing capacity coefficient, K_{sp}. (After Canadian Foundation Engineering Manual, 1985)

for which N_{corr} = average corrected SPT-N value near the pile tip

$$N_{corr} = [0.77 \log_{10} (20/\sigma_v')] N \quad (4.1.3.19)$$

N = measured SPT-N value and σ_v' = effective vertical stress at the pile tip, in tons/ft^2; D = pile width or diameter and q_l = limiting point resistance, in tons per square foot;

$$q_l = 4N_{corr} \text{ for sands} \quad (4.1.3.20)$$

$$q_l = 3N_{corr} \text{ for nonplastic silt} \quad (4.1.3.21)$$

The rationale behind Eq. 4.1.3.18 is that the ultimate unit tip capacity in a cohesionless stratum increases linearly with the embedment ratio (D_b/D) up to a critical embedment ratio of 10 for sands, or 7.5 for silts. At higher embedment ratios, the tip capacity remains constant at its limiting value, q_l.

In bearing strata with highly varying blow counts, Meyerhof (1976) proposed that the average blow count be obtained within the range of depth from 4 pile diameters above to 1 pile diameter below the tip.

Piles bearing on a firm stratum overlying a weaker layer may punch into the lower stratum as shown in Figure 4.8. Meyerhof (1976) suggested that if the distance between the pile tip and the weak deposit, H, is less than 10 pile diameters, the ultimate point resistance will be:

$$q_p = q_o + \frac{(q_l - q_o)H}{10D} \le q_l \quad (4.1.3.22)$$

where q_l is the limiting unit tip resistance in the upper stratum and q_o is the limiting unit tip resistance in the lower stratum.

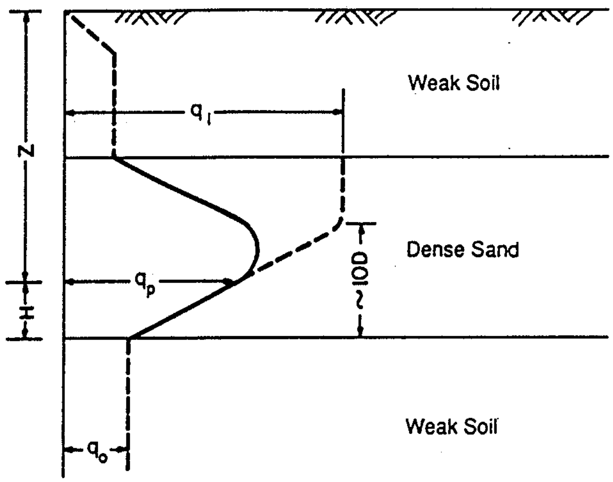


Figure 4.8. Relation between ultimate point resistance of pile and depth in thin sand layer overlying weak soil.

The skin friction of piles in cohesionless soils may be estimated using the following equation (Meyerhof, 1976):

$$q_s = \frac{\overline{N}}{50}, \text{ for driven displacement piles} \quad (4.1.3.23)$$

$$q_s = \frac{\overline{N}}{100}, \text{ for nondisplacement piles (e.g., steel-H piles)} \quad (4.1.3.24)$$

where q_s = unit skin friction for driven piles measured, in tsf; $\overline{N}$ = average (uncorrected) SPT blow count along the pile shaft.

An alternate method of predicting pile capacities using SPT blow counts was proposed by Briaud and Tucker (1984). Their method is more rational in that it considers residual stresses in the pile after driving.

2. *CPT method*—The cone penetration test yields two useful parameters that can be applied to pile capacity prediction: (1) the cone penetration resistance, q_c, which is related to the tip capacity of piles, and (2) sleeve friction, f_s, which can be used to estimate the skin friction capacity. Nottingham and Schmertmann (1975) developed the following procedure for estimating pile capacity.

Nottingham and Schmertmann (1975) found that Begemann's procedure gives a good estimation of end bearing capacity in piles for all soil types. Begemann's procedure for estimating the tip resistance, q_p is outlined in Figure 4.9. The minimum average cone resistance between 0.7 and 4 pile diameters below the elevation of the pile tip is obtained by a trial and error process, with the use of the minimum-path rule (see Figure 4.9). The minimum-path rule is also used to find the value of cone resistance for a distance of eight pile diameters above the tip. The two results are then averaged to give the pile tip resistance.

Nottingham and Schmertmann (1975) presented the following equation for computing the ultimate skin friction of piles:

$$Q_s = K_{s,c}\left[\sum_{L_f=0}^{8D}(L_f/8D)f_s a_s + \sum_{L_f=8D}^{Z} f_s a_s\right] \quad (4.1.3.25)$$

where Q_s = ultimate skin friction capacity of the pile; $K_{s,c}$ = correction factors (K_c for clays and K_s for sands—see Figure 4.10); L_f = depth to point considered; D = pile width or diameter; f_s = unit local sleeve friction resistance from CPT at the point considered; a_s = pile perimeter; and Z = total embedded pile length.

The advantages of using this method is that it (1) corrects for the type of cone penetrometer used (electrical versus mechanical), (2) accounts for the material of the pile, (3) considers the soil type, and (4) corrects for depth of pile embedment.

4.1.3.5 Pile Load Tests

Pile load tests provide the best means of evaluating ultimate load capacities, and should be used whenever possible to verify capacities estimated by means of the methods described earlier. The procedure for the quick load test method is described in ASTM D1143. It is a short duration test and can usually be completed in 1 to 4 hours. The test pile is generally loaded to 200 percent of the design load unless it fails at a lower load. Other load test procedures on piles include the slow maintained load test, the constant rate of penetration test (ASTM D1143), and the tensile (or uplift) test method (ASTM D3689). The slow maintained load test is a long duration test that usually lasts 70 hours or longer. The test pile is loaded to 200 percent of the design load unless it fails at a lower load.

Davisson's graphical procedure provides a logical procedure for defining a conservative (lower bound) ultimate failure load. The method is shown in Figure 4.11 (NAVFAC, 1982).

4.1.3.6 Nondisplacement Piles

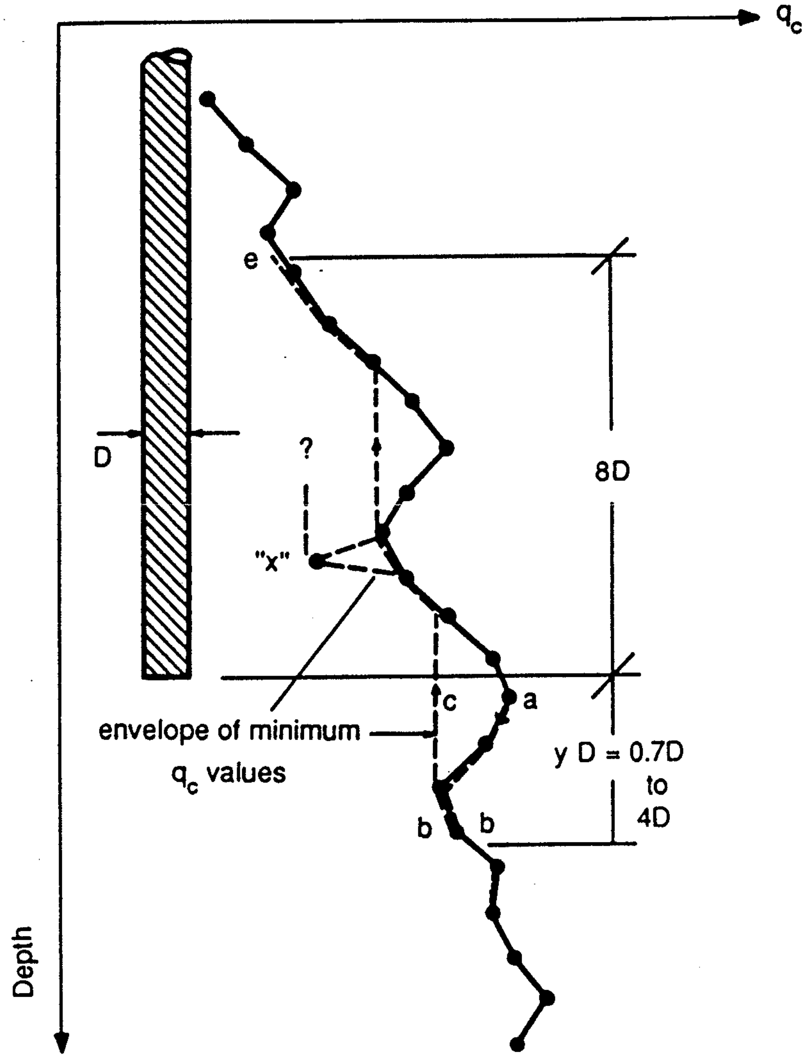
Steel-H piles can fail in two ways. First, they can become plugged when the soil between the flanges adheres fully to the pile. The effective area of the pile in this case is the area of the enveloping rectangle rather than the area of the steel-H section. In this case, the skin friction is the sum of the adhesion at the flanges (e.g., αS_u for saturated clays) and the full soil-to-soil shearing resistance (e.g., S_u for saturated clays) along both sides of the soil plug. The point resistance is calculated using the area of the enveloping rectangle.

Alternatively, steel-H piles can fail without plugging. In this case, the skin friction of the pile is estimated assuming adhesion on the entire perimeter of the steel-H section, and the point resistance is calculated using the area of the steel-H section.

Plugging usually occurs when piles are driven in soft to medium clays and loose to dense sands. Piles usually do not plug in medium to very stiff clays and very dense sands (Duncan, 1988). The case that yields the minimum capacity should be used in design.

Similarly, open-ended pipe piles may or may not plug. In a plugged pipe pile, the skin friction is calculated assuming adhesion on the outside surface only. The gross area of the pipe contributes to the end bearing capacity.

In an unplugged pipe pile, the skin friction is calculated assuming that the soil adheres to both the inside and outside surfaces of the pile. The point bearing capacity is calculated using the cross-sectional area of the steel annulus.



$$q_p = \frac{q_{c1} + q_{c2}}{2}$$

q_{c1} = Average of all values of q_c along path a-b-c over a distance of yD below the pile tip. Sum q_c values measured at each elevation in the downward path a-b. Sum q_c values at every elevation where a cone resistance reading is made, along the upward path b-c, but at each elevation take the minimum of (i) the q_c value at that elevation or (ii) the lowest q_c value between that elevation and the elevation of point b. This method of determining q_c is called the "minimum path" rule. Compute q_{c1} for y-values from 0.7 to 4.0 and use the minimum q_{c1} value obtained.

q_{c2} = Average q_c over a distance of 8D above the pile tip (path c-e). Use the minimum path rule as for path b-c in the q_{c1} computations. Ignore any very extreme peaks or depressions (such as "x" in the diagram above) if the soil is a sand, but include these in minimum path if the soil is a clay.

Figure 4.9. Pile end-bearing computation procedure after Begemann. (After Nottingham and Schmertmann, 1975)

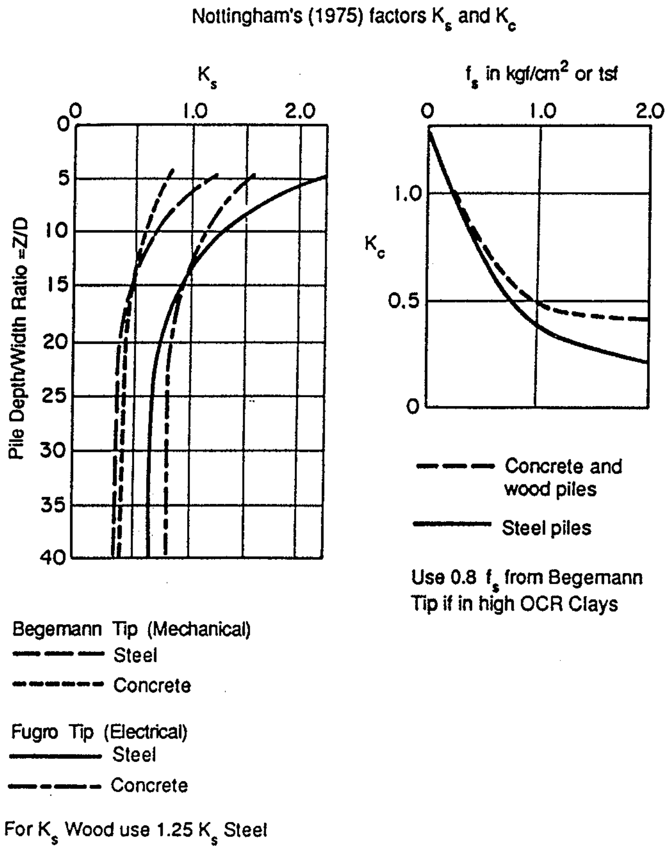


Figure 4.10. Shaft friction correction factors. (After Nottingham and Schmertmann, 1975)

4.2 PILE GROUPS

4.2.1 Bearing Capacity

The design requirements for pile groups are similar to those for single piles, i.e.,

$$\phi_g Q_g \geq \text{group load effect} \qquad (4.2.1.1)$$

where ϕ_g = performance factor for pile group capacity (see Table 4.3) and Q_g = pile group capacity.

The ultimate bearing capacity of a pile group in sand is estimated by summing capacities of all the piles in the group. The group efficiency, defined as the ratio of the ultimate load capacity of the pile group to the sum of the ultimate capacities of the individual piles, is conservatively taken as unity. Evaluation of group capacity of piles in cohesionless soil is the same for the case when the pile cap is, and is not, in contact with the ground.

For pile groups in cohesive soil, the presence and contact of the pile cap with the ground surface must be considered. Pile groups in clay with the cap in firm contact with the ground may fail as a unit consisting of the piles and the block of soil contained within the piles, and the ultimate bearing capacity in this case may be taken as the minimum of the following two values: (1) the sum of the individual pile capacities, or (2) the bearing capacity for block failure of the group.

For a pile group of width X, length Y, and depth Z (Figure 4.12), the bearing capacity for block failure is given by:

$$Q_g = (2X + 2Y)Z\bar{S}_u + XYN_cS_u \qquad (4.2.1.2)$$

where $\bar{S}_u$ = average undrained shear strength along the depth of penetration of the piles; S_u = undrained shear strength at the base of the group; and,

$$N_c = 5(1 + 0.2X/Y)(1 + 0.2Z/X) \quad \text{for } Z/X \leq 2.5 \qquad (4.2.1.3)$$

$$N_c = 7.5(1 + 0.2X/Y) \quad \text{for } Z/X > 2.5 \qquad (4.2.1.4)$$

If the pile cap is not in firm contact with the ground and the clay is normally or slightly overconsolidated or is sensitive, the individual pile capacity must be multiplied by an efficiency factor, η, where $\eta = 0.7$ for a pile spacing of 3D and $\eta = 1.0$ for a pile spacing of 6D. The value of η may be linearly interpolated for intermediate spacings. The group capacity is then calculated as the minimum of the sum of the individual pile capacities multiplied by η or the bearing capacity for block failure as described above.

If the clay is overconsolidated and insensitive, the group should be treated as if the cap were in contact with the ground.

The bearing capacity of a pile group containing batter piles may be estimated by treating the batter piles as vertical piles.

The performance factors for the group capacity calculated using the sum of the individual capacities are the same as those for the single pile capacity. A separate performance factor must be used for the block failure mechanism (see Table 4.3).

4.2.2 Settlement

Settlement of piles in sand and point bearing piles driven to rock are usually small, and they occur fairly rapidly. However, piles in clay may consolidate over a long period of time. The loads causing settlement of pile groups are assumed to act on an equivalent footing located at two-thirds of the depth of embedment of the piles in the layer which provides support (Duncan and Buchignani, 1976), as shown in Figure 4.13.

In estimating settlements of piles in clay, only unfactored permanent loads are considered. However, unfactored live loads must be added to the permanent loads when considering settlement of piles in granular soil.

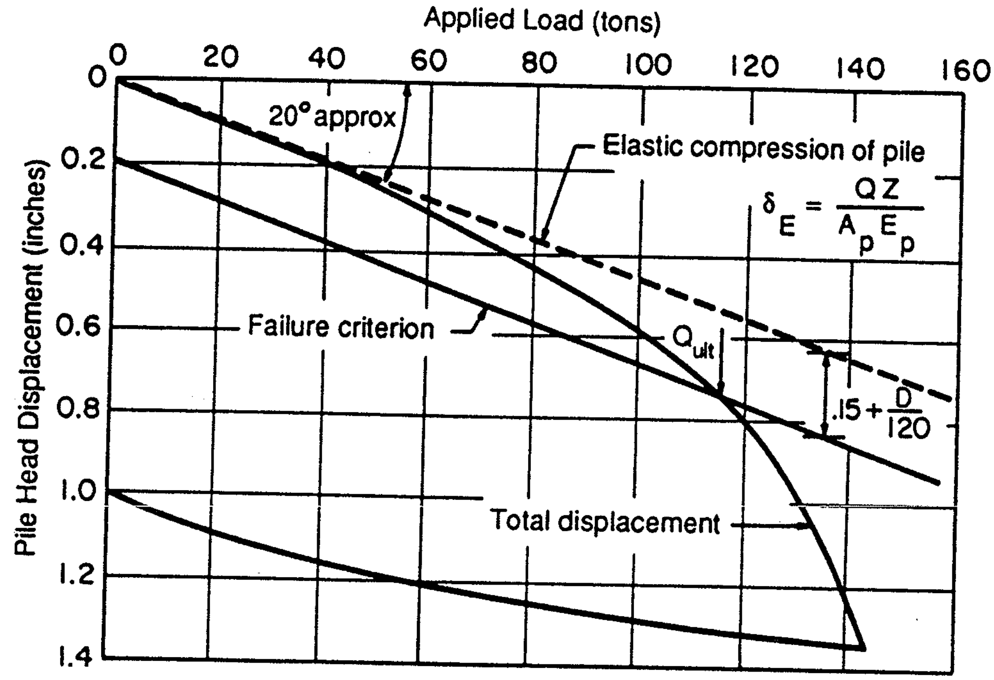
Cohesionless Soil. Meyerhof related the settlement of a pile group (ρ in inches) to the SPT blow count of the soil as follows:

$$\rho = \frac{2q\sqrt{X}\,I}{N_{corr}} \qquad (4.2.2.1)$$

where q = net foundation pressure (including any negative skin friction per unit area), in tons/ft² applied at $2D_b/3$ (see Figure 4.13); X = width of pile group, in ft; I = influence factor of the effective group embedment

$$I = 1 - D'/8X \geq 0.5 \qquad (4.2.2.2)$$

D' = effective depth = $2D_b/3$; N_{corr} = average corrected penetration resistance within the seat of settlement (approximately



TYPICAL TEST PLOT

1. Calculate elastic compression of pile (δ_E) when considered as a free column by:

$$\delta_E = \frac{QZ}{A_p E_p}$$

Q = test load, lbs
Z = pile length, in. (for end-bearing pile)
A_p = cross-sectional area of pile material, sq in
E_p = Young's Modulus for pile material, psi

2. Determine scales of plot such that slope of pile elastic compression line is approximately 20°.

3. Plot pile head total displacement vs. applied load.

4. Failure load is defined as that load which produces a displacement of the pile head equal to:

$$S_f = \delta_E + \left(.15 + \frac{D}{120} \right)$$

S_f = displacement at failure, in.
D = pile diameter, in.

5. Plot failure criterion as described in (4), represented as a straight line, parallel to line of pile elastic compression. Intersection of failure criterion with observed load deflection curve defines failure load, Q_{ult}.

6. Where observed load displacement curve does not intersect failure criterion, the maximum test load should be taken as the failure load.

Figure 4.11. Interpretation of pile load test. (After NAVFAC, 1982)

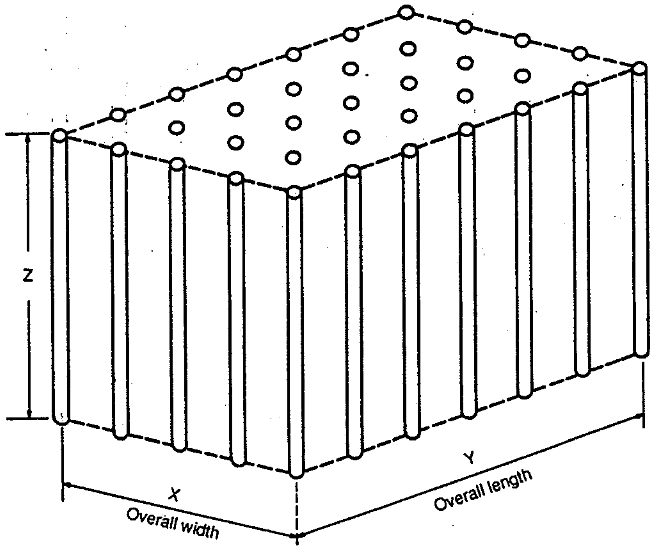


Figure 4.12. Pile group acting as block foundation.

one pile group width below the equivalent footing) as defined in Eq. 4.1.3.19.

The settlement of pile groups in silty sand is usually estimated to be twice the value found using Eq. 4.2.2.1.

Static cone penetration tests may also be used to estimate settlement of pile groups in cohesionless soils. Meyerhof (1976) related the settlement of a pile group to the average static cone resistance as follows:

$$\rho = \frac{qXI}{2q_c} \tag{4.2.2.3}$$

where q_c = average static cone resistance within the seat of settlement and q, X, and I have been defined previously. The units for q, q_c, and X should be consistent.

Cohesive Soil. The settlement of pile groups in clay occurs over a considerable period of time. The long-term settlement of pile groups in clay may be calculated using the methods employed in estimating settlement of shallow foundations. For this purpose, the load carried by a group of friction piles is assumed to be transferred to the soil through an equivalent footing located at two-thirds the pile depth.

The components contributing to the total settlement of a pile group in clay are: immediate settlement, consolidation settlement, and secondary settlement. They can be estimated using the same procedures as used for shallow foundations.

4.2.3 Load Factor Design for Settlement of Pile Groups

The load factor design approach to the settlement of pile groups requires an estimation of the tolerable settlement. The procedure is as follows:

1. Determine the tolerable settlement for the pile group, ρ_{tol}.
2. Estimate the settlement of the pile group, ρ. If the settlement is greater than the tolerable settlement, increase the number of piles, the lengths of the piles, or the pile spacing and repeat step 2.

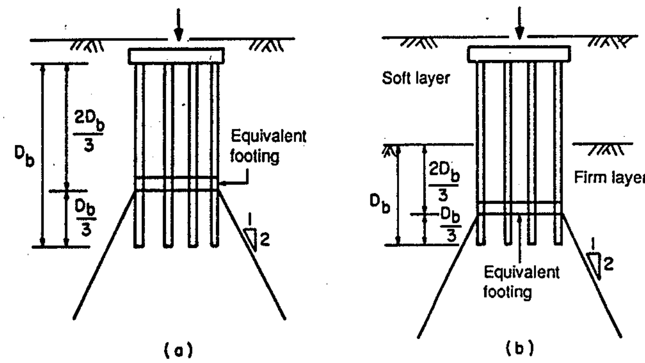


Figure 4.13. Location of equivalent footing. (After Duncan and Buchignani, 1976)

4.3 NEGATIVE SKIN FRICTION

Negative skin friction is the downdrag force induced in piles when the soil around the piles moves downward relative to the piles. Settlement of the soil around the pile may occur because of placement of fill, groundwater fluctuations, pile-driving in the vicinity, and other causes (Poulos and Davis, 1980).

Negative skin friction may be estimated using the rational methods discussed in Section 4.1 (the α, β, and λ methods). The unit negative skin friction, using the α method (Eq. 4.1.3.4) is given by:

$$q_{sn} = \alpha S_u \tag{4.3.1}$$

and the downdrag load is given by:

$$P_{sn} = q_{sn} a_s D_n \tag{4.3.2}$$

where P_{sn} = downdrag load; a_s = pile perimeter; D_n = length of pile embedded in settling soil.

4.3.1 Design Considerations

Downdrag loads can increase the settlement of pile groups but they rarely cause capacity problems. Settlement of pile groups should be checked when downdrag loads (unfactored) act together with dead loads. Temporary live loads and downdrag loads do not act together. This is because temporary live loads will compress the pile elastically and cancel or reduce the downdrag load. When the live load is removed, the pile will rebound elastically, thereby restoring the downdrag load.

If the magnitude of the downdrag load exceeds that of the live load, the structural and soil capacities should be checked for the dead load plus downdrag. The load factor for the downdrag load is the reciprocal of the performance factor for the ultimate skin resistance of the pile. The following criterion expresses this fact:

$$\phi R \geq \gamma_D P_D + \frac{1}{\phi_{qs}} P_{sn} \tag{4.3.1.1}$$

where ϕ = performance factor corresponding to the limit state considered; R = resistance corresponding to the limit state considered; and ϕ_{qs} = performance factor for the ultimate skin resistance of the pile.

4.3.2 Neutral Plane

The neutral plane is defined as the elevation at which the settlement of the pile and the settlement of the soil are the same, as shown in Figure 4.14. Above the neutral plane, the soil loads the shaft in negative skin friction. Below the neutral plane, the pile derives support from the soil. The distribution of the load and resistance in a pile is shown in Figure 4.14(a). A dead load, P_D, acts at the top of the pile. With increasing depth, the load on the pile increases because of negative skin friction. The total load acting on the pile ($P_D + P_{sn}$) increases accordingly. The pile resistance is equal to the tip capacity at the toe, Q_p, and increases upwards as the skin friction, Q_s, increases. This is represented by the curve ($Q_p + Q_s$). The two curves intersect at the neutral plane. This is the location of the maximum load on the pile. The neutral plane of piles end bearing on rock is located at the tip of the piles.

4.3.3 Settlement

Figure 4.14(b) illustrates the procedure for estimating the settlement of the pile cap. The settlement of the pile cap is the sum of the settlement at the neutral plane and the elastic compression of the piles above the neutral plane (Figure 4.14(b)). Unfactored loads are used to estimate the pile group settlement.

4.4 UPLIFT

Uplift of pile foundations may be caused by: swelling soils, frost heave, buoyancy, lateral loads, and upward loads. Piles subjected to uplift must be designed to withstand tensile stresses and pullout from the soil. Pullout resistance is usually adequate in long piles, but piles end-bearing on bedrock at shallow depths may have small pullout resistance.

4.4.1 Single Pile Uplift Capacity

Each pile in a group is either in tension or compression. The load acting on each pile in a group may be estimated using Eq. 4.1.1.3.

Soil Capacity. The ultimate uplift capacity of a single pile is estimated in a manner similar to that for estimating the ultimate shaft capacity for piles in compression (Section 4.1.3). The design requirement for uplift is as follows:

$$\phi_u Q_s \geq P_{x,y} \qquad (4.4.1.1)$$

where Q_s = ultimate uplift capacity due to shaft resistance; $P_{x,y}$ = factored tensile load effect in the pile (see Eq. 4.1.1.3); and ϕ_u = performance factor for uplift capacity (see Table 4.3).

The performance factors for axial compression and uplift are different because: (1) the diameter and, thus, the area of the pile shaft, decreases in tension due to the Poisson effect, thereby

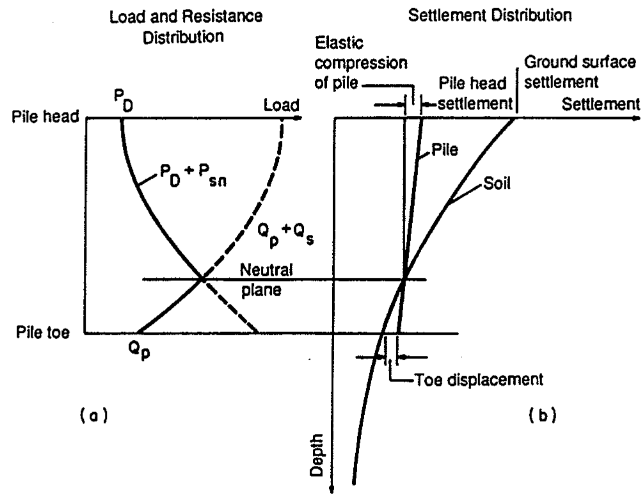


Figure 4.14. Calculation of the location of the neutral plane and the settlement of a pile or a pile group. (After Canadian Foundation Engineering Manual, 1985)

making uplift capacity smaller than compressive load capacity; and (2) piles in tension unload the soil—this reduces the overburden effective stress and, hence, the uplift skin friction resistance of the pile.

The uplift capacity of a pile may be verified by a load test according to ASTM D3689.

Structural Capacity. Fellenius et al. (1989) recommend that tensile loads should be carried entirely by the reinforcement for precast and prestressed concrete piles, and that the tensile strength of concrete should be neglected. The design requirement is as follows:

$$\phi_t f_u A_y \geq P_{x,y} \qquad (4.4.1.2)$$

where f_u = tensile strength of steel; f_u = yield stress of steel, f_y, in the case of the reinforcements in precast concrete piles; f_u = ultimate strength of the tendons in prestressed concrete piles; A_y = total area of steel; and ϕ_t = performance factor for tensile capacity of steel = 0.9 for steel-H and pipe piles, as well as reinforced and prestressed concrete piles.

Equation 4.4.1.2 applies to steel-H and pipe piles, as well as reinforced and prestressed concrete piles.

The parallel-to-grain tensile strengths of timber piles are higher than the compressive strengths. Therefore, the tensile structural capacity of timber piles is not critical if the magnitudes of the uplift loads do not exceed the magnitudes of the compressive loads, and in most cases, this is true.

4.4.2 Pile Group Uplift Capacities

The ultimate uplift capacity of a pile group is usually taken as the minimum of the following two values: the sum of the individual pile uplift capacities, or the uplift capacity of the group considered as a block. The mechanism for the latter is different for piles in clays and sands.

The shaft friction of pile groups in sands deteriorates with time if the piles are subjected to vibratory and lateral loads. Tomlinson (1987) suggested that the weight of the block uplifted

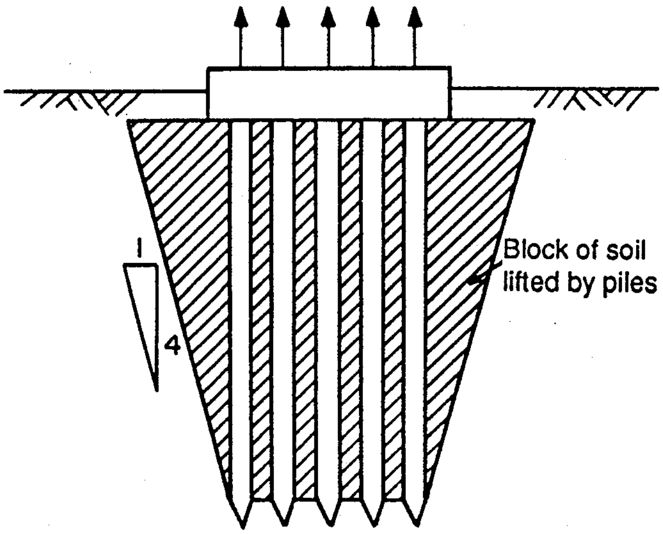


Figure 4.15a. Uplift of group of closely spaced piles in cohesionless soils. (After Tomlinson, 1987)

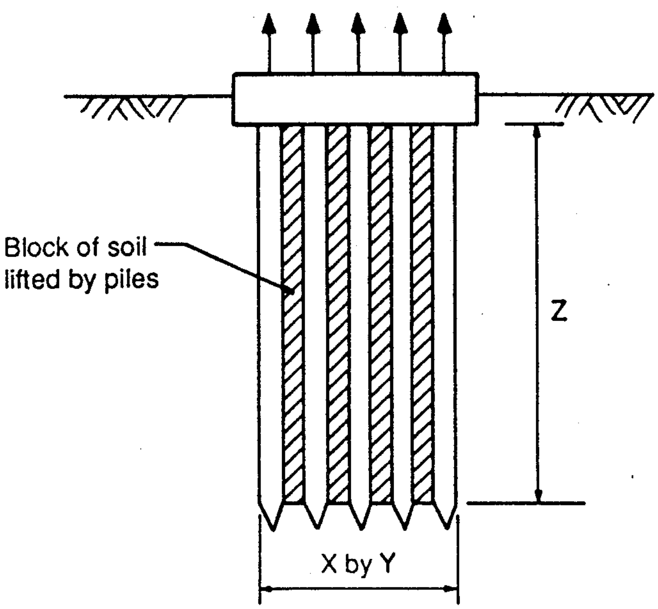


Figure 4.15b. Uplift of group of piles in cohesive soils. (After Tomlinson, 1987)

be estimated using a spread of load of 1 in 4 (Figure 4.15a) from the base of the pile group. Buoyant unit weights should be used for soil below the groundwater level.

In clays, the uplift resistance of the block in undrained shear is given by (Figure 4.15b):

$$Q_{ug} = (2XZ + 2YZ)\bar{S}_u + W_g \qquad (4.4.2.1)$$

where Q_{ug} = ultimate uplift resistance of the group; X = width of the group; Y = length of the group; Z = depth of the block of soil below pile cap; $\bar{S}_u$ = average undrained shear strength along pile shaft; and W_g = weight of the block of soil, piles and pile cap.

Table 4.3. Summary of performance factors for geotechnical ultimate limit states in axially loading of piles.

METHOD/SOIL/CONDITION			PERFORMANCE FACTOR
ULTIMATE BEARING CAPACITY OF SINGLE PILES	SKIN FRICTION	α-method	ϕ_{qs} = 0.70
		β-method	ϕ_{qs} = 0.50
		λ-method	ϕ_{qs} = 0.55
	END BEARING	Clay (Skempton, 1951)	ϕ_{qp} = 0.70
		Sand (Kulhawy, 1983) ϕ' from CPT	ϕ_{qp} = 0.45
		ϕ' from SPT	ϕ_{qp} = 0.35
		Rock (Canadian Geotech. Society, 1985)	ϕ_{qp} = 0.50
	SKIN FRICTION AND END BEARING	SPT-method	ϕ_q = 0.45
		CPT-method	ϕ_q = 0.55
		Load Test	ϕ_q = 0.80
		Pile Driving Analyzer	ϕ_q = 0.70
BLOCK FAILURE	Clay		ϕ_g = 0.65
UPLIFT CAPACITY OF SINGLE PILES	α-method		ϕ_u = 0.60
	β-method		ϕ_u = 0.40
	λ-method		ϕ_u = 0.45
	SPT-method		ϕ_u = 0.35
	CPT-method		ϕ_u = 0.45
	Load Test		ϕ_u = 0.80
GROUP UPLIFT CAPACITY	Sand		ϕ_{ug} = 0.55
	Clay		ϕ_{ug} = 0.55

4.5 SUMMARY OF PERFORMANCE FACTORS

Table 4.3 gives the geotechnical performance factors for bearing capacity and uplift capacity of pile foundations. Settlement of pile foundations should be estimated using unfactored loads.

Foundation design should not be uncoupled from construction considerations. Performance factors can be affected by careful monitoring of pile installation and by how contract documents are written to permit changes in installation procedures. For example, higher performance factors may be used if wave equation analyses are run prior to driving, pile driving blow counts are compared to the wave equation results, and representative piles are monitored with the Pile Driving Analyzer.

4.6 DESIGN EXAMPLES

The design procedures discussed in the previous sections are demonstrated in the following example problems.

EXAMPLE 4.1

Determine whether a 30 ft long HP 14 X 89 pile has adequate structural and bearing capacity to support a dead load of 40 tons and a live load of 34 tons. Electric cone penetration test data for the soil at the site is given in Fig. E4.1.

From Table A1.2, the section properties of HP 14 X 89 are: Depth = D = 13.83 in, Width = b_f = 14.7 in, A = area of steel = 26.1 in^2, Thickness of web (and flange) = $t_w = t_f$ = 0.615 in

The capacity of the pile will be estimated for conditions where: (1) the pile is plugged and (2) the pile is unplugged. The case that yields the minimum capacity will govern the design.

CASE I - ASSUME THAT THE PILE IS UNPLUGGED

1) Design Load (Group I)

Load per pile = $1.3P_D + 2.17P_L$ = (1.3)(40) + (2.17)(34)

= 126 tons/pile

2) Estimate Axial Capacity

a) Structural Capacity

From Table A2.1

$[P_n] = f_yA_y$ = (36)(26.1) = 940 kips/pile = 470 tons/pile

From Table 4.1, ϕ_a = (0.85)(0.78) = 0.66

$\phi_a[P_n]$ = (0.66)(470) = 310 tons/pile > 126 tons/pile.

Therefore the structural capacity of the pile is adequate.

b) Bearing Capacity

Fig. E4.1 shows that the pile will be driven through 3 different soil layers. The ultimate bearing capacity of the pile is the sum of the skin friction in soil layers 1 (clay), 2 (sand) and 3 (clay), and the end bearing resistance of the pile.

End Bearing

End bearing capacity is calculated using the "minimum path rule" described in Fig. 4.8. Values of q_{c1} and q_{c2} are calculated using the procedures described in that figure.

y	yD (m)	Z + yD (m)	q_{c1}
0.7	0.25	9.39	[44+2(74)+44]/4 = 59 tsf
1.0	0.35	9.49	[44+74+3(56)+44]/6 = 55 tsf
1.5	0.53	9.67	[44+74+56+51+6(40)]/10 = 46.5 tsf
2.0	0.70	9.84	[44+74+56+51+40+56+2(68)+56+5(40)]/14 = 50.9 tsf
2.5	0.88	10.02	[44+74+56+51+40+56+68+67+4(61)+56+5(40)]/18 = 53.1 tsf
3.0	1.05	10.19	[44+74+56+51+40+56+68+67+61+5(58)+56+5(40)]/20 = 53.2 tsf
3.5	1.23	10.37	[44+74+56+51+40+56+68+67+61+58+60+7(58)+56+5(40)]/24 = 54 tsf
4.0	1.41	10.55	[44+74+56+51+40+56+68+67+61+58+60+58+59+10(44)+5(40)]/28 = 49.7 tsf

DEPTH m	qc tsf	fs tsf	Fr %
0.1	114	7.13	6.25
0.2	108	6.07	5.62
0.3	112	6.07	5.41
0.4	138	5.28	3.82
0.5	130	5.41	4.16
0.6	118	5.81	4.92
0.7	114	5.15	4.51
0.8	124	4.22	3.40
0.9	130	5.68	4.36
1	122	5.81	4.76
1.1	134	5.81	4.33
1.2	138	7.26	5.26
1.3	106	5.15	4.85
1.4	156	4.75	3.04
1.5	138	5.68	4.11
1.6	156	6.07	3.89
1.7	142	4.88	3.43
1.8	178	6.73	3.78
1.9	166	6.47	3.89
2	152	5.28	3.47
2.1	152	6.6	4.34
2.2	156	6.6	4.23
2.3	170	4.22	2.48
2.4	190	4.36	2.29
2.5	208	3.7	1.77
2.6	168	4.49	2.67
2.7	134	3.83	2.85
2.8	150	3.04	2.02
2.9	170	2.64	1.55
3	250	4.75	1.9
3.1	228	2.77	1.21
3.2	262	4.22	1.61
3.3	330	5.41	1.63
3.4	284	3.7	1.30
3.5	240	3.43	1.42
3.6	234	2.77	1.18
3.7	222	2.77	1.24
3.8	198	1.58	0.79
3.9	150	2.24	1.49
4	152	1.45	0.95
4.1	150	1.58	1.05
4.2	132	1.32	1
4.3	112	0.79	0.70
4.4	122	1.06	0.86
4.5	124	1.45	1.16
4.6	72	0.92	1.27
4.7	56	0.92	1.64
4.8	56	0.79	1.41
4.9	70	0.79	1.12
5	96	0.79	0.82
5.1	98	0.92	0.93
5.2	110	1.45	1.31
5.3	90	1.19	1.32

DEPTH m	qc tsf	fs tsf	Fr %
5.4	128	1.72	1.34
5.5	146	1.72	1.17
5.6	182	1.72	0.94
5.7	212	2.24	1.05
5.8	238	2.11	0.88
5.9	252	1.9	0.75
6	202	2.17	1.07
6.1	98	3.04	3.10
6.2	66	1.72	2.60
6.3	62	2.24	3.61
6.4	50	1.39	2.78
6.5	52	1.58	3.03
6.6	52	1.72	3.30
6.7	54	1.78	3.29
6.8	52	1.72	3.30
6.9	50	1.98	3.96
7	48	1.85	3.85
7.1	45	1.85	4.11
7.2	46	1.72	3.73
7.3	50	1.45	2.9
7.4	58	1.98	3.41
7.5	76	1.98	2.60
7.6	54	2.31	4.27
7.7	58	1.98	3.41
7.8	61	1.72	2.81
7.9	76	1.72	2.26
8	86	1.98	2.30
8.1	100	2.51	2.51
8.2	108	2.77	2.56
8.3	114	2.77	2.42
8.4	108	2.79	2.58
8.5	106	2.9	2.73
8.6	-	-	-
8.7	106	2.77	2.61
8.8	104	4.09	3.93
8.9	82	2.9	3.53
9	64	3.83	5.98
9.1	50	1.78	3.56
9.2	44	1.74	3.95
9.3	74	2.33	3.14
9.4	56	1.33	2.37
9.5	51	1.32	2.58
9.6	40	1.32	3.3
9.7	56	1.04	1.85
9.8	68	1.03	1.51
9.9	67	1.03	1.53
10	61	1.32	2.16
10.1	58	1.03	1.77
10.2	60	1.12	1.86
10.3	58	1.06	1.82
10.4	59	1.12	1.89
10.5	44	1.06	2.40
10.6	52	1.06	2.03

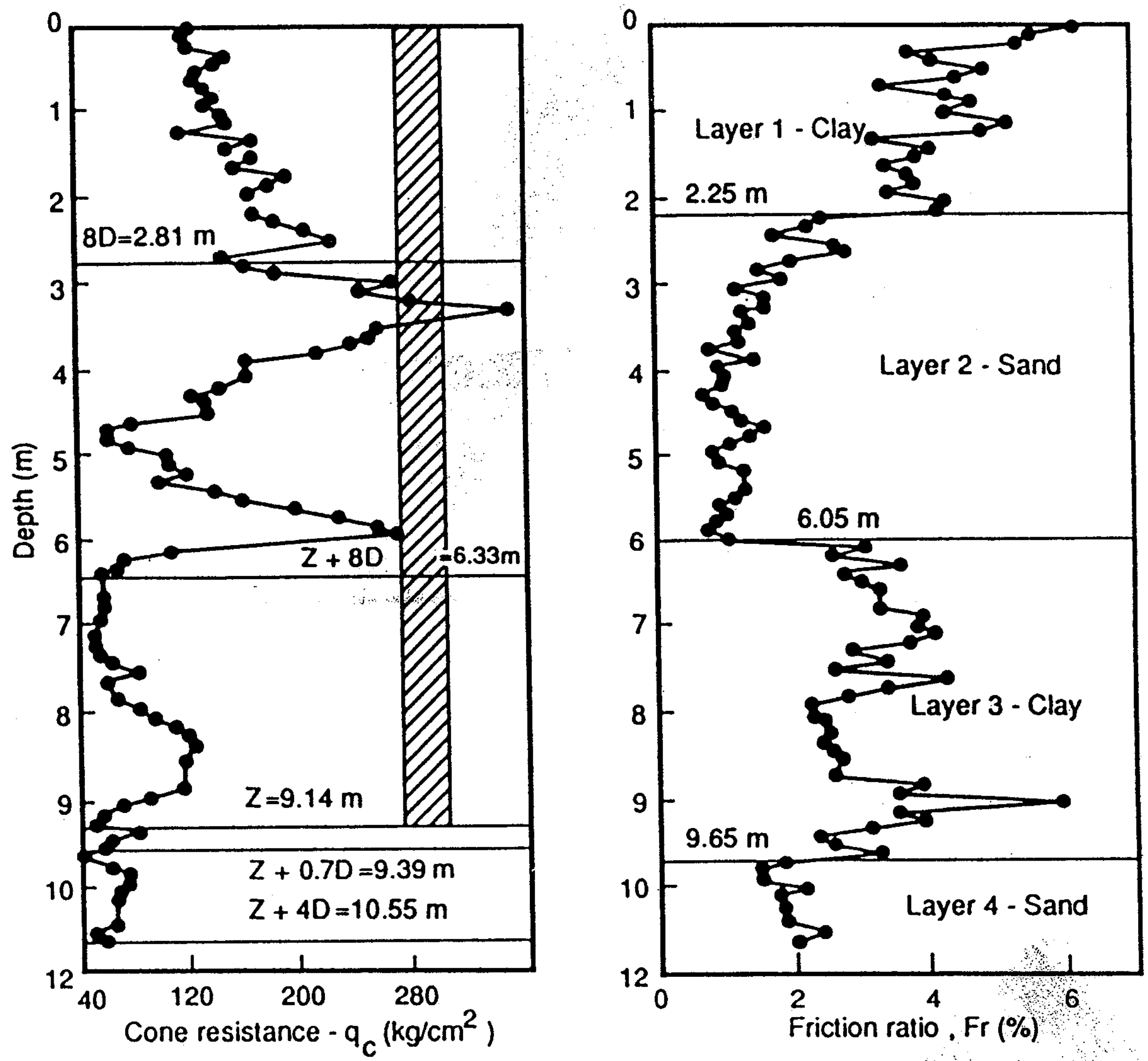


Figure E4.1 Cone penetration test data for example problem 4.1

q_{c2} = 40 tsf since there is no value of q_c for 8 diameters above the tip, that is less than the value of 40 tsf at yD = 0.53 m. below the pile tip, so q_{c2} = 40 tsf. The value of q_p is the average of q_{c1} and q_{c2}:

$$q_p = [46.5 + 40]/2 = 43.3 \text{ tsf}$$

In an unplugged pile, the point resistance is calculated using the cross-sectional area of the steel-H section.

$$Q_p = q_p A_p$$
$$= (43.3)(26.1)/144$$
$$= 7.9 \text{ tons}$$

Skin Friction

The ultimate skin friction of piles can be computed using Equation 4.1.3.25 as follows:

$$Q_s = K_{s,c}\left[\sum_{L_f=0}^{8D} (L_f/8D) f_s a_s + \sum_{L_f=8D}^{Z} f_s a_s\right]$$

In an unplugged pile, the soil adheres to the entire perimeter of the steel-H section.

Pile perimeter, a_s = 2[14.7 + 13.83 + 14.7 - 0.615]/12

= 7.1 ft

LAYER 1: L_f = depth to point considered = 2.25 m = 7.38 ft

$$8D = \frac{8 \times 13.83}{12} = 9.22 \text{ ft}$$

$$L_f/8D = 7.38/9.22 = 0.8$$

From depth of 0 to 2.25 m (7.38 ft), the average f_s = 5.8 tsf

From Fig 4.9, K_c = shaft friction correction factor for piles in clay = 0.2

Q_{s1} = 0.8 X 0.2 X 7.1 X 7.38 X 5.8 = 48.6 tons

LAYER 2: A depth of embedment correction must be made between the top of this layer (2.25 m or 7.38 ft) and depth of 8D (2.81 m or 9.22 ft).

$(L_f/8D)_{7.38\ ft}$ = 7.38/9.22 = 0.8

$(L_f/8D)_{9.22\ ft}$ = 1.0

$(L_f/8D)_{avg}$ = [0.8 + 1.0]/2 = 0.9

The pile penetration to diameter ratio is calculated as follows:

$$\frac{Z}{D} = \frac{30 \times 12}{13.83} = 26.0$$

From Fig. 4.9, K_s = shaft friction correction factor for piles in sand = 0.7

From depth of 2.25 m (7.38 ft) to 2.81 m (9.22 ft), the average f_s = 3.94 tsf

Q_{s2a} = 0.9 X 0.7 X 7.1 X (9.22 - 7.38) X 3.94

= 32.4 tons

From depth of 2.81 m (9.22 ft) to 6.05 m (19.85 ft), the average f_s = 2.04 tsf

Q_{s2b} = 0.7 X 7.1 X (19.85 - 9.22) X 2.04 = 108 tons

LAYER 3:

From depth of 6.05 m (19.85 ft) to 9.14 m (30 ft), the average f_s = 2.23 tsf

From Fig. 4.9, K_c = 0.2

Q_{s3} = 0.2 X 7.1 X (30 - 19.85) X 2.23 = 32.1 tons

Total skin friction Q_s = 48.6 + 32.4 + 108 + 32.1

= 221 tons

Total ultimate pile capacity Q_{ult} = 221 + 7.9

= 229 tons

From Table 4.3, performance factor for CPT method = 0.55

$\phi_q Q_{ult}$ = 0.55 X 229 = 126 tons ≥ 126 tons. Therefore, the bearing capacity of the pile is adequate.

CASE II - ASSUME THAT THE PILE IS PLUGGED

End Bearing

In a plugged pile, the point resistance is calculated using the cross-sectional area of a rectangle 13.83 in. X 14.7 in.

$Q_p = q_p A_p$

= (43.3)(13.83)(14.7)/144

= 61.1 tons

Skin Friction

In a plugged pile, the skin friction is the sum of the adhesion at the flanges and the full soil-to-soil shearing resistance along both sides of the soil plug.

Pile perimeter for adhesion at flanges = 2(14.7)/12

= 2.45 ft

Pile perimeter for soil-to-soil shearing resistance along both sides of the soil plug = 2(13.83)/12

= 2.31 ft

LAYER 1: Assume soil-to-soil shearing resistance is 1.5 times that of adhesion

Q_{s1} = 0.8 X 0.2 X (2.45 + 1.5 X 2.31) X 7.38 X 5.8

= 40.5 tons

LAYER 2: Assume soil-to-soil shearing resistance is 1.5 times that of soil-to-pile friction in sand

From depth of 2.25 m (7.38 ft) to 2.81 m (9.22 ft),

Q_{s2a} = 0.9 X 0.7 X (2.45 + 1.5 X 2.31) X (9.22 - 7.38) X 3.94

= 27.0 tons

From depth of 2.81 m (9.22 ft) to 6.05 m (19.85 ft),

Q_{s2b} = 0.7 X (2.45 + 1.5 X 2.31) X (19.85 - 9.22) X 2.04

= 89.8 tons

LAYER 3: Assume soil-to-soil shearing resistance is 1.5 times that of adhesion

From depth of 6.05 m. (19.85 ft) to 9.14 m. (30 ft)

Q_{s3} = 0.2 X (2.45 + 1.5 X 2.31) X (30 - 19.85) X 2.23

= 26.7 tons

Total Skin Friction Q_s = 40.5 + 27.0 + 89.8 + 26.7

= 184 tons

Total Ultimate Pile Capacity, Q_{ult} = 184 + 61.1

= 245 tons > 229 tons

Assuming that the pile will be plugged results in a higher capacity (245 tons) than assuming that it is not plugged (capacity = 229 tons if the pile is not plugged). Therefore the unplugged condition controls, and the estimated capacity is 229 tons.

EXAMPLE 4.2

Given the dead load of the bridge superstructure and column is 50 kips and the live load is 50 kips, check the adequacy of the pile foundation shown in Fig. E4.2.

1) Determine design load on the piles

Weight of pile cap = (4)(7)(7)(150)/1000 = 29.4 kips

Weight of soil above pile cap = 6[(7)(7) - (4)(2)]120/1000

= 29.5 kips

Dead load due to pile cap and soil above = 29.4 + 29.5

= 59 kips

Total dead load = 50 + 59 = 109 kips

$\gamma_D P_D + \gamma_L P_L$ = (1.3)(109) + (2.17)(50)

= 142 + 109

= 251 kips

2) Select pile type - 12 in. X 12 in. prestressed concrete pile (f_c' = 5000 psi) with six 7/16 in. grade 270 axial strand. Piles are prestressed to 700 psi.

3) Estimate axial capacity of a single pile

a) Structural Capacity

From Table A2.1,

$$P_n = (0.85f_c' - 0.6f_{pre})A_c$$

$$= [(0.85)(5) - (0.6)(0.7)]144$$

$$= 552 \text{ kips/pile}$$

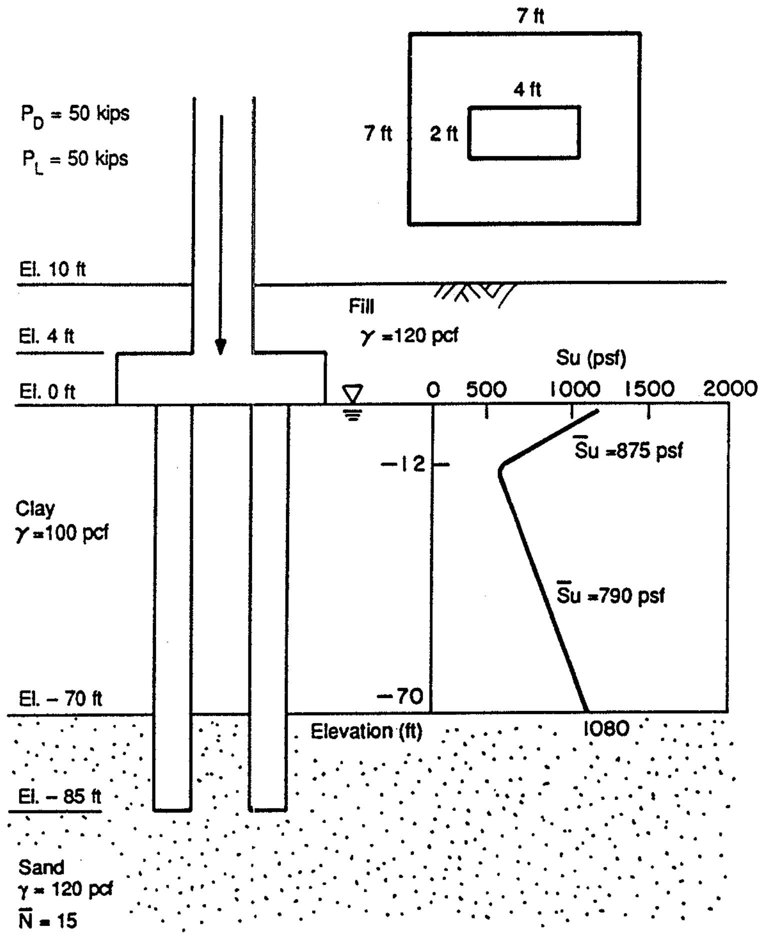


Figure E4.2 Figure for example problem 4.2

From Table 4.1, $\phi_a = (0.85)(0.75) = 0.64$

$$\phi_a P_n = (0.64)(552)$$

$$= 353 \text{ kips/pile}$$

Number of piles needed = 251/353 or 1 pile

b) Bearing Capacity - Assuming that the pile penetrates into the lower sand stratum, the ultimate bearing capacity is the sum of the skin friction of the pile in both clay and sand, and the tip capacity.

Skin Friction of Pile in Clay

Using Fig 4.2,

From elevation 0 to -12 ft, $\overline{S}_u = 0.875$ ksf, $\alpha = 0.8$

From elevation -12 to -70 ft, $\overline{S}_u = 0.79$ ksf, $\alpha = 0.83$

Using Equation 4.1.3.4

$$Q_s = (0.8)(0.875)(12)(4) + (0.83)(0.79)(58)(4)$$

$$= 34 + 152$$

$$= 186 \text{ kips}$$

Skin Friction of Pile in Sand

From elevation -70 to -85 ft, $\overline{N} = 15$

Using Equation 4.1.3.23,

$$Q_s = \frac{15}{50}(4)(15)$$

$$= 18 \text{ tons or } 36 \text{ kips}$$

Tip Capacity of Pile in Sand

$\sigma_v{}'$ at the pile tip = 70(100 - 62.4) + 15(120 - 62.4)

= 3500 psf or 1.75 tsf

$$N_{corr} = [0.77\log_{10}(20/1.75)]15$$

$$= (0.815)(15)$$

$$= 12$$

Since pile penetrates 15 ft into sand stratum, use $q_p = q_l$.

$$q_p = 4N_{corr}$$

$$= (4)(12)$$

$$= 48 \text{ tsf}$$

$$Q_p = (48)(1)$$

$$= 48 \text{ tons or } 96 \text{ kips}$$

Total Factored Pile Capacity

From Table 4.3,

Performance factor for α-method is 0.70.

Performance factor for SPT method is 0.45

$$\phi_q Q_{ult} = (0.70)(186) + (0.45)(36 + 96)$$

$$= 130 + 59$$

$$= 189 \text{ kips}$$

Number of piles needed = 251/189 or 2 piles.

4) Select pile spacing and number of piles

Pile spacing = 3 X pile width = 3 ft

Use 4 piles in the group.

5) Estimate group capacity

The ultimate capacity of pile groups in sand is the number of piles times the factored capacity of a single pile

= (4)(196)

= 784 kips > 251 kips

6) Estimate settlement capacity of pile group

Assume that the tolerable settlement is 2 in.

From Equation 4.2.2.1

$$\rho = \frac{2q\sqrt{X}I}{N_{corr}}$$

$$X = Y = 3 + 1 = 4 \text{ ft}$$

$$I = 1 - \frac{10}{(8)(4)}$$

$$= 0.688$$

$$P_D + P_L = 109 + 50 = 159 \text{ kips}$$

$$q = 159/4^2$$

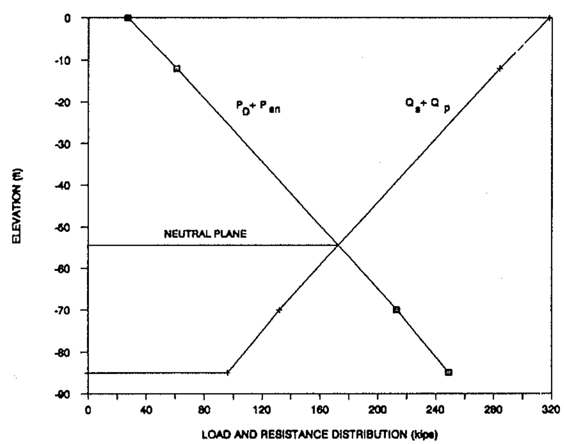
$$= 9.94 \text{ ksf}$$

$$= 4.97 \text{ tsf}$$

$$\rho = \frac{(2)(4.97)\sqrt{4}\ (0.688)}{12}$$

$$= 1.14 \text{ in.} < 2 \text{ in.}$$

7) Check the effect of the downdrag load



The neutral plane occurs at a depth of 54 ft. The load at the neutral plane is 172 kips/pile or 688 kips for the pile group (greater than the working load = 159 kips). Thus, settlement due to downdrag will be greater than the tolerable value of 2 in. The foundation is therefore inadequate. The bearing capacity of the pile group should also be checked to see if the downdrag loads can be adequately supported, but since the tolerable settlement has been exceeded, the foundation must be redesigned with (1) more piles or (2) longer piles.

CHAPTER 5

DESIGN OF PILES FOR LATERAL LOADING

Lateral loads on pile foundations arise because of wind, earthquake, water pressures, earth pressures, and live loads. Pile foundations must be designed to withstand such forces without failing (i.e., without reaching the ultimate limit state) and without deflecting excessively (i.e., without reaching the serviceability limit state).

Batter piles are frequently used to resist lateral loads. Vertical piles alone may suffice in foundations that carry horizontal loads of low magnitudes. Design methodologies for both cases are presented in the following sections.

5.1 BATTER PILES

When lateral loads acting on a foundation are large, batter piles provide an effective way of transmitting loads to the soil. The degree of batter will depend on the type of pile and the magnitude of the lateral loads. Installation by driving is feasible for batters as large 1 horizontal to 2 vertical (Tomlinson, 1987). According to Tomlinson, the greatest efficiency is achieved by using piles battered in opposite directions.

There are situations where the use of batter piles may be undesirable. These include conditions involving large settlements in compressible clays. Settlement induces bending moments in the shafts of batter piles (Tomlinson, 1987).

Tomlinson (1987) described a simple graphical procedure for estimating the compressive and tensile forces in pile groups containing batter piles. The procedure is based on the assumption that (1) the battered piles are pinned at their point of intersection, (2) vertical piles in the group do not carry lateral loads, and (3) batter piles carry only axial loads. Tomlinson's procedure does not consider pile-soil-pile interaction, pile stiffness, soil stiffness, and pile head fixity, all of which can significantly affect the distribution of forces in piles in a pile group. Nevertheless, Tomlinson's graphical procedure is useful for obtaining a preliminary pile layout, and is reasonably accurate if the lateral load is less than 20 percent of the vertical load (Department of the Army, in press).

If the pile group has more than three rows, Tomlinson's simple procedure is not applicable, and, as mentioned previously, it may be inaccurate if the lateral loads are large. More complex methods based on linear elastic and nonlinear elastic soil response are available for analyzing two-dimensional and three-dimensional pile groups. These methods are often very involved and require the use of a computer.

Hrennikoff's (1950) linear elastic procedure may be used to solve for the pile forces and displacements in pile groups that can be modeled in two dimensions. Saul (1968) expanded Hrennikoff's solution to three dimensions. O'Neill, Ghazzaly and Ha (1977) and O'Neill and Tsai (1984) have developed a method of analysis for three-dimensional pile groups that considers nonlinear soil response and pile-soil-pile interaction.

5.2 VERTICAL PILES

The governing criterion in the design of laterally loaded piles is almost always the maximum tolerable deflection or the structural capacity of the pile itself. Mobilizing the ultimate lateral capacity of the soil requires such large displacements that this is not a realistic possibility, and ultimate soil failure does not control the design.

In designing vertical piles to resist lateral loads, both lateral deflection and structural capacity should be considered. Procedures for addressing these issues are described in the following sections.

5.2.1 Lateral Deflection

One of the design objectives is to ensure that the lateral deflection of the pile group does not exceed the tolerable limit. The lateral deflection of a pile group can be related to the lateral deflection of a single pile. Procedures for estimating the lateral deflections of single piles and pile groups are described in the following sections.

5.2.1.1 Single Pile Deflection

Poulos and Davis (1980) described three methods of analyzing the behavior of single piles under lateral load. They include elastic analysis, subgrade reaction analysis, and p-y analysis. Elastic analyses and subgrade reaction analyses approximate the soil behavior as linear; p-y analyses model nonlinear behavior of the soil, but require the use of computer programs and involve considerable engineering time.

The procedure described in this manual is the one developed by Evans and Duncan (1982). The method models nonlinear behavior, but does not require computer analyses. The charts discussed in the following sections are for fixed-head piles. Piles that are embedded in reinforced concrete pile caps are effectively restrained from rotation at the top, and they deflect laterally with negligible rotation at the top of the pile.

Evans and Duncan's Procedure. Evans and Duncan (1982) related lateral deflections to the lateral loads using what they called a characteristic load, P_c. The characteristic load, P_c, embodies the important properties of the pile (diameter, stiffness) and the soil (strength, stiffness) that determine the way the pile and soil respond to lateral loads. The larger the value of P_c, the greater is the capacity of the pile to carry lateral loads, and the smaller is its deflection under a given lateral load.

Charts in dimensionless form were developed for sand and clay (Figures 5.1 and 5.2). These charts show variations of P_{sp}/P_c with Y_{sp}/D; P_{sp} is the unfactored lateral load, Y_{sp} is the pile displacement, and D is the pile width or diameter. The charts

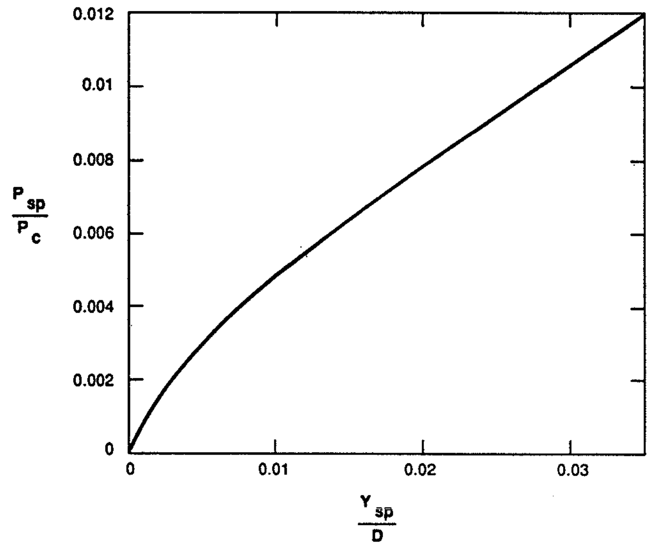


Figure 5.1. Lateral load versus deflection for fixed head piles in sand. (After Evans and Duncan, 1982)

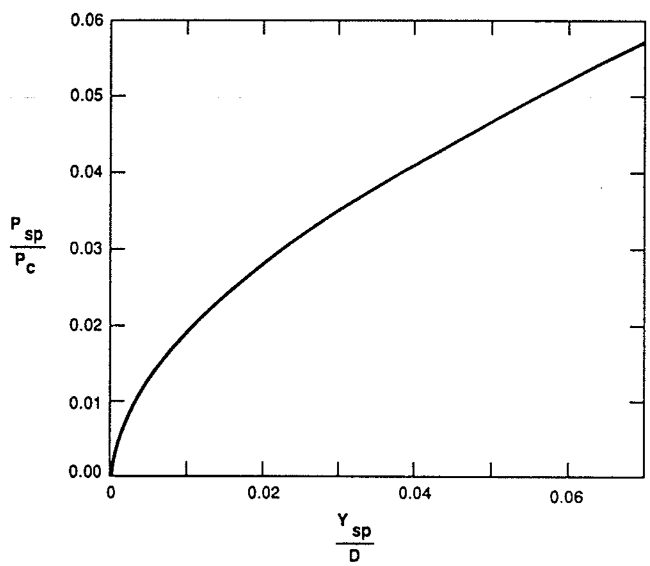


Figure 5.2. Lateral load versus deflection for fixed-head piles in clay. (After Evans and Duncan, 1982)

model the same nonlinear behavior of soil as the p-y method of analysis. The procedure for determining the lateral deflection of a pile, using Figures 5.1 and 5.2, is as follows:

1. Select a pile section having a width or diameter D, Young's modulus E_p, and moment of inertia I_p.

For prestressed and precast concrete piles, the value of Young's modulus can be related to the concrete compressive strength and density as shown in Figure 5.3. The modulus of steel can be taken as 29×10^6 psi. The National Forest Products Association (1982) recommends that the Young's modulus of all species of Douglas Fir and Southern Pine piles be taken as 1.5×10^6 psi. Tables of sectional properties for prestressed concrete, steel-H and pipe piles can be found in Appendix 1.

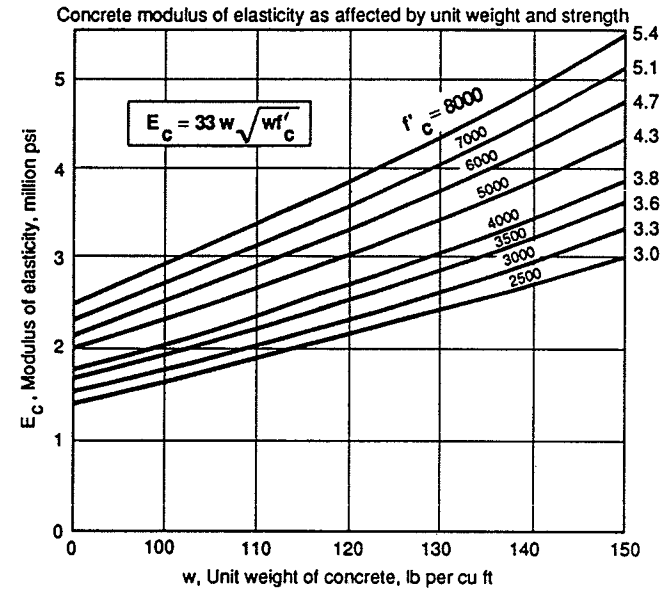


Figure 5.3. Modulus of elasticity of concrete. (After PCI, 1985)

2. Estimate the average undrained shear strength, S_u, for clays, or the average angle of internal friction, ϕ', for sands.

The behavior of the soil close to the ground surface is most important with regard to lateral loads. The properties (S_u for clays, ϕ', and unit weight, γ', for sands) should be averaged over a depth extending about eight pile diameters below the top of the pile. Buoyant unit weights for sands are used below the water table.

3. Determine the characteristic load, P_c, which is defined by the following equations:

For clay

$$P_c = 7.34\, D^2\, (E_p R_I)\, (S_u/E_p R_I)^{0.683} \qquad (5.2.1.1.1)$$

For sand

$$P_c = 1.57\, D^2\, (E_p R_I)\, (\gamma' D \phi' K_p/E_p R_I)^{0.57} \qquad (5.2.1.1.2)$$

where R_I = moment of inertia ratio = I_p/I_{solid}; $I_{solid} = \pi D^4/64$ = moment of inertia of a solid circular cross section; K_p = Rankine passive earth pressure coefficient = $\tan^2(45° + \phi'/2)$; and ϕ' = angle of internal friction of sand, in degrees.

4. Calculate the value of the load ratio P_{sp}/P_c.

5. Use Figure 5.1 for sand or Figure 5.2 for clay to determine the value of Y_{sp}/D.

6. Calculate $Y_{sp} = D\,(Y_{sp}/D)$.

This procedure has been used to develop lateral load-deflection curves for some commonly used pile sections. Charts for prestressed concrete piles (10 in., 12 in., 14 in., 16 in., and 18 in. square) and steel-H piles (HP 10x42, HP 10x57, HP 12x53, HP 12x74, HP 14x73 and HP 14x89) in clay and sand are shown in Figures 5.4 through 5.7. For these piles and soil conditions, deflections can be estimated directly using the charts. For example, a lateral load of 10 kip acting on a 12 in. by 12 in. prestressed concrete pile, driven in clay with an undrained shear strength of 1 ksf, will result in a lateral deflection of about 0.1 in (Figure 5.5).

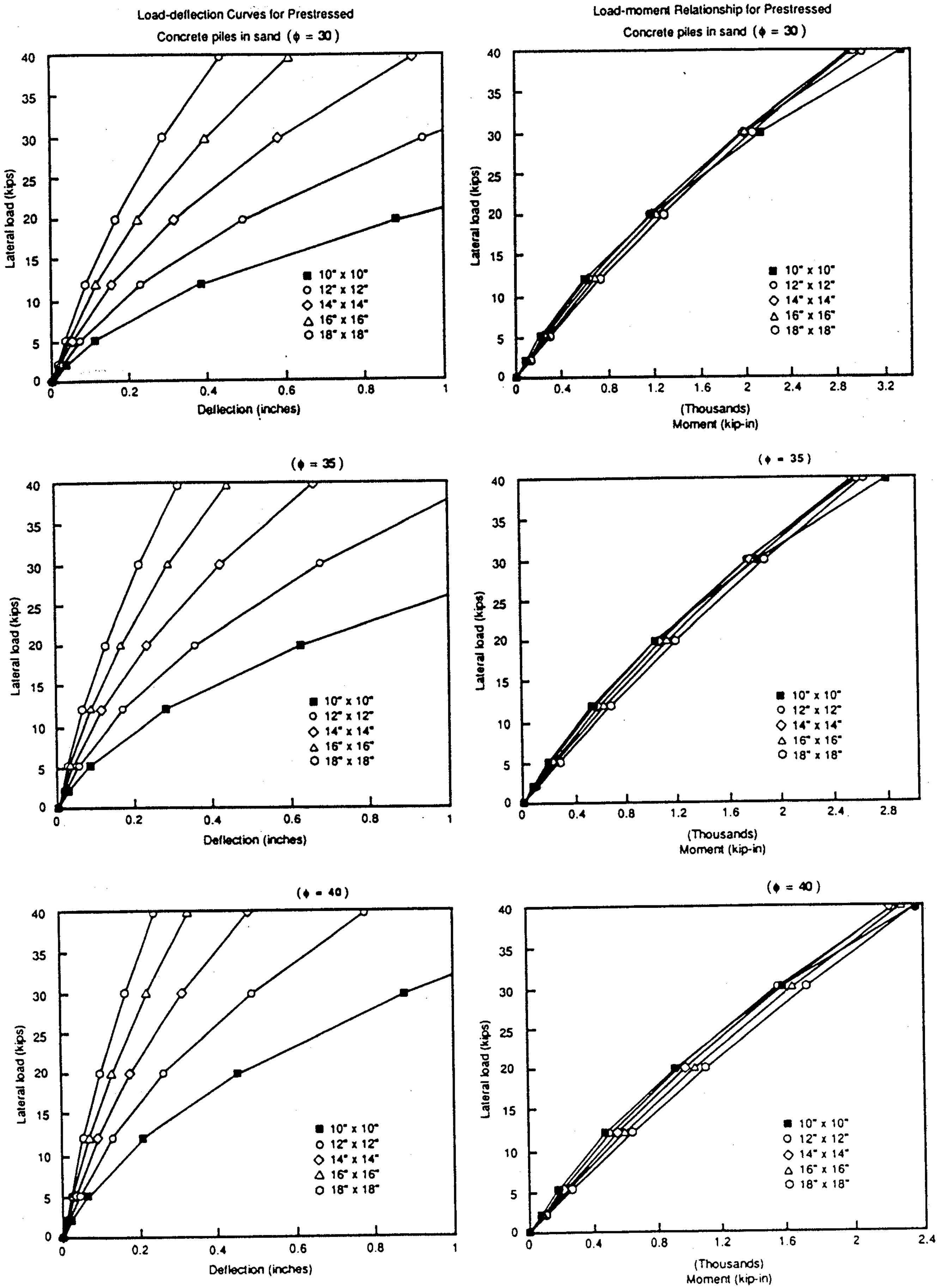


Figure 5.4. Load versus deflection and load versus moment for prestressed concrete piles in sand.

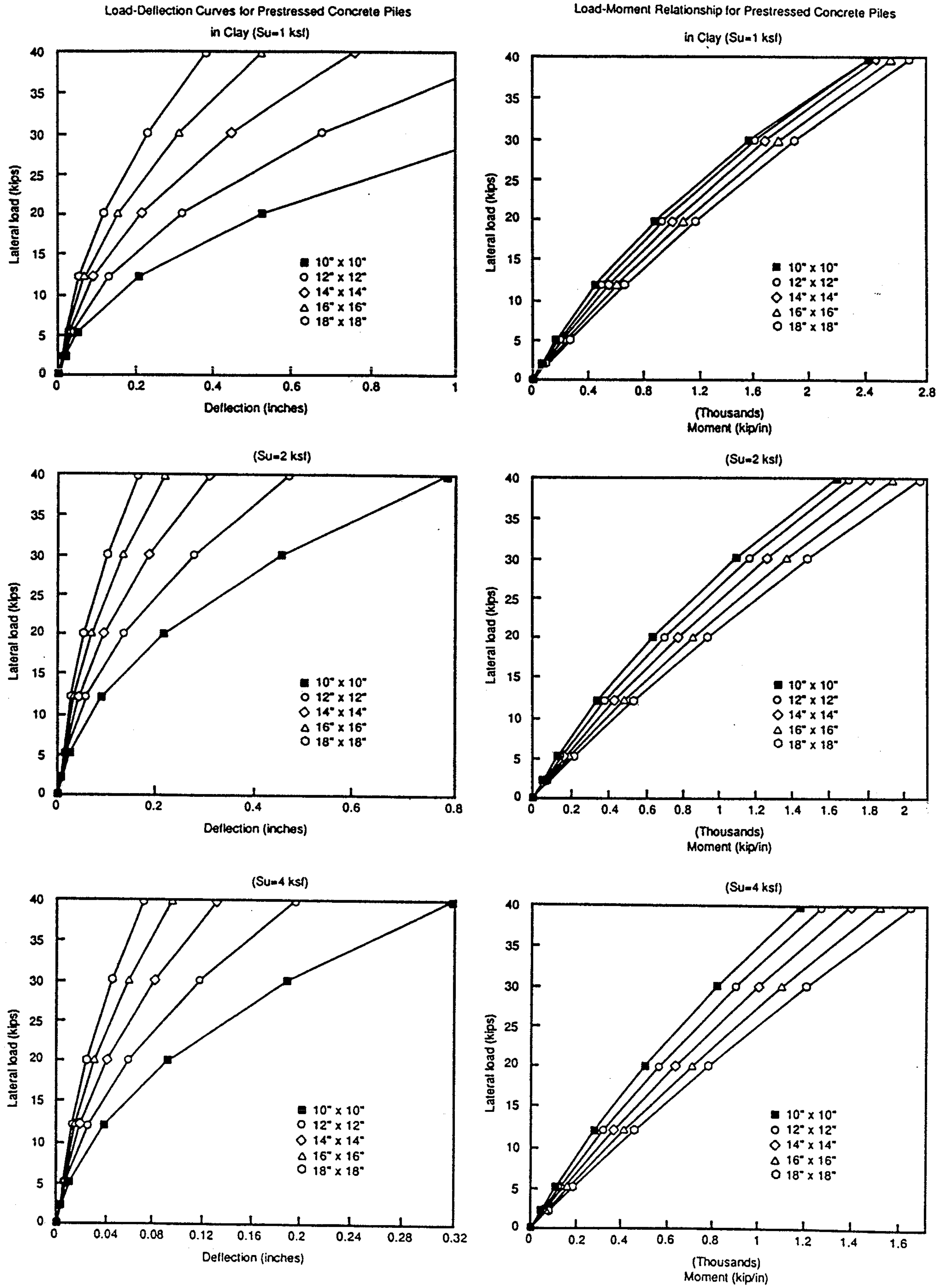


Figure 5.5. Load versus deflection and load versus moment for prestressed concrete piles in clay.

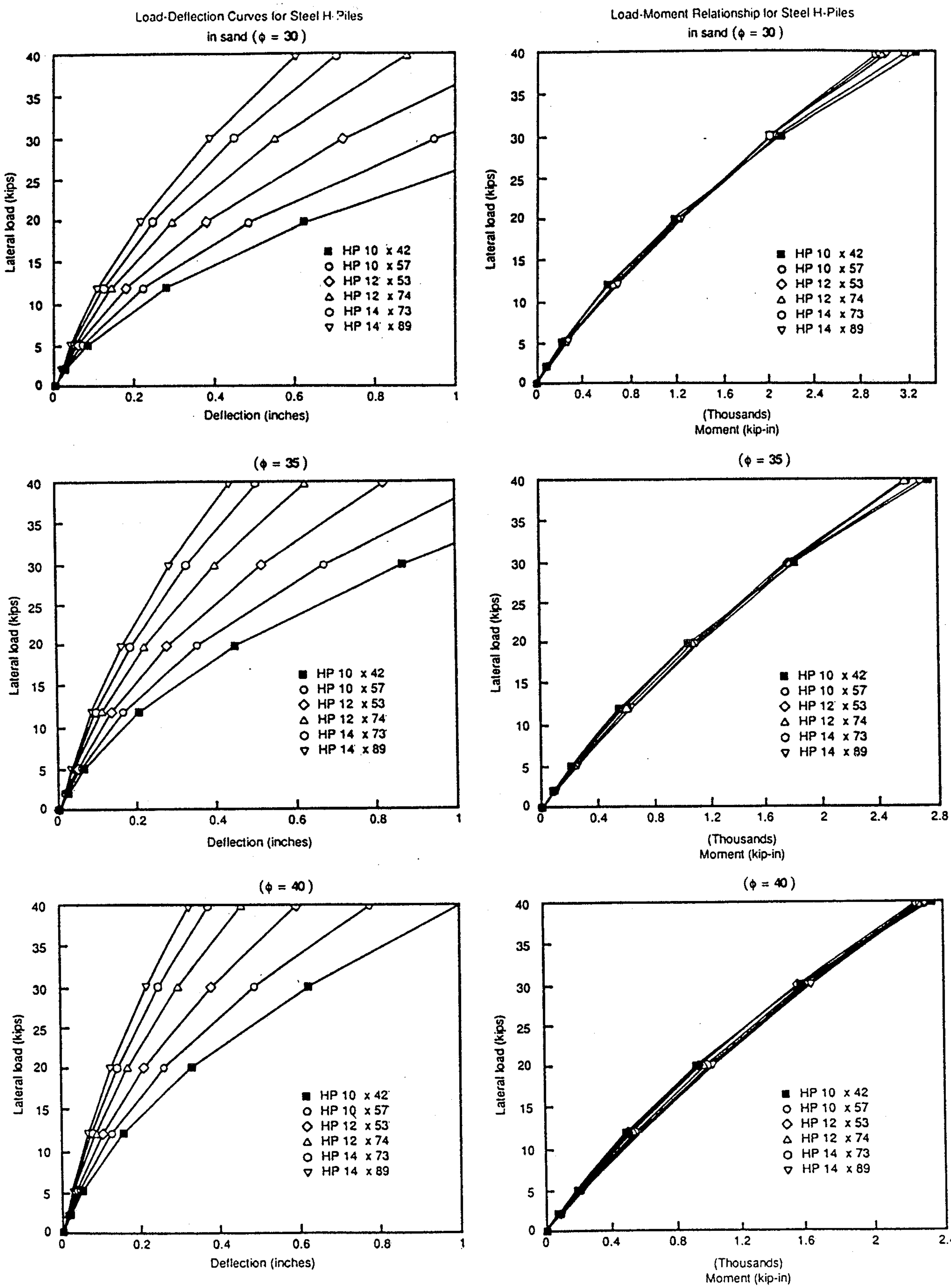


Figure 5.6. Load versus deflection and load versus moment for steel-H piles in sand.

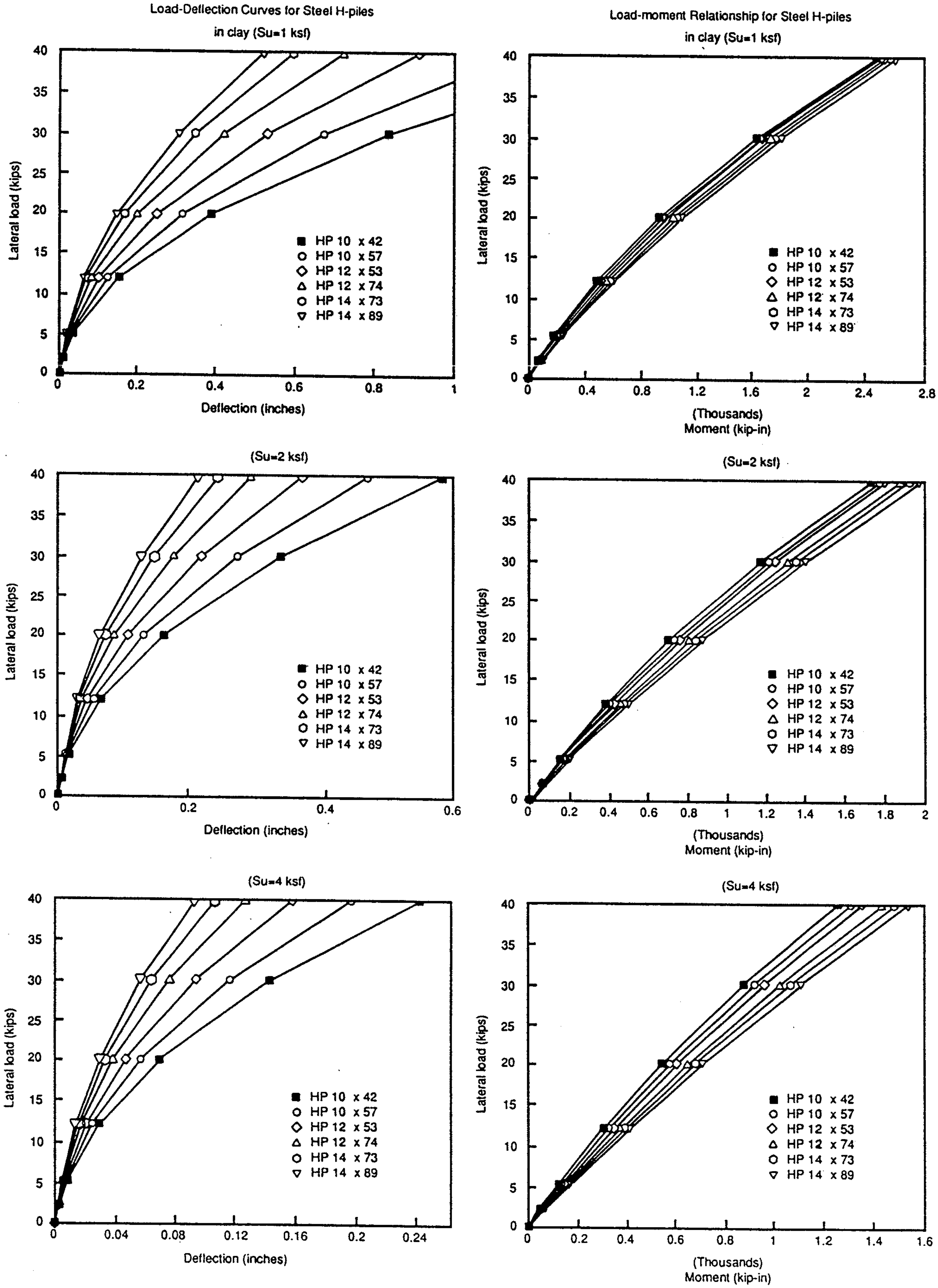


Figure 5.7. Load versus deflection and load versus moment for steel-H piles in clay.

For sands, charts were developed for friction angles of 30 deg., 35 deg., and 40 deg. The water table was assumed to be at or above the ground surface. For intermediate values of friction angle between those shown in the charts, deflections may be estimated by interpolation.

For clays, the load-deflection curves were developed for undrained shear strengths of 1, 2, and 4 ksf. Deflection for intermediate values of undrained shear strengths can be estimated by interpolation.

5.2.1.2 Pile Group Deflection

Excessive horizontal displacements of pile groups may cause distress in bridges and buildings. It is, therefore, necessary both to estimate the largest lateral movement that can be tolerated without damage and to ensure that lateral deflections in service are within the tolerable range.

A pile group will deflect more than a single pile subjected to the same lateral load per pile. This is due to interaction effects whereby deflection of each pile in a group causes deflection of the surrounding soil and thereby increases the deflections of neighboring piles.

The lateral deflection of a group of piles may be estimated using the empirical Eq. 5.2.1.2.1:

$$Y_g = \frac{A + N_{pile}}{B\sqrt{\frac{S}{D} + \frac{P_{sp}}{CP_N}}} Y_{sp} \qquad (5.2.1.2.1)$$

where Y_{sp} = lateral displacement of a single fixed-head pile subjected to a lateral load P_{sp}; N_{pile} = number of piles in group; S = average spacing of piles; D = pile width or diameter; P_{sp} = average lateral load per pile = P_{Yg}/N_{pile}, in which P_{Yg} = lateral load on pile group;

$$P_N = K_p \gamma D^3 \textit{ for sand} \qquad (5.2.1.2.2)$$

$$P_N = S_u D^2 \textit{ for clay} \qquad (5.2.1.2.3)$$

γ = *total* unit weight of sand; K_p = Rankine passive earth pressure coefficient = $\tan^2 (45° + \phi'/2)$; ϕ' = average angle of internal friction of sand within the upper 8 pile diameters; S_u = average undrained shear strength of clay within the upper eight pile diameters. A = 16 for clay and A = 9 for sand; B = 5.5 for clay and B = 3 for sand; C = 3 for clay and C = 16 for sand.

Equation 5.2.1.2.1 was developed through a parametric study of a large number of pile groups using the theories proposed by Focht and Koch (1973). It was developed for uniformly spaced piles, but can be used for groups with nonuniform spacing if the average pile spacing is used in the calculations.

A computer program for calculating the lateral displacement of pile groups using the theory of Focht and Koch has been developed by the writers, and was used to perform the parametric study.

The load factor design approach to the lateral deflection of pile groups requires an estimation of the tolerable lateral displacement. The procedure is as follows:

1. Determine a tolerable lateral displacement, Y_{tol}.
2. Calculate the lateral load per pile $P_{sp} = P_{Yg}/N_{pile}$.
3. Calculate the single pile deflection, Y_{sp}, corresponding to the lateral load per pile, P_{sp}, using either Evans and Duncan's procedure (Section 5.2.1.1) or Figures 5.4 to 5.7.
4. Calculate $k_g = Y_g/Y_{sp}$ using Eq. 5.2.1.2.1 and, $Y_g = k_g Y_{sp}$.
5. If the lateral displacement for the pile group exceeds the tolerable lateral displacement, increase the diameter of the piles, the number of piles, or the pile spacing.

5.3 COMBINED AXIAL LOADS AND BENDING MOMENTS

5.3.1 Estimation of Bending Moment in a Single Pile

Evans and Duncan (1982) developed a simple procedure for estimating the maximum bending moment induced in single piles, M_{sp}, due to a lateral load at the top of the pile. They developed the design charts shown in Figures 5.8 and 5.9 for

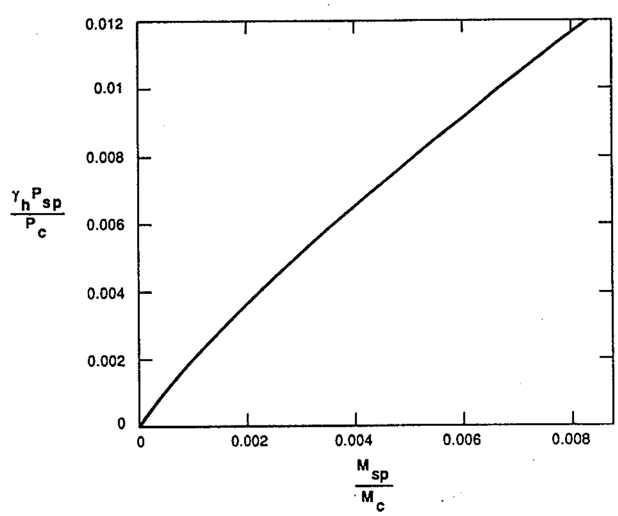


Figure 5.8. Lateral load versus moment for fixed-head piles in sand. (After Evans and Duncan, 1982)

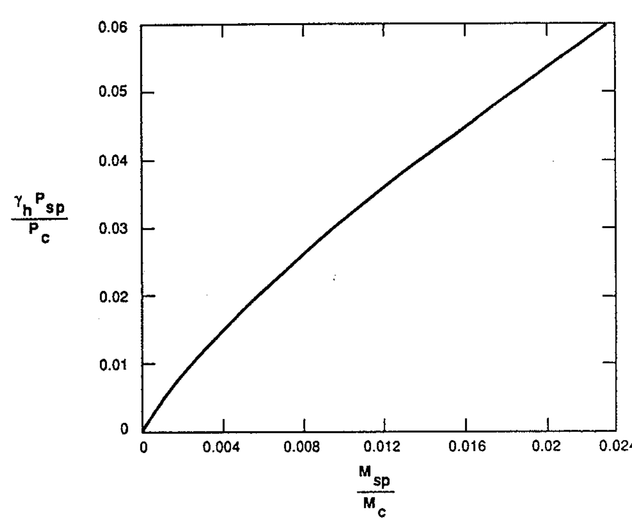


Figure 5.9. Lateral load versus moment for fixed-head piles in clay. (After Evans and Duncan, 1982)

fixed-head piles in sand and clay. These charts show the variation of M_{sp}/M_c with P_{sp}/P_c, where M_{sp} = maximum moment in a single pile and M_c = characteristic moment.

Using these charts, the bending moment in a laterally loaded pile can be estimated as follows:

1. Select a pile section of width (or diameter) D, Young's modulus E_p, and moment of inertia I_p.
2. Estimate the average undrained shear strength, S_u, for clays, or the average angle of internal friction, ϕ', for sands. The behavior is governed by the soil close to the ground surface. The properties (S_u for clays, ϕ' and unit weight, γ', for sands) should be averaged over a depth extending about eight pile diameters below the elevation of the pile top. Buoyant unit weights are used below the water table.
3. Determine the characteristic load, P_c, using Eq. 5.2.1.1.1 for clay or 5.2.1.1.2 for sand.
4. Calculate the factored lateral load, $\gamma_h P_{sp}$ and the value of the load ratio $(\gamma_h P_{sp})/P_c$; γ_h is the lateral load factor.
5. Use Figure 5.8 for fixed-head piles in sand or Figure 5.9 for fixed-head piles in clay to determine the value of M_{sp}/M_c.
6. Determine the characteristic moment, M_c, which is defined by the following equations:

For clay

$$M_c = 3.86\, D^3\, (E_p R_I)\, (S_u/E_p R_I)^{0.46} \tag{5.3.1.1}$$

For sand

$$M_c = 1.33\, D^3\, (E_p R_I)\, (\gamma' D \phi' K_p/E_p R_I)^{0.4} \tag{5.3.1.2}$$

where R_I, K_p, and ϕ' are as defined previously.

7. Calculate $M_{sp} = M_c\,(M_{sp}/M_c)$.

This procedure has been used to develop lateral load–moment curves for some commonly used pile sections. Charts for prestressed concrete and steel-H piles in clay and sand are shown in Figures 5.4 through 5.7. For these piles and soil conditions, bending moments can be estimated directly using the charts. For example, a lateral load of 10 kip acting on a 12 in. by 12 in. prestressed concrete pile driven in clay with an undrained shear strength of 1 ksf will induce a bending moment of 400 kip-in.

5.3.2 Estimation of Bending Moments in Piles Within Pile Groups

As discussed previously, the deflection of any pile in a group causes deflection of the surrounding soil and piles, thus leading to larger deflection for the pile group than for single piles subjected to the same load per pile. The bending moment in a pile within a pile group will consequently be larger than that in a single pile subjected to the same loading. This is because the interaction effects, by causing more deflection, also increase the bending moment in the piles.

Brown et al. (1987, 1988) found that the maximum bending moment in a group of free-head piles occurs in the leading row (or front row) of piles. However, current theories on lateral loading of groups of piles are not able to predict this behavior. A semiempirical procedure that provides a reasonable approximation of the maximum bending moment in the leading row of a group of piles has been developed using the theories of Focht and Koch (1973) and has been confirmed by comparing with field load tests. The increase in moment due to group interaction was studied for a large number of cases by first estimating the pile group deflection using the theory of Focht and Koch (1973), and then "softening" the soil (reduce S_u for clays or ϕ' for sands) until the single pile deflection (calculated using the Evans and Duncan approach) matched the lateral deflection of the pile group. Through this study, the following empirical equation was developed (the equation relates the maximum bending moment of the most severely loaded pile in the group to the maximum bending moment in a single pile):

$$M_g = [Y_g/Y_{sp}]^n M_{sp} \tag{5.3.2.1}$$

where M_{sp} = maximum bending moment in a single fixed-head pile subjected to a lateral load, P_{sp}, calculated using the procedure described in Section 5.3.1; M_g = maximum bending moment in a pile within a pile group; Y_{sp} = lateral deflection of a single fixed-head pile subjected to a lateral load, P_{sp} estimated using the procedure described in Section 5.2.1.1; Y_g = lateral group deflection estimated using Eq. 5.2.1.2.1;

$$n = \frac{\gamma_h P_{sp}}{150 P_N} + 0.25 \textit{ for clay} \tag{5.3.2.2}$$

$$n = \frac{\gamma_h P_{sp}}{300 P_N} + 0.3 \textit{ for sand} \tag{5.3.2.3}$$

P_N is as defined previously in Eqs. 5.2.1.2.2 and 5.2.1.2.3 and γ_h is the load factor for the lateral load.

5.3.3 Structural Capacity of Piles Subjected to Axial Loads and Bending

The structural capacity of a pile is dependent on both moment and axial load. An axial load–moment interaction diagram is an envelope of the combinations of moment and axial load that would cause failure in the pile.

Normalized load–moment interaction diagrams for various types of piles are shown in Figures 5.10 through 5.14. The factored axial load, $\Sigma\gamma_i P_i$ has been normalized by dividing by the factored nominal axial capacity, $\phi_a P_n$. Similarly, the factored bending moment ($\gamma_m M$) has been normalized by dividing by the factored nominal moment capacity, $\phi_m M_n$. The γ-factors account for uncertainties in the loads and moments, and the ϕ-factors account for uncertainties in the structural capacity. Methods of estimating the structural capacities of piles are given in Appendix 2.

The procedure for checking the structural adequacy of piles using the normalized load–moment interaction curves is as follows:

1. Estimate the axial load per pile and calculate the combined axial load effect, $\Sigma\gamma_i P_i$.
2. Determine the nominal axial structural capacity of the pile, P_n. Formulas for calculating the nominal axial structural capacity of piles can be found in Appendix 2.
3. Determine the performance factor for the nominal axial structural capacity, ϕ_a, from Table 5.1 and calculate $\phi_a P_n$.

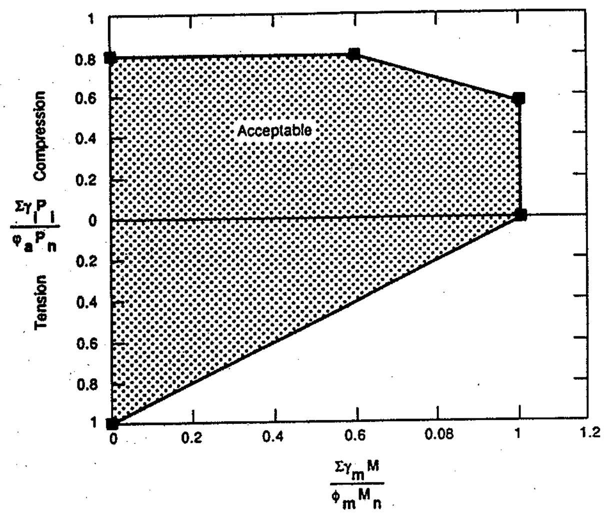


Figure 5.10. Normalized load–moment interaction curve for prestressed concrete piles.

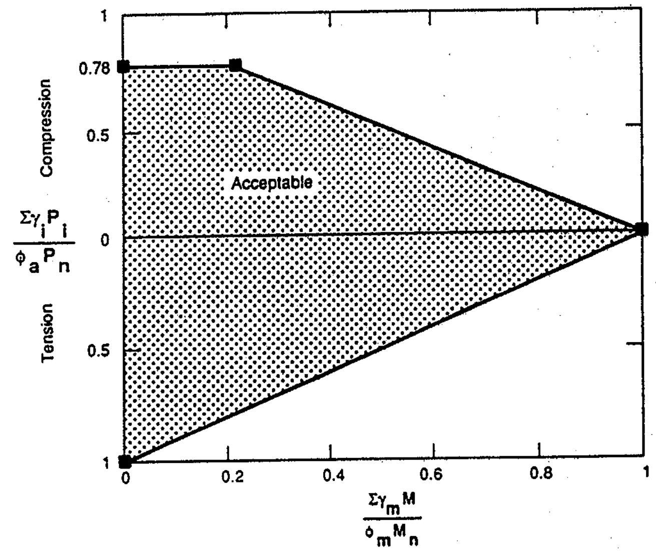


Figure 5.12. Normalized load–moment interaction curve for steel-H piles.

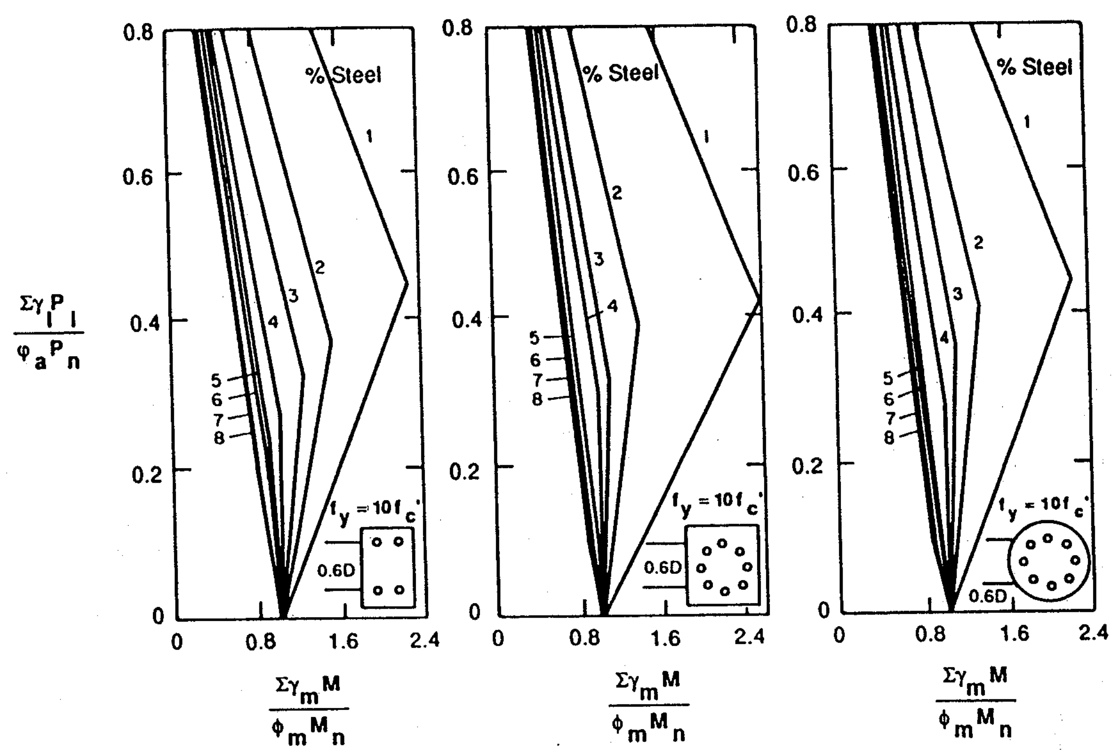


Figure 5.11. Normalized load–moment interaction curves for precast concrete piles.

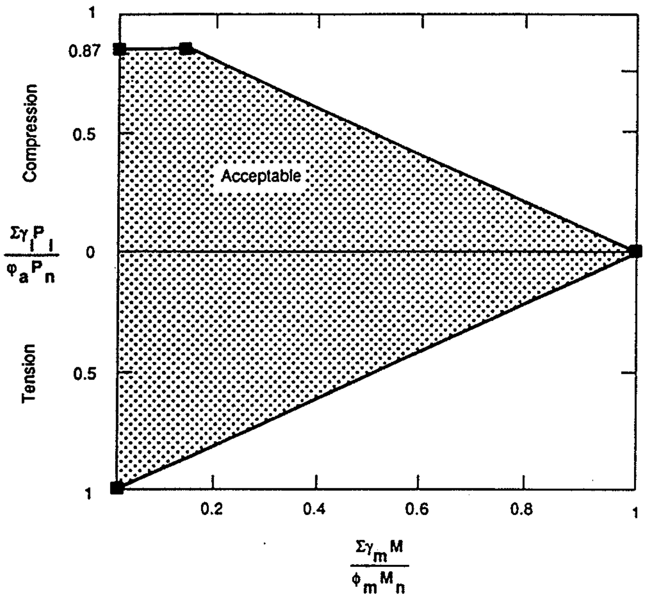


Figure 5.13. Normalized load–moment interaction curve for steel pipe piles.

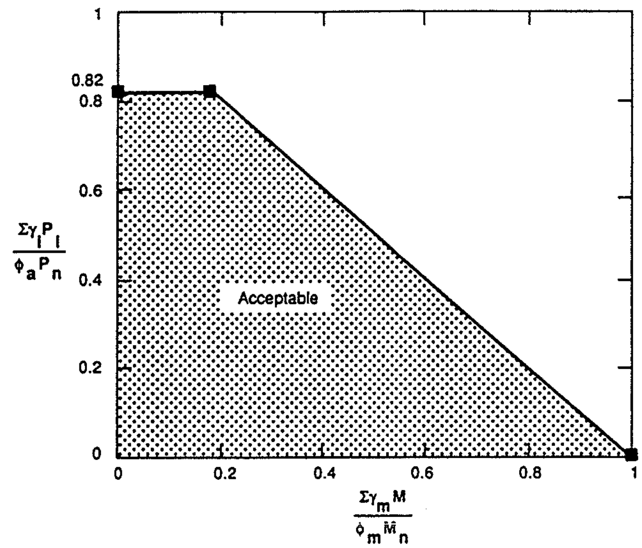


Figure 5.14. Normalized load–moment interaction curve for timber piles.

Table 5.1. Summary of performance factors for the nominal axial structural capacity of piles.

PILE TYPE	PERFORMANCE FACTOR, ϕ_a
Prestressed Concrete Piles	0.75 for spiral columns 0.70 for tied columns
Precast Concrete Piles	0.75 for spiral columns 0.70 for tied columns
Steel H-Piles	0.85
Steel Pipe Piles	0.85
Timber Piles	1.20*

* Davisson et al. (1983) stated that the minimum factor of safety for the structural capacity of piles in axial compression is 1.25. The performance factor is greater than unity because the average load factor for vertical loads (dead and live loads) is greater than the factor of safety.

Table 5.2. Summary of performance factors for the nominal moment capacity of piles.

PILE TYPE	PERFORMANCE FACTOR, ϕ_m
Prestressed Concrete Piles	0.9
Precast Concrete Piles	0.9
Steel H-Piles	0.9
Steel Pipe Piles	0.9
Timber Piles	0.9*

* Davisson et al. (1983) stated that the minimum factor of safety for bending in timber piles is 1.40. The performance factor is obtained by dividing the load factor for lateral loads (= 1.3) by the factor of safety (= 1.4).

4. Calculate the factored design bending moment, $\gamma_m M$, in the pile using factored loads.
5. Estimate the nominal structural moment capacity of the pile, M_n. Formulas to estimate this quantity for piles are given in Appendix 2.
6. Determine the performance factor for the nominal structural moment capacity, ϕ_m, from Table 5.2 and calculate $\phi_m M_n$.
7. Determine the ratios $\Sigma\gamma_i P_i/\phi_a P_n$ and $\Sigma\gamma_m M/\phi_m M_n$, and locate a point at these coordinate values on the normalized load–moment interaction diagram. If the point falls on or close to the interaction curve and inside the area enveloped by the interaction curve and the two axes, the pile chosen is adequate. If it falls outside this region, a larger pile is needed. Steps 2 through 7 should be repeated until the point falls inside and close to the interaction curve. If the point falls inside the region but far away from the interaction curve (e.g., near the origin), the pile chosen has more capacity than required. Steps 2 to 7 can be repeated for smaller pile sections to achieve greater design economy.

5.4 DESIGN EXAMPLES

The design procedures discussed in the previous sections are demonstrated in the following example.

EXAMPLE 5.1

Using the charts developed by Evans and Duncan (1982), determine the lateral deflection of the pile foundation shown in Fig. E5.1 and the structural adequacy of the piles.

(i) Lateral Deflection

Single Pile Deflection

$$P_{SF} = 40/4 = 10 \text{ kips per pile.}$$

$$\gamma_{SF}P_{SF} = 1 \times 10 = 10 \text{ kips per pile for } \gamma_{SF} = 1$$

$$E_p = 4300 \text{ ksi}$$

$$D = 12 \text{ in.}$$

$$I_p = 1728 \text{ in}^4.$$

$$I_{solid} = \pi(12)^4/64 = 1018 \text{ in}^4$$

$$R_I = 1728/1018 = 1.7$$

From Equation (5.2.1.1.1),

$$P_c = \frac{7.34\ (12)^2\ (4300)\ (1.7)}{[144\ (4300)\ (1.7)]^{0.683}}$$

$$= 595 \text{ kips}$$

$$\gamma_{SF}P_{SF}/P_c = 10/595 = 0.017 \qquad \text{for } \gamma_{SF} = 1 \text{ (i.e. unfactored)}$$

From Fig. 5.3,

$$Y_{sp}/D = 0.008$$

$$Y_{sp} = 0.008\ (12) = 0.1 \text{ in.}$$

Similarly, using Fig. 5.6, $Y_{sp} = 0.1$ in.

Pile Group Deflection

Assume that the tolerable lateral deflection is 0.5 in.

From Equation 5.2.1.2.3,

$$P_N = S_u D^2 = (1/144)\ (12)^2 = 1 \text{ kip}$$

From Equation 5.2.1.2.1, the group lateral deflection corresponding to a lateral load per pile of 10 kips may be calculated as follows:

$$Y_g = \frac{16 + 4}{5.5\sqrt{\dfrac{36}{12} + \dfrac{10}{3(1)}}}\ (0.1)$$

$$= 1.445(0.1)$$

$$= 0.14 \text{ in.} < 0.5 \text{ in.}$$

(ii) Structural Capacity

Design bending moment, M_{sp} in a single pile is estimated as follows:

From Fig. 5.10,

$$M_{sp}/M_c = 0.0048 \qquad \text{for } P_{sp}/P_c = 0.017$$

From Equation (5.3.1.1)

$$M_c = \frac{3.86\ (12)^3\ (4300)\ (1.7)}{[144\ (4300)\ (1.7)]^{0.46}}$$

$$= 82\ 760 \text{ kip-in}$$

$$M_{sp} = 0.0048\ (82\ 760)$$

$$= 397 \text{ kip-in.}$$

Similarly, using Fig. 5.6 directly, $M_{sp} = 400$ kip-in.

Design bending moment in the pile group is estimated as follows:

Using Equation (5.3.2.1),

$$Y_g/Y_{sp} = 1.445$$

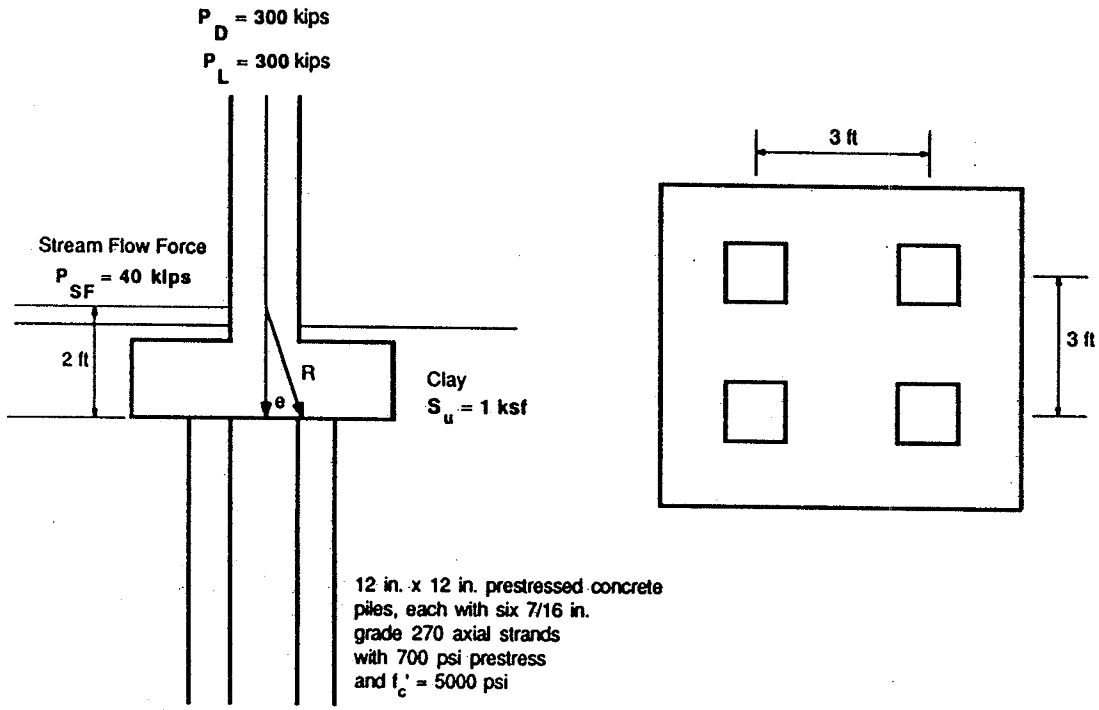


Figure E5.1 Figure for example problem 5.1

$$\gamma_{SF}P_{SF} = 1.3 \times 10 = 13 \text{ kips/pile for } \gamma_{SF} = 1.3.$$

$$n = \frac{13}{150\ (1)} + 0.25$$

$$= 0.337$$

$$M_g = [1.445]^{0.337}\ (397)$$

$$= 1.13\ (397)$$

$$= 449 \text{ kip-in/pile}$$

The structural adequacy of the piles carrying the maximum axial load (i.e. the two piles on the right) is checked as follows:

AASHTO's load combination I consists of dead load, maximum live load and stream flow pressure acting on the pile group.

$$\Sigma\gamma_i P_i = \gamma_D P_D + \gamma_L P_L$$

$$P_D = 300 \text{ kips}$$

$$P_L = 300 \text{ kips}$$

$$\Sigma\gamma_i P_i = (1.3)(300) + (2.17)(300)$$

$$= 1041 \text{ kips}$$

$$\gamma_{SF}P_{SF} = (1.3)(40) \qquad \text{for } \gamma_{SF} = 1.3$$

$$= 52 \text{ kips}$$

Eccentricity of R, $e = (52)(2\text{ft.})/1041$

$$= 0.1 \text{ ft.}$$

$$\Sigma x^2 = (4)(1.5)^2$$

$$= 9 \text{ ft}^2$$

From equation 4.1.1.3, the most heavily loaded pile supports a factored axial load of:

$$P_{x,y} = (1041)[1/4 + (0.1)(1.5)/9]$$

$$= 260.3 + 17.4$$

$$= 278 \text{ kips/pile.}$$

$$P_n = (0.85f_c' - 0.6f_{pre})A_c \qquad \text{from Table A2.1}$$

$$= [0.85\ (5) - 0.6\ (0.7)]\ 144$$

$$= 552 \text{ kips/pile}$$

From Table 5.1, $\phi_a = 0.7$

$$\phi_a P_n = 0.7\ (552)$$

$$= 386 \text{ kips/pile}$$

$$\frac{P_{x,y}}{\phi_a P_n} = \frac{278}{386}$$

$$= 0.72$$

$$\gamma_m M = 1\ (449) \qquad \text{for } \gamma_m = 1$$

$$= 449 \text{ kip-in/pile}$$

Nominal area for 7/16 in. prestressing tendon $A_{ps} = 0.115 \text{ in}^2$

$$M_n = 0.37DA_{ps}f_{pu} \qquad \text{from Table A2.4}$$

$$= 0.37\ (12)\ (6)\ (0.115)\ (270)$$

$$= 827 \text{ kip-in/pile}$$

From Table 5.2, $\phi_m = 0.9$

$$\phi_m M_n = 0.9\ (827)$$

$$= 744 \text{ kip-in/pile}$$

$$\frac{\gamma_m M}{\phi_m M_n} = \frac{449}{744}$$

$$= 0.60$$

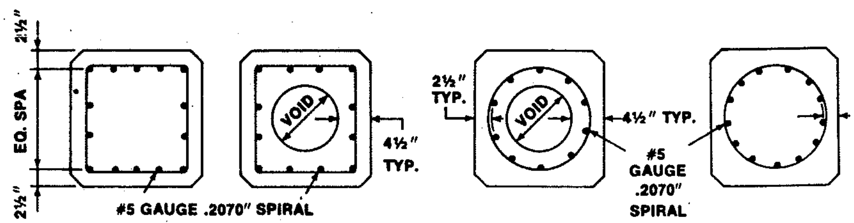
The point (0.60, 0.72) plots inside the interaction diagram of Fig. 5.11. Thus the structural capacity is adequate.

APPENDIX 1

SECTION PROPERTIES OF PRESTRESSED CONCRETE, STEEL-H AND PIPE PILES

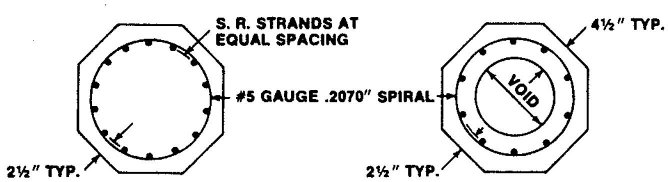
TABLE A1.1 SECTION PROPERTIES OF PRESTRESSED CONCRETE PILES (After Hunt, 1979)

SQUARE PRESTRESSED PILES



Pile Size (1) in.	Area, A_c in.^{2}	Approx. Weight (2) lb./lin. ft.	Min. Effective Prestress Force (3) kips	Strands Per Pile Diameter (4) 7/16-in.	Strands Per Pile Diameter (4) 1/2-in.	Section Modulus in.^{3}	Perimeter in.	Design Bearing Capacity tons (5) Concrete Strength, psi 5000	Design Bearing Capacity tons (5) Concrete Strength, psi 6000
10	100	105	70	4	4	167	40	73	90
12	144	150	101	6	5	288	48	105	129
14	196	205	137	8	6	457	56	143	176
16	256	265	179	11	8	683	64	187	229
18	324	335	227	13	10	972	72	237	290
20	400	415	280	16	12	1333	80	292	358
22	484	505	339	20	15	1775	88	354	433
24	576	600	403	23	18	2304	96	421	516
20 HC	305	320	214	13	10	1261	80	223	273
22 HC	351	365	246	14	11	1647	88	256	314
24 HC	399	415	279	16	12	2097	96	291	357

OCTAGONAL PRESTRESSED PILES



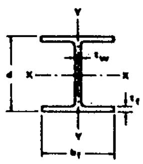
Pile Size (1) (diam.) in.	Area, A_c in.^{2}	Approx. Weight (2) lb./lin. ft.	Min. Effective Prestress Force (3) kips	Strands Per Pile Diameter (4) 7/16-in.	Strands Per Pile Diameter (4) 1/2-in.	Section Modulus in.^{3}	Perimeter in.	Design Bearing Capacity tons (5) Concrete Strength, psi 5000	Design Bearing Capacity tons (5) Concrete Strength, psi 6000
10	83	85	58	4	4	109	33	61	74
12	119	125	83	5	4	189	40	87	107
14	162	170	113	7	5	300	46	118	145
16	212	220	148	9	7	448	53	155	190
18	268	280	188	11	8	638	60	196	240
20	331	345	232	14	10	876	66	242	296
22	401	420	281	16	12	1166	73	293	359
24	477	495	334	19	15	1513	80	348	427
20 HC	236	245	165	10	8	804	66	172	211
22 HC	268	280	188	11	8	1038	73	196	240
24 HC	300	315	210	12	9	1306	80	219	269

NOTES: (For both square and octagonal piles)

NOTES: (For both square and octagonal piles)

(1) Voids in 20", 22" and 24" diameter hollow-core (HC) piles are 11", 13" and 15" diameter, respectively, providing a minimum 4½" wall thickness.
(2) Weights based on 150 lb. per cubic foot of regular concrete.
(3) Minimum effective prestress force based on unit prestress of 700 psi after losses.
(4) Based on 7/16 and 1/2 in. 270 Grade stress-relieved strand with an ultimate strength of 31,000 and 41,300 lb. respectively. If 250 Grade stress-relieved or low-relaxation strand is used, the number of strands per pile should be modified in conformance with strand manufacturer's design information to give the force listed in the fourth column.
(5) Design bearing capacity based on 5,000 and 6,000 psi concrete and an allowable unit stress on the full section of 0.33 f'_c - 0.27 f_{ce}, where f_{ce} is the concrete stress in the pile due to prestressing, after all losses. These bearing capacity values may be increased if higher strength concrete is used.
(6) Circular piles of the same diameter may be used with Engineer's approval.
(7) Details are from Prestressed Concrete Institute standard, which gives more information.

TABLE A1.2 SECTION PROPERTIES OF STEEL - H PILES
(After Pile Buck Annual, 1988)



STANDARD H-PILES

DIMENSIONS AND PROPERTIES FOR DESIGNING

Designation	Area A	Depth d	Flange Width b_f	Flange Thickness t_f	Web Thickness t_w	Section Properties Axis X-X I	Axis X-X S	Axis X-X r	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r
	$in.^2$	in.	in.	in.	in.	$in.^4$	$in.^3$	in.	$in.^4$	$in.^3$	in.
HP14 x 117	34.4	14.21	14.885	0.805	0.805	1220	172	5.96	443	59.5	3.59
x 102	30.0	14.01	14.785	0.705	0.705	1050	150	5.92	380	51.4	3.56
x 89	26.1	13.83	14.695	0.615	0.615	904	131	5.88	326	44.3	3.53
x 73	21.4	13.61	14.585	0.505	0.505	729	107	5.84	261	35.8	3.49
HP13 x 100	29.4	13.15	13.205	0.765	0.765	886	135	5.49	294	44.5	3.16
x 87	25.5	12.95	13 105	0.665	0.665	755	117	5.45	250	38.1	3.13
x 73	21.6	12.75	13.005	0.565	0.565	630	98.8	5.40	207	31.9	3.10
x 60	17.5	12.54	12.900	0.460	0.460	503	80 3	5.36	165	25.5	3.07
HP12 x 84	24.6	12.28	12.295	0.685	0.685	650	106	5.14	213	34.6	2.94
x 74	21.8	12.13	12.215	0.610	0.605	569	93.8	5.11	186	30.4	2.92
x 63	18.4	11.94	12.125	0.515	0.515	472	79.1	5.06	153	25.3	2.88
x 53	15.5	11.78	12.045	0.435	0.435	393	66.8	5.03	127	21.1	2.86
HP10 x 57	16.8	9.99	10.225	0.565	0.565	294	58.8	4.18	101	19.7	2.45
x 42	12.4	9.70	10.075	0.420	0.415	210	43.4	4.13	71.7	14.2	2.41
HP8 x 36	10.6	8.02	8.155	0.445	0.445	119	29.8	3.36	40.3	9.88	1.95

Designation	Nominal Weight per Ft	r_T	$\frac{d}{A_f}$	Compact Section Criteria $\frac{b_f}{2t_f}$	F'_y	$\frac{d}{t_w}$	F''y	F'''y	Torsional Constant J	Warping Constant C_w	Plastic Modulus Z_x	Plastic Modulus Z_y	Surface Area
	lb.	in.			ksi		ksi	ksi	$in.^4$	$in.^6$	$in.^3$	$in.^3$	$ft.^2/ft.$
HP14 x 117	117	4.00	1.19	9.3	49 4	17.7	***	***	8.02	19900	194	91.4	7.11
x 102	102	3.97	1.34	10.5	38 4	19.9	***	***	5.40	16600	169	78.8	7.06
x 89	89	3.94	1.53	11.9	29 6	22 5	***	***	3.60	14200	146	67.7	7.01
x 73	73	3.90	1.85	14.4	20 3	27.0	***	***	2.01	11200	118	54.6	6.96
HP13 x 100	100	3.54	1.30	8.6	56 7	17.2	***	***	6.25	11300	153	66.6	6.38
x 87	87	3.51	1.49	9.9	43 5	19.5	***	***	4.12	9430	131	58.5	6.33
x 73	73	3.47	1.74	11.5	31 9	22.6	***	***	2.54	7680	110	48.3	6.28
x 60	60	3.43	2.11	14.0	21 5	27.3	***	***	1.39	6020	89.0	39.0	6.23
HP12 x 84	84	3.29	1.46	9.0	52 5	17.9	***	***	4.24	7160	120	53.2	5.94
x 74	74	3.26	1.63	10.0	42 1	20.0	***	***	2.98	6160	105	46.6	5.91
x 63	63	3.23	1.91	11.8	30 5	23.2	***	***	1.83	4990	88.3	36.7	5.86
x 53	53	3.20	2.25	13.8	22 0	27.1	***	***	1.12	4080	74.0	32.2	5.82
HP10 x 57	57	2.74	1.73	9.1	51 6	17.7	***	***	1.97	2240	66.5	30.3	4.91
x 42	42	2.69	2.29	12.0	29 4	23.4	***	***	0.81	1540	48.3	21.8	4.83
HP8 x 36	36	2.18	2.21	9.2	50.3	18.0	***	***	0.77	578	33.6	15.2	3.92

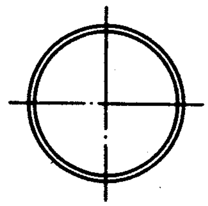
Normal Material Specifications: ASTM A36, ASTM A572 and ASTM A690

★★★ The theoretical maximum yield stress exceeds 65 ksi.

Structural properties are given for use when H-piles are utilized as rakers, wales or as other structural members. See "Manual of Steel Construction," American Institute of Steel Construction, for definitions of terms.

TABLE A1.3

The following charts list the dimensions and physical properties of some of the more commonly used sizes of Pipe Piling.



PIPE PILES

Dimensions and Properties for Designing

(After Pile Buck Inc., 1988)

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Foot	Section Properties			Area of Exterior Surface	Inside Cross-Sectional Area	Inside Volume	External Collapse Index
				I	S	r				
in.	in.	in^2	lb.	in.^{4}	in.^{3}	in.	ft^2/ft	in.^{2}	yd^3/ft	*
PP10	.109	3.39	11.51	41.4	8.28	3.50	2.62	75.2	.0193	62
	.120	3.72	12.66	45.5	9.09	3.49	2.62	74.8	.0192	83
	.134	4.15	14.12	50.5	10.1	3.49	2.62	74.4	.0191	116
	.141	4.37	14.85	53.1	10.6	3.49	2.62	74.2	.0191	135
	.150	4.64	15.78	56.3	11.3	3.48	2.62	73.9	.0190	163
	.164	5.07	17.23	61.3	12.3	3.48	2.62	73.5	.0189	214
	.172	5.31	18.05	64.1	12.8	3.48	2.62	73.2	.0188	247
	.179	5.52	18.78	66.6	13.3	3.47	2.62	73.0	.0188	279
	.188	5.80	19.70	69.8	14.0	3.47	2.62	72.7	.0187	324
	.203	6.25	21.24	75.0	15.0	3.46	2.62	72.3	.0186	409
	.219	6.73	22.88	80.5	16.1	3.46	2.62	71.8	.0185	515
	.230	7.06	24.00	84.3	16.9	3.46	2.62	71.5	.0184	588
	.250	7.66	26.03	91.1	18.2	3.45	2.62	70.9	.0182	719
PP10-3/4	.109	3.64	12.39	51.6	9.60	3.76	2.81	87.1	.0224	50
	.120	4.01	13.62	56.6	10.5	3.76	2.81	86.8	.0223	67
	.125	4.17	14.18	58.9	11.0	3.76	2.81	86.6	.0223	76
	.134	4.47	15.19	63.0	11.7	3.75	2.81	86.3	.0222	93
	.141	4.70	15.98	66.1	12.3	3.75	2.81	86.1	.0221	109
	.150	5.00	16.98	70.2	13.1	3.75	2.81	85.8	.0221	131
	.156	5.19	17.65	72.9	13.6	3.75	2.81	85.6	.0220	148
	.164	5.45	18.54	76.4	14.2	3.74	2.81	85.3	.0219	172
	.172	5.72	19.43	80.0	14.9	3.74	2.81	85.0	.0219	199
	.179	5.94	20.21	83.1	15.5	3.74	2.81	84.8	.0218	224
	.188	6.24	21.21	87.0	16.2	3.73	2.81	84.5	.0217	260
	.203	6.73	22.87	93.6	17.4	3.73	2.81	84.0	.0216	328
	.219	7.25	24.63	100	18.7	3.72	2.81	83.5	.0215	414
	.230	7.60	25.84	105	19.6	3.72	2.81	83.2	.0214	480
	.250	8.25	28.04	114	21.2	3.71	2.81	82.5	.0212	605
	.279	9.18	31.20	126	23.4	3.70	2.81	81.6	.0210	781
	.307	10.1	34.24	137	25.6	3.69	2.81	80.7	.0208	951
	.344	11.2	38.23	152	28.4	3.68	2.81	79.5	.0205	1,180
	.365	11.9	40.48	161	29.9	3.67	2.81	78.9	.0203	1,320
	.438	14.2	48.24	189	35.2	3.65	2.81	76.6	.0197	1,890
	.500	16.1	54.74	212	39.4	3.63	2.81	74.7	.0192	2,380

Material Specifications - ASTM A252

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ration and is for general guidance only. The higher the number, the greater is the resistance to collapse.

TABLE A1.3 CONT'D

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Foot	Section Properties I	Section Properties S	Section Properties r	Area of Exterior Surface	Inside Cross-Sectional Area	Inside Volume	External Collapse Index
in.	in.	in^2	lb.	$in.^4$	$in.^3$	in.	ft^2/ft	$in.^2$	yd^3/ft	*
PP12	.134	5.00	16.98	87.9	14.7	4.20	3.14	108	.0278	67
	.141	5.25	17.86	92.4	15.4	4.19	3.14	108	.0277	78
	.150	5.58	18.98	98.0	16.3	4.19	3.14	108	.0277	94
	.164	6.10	20.73	107	17.8	4.19	3.14	107	.0275	123
	.172	6.39	21.73	112	18.6	4.18	3.14	107	.0274	142
	.179	6.65	22.60	116	19.4	4.18	3.14	106	.0274	161
	.188	6.98	23.72	122	20.3	4.18	3.14	106	.0273	186
	.203	7.52	25.58	131	21.8	4.17	3.14	106	.0272	235
	.219	8.11	27.55	141	23.4	4.17	3.14	105	.0270	296
	.230	8.50	28.91	147	24.6	4.16	3.14	105	.0269	344
	.250	9.23	31.37	159	26.6	4.16	3.14	104	.0267	443
	.281	10.3	35.17	178	29.6	4.14	3.14	103	.0264	616
	.312	11.5	38.95	196	32.6	4.13	3.14	102	.0261	784
PP12-3/4	.109	4.33	14.72	86.5	13.6	4.47	3.34	123	.0317	30
	.125	4.96	16.85	98.8	15.5	4.46	3.34	123	.0316	45
	.134	5.31	18.06	106	16.6	4.46	3.34	122	.0315	56
	.141	5.59	18.99	111	17.4	4.46	3.34	122	.0314	65
	.150	5.94	20.19	118	18.5	4.46	3.34	122	.0313	78
	.156	6.17	20.98	122	19.2	4.45	3.34	122	.0313	88
	.164	6.48	22.04	128	20.1	4.45	3.34	121	.0312	103
	.172	6.80	23.11	134	21.1	4.45	3.34	121	.0311	118
	.179	7.07	24.03	140	21.9	4.45	3.34	121	.0310	134
	.188	7.42	25.22	146	23.0	4.44	3.34	120	.0309	155
	.203	8.00	27.20	158	24.7	4.44	3.34	120	.0308	196
	.219	8.62	29.31	169	26.6	4.43	3.34	119	.0306	246
	.230	9.05	30.75	177	27.8	4.43	3.34	119	.0305	286
	.250	9.82	33.38	192	30.1	4.42	3.34	118	.0303	368
	.281	11.0	37.42	214	33.6	4.41	3.34	117	.0300	526
	.312	12.2	41.45	236	37.0	4.40	3.34	115	.0297	684
	.330	12.9	43.77	248	39.0	4.39	3.34	115	.0295	776
	.344	13.4	45.58	258	40.5	4.39	3.34	114	.0294	848
	.375	14.6	49.56	279	43.8	4.38	3.34	113	.0291	1,010
	.406	15.7	53.52	300	47.1	4.37	3.34	112	.0288	1,170
	.438	16.9	57.59	321	50.4	4.36	3.34	111	.0285	1,350
	.500	19.2	65.42	362	56.7	4.33	3.34	108	.0279	1,760
PP14	.134	5.84	19.84	140	20.0	4.90	3.67	148	.0381	42
	.141	6.14	20.87	147	21.1	4.90	3.67	148	.0380	49
	.150	6.53	22.19	157	22.4	4.90	3.67	147	.0379	59
	.156	6.78	23.07	163	23.2	4.89	3.67	147	.0378	66
	.164	7.13	24.23	171	24.4	4.89	3.67	147	.0378	77
	.172	7.47	25.40	179	25.5	4.89	3.67	146	.0377	89
	.179	7.77	26.42	186	26.5	4.89	3.67	146	.0376	101
	.188	8.16	27.73	195	27.8	4.88	3.67	146	.0375	117
	.203	8.80	29.91	209	29.9	4.88	3.67	145	.0373	147
	.210	9.10	30.93	216	30.9	4.88	3.67	145	.0373	163
	.219	9.48	32.23	225	32.2	4.87	3.67	144	.0372	185
	.230	9.95	33.82	236	33.7	4.87	3.67	144	.0370	215
	.250	10.8	36.71	255	36.5	4.86	3.67	143	.0368	277
	.281	12.1	41.17	285	40.7	4.85	3.67	142	.0365	395
	.312	13.4	45.61	314	44.9	4.84	3.67	141	.0361	542
	.344	14.8	50.17	344	49.2	4.83	3.67	139	.0358	691
	.375	16.1	54.57	373	53.3	4.82	3.67	138	.0355	835
	.438	18.7	63.44	429	61.4	4.80	3.67	135	.0348	1,130
	.469	19.9	67.78	457	65.3	4.79	3.67	134	.0345	1,280
	.500	21.2	72.09	484	69.1	4.78	3.67	133	.0341	1,460

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ration and is for general guidance only. The higher the number, the greater is the resistance to collapse.

Material Specifications - ASTM A252

TABLE A1.3 CONT'D

Designation and Outside Diameter	Wall Thickness	Area A	Weight per Foot	Section Properties I	S	r	Area of Exterior Surface	Inside Cross-Sectional Area	Inside Volume	External Collapse Index
in.	in.	in^2	lb.	in.^{4}	in.^{3}	in.	ft^2/ft	in.^{2}	yd^3/ft	*
PP16	.134	6.68	22.71	210	26.3	5.61	4.19	194	.0500	28
	.141	7.02	23.88	221	27.6	5.61	4.19	194	.0499	33
	.150	7.47	25.39	235	29.3	5.60	4.19	194	.0498	39
	.156	7.76	26.40	244	30.5	5.60	4.19	193	.0497	44
	.164	8.16	27.74	256	32.0	5.60	4.19	193	.0496	52
	.172	8.55	29.08	268	33.5	5.60	4.19	193	.0495	60
	.179	8.90	30.25	278	34.8	5.59	4.19	192	.0494	67
	.188	9.34	31.75	292	36.5	5.59	4.19	192	.0493	78
	.203	10.1	34.25	314	39.3	5.59	4.19	191	.0491	98
	.219	10.9	36.91	338	42.3	5.58	4.19	190	.0489	124
	.230	11.4	38.74	354	44.3	5.58	4.19	190	.0488	144
	.250	12.4	42.05	384	48.0	5.57	4.19	189	.0485	185
	.281	13.9	47.17	429	53.6	5.56	4.19	187	.0481	264
	.312	15.4	52.27	473	59.2	5.55	4.19	186	.0478	362
	.344	16.9	57.52	519	64.8	5.54	4.19	184	.0474	487
	.375	18.4	62.58	562	70.3	5.53	4.19	183	.0470	617
	.438	21.4	72.80	649	81.1	5.50	4.19	180	.0462	874
	.469	22.9	77.79	691	86.3	5.49	4.19	178	.0458	1,000
	.500	24.3	82.77	732	91.5	5.48	4.19	177	.0455	1,130

* The External Collapse Index is a non-dimensional function of the diameter to wall thickness ration and is for general guidance only. The higher the number, the greater is the resistance to collapse.

Material Specifications - ASTM A252

APPENDIX 2

AXIAL AND MOMENT CAPACITIES OF PILES

AXIAL AND MOMENT CAPACITIES OF PILES

In this section, methods of estimating the structural axial and moment capacities of piles are discussed.

PRESTRESSED CONCRETE PILES

1) [P_n]

The value of the axial structural capacity, [P_n], can be estimated using the expressions in Table A2.1 for a pile of known concrete compressive strength, effective prestress and cross-sectional area. Section properties of some common prestressed concrete piles are found in Appendix 1.

2) [M_n]

The nominal moment capacity, [M_n] varies depending on (i) the geometry of the section, (ii) the concrete compressive strength and (iii) the level of prestress (which can be thought of as a type of axial load). However for pile lengths between 40 and 140 ft, the tendons are stressed typically between 700 and 1200 psi (Pile Buck Annual, 1988). Typical concrete compressive strengths for prestressed concrete piles vary between 5000 and 8000 psi. The PCI Design Handbook (1985) contains expressions for the ultimate moment capacity that assume a certain prestress level and concrete strength. These have been tabulated for different shapes in Table A2.4. A comparison below shows that the maximum difference between the ultimate moment capacities from a pile manufacturer (Santa Fe Pomeroy) and the PCI equations is not more than about 20% for the following square sections.

	CALCULATED USING PCI EQUATION IN TABLE A2.4 WITH ϕ_m = 0.9 (kip-ft)	FROM SANTA FE POMEROY CHARTS (kip-ft)	% DIFFERENCE
10 in square	34	29	17
12 in square	62	55	13
14 in square	96	87	10
16 in square	147	142	4
18 in square	206	198	4

The prestressing tendons in prestressed concrete piles are usually made of Grade 270 steel with a diameter of either 7/16 in. (nominal area = 0.115 in^2.) or 1/2 in. (nominal area = 0.153 in^2.).

PRECAST CONCRETE PILES

1) [P_n]

The axial capacity, [P_n], can be determined from the second equation in Table A2.1. The capacity depends on the compressive strength of concrete, the yield strength of steel, the area of concrete and the area of steel.

2) [M_n]

The nominal moment capacity, [M_n] is a function of (i) the shape of the cross section, (ii) the percentage of steel, (iii) the reinforcement layout, (iv) the concrete compressive

strength and (v) the pile dimensions. No general expression exists for the ultimate moment capacity for precast concrete piles. A table of dimensionless ultimate moment capacities is provided for various precast shapes and percentage area of steel (Table A2.5). The limitation of this table is that the yield strength of steel, f_y, must be approximately 10 times the compressive strength of the concrete, f_c'.

STEEL PILES

The type of steel piles referred to in this section are H piles and pipe piles (without concrete fill). According to Davisson et al. (1983), "open-end pipe piles are seldom used in bridge foundation applications". Pipe sections, with superior column charcteristics, are advantageous when piles have free-standing portions. Otherwise, according to Davisson et al., steel-H piles are preferred because they are more cost-effective. Sectional properties of both steel-H and pipe piles can be found in Appendix 1.

1) $[P_n]$

The axial structural capacity in both steel-H piles and pipe piles is simply the product of the yield stress of steel and the cross-sectional steel area (Table A2.1).

2) $[M_n]$

The nominal moment capacity is the product of the yield strength of steel and its plastic section modulus, Z_p.

TIMBER PILES

In the United States, Southern Pine is usually used in the eastern half of the country while Douglas Fir is more prominent in the western part (Forest Products Laboratory, 1989). Factors that influence timber strength include (i) moisture, (ii) temperature (treatment process), (iii) duration of loading and (iv) wood imperfection features such as knots, slope of grain, shakes, checks and splits (Davisson et al., 1983).

1) $[P_n]$

The axial capacity is the product of the 5% exclusion limit in compression parallel to grain for green clear wood specimens (S_c) obtained from Table A2.2, the area of the pile (A_t) and a factor accounting for the treatment conditioning of the timber pile (k_c) obtained from Table A2.3 (Davisson et al., 1983) i.e.

$$P_n = k_c A_t S_c$$

2) $[M_n]$

Similarly, the nominal moment capacity of wood piles is the product of the 5% exclusion limit for modulus of rupture of green small clear wood specimens (S_b) (Table A2.2), the elastic section modulus of the pile (Z_e) and a treatment conditioning factor (k_b) obtained from Table A2.6 (Davisson et al., 1983).

$$M_n = k_b \, Z_e \, S_b$$

If the pile diameter is greater than 12 inches, the expression above has to be multiplied by $[12/D]^{1/9}$ where D is the diameter in inches.

TABLE A2.1 Expressions for Nominal Axial Structural Capacity, $[P_n]$, of Piles in the Absence of Bending Moments (After PCI, 1985 and Davisson et al., 1983)

PILE TYPE	P_n
PRESTRESSED CONCRETE	$(0.85f_c' - 0.6f_{pre})A_c$
PRECAST CONCRETE	$0.85f_c'A_c + f_yA_y$
STEEL-H PILES	f_yA_y
STEEL PIPE PILES	f_yA_y
TIMBER	$k_cS_cA_t$

f_c' = 28 day concrete cylinder strength
f_y = yield stress of steel
f_{pre} = effective prestress in the concrete
A_c = cross-sectional area of concrete
A_y = cross-sectional area of steel
A_t = cross-sectional area of timber
S_c = 5% exclusion limit in compression parallel to grain for green small clear wood specimens (see Table A2.2)
k_c = factor to account for the treatment condition of the timber pile and where along the pile the ultimate axial load is desired (see Table A2.3)

TABLE A2.2 5% Exclusion Values for Compression Parallel to Grain and Modulus of Rupture for Timber Piles (After Davisson et al., 1983)

		COMPRESSION PARALLEL TO GRAIN (psi) S_c	MODULUS OF RUPTURE (psi) S_b
DOUGLAS FIR	COAST	2577	5499
	INTERIOR WEST	2558	5538
	INTERIOR NORTH	2479	5525
	INTERIOR SOUTH	2308	5290
SOUTHERN PINE	LOBLOLLY	2504	5328
	LONGLEAF	3158	6391
	SHORTLEAF	2599	5515
	SLASH	2923	6838

TABLE A2.3 k_c Factor to Account for the Treatment Condition of Timber Piles when Calculating the Axial Compressive Strength Parallel to Grain (After Davisson et al., 1983)

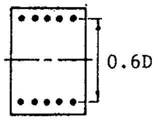
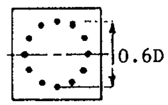
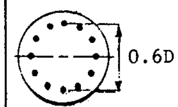
LOCATION	PILE LENGTH	TREATMENT CONDITIONING			
		Untreated or Air-Seasoned	Kiln Dried	Boulton Process	Steamed
Pile Butt	All Lengths	0.534	0.473	0.457	0.396
Pile Tips	≤ 50ft.	0.473	0.427	0.412	0.366
	> 50ft.	0.442	0.396	0.366	0.335

TABLE A2.4 Expressions for the Nominal Moment Capacity, $[M_n]$, of Piles in the Absence of Axial Loads

	M_n
PRESTRESSED CONCRETE (after PCI Design Handbook, 1985)	$0.37DA_{ps}f_{pu}$ - solid square piles $0.32DA_{ps}f_{pu}$ - solid circular and octagonal piles $0.38DA_{ps}f_{pu}$ - hollow square piles $0.34DA_{ps}f_{pu}$ - hollow circular and octagonal piles
PRECAST CONCRETE	See Table A2.5
STEEL H-PILES	f_yZ_p
STEEL PIPE PILES	f_yZ_p
TIMBER PILES	$k_bS_bZ_e$

f_{pu} = ultimate strength of the tendons in prestressed concrete piles
f_y = yield stress of steel
S_b = 5% exclusion value for the modulus of rupture of timber piles (see Table A2.2)
D = pile width or diameter
A_{ps} = nominal area of prestressing tendons
Z_e = elastic section modulus of timber piles
Z_p = plastic section modulus of steel piles
k_b = factor to account for the treatment conditioning of the timber pile and where along the pile the ultimate moment capacity is required (see Table A2.6)

TABLE A2.5 Nominal Moment Capacity, $[M_n]$, for Precast Concrete Piles. Chart Applies to any Value of f_c' and f_y provided $f_y = 10f_c'$.

RATIO OF AREA OF STEEL TO GROSS CROSS-SECTIONAL AREA	$\frac{M_n}{f_c' D A_g}$		
			
0.01	0.043	0.037	0.037
0.02	0.074	0.074	0.067
0.03	0.106	0.102	0.088
0.04	0.137	0.120	0.107
0.05	0.167	0.139	0.126
0.06	0.194	0.157	0.144
0.07	0.226	0.176	0.161
0.08	0.254	0.193	0.176

f_c' = 28 day concrete cylinder strength
f_y = yield stress of steel
D = pile width or diameter
A_g = gross cross-sectional area of concrete

TABLE A2.6 k_b Factor to Account for the Treatment Condition of Timber Piles when Calculating the Nominal Moment Capacity (After Davisson et al., 1983)

LOCATION	PILE LENGTH	TREATMENT CONDITIONING			
		Untreated or Air-Seasoned	Kiln Dried	Boulton Process	Steamed
Pile Butt	All Lengths	0.490	0.448	0.420	0.364
Pile Tips	≤ 50ft	0.448	0.406	0.378	0.336
	> 50ft	0.378	0.350	0.322	0.280

APPENDIX 3

CORRELATIONS FOR ESTIMATING THE FRICTION ANGLE OF SANDS FROM SPT BLOW-COUNTS AND CONE RESISTANCE

CORRELATIONS FOR ESTIMATING THE FRICTION ANGLE OF SANDS FROM SPT BLOW COUNT AND CONE RESISTANCE

The friction angle of sands is usually not measured in the laboratory because it is virtually impossible to obtain undisturbed samples for testing. The friction angle of sands can instead be correlated to the blow-counts from the standard penetration test (SPT) or the cone resistance from a cone penetration test (CPT).

SPT CORRELATION

Peck et al. (1974) developed the relationship between the friction angle of sand and the corrected SPT-N value that is shown in Fig. A3.1. The SPT-N values shown on the horizontal axis in this figure are corrected to eliminate the influence of the overburden pressure at the depth where the penetration test is performed. This is achieved using a correction factor (C_N) which relates the SPT-N value measured at a given overburden pressure to the value corresponding to an overburden pressure of 1 ton/ft^2. Peck, Hansen and Thornburn (1974) expressed the value of this correction factor in the following form:

$$C_N = 0.77\log_{10}(20/\sigma_v')$$

where σ_v' = vertical effective stress in tons/ft^2

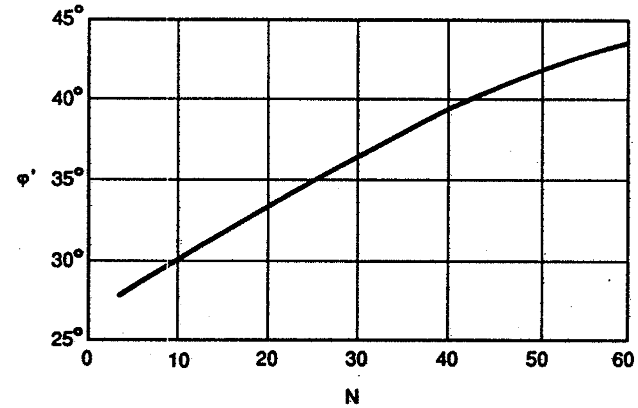


Figure A3.1

Approximate relationship between the friction angle of sand and the SPT-N value (After Peck Hansen and Thornburn, 1974)

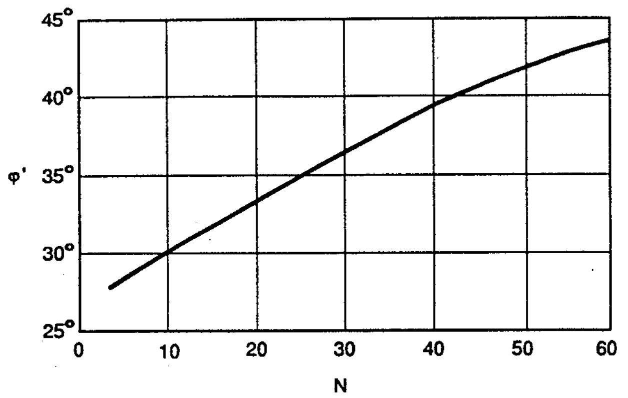


Figure A3.1

Approximate relationship between the friction angle of sand and the SPT-N value (After Peck Hansen and Thornburn, 1974)

The corrected value of N (N_{corr}) is calculated by multiplying the measured value of N by the correction factor:

$$N_{corr} = C_N N$$

where N_{corr} = corrected SPT-N value

N = measured SPT-N value

CPT CORRELATION

Durgunoglu and Mitchell (1975) developed a correlation between the friction angle of sands and the cone resistance, q_c, as shown in Fig. A3.2.

The biggest drawback of the cone penetration test is that no sample is obtained. The test is best used in conjunction with conventional drilling and sampling operations where samples are obtained that can be used for classification. It is also possible to establish a rough idea of soil type based on the relationship of friction ratio (FR) and cone resistance (q_c), and various empirical correlations have been established for this purpose. Campanella and Robertson (1983) developed Fig. A3.3 for this purpose. This diagram can be used to supplement conventional classifications based on samples.

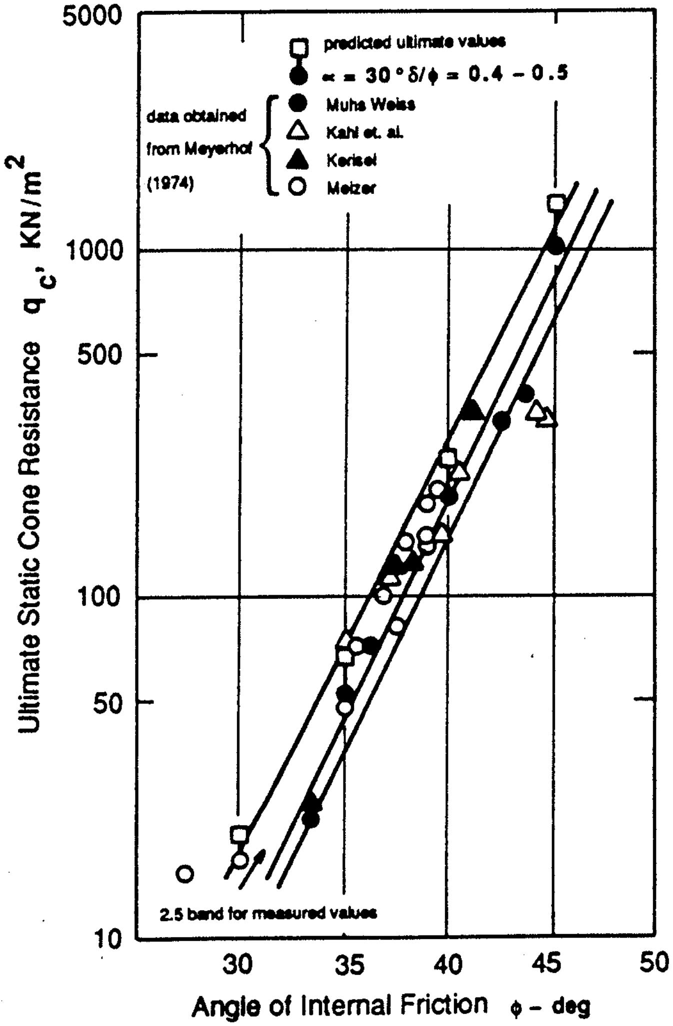


Figure A3.2

Ultimate cone resistance as a function of friction angle for several sands (After Durgunoglu and Mitchell, 1975)

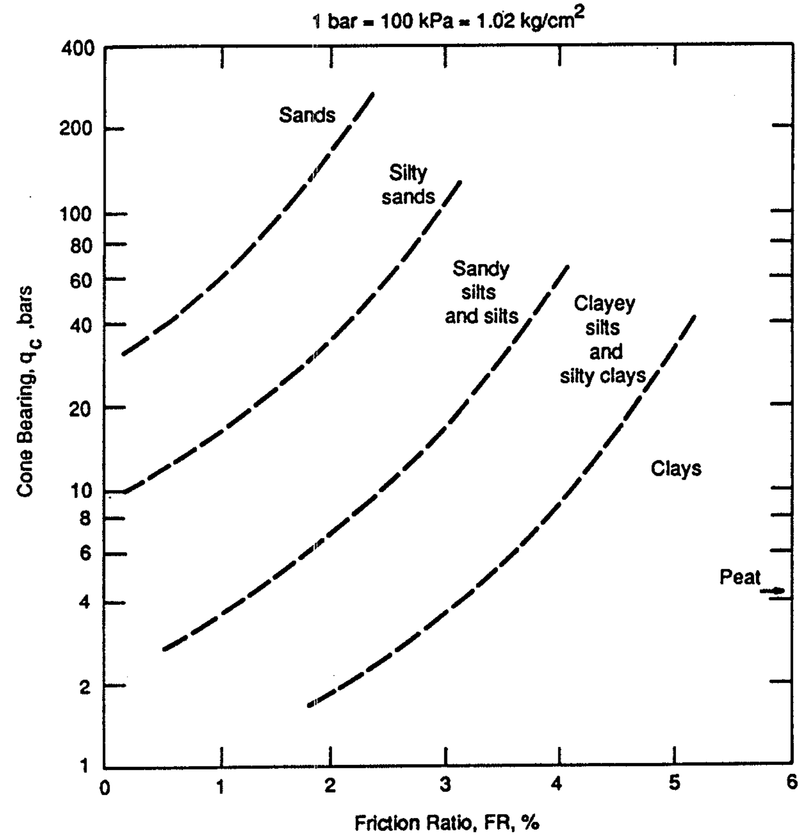


Figure A3.3 Simplified classification chart for standard electric friction cone (After Robertson and Campanella, 1983)

APPENDIX 4

ECCENTRICITY FACTORS FOR DRIVEN PILES

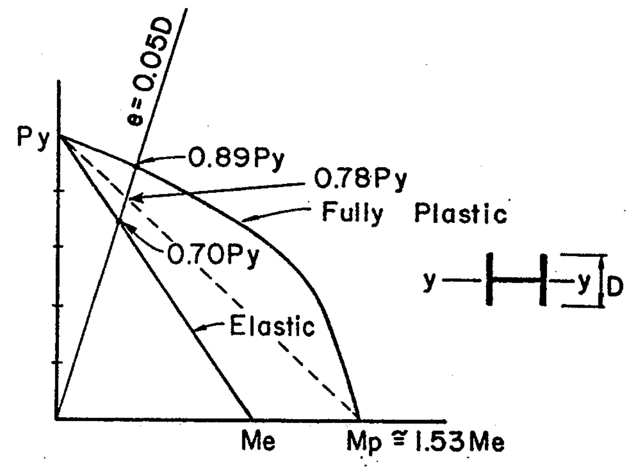
ECCENTRICITY FACTORS FOR DRIVEN PILES

The bases for recommending the eccentricity factors in Table 4.1 for prestressed concrete, precast concrete, steel-H, steel pipe and timber piles are described in this appendix.

The eccentricity factors for precast and prestressed concrete piles are similar to those recommended by AASHTO. The eccentricity factor for timber piles in Table 4.1 (0.82) is the value recommended by Davisson (1983). In the case of steel-H piles and steel pipe piles, the values shown in Table 4.1 are derived based on Davisson's (1983) recommendations.

Steel-H Piles

The moment-thrust interaction curves recommended by Davisson (1983) for steel-H piles along the weak axes are shown as solid lines in Fig. A4.1. The normalized load-moment interaction diagram of Fig. 5.12 reflects a load-moment curve that is intermediate between an elastic section and a plastic section, as indicated by the dashed line in Fig. A4.1. Therefore, the value of eccentricity factor recommended for steel-H piles (0.78) is in between those for the elastic section (0.70) and the plastic section (0.89).



M_e = nominal structural moment capacity of a pile that behaves elastically

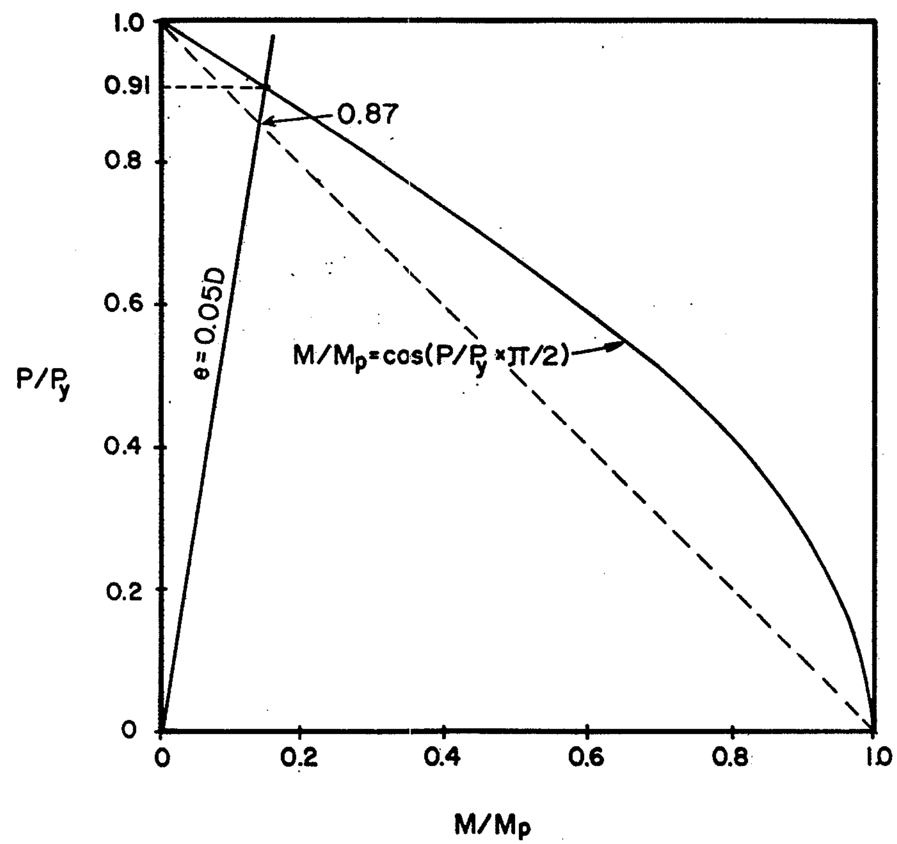
M_p = nominal structural moment capacity of a pile that behaves plastically

P_y = axial structural capacity

Figure A4.1 Load-Moment Interaction Curve for Steel-H Piles Along the Weak Axis (After Davisson, 1983)

Steel Pipe Piles

The moment-thrust interaction curve recommended by Davisson (1983) for steel pipe piles is shown as a solid line in Fig. A4.2. For this curve, the eccentricity factor is 0.91. The normalized load-moment interaction diagram for steel pipe piles in Fig. 5.13 is derived by approximating the straight dashed line in Fig. A4.2 to be the load-moment curve. This is the basis for selecting an eccentricty factor of 0.87 for steel pipe piles.



M_p = nominal structural moment capacity of a pile that behaves plastically

P_y = axial structural capacity

M = bending moment

P = axial load

Figure A4.2 Load-Moment Interaction Curve for Steel-Pipe Piles (After Davisson, 1983)

REFERENCES

1. Alizadeh M., and Davisson M.T., "Lateral Load Tests on Piles - Arkansas River Project", ASCE JSMFED, Vol. 96, No. 9, 1970, pp. 1583-1604.

2. American Association of State Highway and Transportation Officials, "Standard Specifications for Highway Bridges", Fourteenth Edition, 1989.

3. Barker R.M., Duncan J.M. and Rojiani K.B., "Load Factor Design Criteria for Highway Structure Foundations", Interim Rept. for NCHRP, VPI&SU, Oct. 1988, 140 pp.

4. Briaud J.L. and Tucker L., "Piles in Sand: A Method Including Residual Stresses", ASCE JGED, Vol. 110, No. 11, Nov. 1984, pp. 1666-1680.

5. Brown D.A., Reese L.C., O'Neill, M.W., "Cyclic Lateral Loading of a Large-Scale Pile Group", ASCE JGED, Vol. 113, No. 11, Nov. 1987, pp. 1326-1343.

6. Brown D.A., Morrison, C., Reese L.C., "Lateral Load Behavior of Pile Group in Sand", ASCE JGED, Vol. 114, No. 11, Nov. 1988, pp.1261-1276.

7. Canadian Geotechnical Society, "Canadian Foundation Engineering Manual", 2nd Edition, Bitech Publishers Ltd., 1985, 460 pp.

8. Carson A.B., "Foundation Construction", McGraw Hill Book Co., New York, 1965, 424 pp.

9. Circeo M.R., "Skin Friction Mobilization in Axially Loaded Piles and Piers", Rept. submitted to VPI&SU in partial fulfillment for the M.S. Degree in Civ. Eng., Dec. 1986, 61 pp.

10. Davisson M.T., Manuel F.S. and Armstrong R.M., "Allowable Stresses in Piles", FHWA Rept. No. FHWA/RD-83/059, December, 1983, 191 pp.

11. Davisson M.T., and Robinson K.E., "Bending and Buckling of Partially Embedded Piles." Proc. 6th Int. Conf. S.M. and F.E., Montreal, Canada, 1965, pp. 243-246.

13. Department of the Army, "Design of Pile Foundations", U.S. Army Corps of Engineers, Washington D.C. 20314-1000, Publication No. EM 1110-2-2906, in press.

14. Duncan J.M., "Class Notes for Deep Foundation Course - CE 5530", Virginia Polytechnic Institute and State University, Spring 1988.

15. Duncan J.M. and Buchignani A.L., "An Engineering Manual for Settlement Studies", Geotechnical Engineering Report, Univ. of Calif. Berkeley, 1976, 94 pp.

16. Durgunoglu T. and Mitchell J.K., "Static Penetration Resistance of Soils: II - Evaluation of Theory and Implications for Practice", Proc. ASCE Spec. Conf. on In Situ Measurement of Soil Parameters, Vol. I, 1975, pp 172-189.

17. Esrig M.E. and Kirby R.C., "Advances in General Effective Stress Method for the Prediction of Axial Capacity for Driven Piles in Clay", 11th Annual Offshore Technology Conference, Houston, 1979, pp. 437-449.

18. Evans Jr. L.T. and Duncan J.M., "Simplified Analysis of Laterally Loaded Piles", UC Berkeley Rept. No. UCB/GT/82-04, July, 1982, 245 pp.

19. Fellenius B.H., Samson L and Tavenas F., "Geotechnical Guidelines - Pile Design", Public Works Canada, Marine Works Sector, Ottawa Ontario K1A OM2, Canada, August 1989.

20. FHWA, "Interim Procedures for Evaluating Scour at Bridges", FHWA Publication to be incorporated in "Hydraulic Engineering Circular No. 18 (HEC-18), "Scour at Bridges" which is scheduled for Publication in 1989, Sept. 1988, 62 pp.

21. Focht J.A. and Koch K.J., "Rational Analysis of the Lateral Performance of Offshore Pile Groups", Proc. of the 5th Offshore Technology Conference, Houston, Texas, Vol. 2, Paper OTC 1896, 1973, pp. 701-708.

22. Forest Products Laboratory, "Handbook of Wood and Wood-Based Materials for Engineers, Architects and Builders", U.S. Dept. of Agriculture, Hemisphere Publishing Corp., 1989.

23. Heins C.P. and Firmage D.A., "Design of Modern Steel Highway Bridges", John Wiley and Sons, New York, 1979, 463 pp.

24. Holloway D.M., Audibert J.M.E. and Dover A.R., "Recent Advances in Predicting Pile Driveability", Tenth Annual Offshore Tech. Conf., Houston, Texas, May 1978, pp. 1915-1922.

25. Hrennikoff A., "Analysis of Pile Foundations with Batter Piles", Transactions ASCE, CXV, 1950.

26. Hunt H.W., "Design and Installation of Driven Pile Foundations", Associated Pile and Fitting Corp., P.O. Box 1043, Clifton, NJ 07014, 1979, 217 pp.

27. Issa Mohsen and Yuan Robert L., "Prestressed Concrete Column Behavior" PCI Journal, November/December 1989, pp. 51-67.

28. Kulhawy F.H., Trautmann C.H., Beech J.F., O'Rourke T.D. and McGuire W, "Transmission Line Structure Foundations for Uplift-Compression Loading", EPRI Rept. EL-2870, Electric Power Research Institute, 1983.

29. Lawton E.C., Fragaszy R.J., Higgins J.D., Kilian A.P. and Peters A.J., "Review of Methods for Estimating Pile Capacity", Rept. submitted to the Washington State D.O.T., 37 pp.

30. Meyerhof G.G, "Bearing Capacity and Settlement of Pile Foundations", ASCE JGED, Vol. 102, No. GT3, March 1976, pp 196-228.

31. Moulton L.K., GangaRao H.V.S. and Halvorsen G.T., "Tolerable Movement Criteria for Highway Bridges", FHWA/RD-85/107, Federal Highway Administration, Washington D.C., 1985, 118 pp.

32. National Forest Products Association, "National Design Specification for Wood Construction", Recommended Practice by the National Forest Products and Assoc., 1619 Massachussetts Ave., N.W., Washington D.C. 20036, 1982, 81 pp.

33. NAVFAC DM7.2, "Foundations and Earth Structures", Dept. of the Navy, Naval Facilities Engineering Command, May 1982.

34. Nottingham L. and Schmertmann J., "An Investigation of Pile Capacity Design Procedures", Final Report D629 to Florida Dept. of Transportation from Dept. of Civil Engineering, Univ. of Florida, Sept. 1975, 159 pp.

35. O'Neill M.W., Ghazzaly O.I. and Ha H.B., "Analysis of Three-Dimensional Pile Groups with Non-Linear Soil Response and Pile-Soil-Pile Interaction", 9th Annual Offshore Technology Conference, Houston, May 1977, pp. 245-256.

36. O'Neill M.W. and Tsai C.N., "An Investigation of Soil Nonlinearity and Pile-Soil Pile Interaction in Pile Group Analysis", Research Rept. No. UHUC 84-9, Dept. of Civil Eng., Univ. of Houston, Nov. 1984, prepared for US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

37. Peck R.B., Hanson W.E. and Thornburn T.H., "Foundation Engineering", Second Edition, John Wiley and Sons Inc., NY, 1974.

38. Pile Buck Inc., "The Pile Buck Annual - Handbook/Directory of Pile Driving, Foundation and Marine Construction Techniques, Engineers, Contractors, Manufacturers, Distributors and Suppliers", P.O. Box 1056, Jupiter Fl 33468, 1988, 432 pp.

39. Poulos H.G. and Davis E.H., "Pile Foundation Design and Analysis", John Wiley and Sons, 1980, 397 pp.

40. Prakash S. and Sharma H.D., "Pile Foundations in Engineering Practice", John Wiley & Sons, Inc., 1990, 734 pp.

41. Prestressed Concrete Institute, "PCI Design Handbook - Precast and Prestressed Concrete", 3rd Edition, Prestressed Concrete Institute, 20 North Wacker Dr., Chicago Il 60606, 1985.

42. Reese L.C., Cox W.R., and Koop F.D., "Analysis of Laterally Loaded Piles in Sand", 6th Annual Offshore Technology Conf., Paper No. OTC 2080, 1974.

43. Robertson P.K. and Campanella R.G., " Intepretation of Cone Penetration Tests, Part 1 and Part 2, Soil Mechanics Series No. 60, Dept. of Civ. Eng., Univ. of British Columbia, May 1983, 80 pp.

44. Saul W.E., "Static and Dynamic Analysis of Pile Foundations", ASCE Journal, Structural Division, Vol. 94, No. ST5, May 1968, pp. 1077-1100.

45. Scott C.R., "An Introduction to Soil Mechanics and Foundations", 3rd Edition, Applied Science and Publishers Ltd., 1980, 406 pp.

46. Skempton A.W., "The Bearing Capacity of Clays", Proc. of the Building Research Congress, London, England, Vol. 1, 1951, pp. 180-189.

47. Smith E.A.L., "Pile Driving Analysis by the Wave Equation", Journal of Soil Mechanics and the Foundation Division, ASCE, Vol. 86, No. SM4, pp. 35-61.

48. Teng W.C., "Foundation Design", Prentice-Hall, Inc., Tenth Edition, 1962, 466 pp.

49. Terzaghi K., "Evaluation of coefficient of subgrade reaction", Geotechnique, No. 5, 1955, pp. 297-326.

50. Tomlinson M.J., "Pile Design and Construction Practice", Viewpoint Publication, 1987, 415 pp.

51. Vesic A.S., "Bearing Capacity of Shallow Foundations", Chapter 3 in Foundation Engineering Handbook, Ed. by Winterkorn H. and Fang H.Y., Van Nostrand Reinhold Co., New York, 1975, pp. 121-147.

52. Vijayvergiya V.N. and Focht J.A. Jr., "A New Way to Predict the Capacity of Piles in Clay", 4th Annual Offshore Technology Conference, Houston, Vol. 2, 1972, pp. 865-874.

NOTATIONS AND SYMBOLS

Symbol	Represents
ENGLISH	
a_s	Pile perimeter
A	Constant used in the equation for predicting lateral displacement of pile groups
A_c	Cross-sectional area of concrete
A_g	Gross cross-sectional area of concrete pile
A_p	Area of pile point
A_{ps}	Nominal steel area of prestressing tendons
A_s	Surface area of pile shaft
A_t	Cross-sectional area of timber pile
A_y	Cross-sectional area of steel
B	Constant used in the equation for predicting lateral displacement of pile groups
c	Cohesion
C	Constant used in the equation for predicting lateral displacement of pile groups
C_N	Overburden correction factor for SPT-N values
d	Dimensionless depth factor for estimating tip capacity of piles in rock
d_c	Depth factor
d_q	Depth factor
d_γ	Depth factor
D	Pile width or diameter
D'	Effective depth of pile group
D_b	Depth of embedment of pile into a bearing stratum
D_n	Depth of pile embedded in settling soil and subject to downdrag loading

Symbol	Represents
D_s	Diameter of socket when pile is socketed into rock
e_x	Eccentricity of load on a pile group in the x-direction
e_y	Eccentricity of load on a pile group in the y-direction
E_c	Young's modulus of concrete
E_s	Soil modulus
E_p	Young's modulus of pile
f_c'	28 day concrete cylinder strength
f_{pre}	Effective prestress in the concrete pile
f_{pu}	Ultimate strength of prestressing tendons
f_s	Sleeve friction measured from a CPT
f_u	Tensile strength of steel
f_y	Yield stress of steel
FR	Friction ratio ($= f_s/q_c$)
H	Distance between the pile tip and a weaker underlying soil layer
H_s	Depth of embedment of pile socketed into rock
I	Influence factor for the effective embedment of a pile group
I_p	Moment of inertia of a pile
I_r	Rigidity index of soil
I_{solid}	Moment of inertia of a solid circular cross-section
k_b	Factor to account for the treatment conditioning of timber piles when estimating the ultimate moment capacity
k_c	Factor to account for the treatment conditioning of timber piles when estimating the structural axial capacity

Symbol	Represents
k_g	Ratio of the lateral displacement of a pile group to the lateral displacement of a single pile
K	Coefficient of lateral earth pressure
K_c	Correction factor for sleeve friction in clay
K_p	Rankine passive earth pressure coefficient
K_s	Correction factor for sleeve friction in sand
K_{sp}	Dimensionless bearing capacity coefficient
L_{eq}	Equivalent free standing length of a partially embedded pile
L_f	Depth to point considered when measuring sleeve friction
L_u	Unsupported length of pile extending above ground in a partially embedded pile
M_c	Characteristic moment
M_g	Maximum bending moment in a pile within a pile group
M_n	Nominal structural moment capacity of a pile
M_{sp}	Maximum bending moment in a single fixed-head pile subjected to a lateral load P_{sp}
n	Exponent used in estimating the maximum bending moment in a pile within a pile group
n_h	Coefficient of horizontal subgrade reaction
N	SPT blow count
$\overline{N}$	Average SPT blow count along pile shaft
N_c	Bearing capacity factor
N_{corr}	Corrected SPT-N value
N_{pile}	Number of piles in a pile group
N_q	Bearing capacity factor
N_γ	Bearing capacity factor

Symbol	Represents
P_c	Characteristic load
P_{cr}	Critical buckling load
P_D	Dead load
P_{hg}	Lateral load on pile group (unfactored)
P_g	Factored total axial load acting on a pile group
P_i	Axial load due to load i
P_L	Live load
P_n	Nominal structural axial capacity of a pile
P_N	Normalizing load for estimating lateral deflections and maximum bending moments in laterally loaded pile groups
P_{SF}	Lateral stream flow force (unfactored)
P_{sn}	Downdrag load (unfactored)
P_{sp}	Lateral load per pile (unfactored)
$P_{x,y}$	Factored axial load acting on a pile in a pile group. The pile has coordinates (x,y) with respect to the centroidal origin in the pile group
P_{yg}	Lateral displacement capacity of a pile group
q	Net foundation pressure applied at $2D_b/3$
q_c	Static cone resistance
q_l	Limiting point resistance
q_o	Limiting point resistance in lower stratum
q_p	Ultimate unit point resistance of pile
q_s	Ultimate unit skin resistance of pile
q_{sn}	Unit downdrag load
Q	Test load
Q_g	Pile group capacity

Symbol	Represents
Q_i	Load effect due to load component i
Q_p	Ultimate load carried by pile point
Q_s	Ultimate load carried by pile shaft
Q_{ug}	Ultimate uplift resistance of a pile group
Q_{ult}	Total ultimate pile bearing capacity
Q_ρ	Settlement capacity of a pile
r	Eccentricity factor
r_c	Soil rigidity factor
r_q	Soil rigidity factor
r_γ	Soil rigidity factor
R	Resistance corresponding to the limit state considered
R	Characteristic length of soil-pile system in clays
R_I	Moment of inertia ratio
s_c	Shape factor
s_d	Spacing of discontinuities
s_q	Shape factor
s_γ	Shape factor
S	Average spacing of piles
S_b	5% exclusion value for modulus of rupture of timber piles
S_c	5% exclusion values for compression parallel to grain
S_f	Pile butt settlement at failure
S_u	Undrained shear strength
$\bar{S}_u$	Average undrained shear strength along pile shaft
t_d	Width of discontinuities

Symbol	Represents
T	Characteristic length of soil-pile system in sands
W	Weight of pile
W_g	Weight of block of soil, piles and pile cap
x	Distance of the pile from the centroid of the pile cap in the x-direction
X	Width of pile group
y	Distance of the pile from the centroid of the pile cap in the y-direction
Y	Length of pile group
Y_g	Lateral displacement of a laterally loaded pile group
Y_{sp}	Lateral displacement of a single fixed-head pile subjected to a lateral load P_{sp}
$Y_{g\ tol}$	Tolerable lateral displacement of a pile group
Z	Total embedded pile length
Z_e	Elastic section modulus of timber piles
Z_p	Plastic section modulus of steel piles
<u>GREEK</u>	
α	Adhesion factor applied to S_u
β	Coefficient relating the vertical effective stress and the unit skin friction of a pile when used in Section 4
β	Load factor coefficient for a load component when used in Section 3.2
γ	Total unit weight of soil when used in Section 4 and 5
γ	Load factor when used in Section 3.2
γ'	Effective unit weight of soil
γ_D	Load factor for dead load

Symbol	Represents
γ_h	Load factor for lateral load
γ_i	Load factor for load component i
γ_L	Load factor for live load
γ_m	Moment factor
γ_{SF}	Load factor for stream flow force
δ	Angle of shearing resistance between soil and pile
δ_E	Elastic compression of pile
λ	Empirical coefficient relating the passive lateral earth pressure and the unit skin friction of a pile
η	Efficiency factor for capacity of a pile group when the pile cap is not in firm contact with the ground
μ	Poisson's ratio of soil
ρ	Settlement of a pile group
ρ_{tol}	Tolerable settlement
σ_c	Uniaxial compression strength of rock cores
σ_h'	Horizontal effective stress
σ_v	Total vertical stress
σ_v'	Vertical effective stress
ϕ	Performance factor
ϕ'	Angle of internal friction of soil
ϕ_a	Performance factor for the nominal axial structural capacity of a pile
ϕ_g	Performance factor for the bearing capacity of a pile group failing as a unit consisting of the piles and the block of soil contained within the piles
ϕ_m	Performance factor for the ultimate structural moment capacity of a pile

Symbol	Represents
ϕ_q	Performance factor for the total ultimate bearing capacity of a pile
ϕ_{qs}	Performance factor for the ultimate shaft capacity of a pile
ϕ_{qp}	Performance factor for the ultimate tip capacity of a pile
ϕ_t	Performance factor for the tensile strength of steel
ϕ_u	Performance factor for the uplift capacity of a single pile
ϕ_{ug}	Performance factor for the uplift capacity of pile groups

Part 3—Engineering Manual for Retaining Walls and Abutments

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CONTENTS

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Retaining walls are used to retain soil and water in order to maintain an abrupt change in elevation. Abutments are a particular type of retaining wall that support the ends of bridge superstructures. This engineering manual provides guidelines for the design of retaining walls and bridge abutments founded on spread footings, driven piles, or drilled shafts.

Both conventional allowable stress design (ASD) procedure and Load Factor Design (LFD) procedures are presented in this manual.

A discussion of various types of retaining walls and abutments is presented in Chapter 2. Design considerations and forces on retaining walls are discussed in Chapter 3 and Chapter 4. Chapter 5 deals with design requirements with regard to stability and structural failure.

Design examples are presented in Chapter 6 to illustrate application of the new load factor design method for various types of retaining walls and bridge abutments.

CHAPTER 2

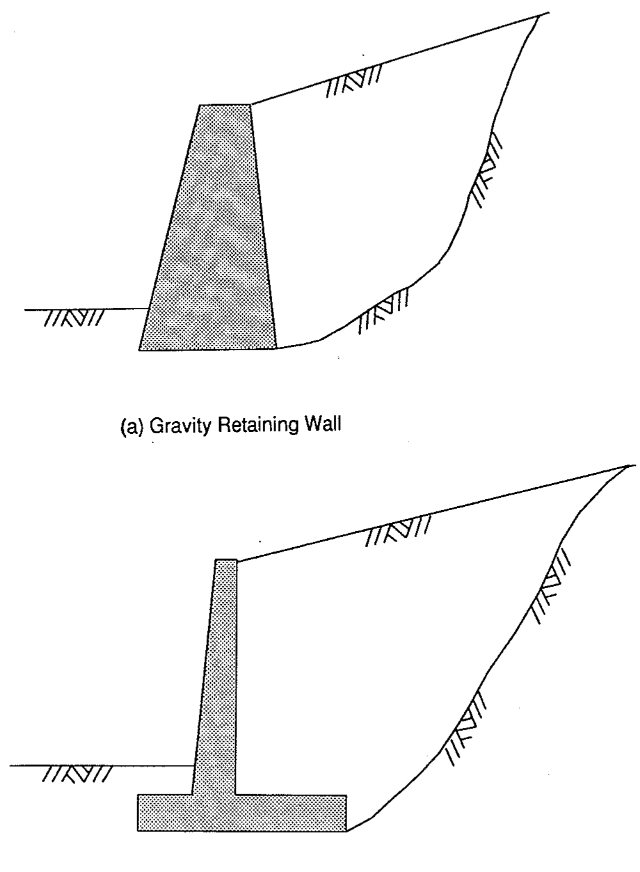
TYPES OF RETAINING WALLS AND ABUTMENTS

2.1 TYPES OF RETAINING WALLS

Retaining walls can be divided into two major categories: conventional retaining walls and alternative, or innovative, types of retaining walls. Conventional retaining walls, such as gravity walls and reinforced concrete walls, have a long history of successful use. During the last two decades, many types of innovative retaining walls have emerged.

2.1.1 Conventional Retaining Walls

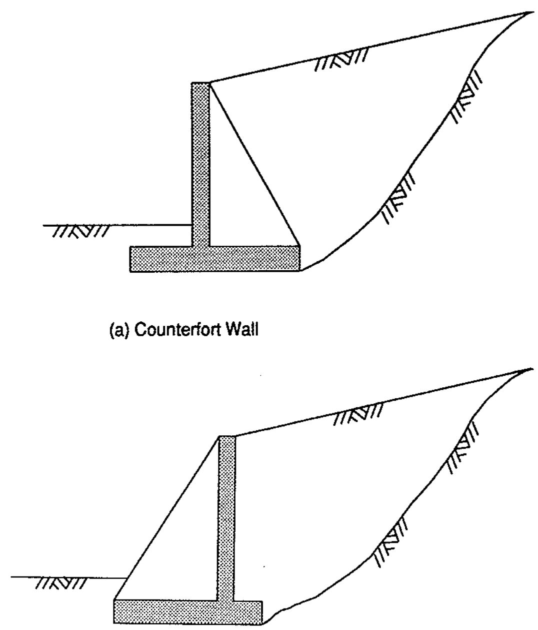
Conventional retaining walls can be subdivided into two principal types: gravity walls and cantilever walls. Gravity walls (Figure 2.1(a)) rely on the mass of wall and the force of gravity to resist the forces exerted by the retained earth and water behind the wall. Cantilever walls (Figure 2.1(b) resist the forces exerted on them by flexural strength. They consist of a wall stem, a toe, a heel, and possibly a shear key. Counterfort and buttressed reinforced concrete walls (Figure 2.2) have been used when the heights of the walls are large, and cantilever walls are not economical.



(a) Gravity Retaining Wall

(b) Cantilever Retaining Wall

Figure 2.1. Typical gravity and cantilever retaining wall.



(a) Counterfort Wall

(b) Buttress Wall

Figure 2.2. Counterfort and buttress retaining walls.

2.1.2 Alternative Retaining Walls

A number of types of alternative retaining walls have been developed in recent years.

Munfakh (1990) classifies the alternative retaining walls into two major categories: (1) walls retaining fill and (2) walls supporting excavations.

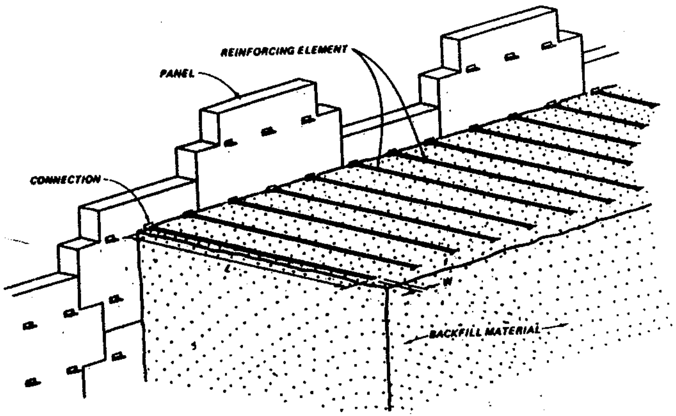
The first category of alternative walls includes embankment-type mechanically stabilized walls (examples are VSL retained earth, geotextile walls, geogrid walls, and welded wire walls) and modular gravity walls (examples are Criblock walls, Doublewals, gabion walls, and Evergreen walls). Some of these mechanically stabilized earth walls and a precast concrete modular gravity wall are shown in Figure 2.3.

The second category of alternative walls are those that support excavations (examples are soil nailing, reticulated micro piles, soil doweling, sheet piles, soldier piles, secant piles, and drilled shaft walls).

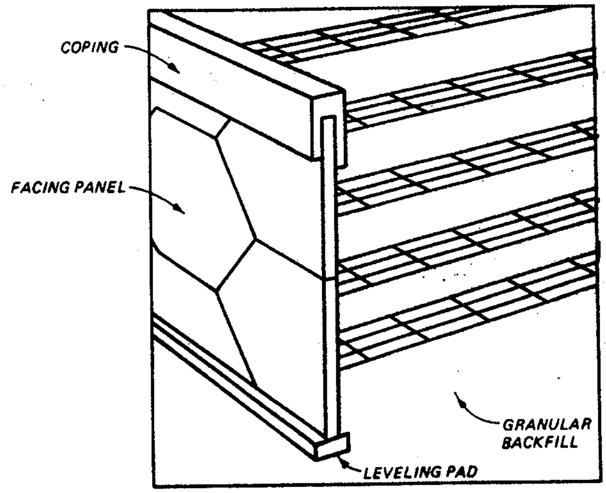
2.2 TYPES OF ABUTMENTS

A proposed revision to the AASHTO bridge specifications (D'Appolonia, 1989) divides abutments into four types: stub abutments, partial depth abutments, full depth abutments, and integral abutments.

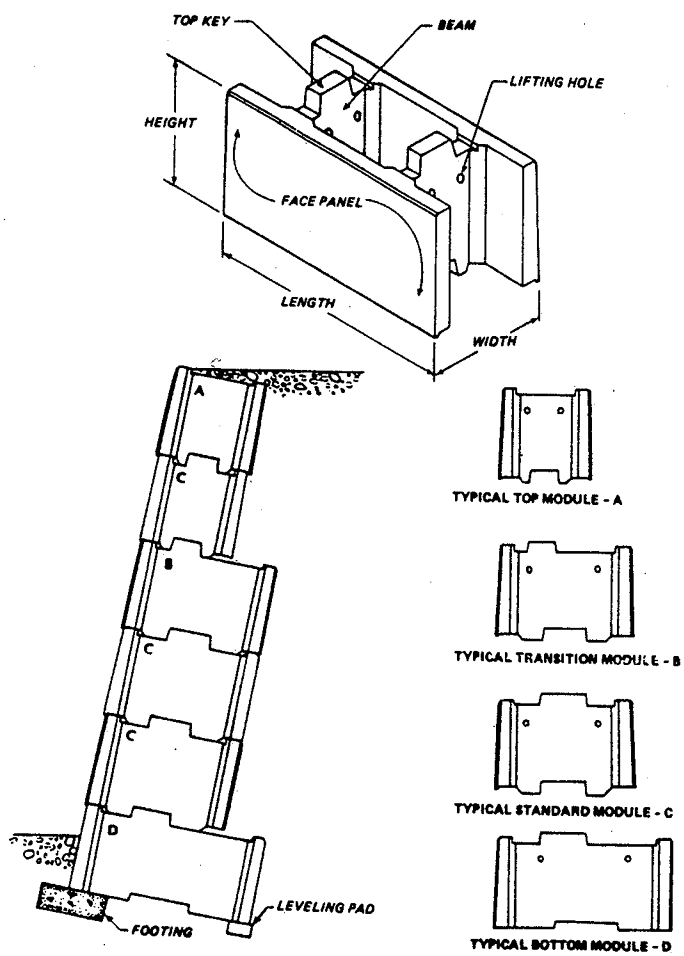
Peck et al. (1974) classify bridge abutments in a different way: gravity abutments, U abutments, spill-through abutments, and pile-bent abutments. A gravity abutment with wing walls (Figure



Schematic diagram of reinforced earth retaining wall



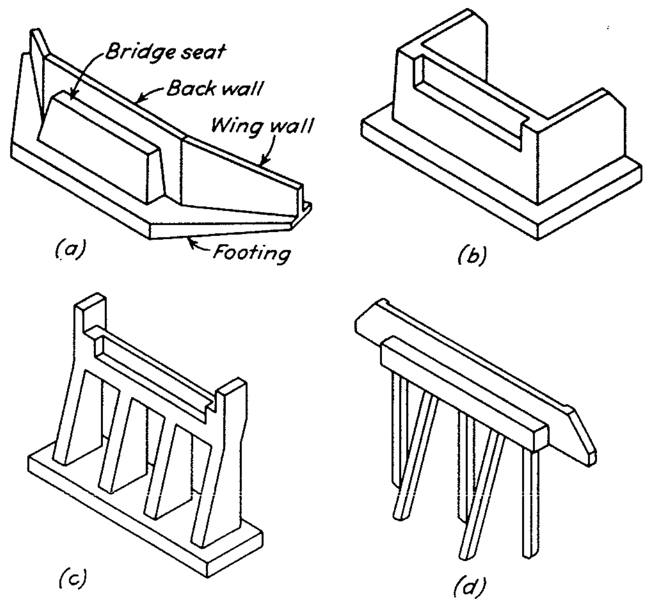
VSL retained earth retaining wall (adapted from VSL Corporation 1984)



Schematic diagram of Doublewal retaining wall (after Doublewal Corporation 1984)

Figure 2.3. Mechanically stabilized earth walls and concrete modular wall. (From Corps of Engineers, 1989)

2.4(a)) is an abutment which consists of a bridge seat, wingwalls, backwall, and footing. A U-abutment (Figure 2.4(b)) is an abutment whose wingwalls are perpendicular to the bridge seat. The spill-through abutment (Figure 2.4(c)) consists of a beam which supports the bridge seat, two or more columns supporting the beam, and a footing supporting the columns. The columns are embedded up to the bottom of the beam in the fill which extends on its natural slope in front of the abutment. The pile-bent abutment with stub wings (Figure 2.4(d) is another type of spill-through abutment, in which a row of driven piles supports the beam.



(a) Typical gravity abutment with wing walls. (b) U abutment. (c) Spill-through abutment. (d) Pile-bent abutment with stub wings.

Figure 2.4. Various types of abutments. (From Peck et al., 1974)

2.3 SELECTION OF RETAINING WALLS AND ABUTMENTS

Selection of the most suitable type of retaining wall or abutment should be based on a number of considerations, including: (1) construction and maintenance cost; (2) cut or fill earthwork situation; (3) traffic maintenance during construction; (4) construction period; (5) safety of construction workers; (6) availability and cost of backfill material; (7) superstructure depth; (8) size of wall; (9) horizontal and vertical alignment changes; (10) area of excavation; (11) aesthetics and similarity to adjacent structures; (12) previous experience with the type of wall or abutment being considered; (13) ease of access for inspection and maintenance; (14) anticipated life, loading conditions, and acceptability of deformations.

Munfakh (1990) and Schnore (1990) have discussed the advantages and disadvantages of various types of walls and abutments. They present decision matrices that include many of the foregoing factors. These tables can be very helpful in selecting the proper wall or abutment for particular site conditions.

CHAPTER 3

GENERAL DESIGN CONSIDERATIONS

3.1 GENERAL

This chapter discusses the considerations involved in design of retaining walls and abutments, including design methods, loads and load combinations, safety factors, load factors and performance factors, tolerable movements, and other design considerations.

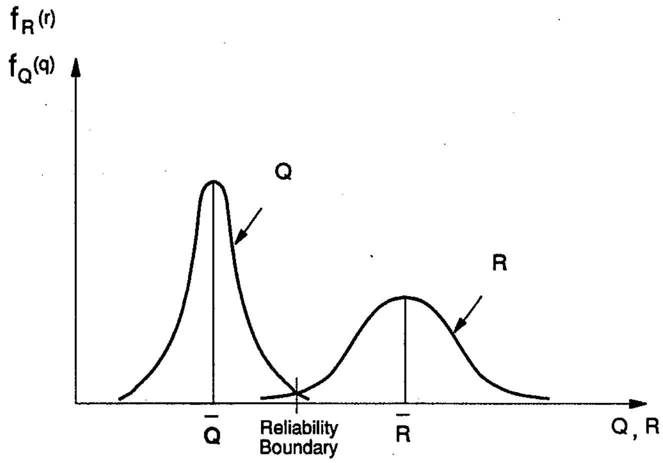
3.2 DESIGN METHODS

The last three decades have seen a trend away from allowable stress design (ASD) that incorporates factors of safety toward strength design methods that incorporate limit state and reliability concepts. Examples of strength design procedures are load factor design (LFD), and load and resistance factor design (LRFD). The current AASHTO bridge specifications make use of LFD as an alternative method for the design of superstructures. In the near future, it is anticipated that the AASHTO specifications will introduce the LFD method for foundation design, so that engineers may use the same philosophy for design of foundations that is used for the superstructure.

3.2.1 Limit States

If retaining walls and abutments fail to satisfy their intended design functions, they are considered to reach "limit states". Limit states can be categorized into two types: ultimate or strength limit states and serviceability limit states.

A retaining wall reaches an ultimate limit state when the strength of at least one of its components is fully mobilized or when the structure becomes unstable. In this ultimate limit state a retaining wall may experience serious distress and structural damage, both local and global. In addition, various failure modes in the soil that supports the wall can also be identified. These are also called ultimate limit states; they include bearing capacity failure, sliding, overturning, and overall instability.



where

Q : Load

R : Resistance

$\bar{Q}$: Mean of Load

$\bar{R}$: Mean of Resistance

Figure 3.1. Probability density functions for load and resistance.

A retaining wall experiences a serviceability limit state when it fails to perform its intended design function fully, because of excessive deformation or deterioration. Serviceability limit states include excessive total or differential settlement, lateral movement, fatigue, vibration, and cracking.

3.2.2 Allowable Stress Design (ASD)

Allowable stress design is a method which ensures safety by restricting values of stress obtained from elastic analysis to values that are not larger than some allowable values. Values of allowable stress are generally selected as some function of the yield strength of a material, and are determined by dividing the yield strength by a global factor of safety.

Because the ASD method uses deterministic values for loads and resistances, the random nature of the loads and resistances, are not accounted for explicitly. The ASD method therefore does not consider the different degree of uncertainty for different types of loads; live load is treated the same as dead load, even though it usually has greater variation during the life of the structure and greater uncertainty. Another shortcoming of the ASD method is its inability to account rationally for uncertainties in the strengths of materials and the ultimate resistances of structural components.

3.2.3 Load Factor Design (LFD)

Load factor design is a strength design or limit state design method. Unlike ASD, this method takes into account the random nature of loads and resistances, and different levels of uncertainty for different types of load.

The design loads are obtained by multiplying the normally expected values, called "nominal" loads, by load factors typically larger than unity. The design strengths or resistances are calculated by multiplying nominal strengths by resistance factors that have values smaller than unity. For an acceptable design, the factored resistance has to be greater than or equal to the strength demand resulting from the factored design loads for a particular limit state. A mathematical statement of LFD is given as:

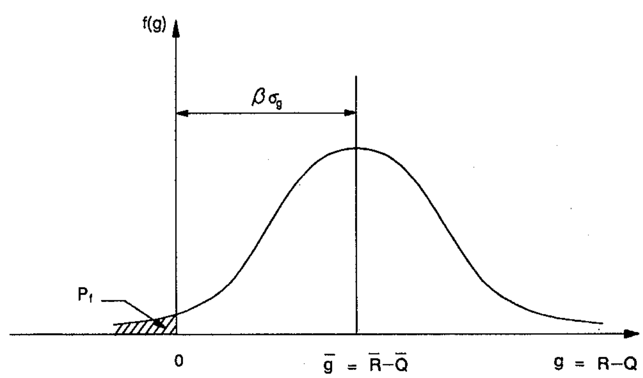
$$\phi\, R_n \geq \text{effect of } \Sigma\, \gamma_i\, Q_i \qquad (3.2.3.1)$$

where ϕ = performance factor, R_n = nominal resistance, γ_i = load factor for load component i, and Q_i = load component i.

In LFD the values of the load and performance factors are based on semiquantitative considerations of probabilities, judgment, and previous experience with other design methods.

3.2.4 Load and Resistance Factor Design (LRFD)

Load and resistance factor design is also a strength design method and has the same design format as LFD (Eq. 3.2.3.1). However, the manner in which load and resistance factors are derived is different. LRFD formally uses reliability theory. Loads and resistances are considered as random variables, and are represented by their means and standard deviations. The load and resistance factors used in LRFD depend on the value of the safety index, which is directly related to probability of failure. The safety index is chosen such that the probability of failure of a structure or foundation is small. Probability density functions for load and resistance are shown in Figure 3.1, and the relationship between safety index and probability of failure is presented in Figure 3.2.



where

Q : Load

R : Resistance

$\bar{Q}$: Mean of Load

$\bar{R}$: Mean of Resistance

P_f: Probability of Failure

σ_g: Standard Deviation of g = $\sqrt{\sigma_R^2 + \sigma_Q^2}$

β: Safety Index = $\dfrac{\bar{g}}{\sigma_g}$

Figure 3.2. Definition of safety index.

3.3 SAFETY FACTORS, LOAD FACTORS, AND PERFORMANCE FACTORS

3.3.1 Safety Factors

In ASD, a global safety factor is used to ensure safety of a structure or foundation. Table 3.1 gives typical values of safety factors used in geotechnical engineering.

3.3.2 Load Factors

Load factors are applied to the loads to account for the uncertainties involved in the process of selecting loads and load effects. The load factors used in the current AASHTO bridge specifications (1989) are given in Table 3.2.

3.3.3 Performance Factors

Performance factors are used to reduce various types of resistance to account for uncertainties in structural properties, soil properties, variability in workmanship, and inaccuracies in the design equations used to estimate the capacity. These factors are used for design at the ultimate limit state. Suggested values of performance factors for shallow foundations, driven piles, and drilled shafts are given in Tables 3.3 to 3.5.

Table 3.1. Typical safety factors in foundation design. (After Meyerhof, 1984)

Failure Type	Item	Safety Factor*
Shearing	Earthworks	1.3 to 1.5
	Earth retaining structures, excavations	1.5 to 2.0
	Foundations	2.0 to 3.0
Seepage	Uplift, heave	1.5 to 2.0
	Exit gradient, piping	2.0 to 3.0

* The lower values are used when uncertainty in design is small and consequences of failure are minor; higher values are used when uncertainty in design is large and consequences of failure are major.

3.4 DESIGN LOADS AND LOAD COMBINATIONS

Various combinations of different loads are used for the design of superstructures and foundations. These loads include dead load, live load, lateral earth pressure, wind load, earthquake load, ice load, and strain-related loads such as creep and shrinkage in concrete. The AASHTO load factors and load combinations for ultimate limit states and serviceability limit states, which are given in Table 3.2, are expressed in the following format:

$$\begin{aligned}\text{Group N} = \gamma[&\beta_D D + \beta_L(L+I) + \beta_C CF + \beta_E E \\ &+ \beta_B B + \beta_{SF} SF + \beta_W W + \beta_{WL} WL \\ &+ \beta_{LF} LF + \beta_R(R+S+T) \\ &+ \beta_{EQ} EQ + \beta_{ICE} ICE] \qquad (3.4.1)\end{aligned}$$

where N = loading group number, γ = load factor, β = coefficient, D = dead load, L = live load, I = impact due to live load, E = earth pressure, B = buoyancy, W = wind load on structure, WL = wind load on live load, LF = longitudinal force from live load, CF = centrifugal force, R = rib shortening, S = shrinkage, T = temperature, EQ = earthquake, SF = stream flow pressure, and ICE = ice pressure.

3.5 OTHER CONSIDERATIONS

3.5.1 Backfill

Various types of soil can be used as backfill for walls and abutments. Free-draining soils such as clean sands and gravels are the most desirable materials for backfill. Clay soils cause many problems, including seasonal volume changes, slow drainage, settlement, creep movements, and cracks due to shrinkage. Shrinkage cracks that develop in clay may be filled with water so that the walls may be subject to unexpectedly high lateral pressures. Because of these disadvantages, clay soils are not recommended for backfill.

The backfill material should be compacted to minimize settlements due to self-weight and surcharge loads. Compaction increases the lateral forces exerted on the wall, and these should be considered in design. Earth pressures due to compaction are discussed in more detail in Chapter 4.

Soil densities and strengths of backfill materials can be determined from laboratory tests or estimated based on correlations with index properties and specified values of relative compaction. Friction angles for sandy and gravelly soils can be estimated using the correlations shown in Figure 3.3.

3.5.2 Drainage

One of the major reasons for unsatisfactory performance of retaining walls is an improperly designed or poorly constructed drainage system.

The main purpose of a drainage system is to prevent excessive water pressures from acting against the wall. Various types of drainage systems can be used based on the type of retaining wall, the type of backfill material, the ground water condition, and possible frost effects. An inclined drainage system is more effective than a vertical drainage system located at the back of the wall. The effect of drain location on excess hydrostatic pressures on a potential failure plane in the backfill is shown in Figure 3.4.

A drainage blanket or a prefabricated drainage composite can be used as drain material. Longitudinal drains within drainage blankets should be located at adequate intervals to convey the released water from behind the wall to a free exit. Where weepholes are used they should be at least 3 in. in diameter and less than 10 ft apart horizontally and vertically. Granular material or filter fabric should be used to prevent weepholes from plugging. Examples of various types of drainage systems are shown in Figures 3.5 and 3.6.

Terzaghi and Peck (1948) suggested a drainage system for clay backfill materials, as shown in Figure 3.7, to prevent changes in water content of the clay, so as to reduce potential cracking and swelling.

Table 3.2. Load factors and load combinations of AASHTO bridge specifications. (From AASHTO, 1989)

Col. No.		1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14	
			β FACTORS														
GROUP		γ	D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE	%	
SERVICE LOAD	I	1.0	1	1	0	1	β_E	1	1	0	0	0	0	0	0	100	
	IA	1.0	1	2	0	0	0	0	0	0	0	0	0	0	0	150	
	IB	1.0	1	0	1	1	β_E	1	1	0	0	0	0	0	0	**	
	II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	0	125	
	III	1.0	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125	
	IV	1.0	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125	
	V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	0	140	
	VI	1.0	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140	
	VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	0	133	
	VIII	1.0	1	1	0	1	1	1	1	0	0	0	0	0	1	140	
	IX	1.0	1	0	0	0	1	1	1	1	0	0	0	0	1	150	
	X	1.0	1	1	0	0	β_E	0	0	0	0	0	0	0	0	100	Culvert
LOAD FACTOR DESIGN	I	1.3	β_D	1.67*	0	1.0	β_E	1	1	0	0	0	0	0	0	Not Applicable	
	IA	1.3	β_D	2.20	0	0	0	0	0	0	0	0	0	0	0		
	IB	1.3	β_D	0	1	1.0	β_E	1	1	0	0	0	0	0	0		
	II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0		
	III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0		
	IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0		
	V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0		
	VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0		
	VII	1.3	β_D	0	0	0	β_E	1	1	0	0	0	0	1	0		
	VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1		
	IX	1.20	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1		
	X	1.30	1	1.67	0	0	β_E	0	0	0	0	0	0	0	0		Culvert

$(L + I)_n$ - Live load plus impact for AASHTO Highway H or HS loading

$(L + I)_p$ - Live load plus impact consistent with the overload criteria of the operation agency.

* 1.25 may be used for design of outside roadway beam when combination of sidewalk live load as well as traffic live load plus impact governs the design, but the capacity of the section should not be less than required for highway traffic live load only using a beta factor of 1.67. 1.00 may be used for design of deck slab with combination of loads as described in Article 3.24.2.2.

$$** \text{ Percentage} = \frac{\text{Maximum Unit Stress (Operating Rating)}}{\text{Allowable Basic Unit Stress}} \times 100$$

For Service Load Design

% (Column 14) Percentage of Basic Unit Stress

No increase in allowable unit stresses shall be permitted for members or connections carrying wind loads only.

β_E = 1.00 for vertical and lateral loads on all other structures.

For culvert loading specifications, see Article 6.2.

β_E = 1.0 and 0.5 for lateral loads on rigid frames (check both loadings to see which one governs). See Article 3.20.

For Load Factor Design

β_E = 1.3 for lateral earth pressure for retaining walls and rigid frames excluding rigid culverts.
β_E = 0.5 for lateral earth pressure when checking positive moments in rigid frames. This complies with Article 3.20.
β_E = 1.0 for vertical earth pressure
β_D = 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity . . . . . For
β_D = 1.0 when checking member for maximum axial load and minimum moment . . . . Column Design
β_D = 1.0 for flexural and tension members
β_E = 1.0 for Rigid Culverts
β_E = 1.5 for Flexible Culverts

For Group X loading (culverts) the β_E factor shall be applied to vertical and horizontal loads.

Table 3.3. Performance factors and safety factors for shallow foundations. (After Tan et al., 1991)

Type of Limit State	Performance Factor
1. Bearing Capacity	
a. Sand	
Semi-empirical Procedure (SPT)	0.45
Semi-empirical Procedure (CPT)	0.55
Rational Method:	
using ϕ_f estimated from SPT	0.35
using ϕ_f estimated from CPT	0.45
b. Clay	
Semi-empirical Procedure (CPT)	0.50
Rational Method:	
using shear strength in lab tests	0.60
using shear strength from field vane tests	0.60
using shear strength estimated from CPT data	0.50
c. Rock	
Semi-empirical procedure	0.60
2. Sliding	
a. Precast concrete placed on sand:	
using ϕ_f estimated from SPT	0.90
using ϕ_f estimated from CPT	0.90
b. Concrete Cast in place on sand:	
using ϕ_f estimated from SPT	0.80
using ϕ_f estimated from CPT	0.80
c. Clay (where shear strength is less than 0.5 times normal pressure):	
using shear strength in lab	0.85
using shear strength from field vane test	0.85
using shear strength estimated from CPT data	0.80
d. Clay (where the strength is greater than 0.5 times normal pressure)	0.85

Table 3.4. Performance factors for driven piles.

	METHOD/SOIL/CONDITION		PERFORMANCE FACTOR
ULTIMATE BEARING CAPACITY OF SINGLE PILES	SKIN FRICTION	α-method	0.70
		β-method	0.50
		λ-method	0.55
	END BEARING	Clay (Skempton, 1951)	0.70
		Sand (Kulhawy, 1983)	
		ϕ_f from CPT	0.45
		ϕ_f from SPT	0.35
		Rock (Canadian Geotech. Society, 1985)	0.50
	SKIN FRICTION AND END BEARING	SPT-method	0.45
		CPT-method	0.55
		Load Test	0.80
		Pile Driving Analyzer	0.70
BLOCK FAILURE	Clay		0.65
UPLIFT CAPACITY OF SINGLE PILES	α-method		0.60
	β-method		0.40
	λ-method		0.45
	SPT-method		0.35
	CPT-method		0.45
	Load Test		0.80
GROUP UPLIFT CAPACITY	Sand		0.55
	Clay		0.55

According to the Corps of Engineers (1989), even when a drainage system is used, walls should still have a sufficient factor of safety to account for the possibility that the drain system may not work properly.

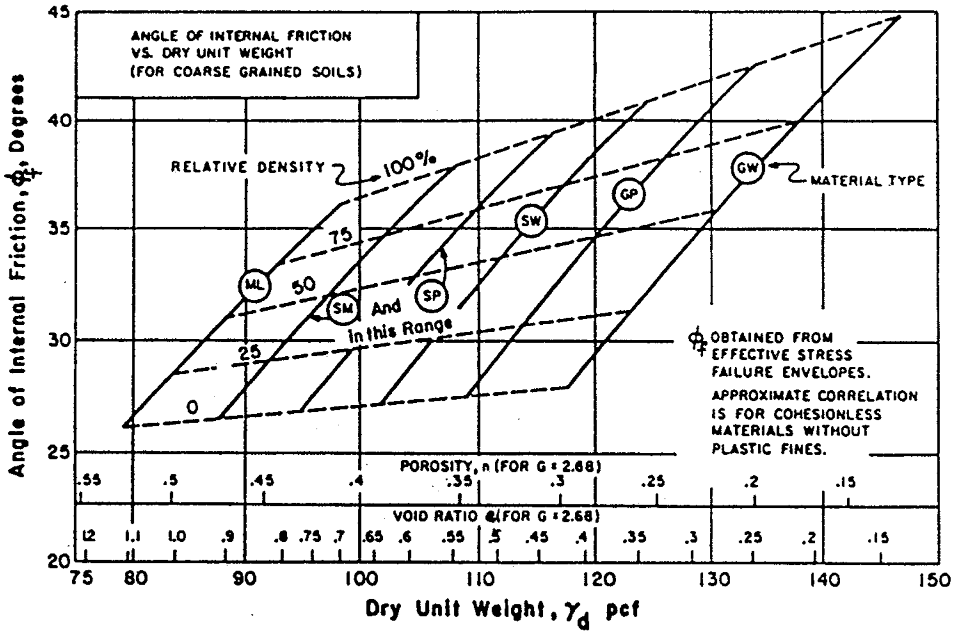
3.5.3 Contraction and Expansion Joints

Attention should be given to the effects of shrinkage and thermal stresses when the length of a retaining wall or abutment is long. Joints are provided to allow the concrete to move slightly and to control shrinkage and thermal stresses. There are two broad types of joints: expansion joints and contraction joints.

Table 3.5. Performance factors for drilled shafts.

	METHOD/SOIL/CONDITION		PERFORMANCE FACTOR
ULTIMATE BEARING CAPACITY OF SINGLE DRILLED SHAFTS	SIDE RESISTANCE IN CLAY	α-method (Reese & O'Neill)	0.65
	BASE RESISTANCE IN CLAY	Total Stress (Reese & O'Neill)	0.55
	SIDE RESISTANCE IN SAND	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See Discussion in Article 4.2.2
	BASE RESISTANCE IN SAND	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See Discussion in Article 4.2.2
	SIDE RESISTANCE IN ROCK	Carter & Kulhawy	0.55
		Horvath and Kenney	0.65
	BASE RESISTANCE IN ROCK	Canadian Geotechnical Society	0.50
		Pressuremeter Method (Canadian Geotechnical Society)	0.50
	SIDE RESISTANCE AND END BEARING	Load Test	0.80
BLOCK FAILURE	Clay		0.65
UPLIFT CAPACITY OF SINGLE DRILLED SHAFTS	CLAY	α-method (Reese & O'Neill)	0.55
		Belled Shafts (Reese & O'Neill)	0.50
	SAND	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See Discussion in Section 4.2.2
	ROCK	Carter & Kulhawy	0.45
		Horvath & Kenney	0.55
		Load Test	0.80
GROUP UPLIFT CAPACITY		Sand	0.55
		Clay	0.55

* Section 4.2.2 of the Drilled Shafts Engineering Manual



where ML: Silt
SM: Silty sand
SP: Poorly graded sand
SW: Well-graded sand
GP: Poorly graded gravel
GW: Well-graded gravel

Figure 3.3. Approximate frictional angle of granular backfill. (From NAVFAC, 1982)

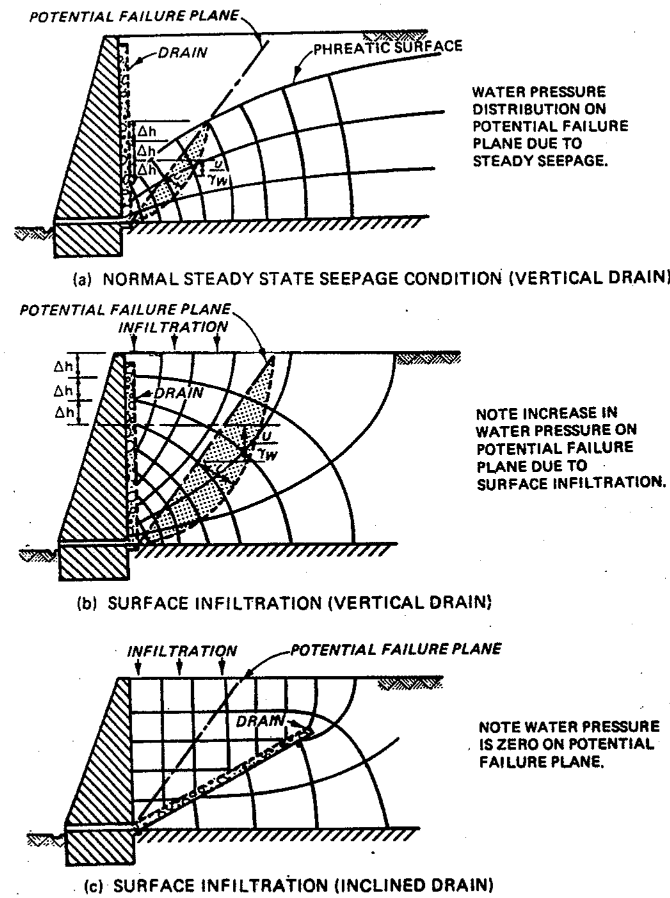
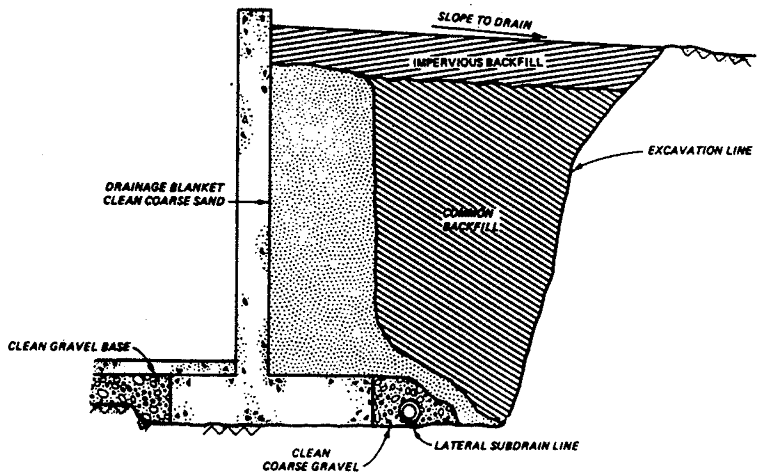
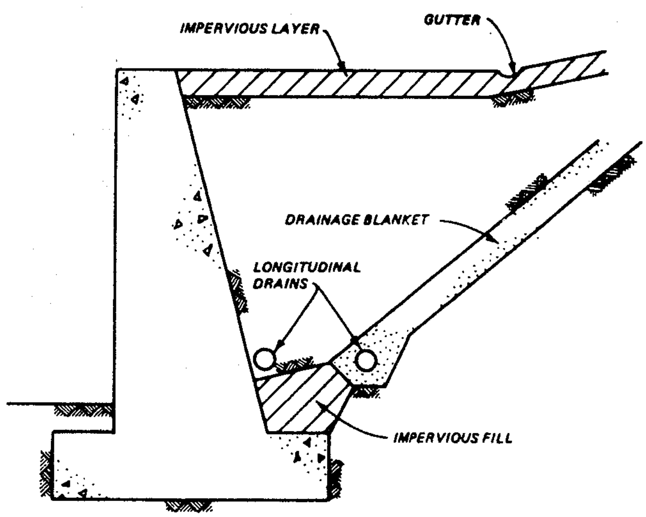


Figure 3.4. Effect of drain location on excess hydrostatic pressures on the failure plane. (From Geotechnical Control Office, 1982)



NOT TO SCALE

Drainage blanket located adjacent to retaining wall (after Sibley 1967)



NOT TO SCALE

Inclined drainage blanket (after Department of the Navy 1982a)

Figure 3.5. Drainage systems using drainage blanket. (After Sibley 1967, and NAVFAC 1982)

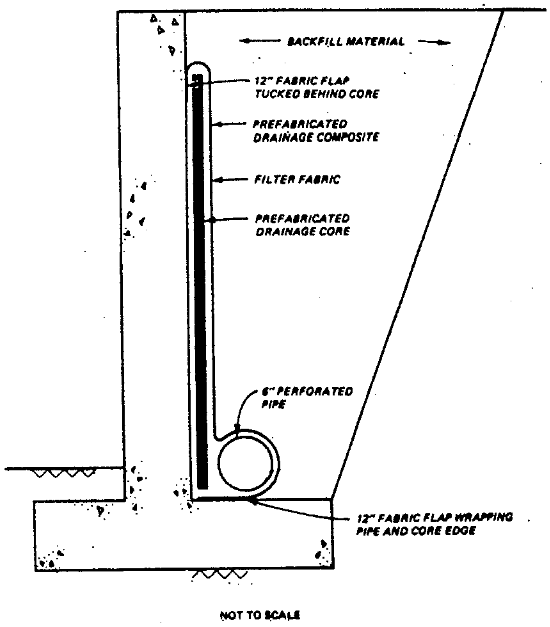
Expansion (or isolation) joints are used to permit differential horizontal and vertical movement between adjacent wall segments. They consist of a completely separated joint filled with compressible material that allows the wall to expand and contract with changes in moisture and temperature.

Contraction (or control) joints are used to control the location of tension cracks by creating a plane of weakness in the wall and forcing the cracking to occur at those locations. The weakened plane is made with narrow grooves placed on the outside of the wall stem. The grooves are usually filled with a sealant to prevent penetration of moisture.

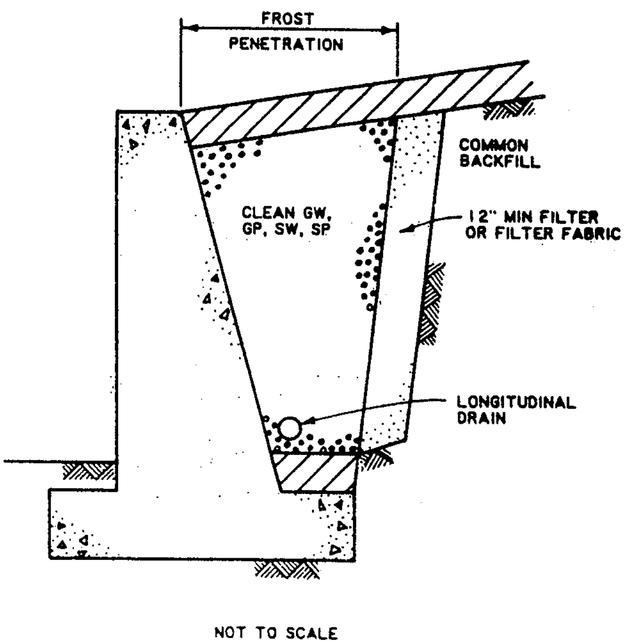
The AASHTO bridge specifications (1989) require that contraction joints be placed at 30-ft intervals or less, and expansion joints should be placed at intervals of 90 ft or less in concrete walls.

3.5.4 Reinforcement for Shrinkage and Thermal Stresses

The AASHTO bridge specifications (1989) require a minimum total reinforcement area of $\frac{1}{8}$ in.^{2} per ft in both directions. This



Prefabricated drainage composite used as drain adjacent to retaining wall (adapted from Carrol and Murphy 1985)



Drainage system to prevent frost penetration behind retaining wall (after Department of the Navy 1982a)

Figure 3.6. Various drainage systems using filter fabric. (After Carrol and Murphy 1985, and NAVFAC 1982)

reinforcement is provided near exposed surfaces, except in gravity walls, to resist the formation of cracks due to temperature change and shrinkage.

The maximum bar spacing is three times the wall thickness or 18 in., whichever is less, according to Art. 8.20 of the AASHTO specifications (1989).

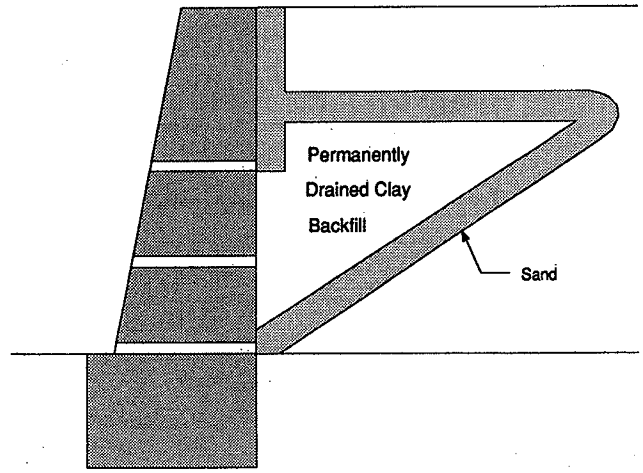


Figure 3.7. Drainage system for clay type backfill. (After Terzaghi and Peck, 1948)

3.5.5 Junctions of Wingwalls and Abutments

Wingwalls are used to retain the roadway embankment and to protect it from erosion. Wingwalls are usually designed as cantilever retaining walls subject to the earth pressure produced by the retained soil.

Attention should be paid to the junction of the wingwall and abutment to prevent cracks caused by differential movements between them. Differential movements may occur because the abutment supports loads from the bridge superstructure, and may thus have a tendency to settle and move laterally more than the wingwall.

The current AASHTO specification (1989) requires that reinforcement be spaced across the junction between abutments and wingwalls to connect them structurally. It also specifies the use of appropriate development lengths for the bars embedded in each side of the joint, and that the length of bars be varied to avoid planes of weakness in the concrete at each end. If no bars are used in the junction, an expansion joint with a shear key should be provided.

3.5.6 Frost

Water in soil expands when it freezes and ice lenses may develop, resulting in frost heave in the frozen soil. Foundations should be built below the frost line to avoid possible frost damage.

The maximum frost depth can be estimated from local experience, or from a frost-depth contour map such as the one shown in Figure 3.8. Figure 3.9 may also be used to estimate approximate frost depth using a freezing index proposed by Corps of Engineers (1949) and Brown (1964).

$$\text{Freezing Index} = N_{32^\circ F} \times (32^\circ F - T) \qquad (3.5.6.1)$$

where $N_{32^\circ F}$ = number of days below 32°F, and T = annual average daily temperature in Fahrenheit.

For example, if $N_{32^\circ F}$ is 25 days and T is 22°F for a particular

site, the freezing index is (25 days) (32° − 22°) = 250. From Figure 3.9, the corresponding frost depth is obtained to be about 20 in.

3.5.7 Seasonal Volume Change

Volume changes due to seasonal moisture variations occur in clayey soils, particularly in areas subject to a long dry season and periodic heavy rains. Seasonal volume change has been correlated with Plasticity Index, I_p. Figure 2.6 in Part 1 shows the relationship between Plasticity Index and volume change potential.

3.5.8 Undermining of Abutments by Scour

Bridge abutments and piers in streams, flood plains, or adjacent to water are subject to undermining by scouring action. This scouring action has caused many bridge failures.

Three types of scour in a river are shown in Figure 3.10 (Sowers, 1962). The first type of scour (a) is due to the lateral shifting of the channel. Continuous scouring occurs at the outside of each bend in a meandering river because of higher velocity of the stream, while sedimentation occurs at the inside of each bend. The abutment close to the outside of a bend should be protected from undermining by placing concrete or asphaltic mats over the river bank or by founding the abutment below the greatest possible depth of scour.

The second type of scour (b) is due to the erosion of the river bed which occurs during periods of high flow. High velocity flow moves heavy bed materials, so that the bed is lowered. The scouring action is especially large where the width of the flow path is narrow. In general, this normal scour is proportional to the rise in the water surface, and therefore the maximum depth of scour can be estimated by observation of the river bed during periods of high flow.

The third type of scour (c) is the localized and accelerated scour that results from such obstructions as bridge piers, which cause contraction of the channel cross section, and generates higher velocity of flow. The depth of scour in this case depends on factors such as the configuration of the pier, the angle between the flow and the pier, the contraction of the waterway, and the volume of debris that may be caught by the bridge.

Detailed information on scour prediction can be obtained from the FHWA Technical Advisory T5140.20 titled "Scour at Bridges" (1988).

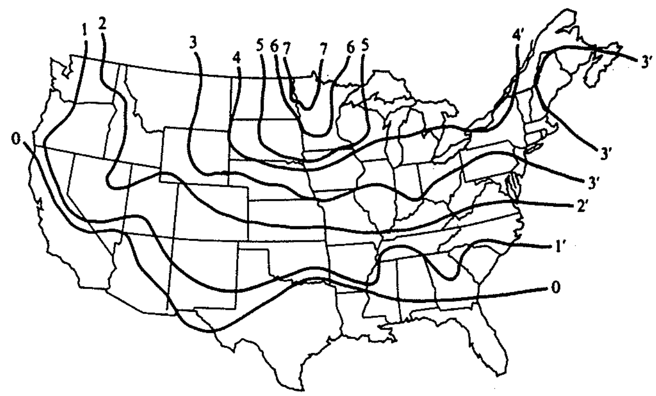
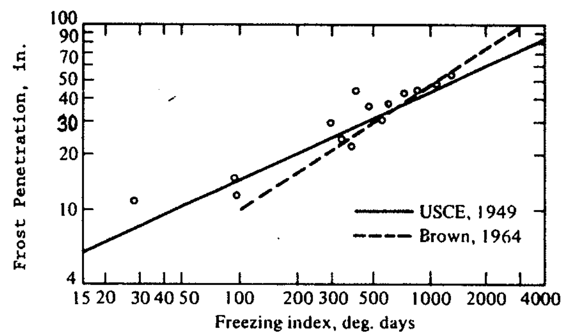


Figure 3.8. Approximate frost-depth contours for the United States. (From Bowles, 1989)



Freezing Index = No. of days x (32° - avg. daily temp. °F)

Figure 3.9. Design curves for maximum frost penetration based on the freezing index. (From Corps of Engineers 1949, and Brown 1964)

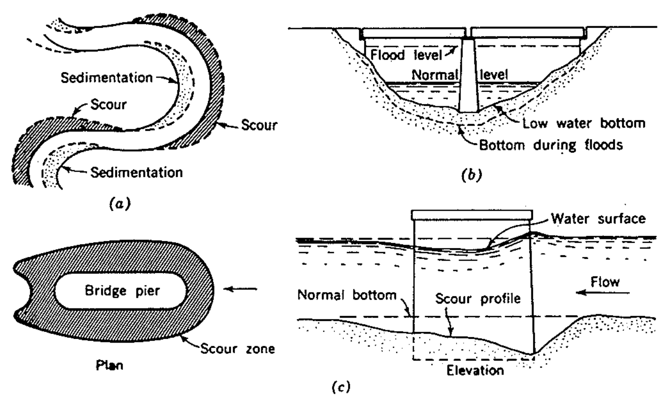


Figure 3.10. Forms of scour in rivers: (a) lateral shift of a stream caused by bank erosion and deposition; (b) normal bottom scour during floods; and (c) accelerated scour caused by a bridge pier. (From Sowers, 1962)

CHAPTER 4

FORCES ON RETAINING WALLS AND ABUTMENTS

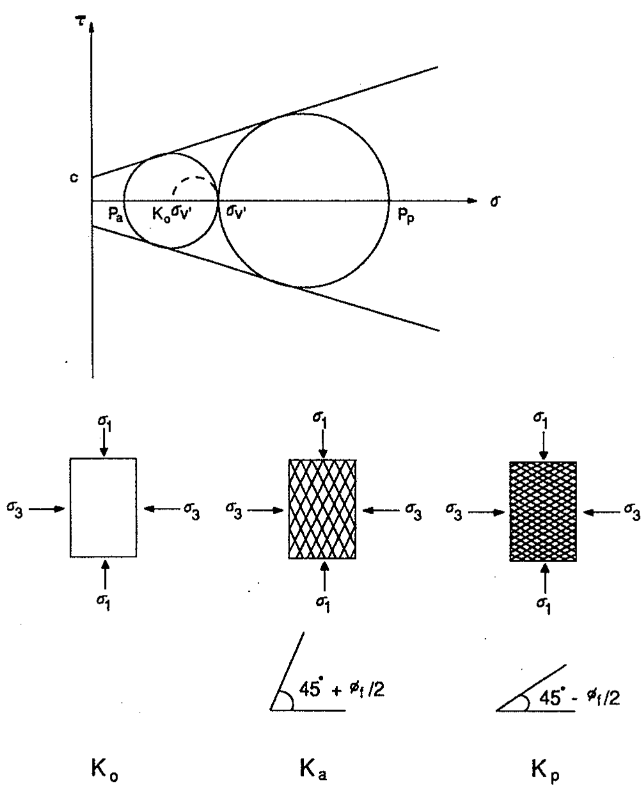
4.1 GENERAL

This chapter is concerned with the considerations involved in estimating static and seismic earth pressures on retaining walls and abutments.

4.2 STATIC EARTH PRESSURE

Static earth pressures exerted on a retaining wall can be classified into at-rest, active, and passive. The stresses at any point within a soil mass can be illustrated by Mohr circles, as shown in Figure 4.1, on a coordinate which consists of effective normal stress, σ_v' and shear stress, τ.

The at-rest condition develops when any possible combinations of normal stresses and shear stresses in a soil mass are within the limiting envelope (the dotted half-circle in Figure 4.1).



Where P_a : Static active earth pressure force,
P_p : Static passive earth pressure force,
K_o : Coefficient of at-rest earth pressure,
K_a : Coefficient of static active earth pressure,
K_p : Coefficient of static passive earth pressure.
c : Cohesion

Figure 4.1. Mohr's circles for earth pressure coefficients.

Failure conditions, called states of limiting or plastic equilibrium, may evolve as the maximum shear strength is fully mobilized. These states are known as active and passive conditions, and they occur when shear stresses reach the maximum possible values due to lateral extension or compression in a soil mass.

Each of these earth pressure states corresponds to different conditions with regard to the direction and the magnitude of wall movement, as discussed below.

4.2.1 At-Rest Earth Pressure

Walls that do not move, or that move very little, are acted on by at-rest earth pressures. At-rest pressures increase linearly with depth (see Figure 4.2) and can be expressed as:

$$p_h = K_o \gamma z \qquad (4.2.1.1)$$

where p_h = at-rest pressure, force/length^2; K_o = coefficient of lateral earth pressure at-rest; γ = unit weight of soil, force/length^3 (for depths below water table use submerged unit-weight,

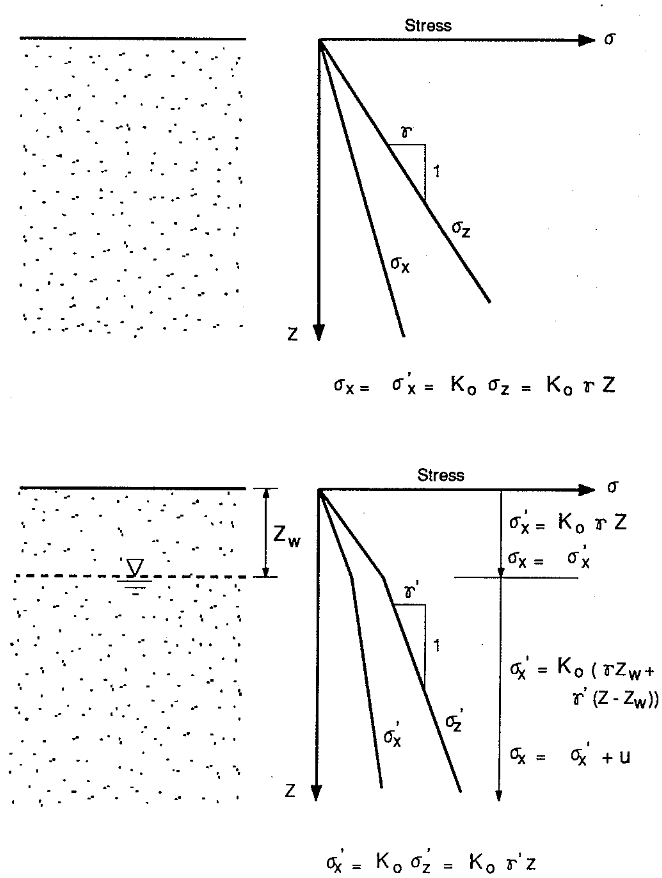


Figure 4.2. At-rest earth pressure distribution, homogeneous soil. (After Clough and Duncan, 1991)

γ'); and z = depth below the surface of the fill, length.

For normally consolidated soils, the coefficient, K_o, can be estimated by the following empirical correlation (Jaky, 1944):

$$K_o = 1 - \sin \phi' \tag{4.2.1.2}$$

where ϕ' = drained friction angle of soil, degrees.

Because Eq. 4.2.1.2 is based on level backfills, the following equation for sloping backfills is recommended by the Danish Code (1978):

$$K_{oi} = K_o (1 + \sin i) \tag{4.2.1.3}$$

where $K_{oi} = K_o$ for sloping backfills, K_o = coefficient of earth pressure at-rest for normally consolidated soils (Eq. 4.2.1.2) and i = the sloping angle of backfill, degrees.

For overconsolidated soils, values of coefficient of at-rest earth pressure can be estimated using the empirical relationship (Mayne and Kulhawy, 1982):

$$K_{ou} = K_o (OCR)^{\sin \phi'} \tag{4.2.1.4}$$

where K_{ou} = coefficient of at-rest earth pressure for overconsolidated soils; K_o = coefficient of earth pressure at-rest for normally consolidated soils (Eq. 4.2.1.2); ϕ' = drained friction angle of soil, degrees; and OCR = overconsolidation ratio, or maximum previous vertical pressure divided by current effective vertical pressure.

Typical values of K_{ou} corresponding to various values of OCR are presented in Table 4.1.

Table 4.1. Typical coefficients of lateral earth pressure at-rest. (After Clough and Duncan, 1991)

		Coefficient of Lateral Earth Pressure			
Soil Type	ϕ_f	OCR = 1	OCR = 2	OCR = 5	OCR = 10
Loose Sand	33.5°	0.45	0.65	1.10	1.50
Medium Sand	36.5°	0.40	0.60	1.05	1.55
Dense Sand	40.5°	0.35	0.55	1.00	1.50
Silt	29.5°	0.50	0.70	1.10	1.60
Lean Clay, CL	23.5°	0.60	0.80	1.20	1.65
Highly Plastic Clay, CH	20.5°	0.65	0.80	1.10	1.40

Table 4.2. Approximate magnitudes of movements required to reach minimum active and maximum passive earth pressure conditions. (After Clough and Duncan, 1991)

	Values of Δ/H *	
Type of Backfill	Active	Passive
Dense sand	0.001	0.01
Medium dense sand	0.002	0.02
Loose sand	0.004	0.04
Compacted silt	0.002	0.02
Compacted lean clay	0.01 **	0.05 **
Compacted fat clay	0.01 **	0.05 **

* Δ = movement of top of wall required to reach minimum active or maximum passive pressure, by tilting or lateral translation

H = height of wall

** Under stress conditions close to the minimum active or maximum passive earth pressures, cohesive soils creep continually. The movements shown would produce active or passive pressures only temporarily. With time the movements would continue if pressures remain constant. If movement remains constant, active pressures will increase with time, approaching the at-rest pressure, and passive pressures will decrease with time, approaching values on the order of 40% of the maximum short-term passive pressure.

4.2.2 Wall Movements and Earth Pressures

The earth pressures that act on a wall vary with wall displacement. That is, when a retaining wall moves away from the backfill, the earth pressure decreases (active pressure) and when it moves toward the backfill, the earth pressure increases (passive pressure). Table 4.2, obtained through experimental data and finite element analyses (Clough and Duncan, 1991), gives approximate magnitudes of wall movements required to reach minimum active and maximum passive earth pressure conditions. Observations from the table can be summarized as follows (Clough and Duncan, 1991): (1) The required movements for the extreme conditions are approximately proportional to the wall height. (2) The movement required to reach the maximum passive pressure is about 10 times as great as that required to reach the minimum active pressure, for walls of the same height. (3) The movement required to reach the extreme conditions for dense and incompressible soils is smaller than those for loose and compressible soils.

For any cohesionless backfill, conservative and simple guidelines for the maximum movements required to reach the extreme cases are provided by Clough and Duncan (1991). For example, for minimum active pressure the movement is no more than about 1 in. in 20 ft (Δ/H = 0.004); and for maximum passive pressure, about 1 in. in 2 ft (Δ/H = 0.04).

As shown in Figure 4.3, the value of the earth pressure coefficient varies with wall displacement and eventually remains constant after sufficiently large displacements. The change of pressures also varies with the type of soils, i.e., the pressures in the dense sand change more quickly with wall movement.

The earth pressure coefficients shown in Figure 4.4 are for a backfill compacted to a medium dense condition. Compared with the values in Figure 4.3, the curve is shifted upward so that the required movement for the minimum active earth pressure is increased and the required movement for the maximum passive earth pressure is decreased. However, in spite of the effect of compaction, the conservative guidelines of 1 in. in 20 ft for active condition and 1 in. in 2 ft for passive condition still can be used to estimate the required movements for the extreme earth pressure conditions.

4.2.3 Methods for Estimating K_a and K_p

Coulomb (1776) and Rankine (1856) developed simple methods for calculating the active and passive earth pressures exerted on retaining structures. Caquot and Kerisel (1948) developed the more generally applicable log spiral theory. Where the move-

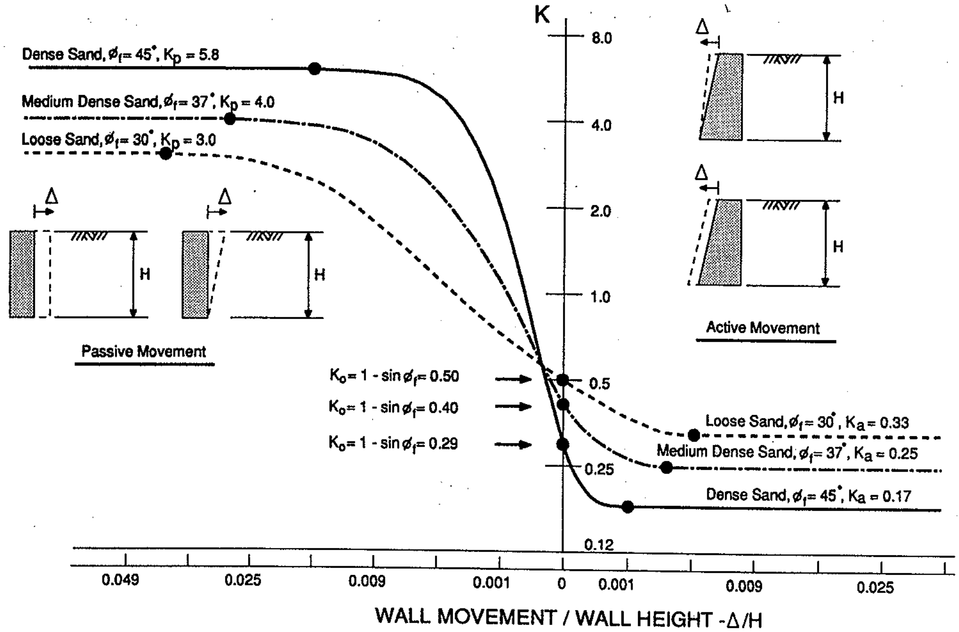


Figure 4.3. Relationship between wall movement and earth pressure. (After Clough and Duncan, 1991)

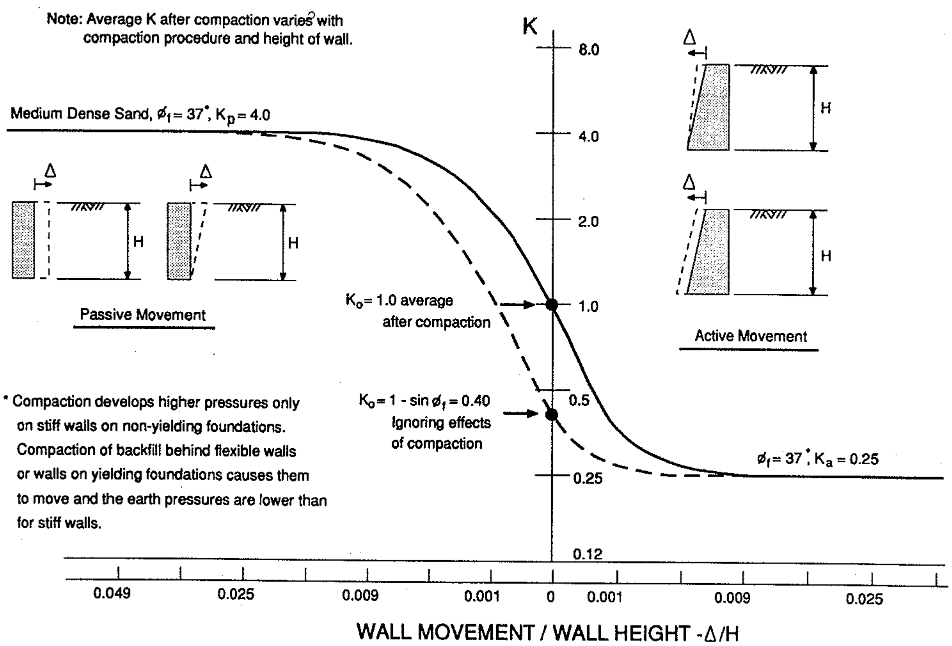


Figure 4.4. Relationship between wall movement and earth pressure for a wall with compacted backfill. (After Clough and Duncan, 1991)

ments of walls are sufficiently large so that the shear strength of the backfill soil is fully mobilized, and where the strength properties of the backfill can be estimated with sufficient accuracy, these methods of calculation are useful for practical purposes.

Coulomb's trial wedge method can be used for irregular backfill configurations, and Rankine's theory and the log spiral analysis can be used for more regular configurations. Each of these methods will be discussed next.

4.2.4 Coulomb Theory

The Coulomb theory, the first rational solution to the earth pressure problem, is based on the concept that the lateral force exerted on a wall by the backfill can be evaluated by analysis of the equilibrium of a wedge-shaped mass of soil bounded by the back of the wall, the backfill surface, and a surface of sliding through the soil. The assumptions in this analysis are: (1) the surface of sliding through the soil is a straight line; (2) the full strength of the soil is mobilized to resist sliding (shear failure) through the soil.

Active Pressure. A graphical illustration for the mechanism of active failure according to the Coulomb theory is shown in Figure 4.5(a). The active earth pressure force can be expressed as:

$$P_a = \frac{1}{2}\gamma H^2 \frac{\cos^2(\phi_f - \beta)}{\cos^2\beta\,\cos(\beta + \delta)\left[1 + \sqrt{\dfrac{\sin(\phi_f + \delta)\sin(\phi_f - i)}{\cos(\beta + \delta)\cos(\beta - i)}}\right]^2} \quad (4.2.4.1)$$

where P_a = active earth pressure force, force/length, $= \frac{1}{2}\gamma H^2 K_a$; K_a = coefficient of active earth pressure; γ = unit weight of backfill soil, force/length^3; H = wall height, length; ϕ_f = the internal friction angle of soil, degrees; β = the slope of stem face, degrees; δ = the friction angle between wall and soil, degrees; and i = the slope of backfill surface, degrees.

Passive Pressure. The Coulomb theory can be used to evaluate passive resistance, using the same basic assumptions. Figure 4.5(b) shows the failure mechanism for the passive case. The passive earth pressure force, P_p, can be expressed as follows:

$$P_p = \frac{1}{2}\gamma H^2 \frac{\cos^2(\phi_f + \beta)}{\cos^2\beta \cos(\beta - \delta)\left[1 - \sqrt{\frac{\sin(\phi_f + \delta)\sin(\phi_f + i)}{\cos(\beta - \delta)\cos(\beta - i)}}\right]^2} \quad (4.2.4.2)$$

The basic assumption in the Coulomb theory is that the surface of sliding is a plane. This assumption does not affect appreciably the accuracy for the active case. However, for the passive case, values of P_p calculated by the Coulomb theory can be much larger than can actually be mobilized, especially when the value of δ exceeds about half of ϕ_f.

Wall Friction. Friction between the wall and backfill has an important effect on the magnitude of earth pressures and an even more important effect on the direction of the earth pressure force. Table 4.3 presents values of the maximum possible wall friction angle for various wall materials and soil types.

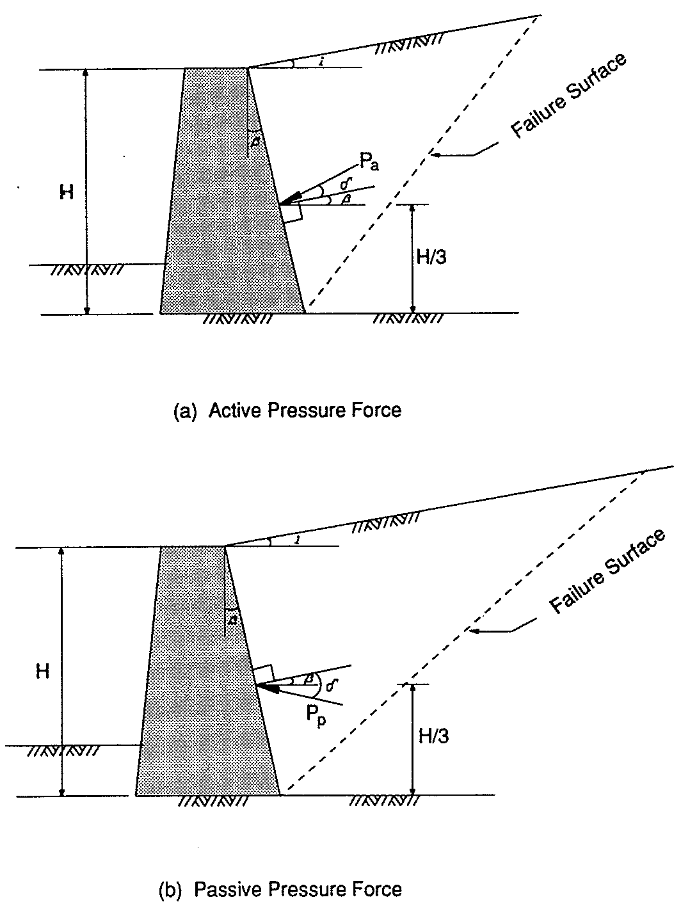


Figure 4.5. Coulomb theory for active and passive earth pressures.

Table 4.3. Ultimate friction factors, friction angles, and adhesion for dissimilar materials. (From NAVFAC, 1982)

INTERFACE MATERIALS	FRICTION FACTOR tan δ	FRICTION ANGLE, δ (degrees) (1)
Mass concrete on the following foundation materials:		
- Clean sound rock	0.70	35
- Clean gravel, gravel-sand mixtures, coarse sand	0.55 to 0.60	29 to 31
- Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55	24 to 29
- Clean fine sand, silty or clayey fine to medium sand	0.35 to 0.45	19 to 24
- Fine sandy silt, nonplastic silt	0.30 to 0.35	17 to 19
- Very stiff and hard residual or preconsolidated clay	0.40 to 0.50	22 to 26
- Medium stiff and stiff clay and silty clay	0.30 to 0.35	17 to 19
(Masonry on foundation materials has same friction factors)		
Steel sheet piles against the following soils:		
- Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40	22
- Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30	17
- Silty sand, gravel or sand mixed with silt or clay	0.25	14
- Fine sandy silt, nonplastic silt	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
- Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40 to 0.50	22 to 26
- Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30 to 0.40	17 to 22
- Silty sand, gravel or sand mixed with silt or clay	0.30	17
- Fine sandy silt, nonplastic silt	0.25	14
Various structural materials:		
- Masonry on masonry, igneous and metamorphic rocks:		
* Dressed soft rock on dressed soft rock	0.70	35
* Dressed hard rock on dressed soft rock	0.65	33
* Dressed hard rock on dressed hard rock	0.55	29
- Masonry on wood (cross grain)	0.50	26
- Steel on stee at sheet pile interlocks	0.30	17

INTERFACE MATERIALS	SOIL COHESION, c (psf)	ADHESION, c_a (psf)
Very soft cohesive soil	(0 - 250)	0 - 250
Soft cohesive soil	(250 - 500)	250 - 500
Medium stiff cohesive soil	(500 - 1,000)	500 - 750
Stiff cohesive soil	(1,000 - 2,000)	750 - 950
Very stiff cohesive soil	(2,000 - 4,000)	950 - 1,300

(1) δ = friction angle between two dissimilar materials.

4.2.5 Rankine Theory

The Rankine theory is applicable to conditions where the wall friction angle, δ, is equal to the slope of the backfill surface, i.

As in the case of the Coulomb theory, it is assumed that the strength of the soil is fully mobilized.

Active Pressure. The active earth pressure considered in the Rankine theory is illustrated in Figure 4.6(a) for a level backfill condition. The coefficient of active earth pressure, K_a, can be expressed as:

$$K_a = \cos i \frac{\cos i - \sqrt{\cos^2 i - \cos^2 \phi_f}}{\cos i + \sqrt{\cos^2 i - \cos^2 \phi_f}} \tag{4.2.5.1}$$

When the ground surface is horizontal, i.e., when $i = 0$, K_a can be expressed as:

$$K_a = \frac{1 - \sin \phi_f}{1 + \sin \phi_f} \tag{4.2.5.2}$$

The active earth pressure, p_a, can be expressed as:

$$p_a = K_a \gamma z - 2 c \sqrt{K_a} \tag{4.2.5.3}$$

where p_a = the active pressure, force/length²; K_a = the active pressure coefficient; γ = the unit weight of soil, force/length³; c = the cohesion, force/length²; and z = the depth below the ground surface, length.

The variation of active pressure with depth is linear, as shown in Figure 4.6(b). If the backfill is cohesive, the soil is theoretically in a tension zone down to a depth of $2\ c/\gamma\ \sqrt{K_a}$. However, a tension crack is likely to develop in that zone, and may be filled with water, so that hydrostatic pressure will be exerted on the wall, as shown in Figure 4.6(c).

Passive Pressure. The Rankine theory can also be applied to passive pressure conditions. The passive earth pressure coefficient, K_p, can be expressed as:

$$K_p = \cos i \frac{\cos i + \sqrt{\cos^2 i - \cos^2 \phi_f}}{\cos i - \sqrt{\cos^2 i - \cos^2 \phi_f}} \tag{4.2.5.4}$$

When the ground surface is horizontal, K_p can be expressed as:

$$K_p = \frac{1 + \sin \phi_f}{1 - \sin \phi_f} \tag{4.2.5.5}$$

The passive pressure at depth z can be expressed as:

$$p_p = K_p \gamma z + 2 c \sqrt{K_p} \tag{4.2.5.6}$$

where p_p = the passive pressure, force/length²; K_p = the passive pressure coefficient.

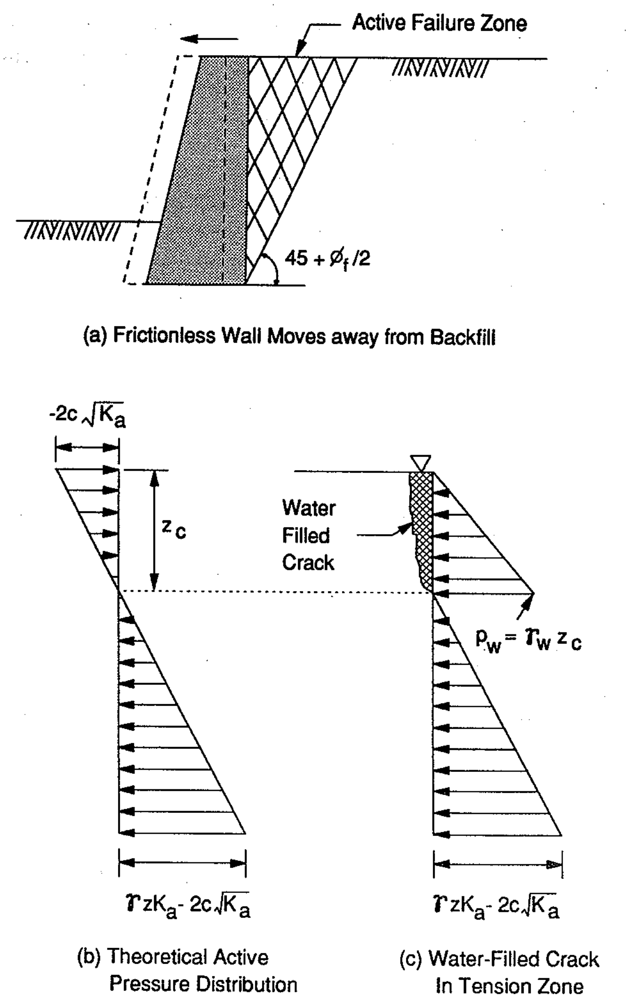


Figure 4.6. Active pressure, frictionless wall. (After Clough and Duncan, 1991)

4.2.6 Log Spiral Analysis

The failure surface in most cases is more closely approximated by a log spiral than a straight line, as shown in Figure 4.7.

Active and passive pressure coefficients, K_a and K_p, obtained from analyses using log spiral surfaces are given in Tables 4.4 and 4.5 (Caquot and Kerisel, 1948). Values of K_a and K_p for walls with level backfill and vertical stem are also shown in Figure 4.8. These values are based on the log spiral analyses performed by Caquot and Kerisel.

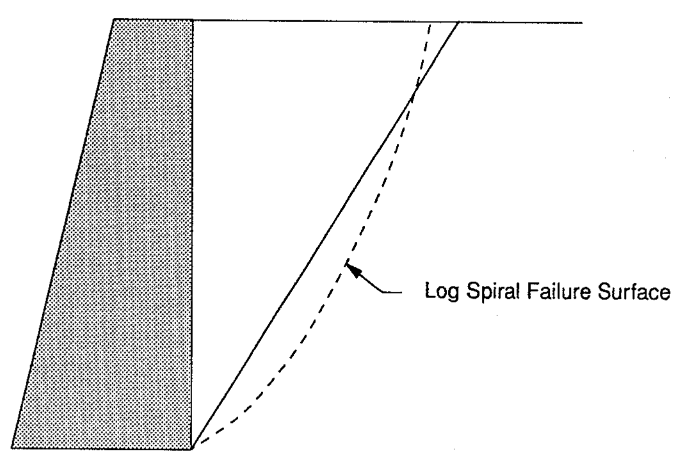


Figure 4.7. Comparison of log spiral and straight line failure surfaces for active conditions. (From Clough and Duncan, 1991)

Table 4.4. Values of K_a for log spiral failure surface. (After Caquot and Kerisel, 1948)

$\delta°$	$i°$	$\beta°$	$\phi_f°$					
			20	25	30	35	40	45
0	-15	-10	0.37	0.30	0.24	0.19	0.14	0.11
		0	0.42	0.35	0.29	0.24	0.19	0.16
		10	0.45	0.39	0.34	0.29	0.24	0.21
	0	-10	0.42	0.34	0.27	0.21	0.16	0.12
		0	0.49	0.41	0.33	0.27	0.22	0.17
		10	0.55	0.47	0.40	0.34	0.28	0.24
	15	-10	0.55	0.41	0.32	0.23	0.17	0.13
		0	0.65	0.51	0.41	0.32	0.25	0.20
		10	0.75	0.60	0.49	0.41	0.34	0.28
ϕ_f	-15	-10	0.31	0.26	0.21	0.17	0.14	0.11
		0	0.37	0.31	0.26	0.23	0.19	0.17
		10	0.41	0.36	0.31	0.27	0.25	0.23
	0	-10	0.37	0.30	0.24	0.19	0.15	0.12
		0	0.44	0.37	0.30	0.26	0.22	0.19
		10	0.50	0.43	0.38	0.33	0.30	0.26
	15	-10	0.50	0.37	0.29	0.22	0.17	0.14
		0	0.61	0.48	0.37	0.32	0.25	0.21
		10	0.72	0.58	0.46	0.42	0.35	0.31

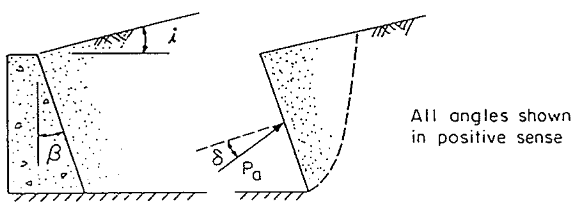
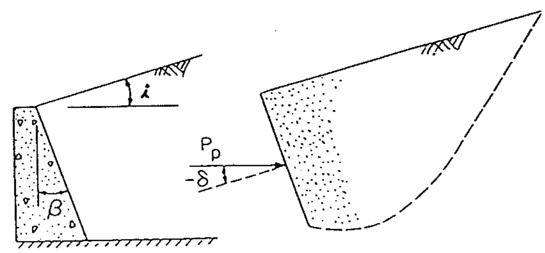


Table 4.5. Values of K_p for log spiral failure surface. (After Caquot and Kerisel, 1948)

$\delta°$	$i°$	$\beta°$	$\phi_f°$					
			20	25	30	35	40	45
0	-15	-10	1.32	1.66	2.05	2.52	3.09	3.95
		0	1.09	1.33	1.56	1.82	2.09	2.48
		10	0.87	1.03	1.17	1.30	1.33	1.54
	0	-10	2.33	2.96	3.82	5.00	6.68	9.20
		0	2.04	2.46	3.00	3.69	4.59	5.83
		10	1.74	1.89	2.33	2.70	3.14	3.69
	15	-10	3.36	4.56	6.30	8.98	12.2	20.0
		0	2.99	3.86	5.04	6.72	10.4	12.8
		10	2.63	3.23	3.97	4.98	6.37	8.2
$-\phi_f°$	-15	-10	1.95	2.90	4.39	6.97	11.8	22.7
		0	1.62	2.31	3.35	5.04	7.99	14.3
		10	1.29	1.79	2.50	3.58	5.09	8.86
	0	-10	3.45	5.17	8.17	13.8	25.5	52.9
		0	3.01	4.29	6.42	10.2	17.5	33.5
		-10	2.57	3.50	4.98	7.47	12.0	21.2
	15	-10	4.95	7.95	13.5	24.8	50.4	11.5
		0	4.42	6.72	10.8	18.6	39.6	73.6
		10	3.88	5.62	8.51	13.8	24.3	46.9



4.2.7 Selection of Earth Pressure Coefficients

Selecting a proper earth pressure coefficient is essential for a successful wall design. A number of methods previously discussed can be used to decide the magnitude of the coefficients. The procedures for estimating the coefficient using these methods are given in Example 1 in Chapter 6.

An alternative method is to use a hydrostatic fluid pressure which is equivalent to the product of an earth pressure coefficient and the unit weight of the soil. This equivalent fluid pressure method is discussed in Section 4.3 and illustrated in Examples 1 and 3 in Chapter 6.

A decision on what type of earth pressure coefficient should be used is based on the direction and the magnitude of the wall movement. Table 4.2, Figure 4.3 and Figure 4.4. can be useful in this procedure.

The New Zealand Ministry of Works and Development (1979) has recommended the following static earth pressure coefficients for use in design:

1. Counterfort or gravity walls founded on rock or piles: K_o
2. Cantilever walls less than 16 ft high founded on rock or piles: $0.5\ (K_o + K_a)$
3. Cantilever walls higher than 16 ft or any wall founded on a spread footing: K_a

The New Zealand Ministry of Works and Development (1979) also recommended that K_o should be used for the types of abutments that are not mentioned above. Earth pressures on integral abutments or framed abutments may be higher than active due to thermal movements of the bridge superstructure.

4.2.8 Location of Horizontal Resultant

In conventional designs and analyses, the horizontal resultant is assumed to be located at one-third of total height from the bottom of the wall. However, several experimental tests performed by many researchers including Terzaghi (1934), Clausen and Johansen (1972), and Sherif et al. (1982) have shown that the point of application is above the lower third point.

According to a study by Duncan et al. (1990), Terzaghi found that the resultant was applied at 0.40H to 0.45H from the bottom of the wall, where H is the total height of the wall. Clausen and Johansen found the same range of locations as Terzaghi's, and Sherif et al. recommended 0.42H for a wall in static active condition. However, Duncan et al. (1990) suggested that the location of the resultant should be assumed to be 0.40H above the bottom of the wall.

4.3 EQUIVALENT FLUID PRESSURE

Equivalent fluid pressures provide a convenient means of estimating design earth pressures, especially when the backfill material is a clayey soil.

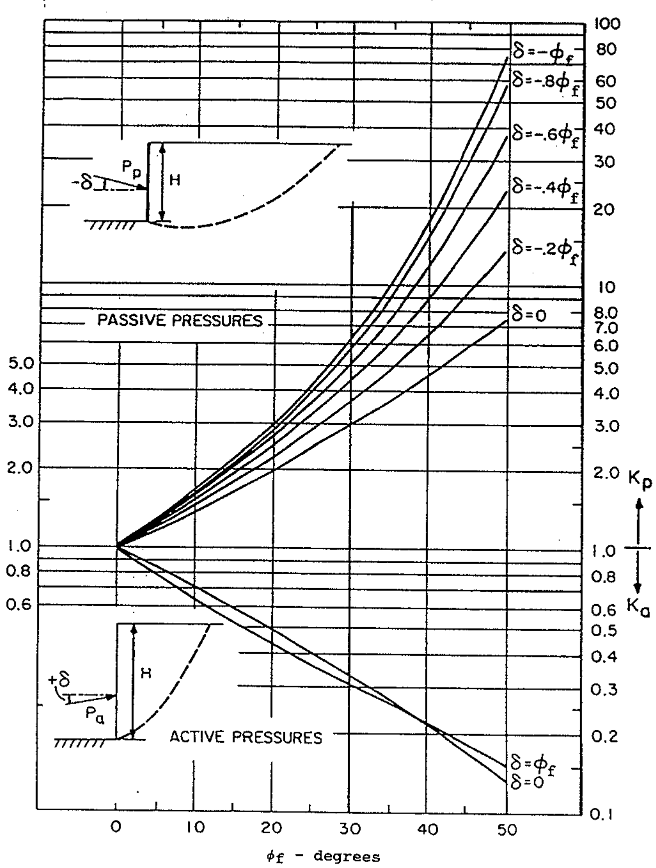


Figure 4.8. Active and passive pressure coefficients for vertical wall and horizontal backfill—based on log spiral failure surfaces. (After Caquot and Kerisel, 1948)

The lateral earth pressure at depth z can be expressed as:

$$p_h = \gamma_{eq}\, z \tag{4.3.1}$$

where p_h = lateral earth pressure, force/length^2; γ_{eq} equivalent fluid unit weight, force/length^3 (unit weight of a fluid that would exert the same pressure as the backfill soil); and z = depth below the surface of backfill, length.

Some typical equivalent fluid unit weights and corresponding pressure coefficients are given in Table 4.6. These are appropriate for use in designing walls up to about 20 ft in height. Values are presented for at-rest condition and for walls that can tolerate movements of 1 in. in 20 ft, and for level and sloped backfill.

4.4 EFFECT OF SURCHARGES

When vertical loads act on a surface of the backfill near a retaining wall or an abutment, the lateral and vertical earth pressure used for the design of the wall should be increased.

4.4.1 Uniform Surcharge Load

A surcharge load uniformly distributed over a large ground

Table 4.6. Coefficients and unit weights for equivalent fluid pressure. (From Clough and Duncan, 1991)

Type of Soil	Equivalent Fluid Unit Weights and Pressure Coefficients							
	Level Backfill				Backfill 2(H) on 1(V)			
	At-Rest		Δ/H = 1/240		At-Rest		Δ/H = 1/240	
	γ_{eq} (pcf)	K	γ_{eq} (pcf)	K	γ_{eq} (pcf)	K	γ_{eq} (pcf)	K
Loose Sand or Gravel	55	0.45	40	0.35	65	0.55	50	0.45
Medium Dense Sand or Gravel	50	0.40	35	0.25	60	0.50	45	0.35
Dense Sand or Gravel	45	0.35	30	0.20	55	0.45	40	0.30
Compacted Silt (ML)	60	0.50	40	0.35	70	0.60	50	0.45
Compacted Clay (CL)	70	0.60	45	0.40	80	0.70	55	0.50
Lean Compacted Fat Clay (CH)	80	0.65	55	0.50	90	0.75	65	0.60

$p_h = \gamma_{eq}\, z + K\, q_s$

where γ_{eq} = equivalent fluid unit weight,
z = depth below ground surface,
K = horizontal earth pressure coefficient,
q_s = uniform surcharge pressure.

surface area increases both the vertical and the lateral pressures. The increase in the vertical pressure, Δp_v, is the same as the applied surcharge pressure, q_s. That is,

$$\Delta p_v = q_s \tag{4.4.1.1}$$

and the amount of increase in the lateral pressure, Δp_h is

$$\Delta p_h = K\, q_s \tag{4.4.1.2}$$

where K = an earth pressure coefficient, dimensionless; K = K_a for active pressure; K = K_o for at-rest condition; and K = K_p for passive pressure.

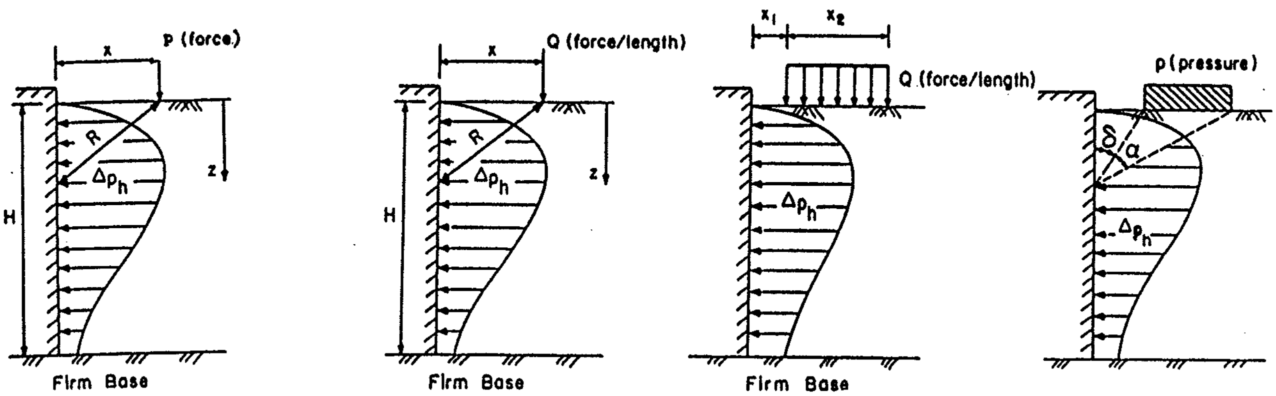
Because the applied area is infinitely large, the increases in both vertical and horizontal pressures are constant over the height of the wall. Therefore, the horizontal resultant force due to a surcharge load is located at mid-height of the wall.

4.4.2 Point Load, Line Load, and Strip Load

The theory of elasticity can be used to estimate the increased earth pressures induced by various types of surcharge loads. Equations for earth pressures due to point loads, line loads, and strip loads are presented in Figure 4.9.

4.5 EFFECT OF WATER

When water can pond behind a wall, the wall must be designed to withstand hydrostatic water pressure in addition to the earth pressure. The hydrostatic water pressure, p_w can be evaluated using the following expression:



(a) Point Load

(b) Infinitely long line load parallel to wall

(c) Finite line load perpendicular to wall

(d) Uniformly loaded strip parallel to wall

$$\Delta p_h = \frac{P}{\pi R} \frac{3Zx^2}{R^3} \frac{R(1-2\nu)}{R+Z}$$

$$\Delta p_h = \frac{4Q}{\pi} \frac{x^2 z}{R^4}$$

$$\Delta p_h = \frac{Q}{\pi z}\left(\frac{1}{\left[1+\left(\frac{z}{x_2}\right)^2\right]^{3/2}} - \frac{1-2\nu}{\left[1+\left(\frac{z}{x_2}\right)^2\right]^{1/2} + \frac{z}{x_2}} - \frac{1}{\left[1+\left(\frac{z}{x_1}\right)^2\right]^{3/2}} + \frac{1-2\nu}{\left[1+\left(\frac{z}{x_1}\right)^2\right]^{1/2} + \frac{z}{x_1}} \right)$$

$$\Delta p_h = \frac{2p}{\pi}\left(\alpha - \sin\alpha \cos(\alpha + 2\delta)\right)$$

where ν = Poisson's ratio.

Figure 4.9. Earth pressure due to point loads, line loads, and strip loads. (From Clough and Duncan, 1991)

$$p_w = \gamma_w{}^* (z - z_w) \qquad (4.5.1)$$

where γ_w = unit weight of water, force/length^3; z_w = depth to groundwater table, length; and z = depth below surface of backfill, length.

The submerged unit weight of the soil, γ', should be used to determine the lateral earth pressure below the groundwater table. Figure 4.10 shows the effect of ponded water on the pressures exerted on a retaining wall.

groundwater table, length; and z = depth below surface of backfill, length.

The submerged unit weight of the soil, γ', should be used to determine the lateral earth pressure below the groundwater table. Figure 4.10 shows the effect of ponded water on the pressures exerted on a retaining wall.

4.6 EARTH PRESSURES DUE TO COMPACTION

When heavy static or dynamic compaction equipment is used within a distance of one-half its height behind the wall, the effect of additional earth pressure produced by compaction should be considered. Detailed information on the procedures for estimating earth pressures due to compaction by rollers, vibratory plates, and rammers are presented in Chapter 6 of *Foundation Engineering Handbook* (Clough and Duncan, 1991).

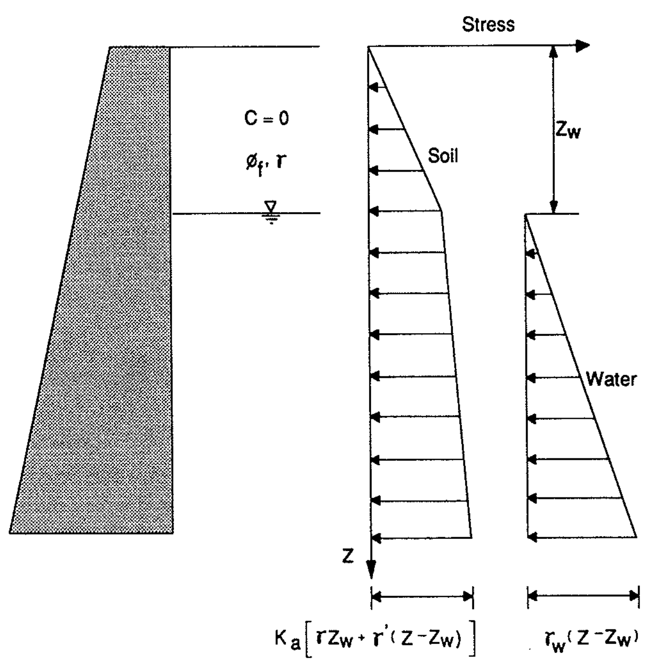


Figure 4.10. Active pressures for frictionless wall in presence of groundwater table. (From Clough and Duncan, 1991)

4.7 SEISMIC EARTH PRESSURE

Retaining walls and abutments in seismic areas may experience problems such as excessive settlement or slumping of the backfill and excessive movements of the wall during earthquakes. Theoretical and practical methods for analysis of the earthquake forces have been proposed by many investigators based on case histories, past experience, and experiments.

4.7.1 Mononobe-Okabe Theory

The most generally used method for obtaining seismic earth forces is a pseudostatic approach developed by Mononobe and Okabe in the 1920s. The Mononobe-Okabe (M-O) analysis is an extension of the sliding-wedge theory of Coulomb. The M-O method has been discussed in detail by many authors, including Seed and Whitman (1970) and Elms and Martin (1979). The method involves the following assumptions: (1) The wall is free to move sufficiently to mobilize minimum active pressure conditions. (2) The backfill is completely drained and cohesionless. (3) The failure surfaces are plane. (4) The accelerations are uniform throughout the soil mass.

4.7.2 Equations

The active and passive forces exerted on the wall because of the combined static and earthquake pressures are as follows.

For active pressure (Figure 4.11(a)),

$$P_{ae} = \frac{1}{2} K_{ae}\, \gamma\, (1 - k_v)\, H^2 \qquad (4.7.2.1)$$

where:

$$K_{ae} = \frac{\cos^2(\phi_f - \beta - \theta)}{\cos\theta \cos^2\beta \cos(\delta + \beta + \theta)\left[1 + \sqrt{\dfrac{\sin(\phi_f + \delta)\sin(\phi_f - i - \theta)}{\cos(\delta + \beta + \theta)\cos(i - \beta)}}\right]^2} \qquad (4.7.2.2)$$

For passive pressure (Figure 4.11(b)),

$$P_{pe} = \frac{1}{2} K_{pe}\, \gamma\, (1 - k_v)\, H^2 \qquad (4.7.2.3)$$

where:

$$K_{pe} = \frac{\cos^2(\phi_f + \beta - \theta)}{\cos\theta \cos^2\beta \cos(\delta - \beta + \theta)\left[1 - \sqrt{\dfrac{\sin(\phi_f + \delta)\sin(\phi_f + i - \theta)}{\cos(\delta - \beta + \theta)\cos(i - \beta)}}\right]^2} \qquad (4.7.2.4)$$

in which γ = unit weight of soil, force/length^3; k_v = vertical acceleration coefficient; k_h = horizontal acceleration coefficient; H = wall height, length; ϕ_f = soil friction angle, degrees; β = slope of wall stem, degrees; $\theta = \arctan[(k_h)/(1 - k_v)]$, degrees; δ = wall friction angle, degrees; and i = slope of backfill surface, degrees.

4.7.3 Location of Horizontal Resultant

The M-O analysis does not specify the location of the horizontal resultant. However, the height of the resultant measured from the bottom of the wall increases as the effect of the earthquake shaking increases. As shown in Figure 4.12, the static component P_a is placed at the lower one-third point of the wall while the dynamic component ΔP_{ae}, which is the difference between P_{ae} and P_a, is placed at the upper one-third. Considering this fact, it has been suggested by Elms and Martin (1979), and the commentary of AASHTO's guide for seismic design (1983), that the location of the total resultant force, P_{ae}, can be assumed to act at one-half of the wall height.

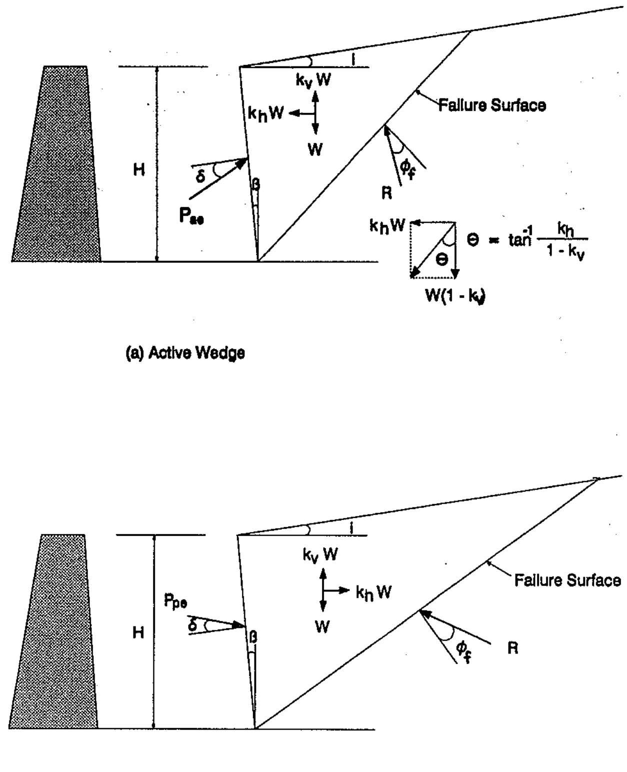


Figure 4.11. Active and passive seismic wedges.

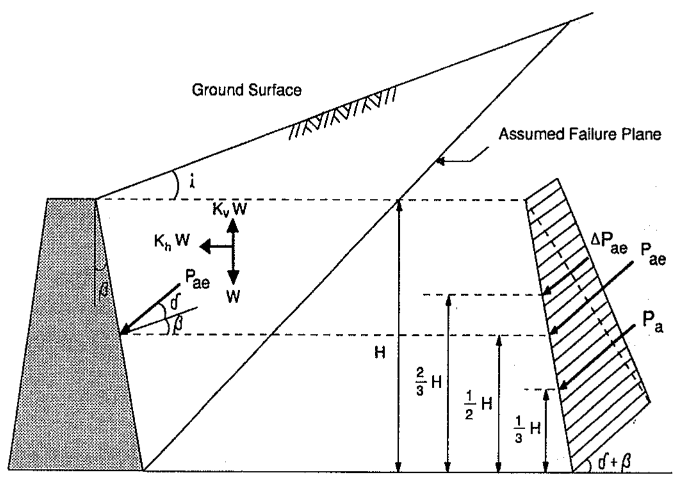


Figure 4.12. Location of resultant forces.

4.7.4 Selection of Acceleration Coefficients

Elms and Martin (1979) pointed out that it is desirable for an efficient design to allow some movement of the wall rather than no movement. The AASHTO seismic guide (1983) recommends that a value of $k_h = 0.5\alpha$ be used for most cases if the wall is designed to move up to 10α (in.) where α is peak ground acceleration coefficient for a site. However, if no horizontal displacement of the wall is allowed, the AASHTO guide (1983) recommends a coefficient of $k_h = 1.5\alpha$ be used.

As shown in Figure 4.13, the AASHTO bridge specifications (1989) classifies the United States into four zones according to the degree of seismic risk, using numerical values ranging from zero for nonseismic areas to three for areas subject to the most severe seismic effects. The corresponding values of A ($= \alpha$ g) are as follows: zone I—A = 0.09 g, zone II—A = 0.22 g, zone III—A = 0.50 g, where g = 32.2 ft/sec^2.

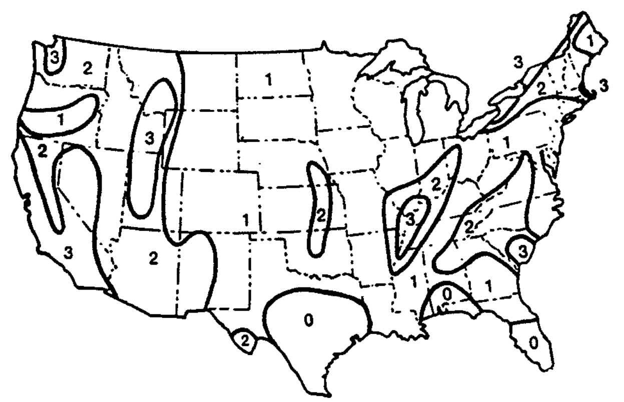


Figure 4.13. Seismic risk map of the United States. (From AASHTO, 1989)

CHAPTER 5

DESIGN REQUIREMENTS FOR RETAINING WALLS AND ABUTMENTS

5.1 GENERAL

Retaining walls and abutments are subject to various limit states or types of failure, as illustrated in Figure 5.1. Failures can occur within soils or the structural members. Sliding failure (Figure 5.1(a)) occurs when the lateral earth pressure exerted on the wall exceeds the frictional sliding capacity of the wall. If the bearing pressure is larger than the capacity of the foundation soil or rock, bearing failure (Figure 5.1(b)) results. Deep-seated sliding failure (Figure 5.1(c)) may develop in clayey soils. Structural failure (Figure 5.1(d)) also should be checked.

This chapter presents the conventional ASD methods and proposed LFD design requirements for stability of retaining walls and abutments. Design of individual structural elements of the wall will also be discussed. In Chapter 6, these design requirements will be illustrated with a series of sample problems.

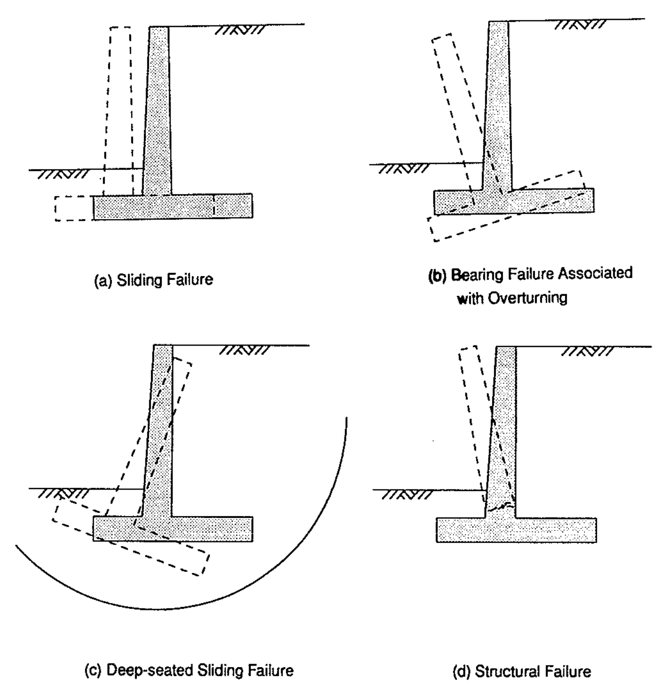


Figure 5.1. Failure modes for retaining walls.

5.2 BASIC DESIGN CRITERIA FOR RETAINING WALLS

For design purposes, retaining walls on spread footings can be classified into three categories (Duncan et al., 1990): (1) walls with clayey soils in the backfill or foundation, (2) walls with granular backfills and foundations of sand or gravel, and (3) walls with granular backfills and foundations on rock.

For each category, design procedures and stability criteria for the ASD method and the LFD method are summarized in Figures 5.2 through 5.4. The uniform pressure distributions shown in the figures are based on the method presented by Meyerhof (1953). Gravity walls are used in the figures, but the procedures and criteria are equally applicable to semigravity, cantilever, and other types of retaining walls or abutments that develop similar foundation pressures when subjected to lateral forces.

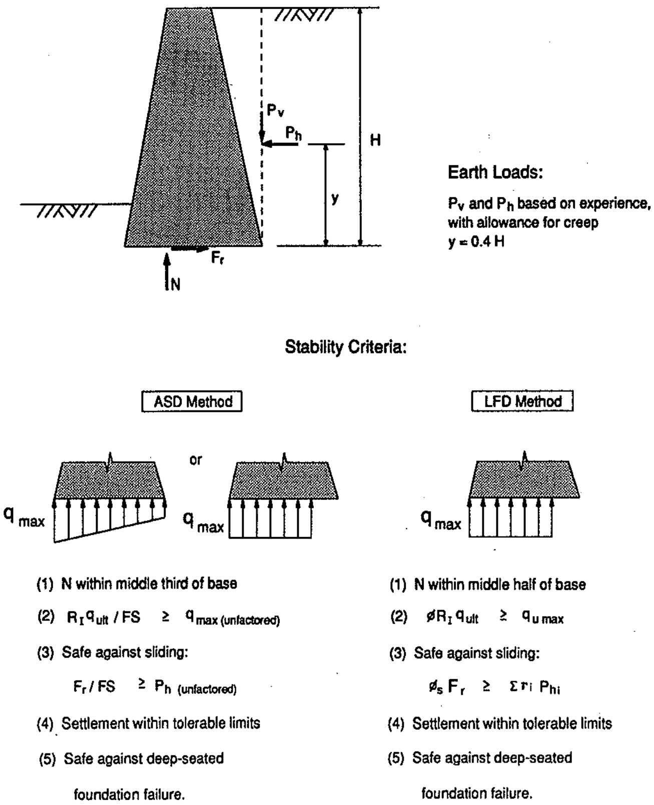


Figure 5.2. Earth loads and stability criteria for walls with clayey soils in the backfill or foundation. (After Duncan et al., 1990)

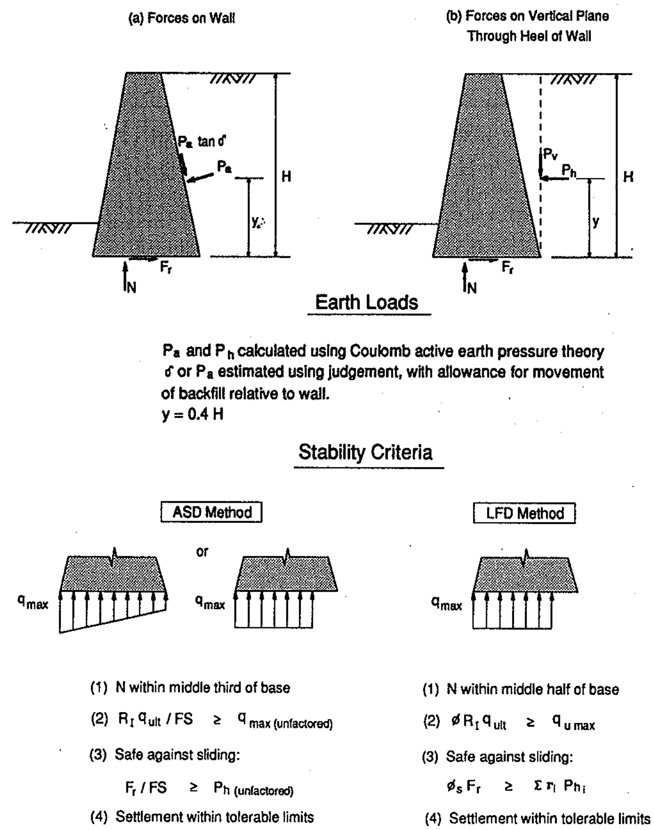


Figure 5.3. Earth loads and stability criteria for walls with granular backfills and foundations on sand or gravel. (After Duncan et al., 1990)

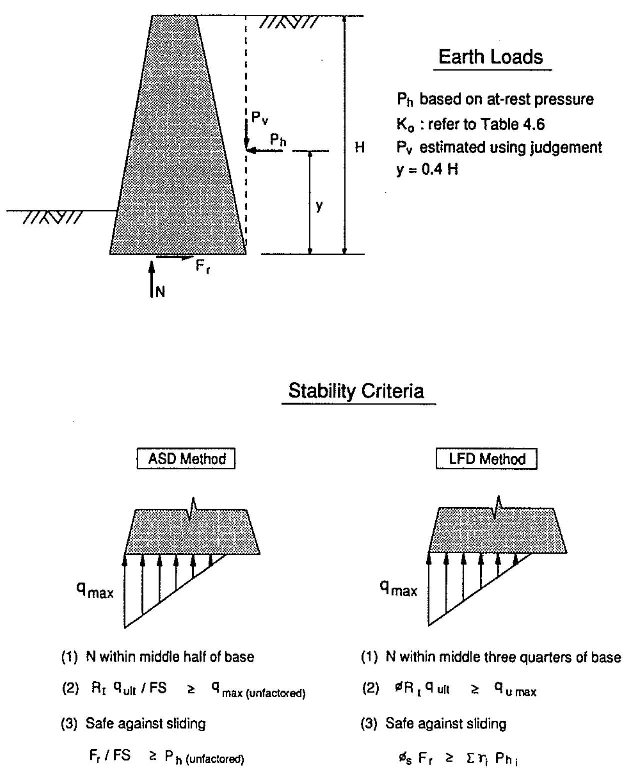


Figure 5.4. Earth loads and stability criteria for walls with granular backfills and foundations on rock. (After Duncan et al., 1990)

5.3 PROCEDURE FOR DESIGN OF RETAINING WALLS

In both ASD and LFD, a series of steps must be followed to obtain a satisfactory design. These are:

1. Select preliminary proportions of the wall.
2. Determine loads and earth pressures.
3. Calculate magnitude of reaction forces on base.
4. Check stability and safety criteria: (a) location of normal component of reactions, (b) adequacy of bearing pressure, and (c) safety against sliding.
5. Revise proportions of wall and repeat steps 2 through 4 until stability criteria are satisfied; then check: (a) settlement within tolerable limits and (b) safety against deep-seated foundation failure.
6. If proportions become unreasonable, consider a foundation supported on driven piles or drilled shafts.
7. Compare economics of completed design with other wall systems.

5.3.1 Step 1—Preliminary Proportions

Figure 5.5 shows commonly used dimensions for a gravity retaining wall and a cantilever wall. These proportions can be used when scour is not a concern to obtain dimensions for a first trial.

5.3.2 Step 2—Loads and Earth Pressures

Design loads for a retaining wall or an abutment are obtained by using group load combinations described in Table 3.2. Methods for calculating earth pressures exerted on the wall are discussed in Chapter 4. The use of equivalent fluid pressures presented in Table 4.6 gives satisfactory earth pressures if conditions are not unusual.

5.3.3 Step 3—Reaction Forces on Base

Figure 5.6 illustrates a typical cantilever wall subjected to various loads causing reaction forces that are normal to the base, N, and tangent to the base, F_r. These reaction forces are determined by simple statics for each load combination being investigated.

5.3.4 Step 4—Stability Criteria

1. The location of the resultant on the base is determined by balancing moments about the toe of the wall. The criteria for foundations on soil for the location of the resultant is that it must lie within the middle one-third for ASD and the middle-half for LFD. This criterion replaces the check on the ratio of stabilizing moment to overturning moment. For foundations on rock, the acceptable location of the resultant has a greater range than for foundations on soil (see Figure 5.4).

As shown in Figure 5.6, the location of the resultant, X_o, is obtained by:

$$X_o = \frac{\text{Summation of moments about point o}}{N} \qquad (5.3.4.1)$$

where N = the vertical resultant force, force/length.

The eccentricity of the resultant, e, with respect to the centerline of the base is:

$$e = \frac{B}{2} - X_o \qquad (5.3.4.2)$$

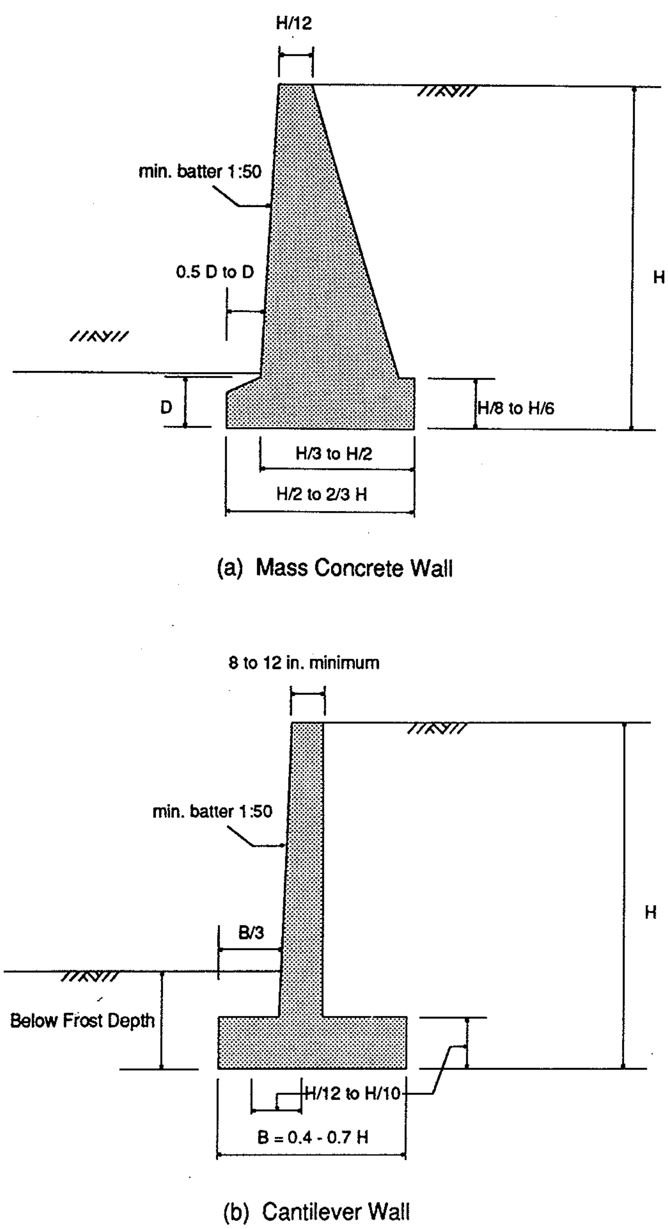


Figure 5.5. Preliminary dimensions for gravity walls and cantilever walls. (After Clayton and Milititsky, 1986)

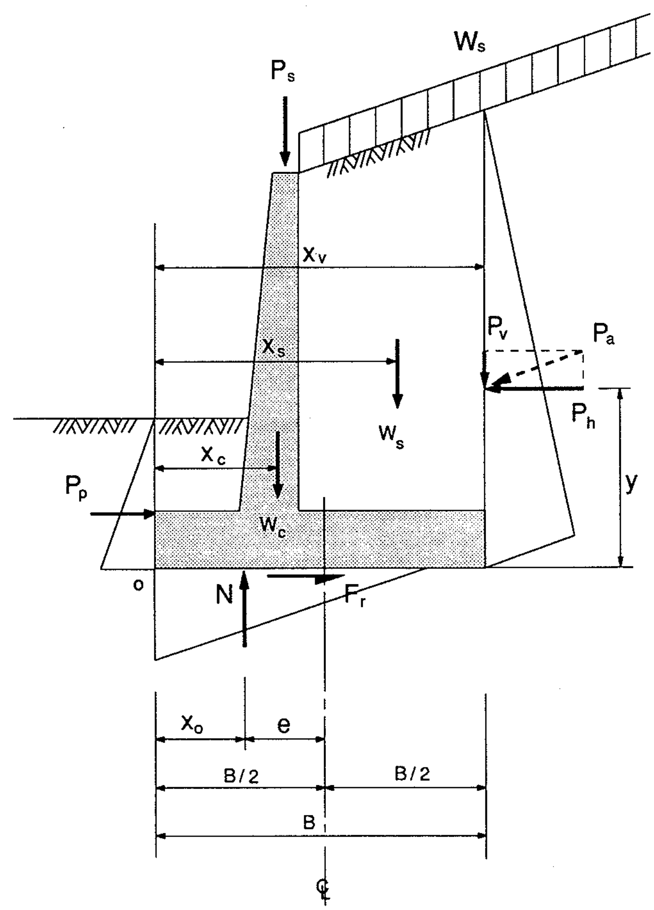


Figure 5.6. Forces on a typical retaining wall or abutment.

where B = base width, length.

2. Safety against bearing failure is obtained by applying a safety factor to the ultimate bearing capacity in the ASD method or by applying a performance factor to the ultimate bearing capacity in the LFD method. The ultimate bearing capacity can be calculated from in situ tests or semiempirical procedures as presented in the engineering manual for shallow foundations (Part 1).

Safety against bearing failure is checked by:

$$\textit{In ASD: } R_I q_{ult} / FS \geq q_{max} \tag{5.3.4.3}$$

$$\textit{In LFD: } \phi\ R_I q_{ult} \geq q_{umax} \tag{5.3.4.4}$$

where q_{ult} = ultimate bearing capacity, force/length^2; R_I = reduction factor due to inclined loads; FS = factor of safety; ϕ = performance factor; q_{max} = maximum bearing pressure due to unfactored loads, force/length^2, and q_{umax} = maximum bearing pressure due to factored loads, force/length^2.

Shape of Bearing Pressure Distribution. The shape of the bearing pressure distribution for soil foundations can be treated as a triangle, a trapezoid, or a rectangle; and for rock foundations, treated as a triangle or a trapezoid (see Figures 5.2 through 5.4). Therefore, the resultant, N, will pass through the centroid of a triangular or trapezoidal stress distribution or the middle of a uniformly distributed stress block, as shown in Figure 5.7.

Maximum Bearing Pressure. The following equations are used to compute the maximum soil pressures, q_{max} and q_{umax} (both indicated as $q_{(u)max}$, where (u) is not present in ASD), per unit length of a rigid footing.

For a triangular shape of bearing pressure:

when the resultant is within the middle third of base:

$$q_{(u)max} = \frac{N_{(u)}}{B} + \frac{6\ N_{(u)}\ e}{B^2} \tag{5.3.4.5}$$

when the resultant is outside of the middle third of base:

$$q_{(u)max} = \frac{2\ N_{(u)}}{3\ X_o} \tag{5.3.4.6}$$

For a uniform distribution of bearing pressure:

$$q_{(u)max} = \frac{N_{(u)}}{2\ X_o} \tag{5.3.4.7}$$

where $N_{(u)}$ = unfactored (factored) vertical resultant, force/length; X_o = location of the resultant measured from toe, length; and e = eccentricity of $N_{(u)}$, length.

3. Sliding stability is checked in the ASD method by satisfying the following criteria:

$$\frac{F_r}{FS} \geq P_h \tag{5.3.4.8}$$

where FS = 1.5 for sands and 2.0 for clays whose shear strength is less than 0.5 times the normal pressure; F_r = frictional resistance, force/length, or, = N tan δ_b + c_a B_e; N = resultant on base required for vertical equilibrium, force/length; δ_b = friction

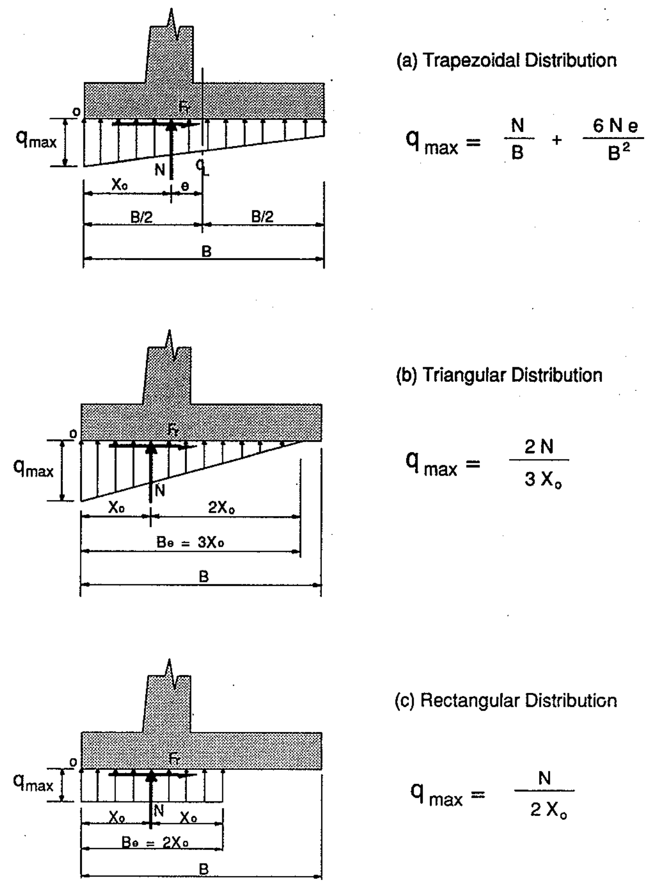


Figure 5.7. Various shapes of stress distributions and maximum bearing pressures.

angle between base and soil, degrees = ϕ_f of base soil for cast-in-place concrete, or = the values given in Table 4.3; c_a = adhesion, force/length^2; c = cohesion of the base soil, force/length^2; B_e = effective length of base in compression, length; and P_h = horizontal earth pressure force causing sliding, force/length.

In the LFD method, sliding stability is checked by

$$\phi_s\ F_{ru} \geq \Sigma\ \gamma_i\ P_{hi} \tag{5.3.4.9}$$

where F_{ru} = N_u tan δ_b + c_a B_e; N_u = factored vertical resultant, force/length; ϕ_s = performance factor for sliding (values given in Table 3.3); γ_i = load factor for force component i = $\gamma\beta_i$ shown in Table 3.2; and P_{hi} = horizontal earth pressure force i causing sliding, force/length.

As shown in Figure 5.6, the lateral pressure, P_h, causes the wall to slide and is resisted by friction and adhesion between the base and foundation soil. According to the proposed revision to the AASHTO Specifications prepared by D'Appolonia (1989), the passive earth pressure, P_p, generated by the soil in front of the wall may be included to resist sliding if it is ensured that the soil in front of the wall will exist permanently; for example, if the wall is embedded deeply below a covering such as a sidewalk or pavement. When passive earth pressure is used, a safety factor of 2.0 or larger is recommended by Lambe and Whitman (1969).

However, sliding failure occurs in many cases before the passive earth pressure is fully mobilized. Therefore, it is safer to ignore the effect of the passive earth pressure.

5.3.5 Step 5—Revise Proportions

When the preliminary wall dimensions are found inadequate, the wall dimensions should be adjusted by a trial and error method. A sensitivity study done by the authors shows that the stability can be improved by varying the location of the wall stem, the base width, and the wall height.

Some suggestions for correcting each stability or safety problem are presented as follows:

1. *Bearing failure or eccentricity criterion not satisfied:* (a) increase the base width; (b) relocate the wall stem moving it toward the heel; (c) minimize P_h by replacing a clayey backfill with granular material or by reducing porewater pressures behind the wall stem with a well-designed drainage system (see Section 3.5); and (d) provide an adequately designed reinforced concrete approach slab supported at one end by the abutment so that no horizontal pressure due to live load surcharge need be considered.
2. *Sliding stability criterion not satisfied:* (a) increase the base width; (b) minimize P_h as described above; (c) use an inclined base (heel side down) to increase horizontal resistance; (d) provide an adequately designed approach slab as mentioned above; and (e) use a shear key. (The main function of a shear key is to generate additional passive soil pressure to increase sliding resistance. The considerations discussed earlier concerning the amount of displacement required to mobilize passive earth pressure are also important with regard to the resistance that can be achieved through the use of a key. Depending on local experience or job conditions, the design engineer must decide whether some passive earth pressure should be included or not.)
3. *Settlement and overall stability check:* Once the proportions of the wall have been selected to satisfy the bearing pressure, eccentricity, and sliding criteria, the requirements on settlement and overall slope stability must be checked.

(a) Settlement should be checked for walls founded on compressible soils to ensure that the predicted settlement is less than the settlement that the wall or structure it supports can tolerate. The magnitude of settlement can be estimated using the methods described in the engineering manual for shallow foundations (Part 1). For a determination of the allowable settlement, refer to the engineering manual for estimating tolerable movements of bridges (Part 5).

(b) The overall stability of slopes with regard to the most critical sliding surface should be evaluated if the wall is underlain by weak soil. This check is based on limiting equilibrium methods which employ the modified Bishop, simplified Janbu or Spenser analysis. This subject is discussed in a number of design manuals and papers including NAVFAC DM-7.1 (1982a) and a manual by the Corps of Engineers (1989), and will not be covered in this manual.

5.3.6 Step 6—Consider Deep Foundations

Driven piles and drilled shafts can be used when the configuration of the wall is unreasonable or uneconomical. Engineering manuals for driven piles (Part 2) and drilled shafts (Part 4) may be consulted with regard to design of deep foundations to withstand vertical and lateral loads.

5.3.7 Step 7—Compare with Alternative Wall Systems

When a design is completed, it should be compared with other types of walls that may result in a more economical design. Detailed information can be found in Section 2.1.2 of this manual and Section 10 of a manual by the Corps of Engineers (1989).

5.4 DESIGN OF STRUCTURAL MEMBERS

5.4.1 Wall Stems

The wall stem of a cantilever wall is designed as a vertical cantilever supported by the base. The wall stem of a counterfort or buttress wall can be designed as a horizontal fixed or continuous beam supported by counterforts or buttresses. The wall can also be designed as a plate supported on three sides by the base slab and counterforts (or buttresses) and free at the top.

Wall stems are subjected to axial loads and bending. Axial loads are due to the stem weight, the loads from structures supported on the stem, and the vertical component of earth pressure exerted on the stem. Bending moments are generated from the horizontal component of earth pressure, surcharge loads, and eccentric vertical loads.

5.4.2 Base Slab

The base slab of a cantilever wall is designed as a cantilever fixed at the wall. The base slab of a counterfort or buttress wall is designed as a fixed or continuous beam spanning between the counterforts or buttresses.

The base of a cantilever wall is generally divided into two parts by the location of the wall stem: the edge of the base about which the wall stem tends to rotate is called the toe and the other edge is called the heel.

The maximum bending moments for the design of the base slabs are taken at the face of the wall stem for the toe and the back of the wall stem for the heel. The maximum design shear forces for the base slabs are taken at a distance equal to the effective depth of the slab from the face of the wall stem for the toe section and at the back of the wall stem for the heel section.

5.4.3 Counterforts and Buttresses

A counterfort undergoes tension when subject to active pressure and, thus, is designed as a T-beam, while a buttress undergoes compression and is designed as a rectangular beam. At the junction of the counterfort with the wall stem and the footing base, tension reinforcement should be provided with a combination of horizontal and vertical bars or stirrups.

5.4.4 Reinforcement

Detailed information on development of flexural reinforcement is given in Section 8, Part D, articles 8.17 through 8.32 in the current AASHTO specifications (1989).

Required reinforcement and bar spacing for temperature change and shrinkage are mentioned in Section 3.5.4 of this manual.

CHAPTER 6

DESIGN EXAMPLES

6.1 GENERAL

The design procedures presented in the previous chapters are demonstrated in three example problems. In each example, results from the ASD method are compared to those from the LFD method.

Example 1 concerns the design of a retaining wall to be built on a shallow foundation on competent rock. Because of the rock foundation, the bearing pressure distribution used in this example is triangular for both the ASD method and the LFD method. The design procedures are described in Section 5.3.

Example 2 is presented to illustrate a typical retaining wall with level backfill which is subjected to live load surcharge. The wall is founded on a shallow footing on sand. The bearing pressure distribution for this case is rectangular for both the ASD method and the LFD method.

Example 3 concerns a bridge abutment founded on a shallow foundation on medium dense sand. Rectangular distribution of the bearing pressure is used also in this example. Because the foundation soil is sand, the abutment tends to move vertically as well as laterally. With present information of this project (NCHRP Project 24-4), only vertical movement of retaining walls and abutments can be estimated.

As mentioned in Chapter 5, the effect of inclined loads on the ultimate bearing capacity must be considered in the design of wall footings. In Example 1, the lateral earth pressure is large, and therefore the inclined load effect is significant: the ultimate bearing capacity is reduced to about 30 percent due to load inclination.

In the current AASHTO specifications, different values of β_i are given so that possible critical load combinations are not overlooked. For example, when checking for maximum eccentricity in column design, $\beta_D = 0.75$ or $\gamma\beta_D \approx 1.0$. The implied philosophy is that if dead load tends to counteract a destabilizing effect, a lower load factor should be used. This philosophy has been incorporated in the examples of this manual by including additional load combinations that are identified by a lower case "a" suffix.

In regard to earth pressures, the current AASHTO specifications are very specific that β_E is 1.0 for vertical earth pressure. The authors recommend that this value be used even when the weight of a soil mass has a stabilizing effect. The value of β_E for horizontal earth pressure loads is 1.3. When vertical and horizontal components are taken of the same force, as in the case of active pressures due to a sloping backfill or an inclined wall surface, the authors recommend that the same load factor of $\beta_E = 1.3$ be used for both components.

6.2 EXAMPLE 1—DESIGN OF A RETAINING WALL

Problem Statement. The stability and safety criteria for a 25-ft high reinforced concrete cantilever wall retaining a gravel backfill is to be checked. The retaining wall is to be founded on a spread footing on rock. The surface of the backfill will slope at 1 (vertical) to 4 (horizontal). The angle of internal friction, ϕ_f, of the backfill is 35 deg, and the unit weight, γ, is 120 lb/ft^3. The bearing material is massive rock, and the estimated ultimate bearing capacity, q_{ult}, is 80 kip/ft^2. Because the footing is founded on rock, consideration for frost depth is not necessary.

6.2.1 Preliminary Wall Proportions

Based on the dimensions shown in Figure 5.5, a preliminary configuration was chosen, which is shown in Figure 6.1. The base width, B, is 13 ft and the thickness of the footing is 2.5 ft. The top thickness of the stem is 1 ft and the thickness of the stem base is 2.5 ft.

6.2.2 Determination of Loads and Earth Pressures

6.2.2.1 Load Combinations

The loadings in this retaining wall example are dead load and earth load. The general expression of the AASHTO group loading combinations is: Group N $= \gamma\,(\beta_D D + \beta_E E)$.

Group I is the governing group in this example for both the ASD and the LFD methods. Therefore, design loads are obtained by:

For ASD Group I $= 1.0\text{ D} + 1.0\text{ E}_v + 1.0\text{ E}_h$

For LFD (a) Group I $= 1.3\text{ D} + 1.3\text{ E}_v + 1.69\text{ E}_h$

(b) Group Ia $= (1.3)(0.75)\text{ D} + 1.3\text{ E}_v + 1.69\text{ E}_h$

where E_v and E_h represent the loads causing vertical earth pressure and horizontal earth pressure, respectively. The percentage of the basic unit stress for Group I in ASD is 100 percent.

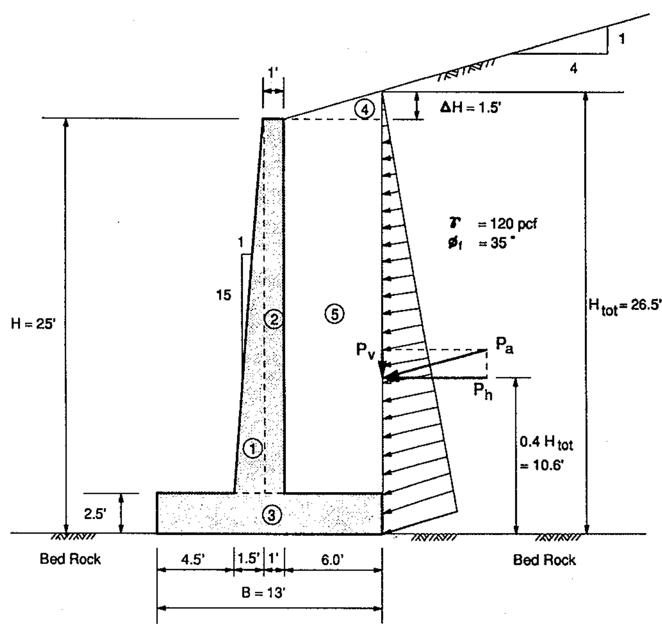


Figure 6.1. A cantilever retaining wall in Example 1.

6.2.2.2 Lateral Earth Pressure Force

A. Required wall movements. Assume that the tolerable lateral movement of the wall is as much as 2 in. after compaction: $\Delta/H = (2 \text{ in.})/(25 \text{ ft} \times 12) = 0.007$. From Table 4.2 and Figure 4.4, the approximate magnitude of movement required to reach minimum active for medium dense sand is 0.002. Therefore, active earth pressures can be used for design.

B. Various methods for estimating active earth pressures. (a) If the Rankine equation is used for calculating the lateral earth pressure force, with the backfill slope, i = arctan (¼) = 14 deg and internal friction angle, ϕ_f = 35 deg, then

$$K_a = \cos i \frac{\cos i - \sqrt{\cos^2 i - \cos^2\phi_f}}{\cos i + \sqrt{\cos^2 i - \cos^2\phi_f}} \quad \text{(from Eq. 4.2.5.1)}$$

$$= (0.9701)\frac{0.9701 - \sqrt{0.9411 - 0.6710}}{0.9701 + \sqrt{0.9411 - 0.6710}} = \underline{0.29}$$

$$P_a = 1/2\ \gamma\ K_a\ H_{tot}^2$$

$$= (1/2)(0.120)(0.29)(26.5)^2 = \underline{12.2} \text{ kip/ft}$$

where $H_{tot} = H + \Delta H = 25 \text{ ft} + 1.5 \text{ ft} = 26.5$ ft.

The vertical and horizontal components of P_a are:

$$P_v = P_a \sin i = (12.2)(0.242) = \underline{3.0} \text{ kip/ft}$$

$$P_h = P_a \cos i = (12.2)(0.970) = \underline{11.8} \text{ kip/ft}$$

(b) If the Coulomb equation is used, with δ = 14 deg (assumed to be the angle of backfill slope), i = 14 deg, β = 0, and ϕ_f = 35 deg, calculate from Eq. 4.2.4.1:

$$K_a = \underline{0.29}$$

$$P_a = 1/2\ \gamma\ K_a\ H_{tot}^2$$

$$= (1/2)(0.120)(0.29)(26.5)^2$$

$$= \underline{12.2} \text{ kip/ft}$$

$$P_v = P_a \sin \delta = (12.2)(0.242) = \underline{3.0 \text{ kip/ft}}$$

$$P_h = P_a \cos \delta = (12.2)(0.970) = \underline{11.8} \text{ kip/ft}$$

(c) If log spiral is used, with δ = 14 deg (assumed to be the angle of backfill slope), i = 14 deg, β = 0, and ϕ_f = 35 deg, read from Table 4.4:

$$K_a = \underline{0.32}$$

$$P_a = 1/2\ \gamma\ K_a\ H_{tot}^2$$

$$= (1/2)(0.120)(0.32)(26.5)^2 = \underline{13.5} \text{ kip/ft}$$

$$P_v = P_a \sin \delta = (13.5)(0.242) = \underline{3.3} \text{ kip/ft}$$

$$P_h = P_a \cos \delta = (13.5)(0.970) = \underline{13.1} \text{ kip/ft}$$

(d) If equivalent fluid pressure is used, read from Table 4.6 for medium dense sand or gravel:

γ_{eq} = 35 pcf for level backfill and γ_{eq} = 45 pcf for backfill 2(H) on 1(V). Interpolating the curve for backfill 4(H) on 1(V), estimate γ_{eq}:

$$\gamma_{eq} = \underline{40} \text{ pcf}$$

$$P_h = 1/2\ \gamma_{eq}\ H_{tot}^2$$

$$= (1/2)(0.040)(26.5)^2$$

$$= \underline{14.0} \text{ kip/ft}$$

$$P_v = P_h \tan i = (14.0)(0.249) = \underline{3.5} \text{ kip/ft}$$

$$P_a = (P_h^2 + P_v^2)^{1/2} = \underline{14.4} \text{ kip/ft}$$

All of these methods give essentially comparable values for active earth pressure coefficients. The equivalent fluid pressure methods give the largest horizontal component and vertical component of active earth pressure. However, in this example, the earth pressures from the Rankine equation will be used.

$$P_a = \underline{12.2} \text{ kip/ft}$$

$$P_v = \underline{3.0} \text{ kip/ft}$$

$$P_h = \underline{11.8} \text{ kip/ft}$$

The location of application of the horizontal resultant, y, is 0.4 H_{tot} as shown in Figure 6.1.

$$y = 0.4\ H_{tot} = (0.4)(26.5) = \underline{10.6} \text{ ft}$$

6.2.3 Calculation for Reaction Forces

The next step is to calculate reaction forces that are normal to the base, N, and tangent to the base, F_r.

Vertical and horizontal loads for ASD and LFD are tabulated, as follows. Numbered items refer to the elements of the wall and soil as shown in Figure 6.1.

6.2.3.1 The ASD Method

A. Vertical loads (unfactored):

Items	V_{unf} (k/ft)	Arm about point o (ft)	Moment about point o (k-ft/ft)
1. (1/2)(1.5)(22.5)(0.150)	2.53	5.5	13.92
2. (1.0)(22.5)(0.150)	3.38	6.5	21.97
3. (13.0)(2.5)(0.150)	4.88	6.5	31.72
4. (1/2)(6.0)(1.5)(0.120)	0.54	11.0	5.94
5. (6.0)(22.5)(0.120)	16.20	10.0	162.00
P_v	3.00	13.0	39.00
	$N_{unf} = \Sigma V_{unf}$ = 30.5		ΣM_{Vunf} = 274.6

B. Horizontal loads (unfactored):

Items	H_{unf} (k/ft)	Arm about point o (ft)	Moment about point o (k-ft/ft)
P_h	11.8	10.6	−125.1
	ΣH_{unf} = 11.8		ΣM_{Hunf} = −125.1

6.2.3.2 The LFD Method

A. Vertical loads (factored):

(a) when $\beta_D = 1.0$

Items	V_{unf} (k/ft)	Load Factor $\gamma\beta_i$	V_u (factored) (k/ft)	Arm about point o (ft)	Moment about point o (k-ft/ft)
1.	2.53	1.3	3.29	5.5	18.1
2.	3.38	1.3	4.39	6.5	28.6
3.	4.88	1.3	6.34	6.5	41.2
4.	0.54	1.3	0.70	11.0	7.7
5.	16.20	1.3	21.06	10.0	210.6
P_v	3.00	1.69*	5.07	13.0	65.9
		$N_u = \Sigma V_u = 40.9$			$\Sigma M_{Vu} = 372.1$

*Since P_v is the vertical component of the lateral earth pressure force, P_a, the load factor for E_h ($\gamma\beta_i = 1.3 \times 1.3 = 1.69$) is applied to P_v.

(b) when $\beta_D = 0.75$

Items	V_{unf} (k/ft)	Load Factor $\gamma\beta_i$	V_u (factored) (k/ft)	Arm about point o (ft)	Moment about point o (k-ft/ft)
1.	2.53	0.98	2.48	5.5	13.6
2.	3.38	0.98	3.31	6.5	21.5
3.	4.88	0.98	4.78	6.5	31.1
4.	0.54	1.3	0.70	11.0	7.7
5.	16.20	1.3	21.06	10.0	210.6
P_v	3.00	1.69*	5.07	13.0	65.9
		$N_u = \Sigma V_u = 37.4$			$\Sigma M_{Vu} = 350.4$

*Since P_v is the vertical component of the lateral earth pressure force, P_a, the load factor for E_h ($\gamma\beta_i = 1.3 \times 1.3 = 1.69$) is applied to P_v.

B. Horizontal loads (factored):

Items	V_{unf} (k/ft)	Load Factor $\gamma\beta_i$	V_u (factored) (k/ft)	Arm about point o (ft)	Moment about point o (k-ft/ft)
P_h	11.8	1.69	19.9	10.6	−211.4
			$\Sigma H_u = 19.9$		$\Sigma M_{Hu} = -211.4$

6.2.4 Stability and Safety Criteria Check

A number of stability and safety criteria have to be satisfied as follows.

6.2.4.1 Location of Normal Component of Reactions

For foundations on rock the eccentricity should be smaller than B/4 for ASD and 3B/8 for LFD.

A. The ASD method:

$$X_o = \frac{\Sigma M_{unf}}{\Sigma V_{unf}} = \frac{274.6 - 125.1}{30.5} = 4.90 \text{ ft}$$

$$e = B/2 - X_o$$
$$= 6.5 - 4.90 = \underline{1.60} \text{ ft} < B/4 = \underline{3.25} \text{ ft} \qquad \textbf{OK}$$

B. The LFD method:

(a) when $\beta_D = 1.0$

$$X_o = \frac{\Sigma M_u}{\Sigma V_u} = \frac{372.1 - 211.4}{40.9} = 3.93 \text{ ft}$$
$$e = B/2 - X_o$$
$$= 6.5 - 3.93 = \underline{2.57} \text{ ft} < 3B/8 = \underline{4.88} \text{ ft } \textbf{OK}$$

(b) when $\beta_D = 0.75$

$$X_o = \frac{\Sigma M_u}{\Sigma V_u} = \frac{350.4 - 211.4}{37.4} = 3.72 \text{ ft}$$
$$e = B/2 - X_o$$
$$= 6.5 - 3.72 = \underline{2.78} \text{ ft} < 3B/8 = \underline{4.88} \text{ ft } \textbf{OK}$$

6.2.4.2 Adequacy of Bearing Pressure

Since bearing material is rock, a triangular bearing pressure distribution is used.

A. The ASD method:

$$q_{max} = 2N_{unf}/3X_o = 2(30.5)/3(4.90) = \underline{4.2} \text{ ksf}$$
$$q_{ult(capacity)} = q_{ult}(R_I)/FS$$
$$= (80.0)(0.27)/2.5 = \underline{8.6} \text{ ksf} \qquad \textbf{OK}$$

where R_I is obtained from Table 4.2 of the shallow foundations manual (Part 1) as follows: for the ratio of $H_{unf}/V_{unf} = 11.8/30.5 = 0.38$, and for the ratio of $D_f/B_e = 0/13 = 0$, read $R_I = 0.27$ from the table.

B. The LFD method:

(a) when $\beta_D = 1.0$

$$q_{max} = 2N_u/3X_o = 2(40.9)/3(3.93) = \underline{6.9} \text{ ksf}$$
$$q_{ult(capacity)} = (\phi)(R_I)(q_{ult})$$
$$= (0.6)(0.27)(80.0) = \underline{12.96} \text{ ksf} \qquad \textbf{OK}$$

(b) when $\beta_D = 0.75$

$$q_{max} = 2N_u/3X_o = 2(37.4)/3(3.72) = \underline{6.7} \text{ ksf}$$
$$q_{ult(capacity)} = (\phi)(R_I)(q_{ult})$$
$$= (0.6)(0.27)(80.0) = \underline{12.96} \text{ ksf} \qquad \textbf{OK}$$

where ϕ = performance factor for bearing capacity on rock (Table 3.3); R_I applied in the LFD method is also based on unfactored loading conditions. Thus, the value is same as that of the ASD method.

6.2.4.3 Sliding Stability

From Table 4.3, the friction angle for mass concrete on clean sound rock is 35 deg.

A. The ASD method:

$\Sigma H_{unf} = 11.8$ kip/ft and FS = 1.5

$F_r = N_{unf} \tan \delta_b + C_a B_e$

$= (30.5)(\tan 35 \text{ deg}) + 0 = 21.4$ kip/ft

$F_r/FS = 21.4/1.5 = 14.2 \text{ kip/ft} > \Sigma H_{unf} = 11.8$ kip/ft **OK**

B. The LFD method:

(a) when $\beta_D = 1.0$

$\Sigma H_u = 19.9$ kip/ft

$F_{ru} = N_u \tan \delta_b + C_a B_e = (40.9)(\tan 35 \text{ deg}) + 0 = 28.6$ kip/ft

Assuming that the sliding performance factor for rock, ϕ_s, is 0.8 (**Note:** ϕ_s for a rock foundation was not specified in the Table 3.3 because of lack of data; therefore, it is appropriate that the minimum value in the table, 0.8, be used for the performance factor in this case), then

$\phi_s F_{ru} = (0.8)(28.6)$

$= \underline{22.9} \text{ kip/ft} > \Sigma H_u = \underline{19.9}$ kip/ft **OK**

(b) when $\beta_D = 0.75$

$\Sigma H_u = 19.9$ kip/ft

$F_{ru} = N_u \tan \delta_b + C_a B_e = (37.4)(\tan 35 \text{ deg}) + 0 = 26.2$ kip/ft

Assuming that the sliding performance factor for rock, ϕ_s, is 0.8 (**Note:** ϕ_s for a rock foundation was not specified in the Table 3.3 because of lack of data; therefore, it is appropriate that the minimum value in the table, 0.8, be used for the performance factor in this case), then

$\phi_s F_{ru} = (0.8)(26.2)$

$= \underline{21.0} \text{ kip/ft} > \Sigma H_u = \underline{19.9}$ kip/ft **OK**

6.2.5 Conclusion

The retaining wall satisfies the stability and safety criteria for both ASD and LFD. It would be possible to reduce the base width somewhat and still satisfy the design criteria. Because the wall is founded on rock, it is not necessary to check the settlement criteria.

6.3 EXAMPLE 2—A RETAINING WALL SUBJECTED TO LIVE LOAD SURCHARGE

Problem Statement. The stability and safety for the retaining wall shown in Figure 6.2 is to be checked, using the ASD method and the LFD method.

The wall is founded on medium dense sand with an average standard penetration test (SPT) blow count of 12. The ultimate bearing capacity of the foundation soil is estimated to be 26.0 ksf.

The backfill material is medium dense sand with an angle of internal friction equal to 35 deg. The unit weight of the backfill soil is 120 pcf and the density of the concrete is 150 pcf. The friction angle between the base and foundation soil, δ_b, is 28 deg.

The passive resistance of the soil in front of the retaining wall is to be neglected.

A live load surcharge equal to 2 ft of earth acts on the surface of the backfill.

In this example, the active earth pressures are determined by the Rankine equation. For the zero backfill slope and $\phi_f = 35$ deg, the active earth pressure coefficient, K_a, is;

$$K_a = \frac{1 - \sin \phi_f}{1 + \sin \phi_f} = 0.271$$

6.3.1 Determination of Loads and Earth Pressures

With loadings of types D, L, and E, Eq. 3.4.1 can be reduced to: Gr N = $\gamma[\beta_D D + \beta_L L + \beta_E E]$.

From Table 3.2, the loading combinations applicable to this example are Group I, II, and VI for ASD and LFD.

6.3.1.1 Load Factors

The following load factors are multiplied by the corresponding load component and the products summed for each load combination.

A. For ASD:

		Backfill		Surcharge		
Gr	D	E_v	E_h	E_{vL}	E_{hL}	%*
I	1	1	1	1	1	100
II	1	1	1	0	0	125
VI	1	1	1	1	1	140

*The numbers in this column represent percentages of basic unit stress, which are usually multiplied on the resistance side. However, for convenience in comparison with LFD, the loads in ASD will be divided by these percentages.

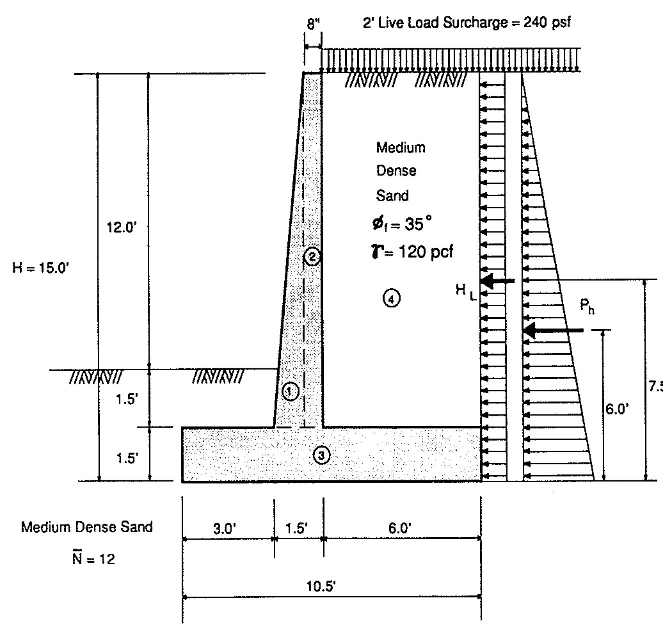


Figure 6.2. A retaining wall subjected to live load surcharge.

B. For LFD:

load factors $= \gamma\beta_i$

		Backfill		Surcharge	
Gr	D	E_v	E_h	E_{vL}	E_{hL}
I	1.3	1.3	1.69	2.17	1.69
Ia	0.98	1.3	1.69	2.17	1.69
II	1.3	1.3	1.69	0	0
IIa	0.98	1.3	1.69	0	0
VI	1.25	1.25	1.63	1.25	1.63
VIa	0.94	1.25	1.63	1.25	1.63

where: E_v = vertical load due to earth
E_h = horizontal load due to earth
E_{vL} = vertical load due to live load surcharge
E_{hL} = horizontal load due to live load surcharge

It can be observed that Group I governs over Group VI in ASD. However, it is not clear at a glance whether Group I or Group VI governs in LFD.

6.3.1.2 Unfactored Loads

Unfactored vertical and horizontal loads are summarized as follows.

A. Vertical loads:

Items	V_{unf} (k/ft)	Arm about o (ft)	Moment about o (k-ft/ft)
1. (1/2)(0.833)(13.5)(0.150)	0.84	3.56	2.99
2. (0.67)(13.5)(0.150)	1.35	4.17	5.63
3. (10.5)(1.5)(0.150)	2.36	5.25	12.39
4. (6.0)(13.5)(0.120)	9.72	7.50	72.90
V_L (due to ω_L)	1.44	7.50	10.80

B. Horizontal loads:

for $K_a = 0.271$, $H = 15.0$ ft

Items	H_{unf} (k/ft)	Arm about o (ft)	Moment about o (k-ft/ft)
P_E (earth pressure)	3.66	6.0 (0.4H)	21.96
H_L (due to ω_L)	0.98	7.5 (0.5H)	7.35

6.3.2 The ASD Method

6.3.2.1 Design Loads

A. Vertical loads and moments due to vertical loads:

(a) vertical loads, V_{asd} (kip/ft)

Items *	(1) D	(2) D	(3) D	(4) E_v	V_L E_{vL}	V_{asd}		
V_{unf}	0.84	1.35	2.36	9.72	1.44	Sum	%	Sum/%
I	0.84	1.35	2.36	9.72	1.44	15.71	100	15.71
II	0.84	1.35	2.36	9.72	0	14.27	125	11.42
VI	0.84	1.35	2.36	9.72	1.44	15.71	140	11.22

where: V_{unf} = unfactored vertical loads
* = notations used by AASHTO

(b) moments, M_{Vasd} (kip-ft/ft)

Items *	(1) D	(2) D	(3) D	(4) E_v	V_L E_{vL}	M_{Vasd}		
M_{Vunf}	2.99	5.63	12.39	72.90	10.80	Sum	%	Sum/%
I	2.99	5.63	12.39	72.90	10.80	104.71	100	104.71
II	2.99	5.63	12.39	72.90	0	93.91	125	75.13
VI	2.99	5.63	12.39	72.90	10.80	104.71	140	74.79

where: M_{Vunf} = moment due to unfactored vertical loads
* = notations used by AASHTO

B. Horizontal loads and moments due to horizontal loads:

(a) horizontal loads, H_{asd} (kip/ft)

Items *	P_E E_h	H_L E_{hL}	H_{asd}		
H_{unf}	3.66	0.98	Sum	%	Sum/%
I	3.66	0.98	4.64	100	4.64
II	3.66	0	3.66	125	2.93
VI	3.66	0.98	4.64	140	3.31

where: H_{unf} = unfactored horizontal loads
* = notations used by AASHTO

(b) moments, M_{Hasd} (kip-ft/ft)

Items *	P_E E_h	H_L E_{hL}	M_{Hasd}		
M_{Hunf}	21.96	7.35	Sum	%	Sum/%
I	21.96	7.35	29.31	100	29.31
II	21.96	0	21.96	125	17.57
VI	21.96	7.35	29.31	140	20.94

where: M_{Hunf} = moment due to unfactored horizontal loads (kip-ft/ft)
* = notations used by AASHTO

6.3.2.2 Stability and Safety Criteria

A. Eccentricity check:

Gr	V_{asd} (k/ft)	H_{asd} (k/ft)	M_{Vasd} (k-ft/ft)	M_{Hasd} (k-ft/ft)	X_o (ft)	e (ft)	e_{max} (ft)	q_t (ksf)	q_{unif} (ksf)
I	15.71	4.64	104.71	29.31	4.80	0.45 <	1.75	1.88	1.64
II	11.42	2.93	75.13	17.57	5.04	0.21 <	1.75	1.22	1.13
VI	11.22	3.31	74.79	20.94	4.80	0.45 <	1.75	1.34	1.17

where: X_o = location of resultant = $(M_{Vasd} - M_{Hasd})/V_{asd}$
e = eccentricity = $B/2 - X_o$
e_{max} = $B/6 = 10.5/6 = 1.75$ ft
q_t = triangular bearing pressure at toe (see Fig. 5.7, with $N = V_{asd}$)
q_{unif} = uniformly distributed bearing pressure at toe (see Fig. 5.7)

B. Bearing capacity check (as previously mentioned, reduction factors, R_I, due to inclined load shall be considered in bearing capacity check):

Gr	V_{asd} (k/ft)	H_{asd} (k/ft)	$H_{asd} \div V_{asd}$	D_f/B_e	R_I	q^v_{ult} (ksf)	q^i_{ult} (ksf)	$q^i_{ult} \div$ FS		q_{max} (ksf)
I	15.71	4.64	0.30	0.31	0.37	26.0	9.62	2.41	>	1.64
II	11.42	2.93	0.26	0.30	0.42	26.0	10.92	2.73	>	1.13
VI	11.22	3.31	0.30	0.31	0.37	26.0	9.62	2.41	>	1.17

where: D_f = depth from the ground surface to the bottom of footing = 3.0 ft
B_e = effective base width = $2X_o$ in this example
R_I = reduction factor due to inclined load
q^v_{ult} = vertical bearing capacity
q^i_{ult} = inclined bearing capacity = $(R_I)(q^v_{ult})$
FS = factor of safety for SPT method = 4.0
q_{max} = maximum bearing pressure due to loadings = q_{unif} in this example

C. Sliding stability check (since uniformly distributed bearing pressures are used, the effective base width, B_e, for calculation of sliding resistance is $2X_o$):

Gr	V_{asd} (k/ft)	$\tan \delta_b$	F_r (k/ft)	FS	F_r/FS (k/ft)		H_{asd}
I	15.71	0.532	8.36	1.5	5.57	>	4.64
II	11.42	0.532	6.08	1.5	4.05	>	2.93
VI	11.22	0.532	5.97	1.5	3.98	>	3.31

where: F_r = $N_{asd} \tan \delta_b + C_a B_e$
N_{asd} = unfactored resultant force normal to base V_{asd}
δ_b = frictional angle between base and soil = 28 deg
C_a = adhesion = 0

6.3.2.3 Conclusions for ASD

It is observed that the sliding stability criterion is closer to controlling the design of the wall in this example than are the eccentricity or bearing capacity checks. All of the stability and safety criteria are satisfied and are governed by Group I. Therefore, using the ASD criteria, the retaining wall is verified to be adequately designed.

6.3.3 The LFD Method

6.3.3.1 Design Loads

A. Vertical loads and moments due to vertical loads:

(a) factored vertical loads, V_u (kip/ft)

Items *	(1) D	(2) D	(3) D	(4) E_v	V_L E_{vL}	V_u
V_{unf}	0.84	1.35	2.36	9.72	1.44	Total
I	1.09	1.76	3.07	12.64	3.12	21.68
Ia	0.82	1.32	2.31	12.64	3.12	20.21
II	1.09	1.76	3.07	12.64	0	18.56
IIa	0.82	1.32	2.31	12.64	0	17.09
VI	1.05	1.69	2.95	12.15	1.80	19.64
VIa	0.79	1.27	2.22	12.15	1.80	18.23

where: V_{unf} = unfactored vertical loads
* = notations used by AASHTO

(b) factored moments, M_{Vu} (kip-ft/ft)

Items *	(1) D	(2) D	(3) D	(4) E_v	V_L E_{vL}	M_{Vu}
M_{Vunf}	2.99	5.63	12.39	72.90	10.80	Total
I	3.89	7.32	16.11	94.77	23.44	145.54
Ia	2.93	5.52	12.14	94.77	23.44	138.80
II	3.89	7.32	16.11	94.77	0	122.09
IIa	2.93	5.52	12.14	94.77	0	115.36
VI	3.74	7.04	15.49	91.13	13.50	130.90
VIa	2.81	5.29	11.65	91.13	13.50	124.38

where: M_{Vunf} = moment due to unfactored vertical loads
* = notations used by AASHTO

B. Horizontal loads and moments due to horizontal loads:

(a) factored horizontal loads, H_u (kip/ft)

Items *	P_E E_h	H_L E_{hL}	H_u
H_{unf}	3.66	0.98	Total
I	6.19	1.66	7.85
Ia	6.19	1.66	7.85
II	6.19	0	6.19
IIa	6.19	0	6.19
VI	5.95	1.59	7.54
VIa	5.95	1.59	7.54

where: H_{unf} = unfactored horizontal loads
* = notations used by AASHTO

(b) factored moments, M_{Hu} (kip-ft/ft)

Items *	P_E E_h	H_L E_{hL}	M_{Hu}
M_{Hunf}	21.96	7.35	Total
I	37.11	12.42	49.53
Ia	37.11	12.42	49.53
II	37.11	0	37.11
IIa	37.11	0	37.11
VI	35.69	11.94	47.63
VIa	35.69	11.94	47.63

where: M_{Hunf} = moment due to unfactored horizontal loads
* = notations used by AASHTO

6.3.3.2 Stability and Safety Criteria

A. Eccentricity check:

Gr	V_u (k/ft)	H_u (k/ft)	M_{Vu} (k-ft/ft)	M_{Hu} (k-ft/ft)	X_o (ft)	e (ft)	e_{max} (ft)	q_t (ksf)	q_{unif} (ksf)
I	21.68	7.85	145.53	49.53	4.43	0.82 <	2.63	3.03	2.45
Ia	20.21	7.85	138.80	49.53	4.42	0.83 <	2.63	2.84	2.29
II	18.56	6.19	122.09	37.11	4.58	0.67 <	2.63	2.44	2.03
IIa	17.09	6.19	115.36	37.11	4.58	0.67 <	2.63	2.25	1.87
VI	19.64	7.54	130.90	47.63	4.24	1.01 <	2.63	2.95	2.32
VIa	18.23	7.54	124.38	47.63	4.21	1.04 <	2.63	2.77	2.17

where: X_o = location of resultant = $(M_{Vu} - M_{Hu})/V_u$
e = eccentricity = $B/2 - X_o$
e_{max} = $B/4 = 10.5/4 = 2.63$ ft
q_t = triangular bearing pressure at toe (see Fig. 5.7)
q_{unif} = uniformly distributed bearing pressure at toe (see Fig. 5.7)

B. Bearing capacity check (as mentioned in Example 1, reduction factors, R_I, from ASD are used for the LFD method):

Gr	R_I (ASD)	q^v_{ult} (ksf)	q^i_{ult} (ksf)	ϕq^i_{ult} (ksf)		q_{max} (ksf)
I	0.37	26.0	9.62	4.33	>	2.45
Ia	0.37	26.0	9.62	4.33	>	2.29
II	0.42	26.0	10.92	4.91	>	2.03
IIa	0.42	26.0	10.92	4.91	>	1.87
VI	0.37	26.0	9.62	4.33	>	2.32
VIa	0.37	26.0	9.62	4.33	>	2.17

where: D_f = depth from the soil surface to the bottom of footing
B_e = $2X_o$
q^v_{ult} = vertical bearing capacity
q^i_{ult} = inclined bearing capacity
ϕq_{ult} = factored bearing capacity ($\phi = 0.45$)
q_{max} = maximum bearing pressure due to loadings = q_{unif} in this example.

C. Sliding stability check (because the uniformly distributed bearing pressures are used, the effective base width for calculation of sliding resistance is $2X_o$):

Gr	V_u (k/ft)	$\tan \delta_b$	F_{ru} (k/ft)	ϕ_s	$\phi_s F_{ru}$ (k/ft)		H_u (k/ft)
I	21.68	0.532	11.53	0.80	9.22	>	7.85
Ia	20.21	0.532	10.75	0.80	8.60	>	7.85
II	18.56	0.532	9.87	0.80	7.90	>	6.19
IIa	17.09	0.532	9.09	0.80	7.27	>	6.19
VI	19.64	0.532	10.45	0.80	8.36	>	7.54
VIa	18.23	0.532	9.70	0.80	7.76	>	7.54

where: $F_{ru} = N_u \tan \delta_b + C_a B_e$
N_u = factored resultant force normal to base V_u
δ_b = frictional angle between base and soil = 28 deg
C_a = adhesion = 0
ϕ_s = sliding performance factor for SPT data

6.3.3.3 Conclusions for LFD

The design of the wall is controlled by the sliding stability criterion. Group VIa governs with respect to the eccentricity and sliding stability checks. Bearing capacity is governed by Group I. However, all of the stability and safety criteria are satisfied in the LFD method. Thus, using the LFD criteria, the retaining wall is acceptable.

6.3.4 Serviceability Limit State Check

The vertical movement (or settlement) of the retaining wall is checked according to the procedures described in the engineering manual for shallow foundations (Part 1). Two methods are used to estimate settlement of the retaining wall: the Terzaghi and Peck method and the D'Appolonia et al. method.

The minimum average SPT blow count, $\overline{N}$, within the range of depth from the footing base to the depth B below the bottom of footing is 12.

Because it is a serviceability condition, unfactored loads are used to estimate settlement. Group I governs for all criteria of the ASD method in this example. From the tabulated results of the ASD method, the effective base width, $B_e = 2X_o = 9.60$ ft and the uniform bearing pressure, p = 1.64 ksf or 0.82 tsf.

6.3.4.1 Terzaghi and Peck Method

From Figure 5.1 (in Part 1) read the bearing pressure corresponding to 1-in. settlement with $B = B_e = 9.60$ ft and the minimum average SPT blow counts, $\overline{N} = 12$; thus, p1 in. = 1.2 tsf.

Considering that the water table is below the foundation at least 2B and has no effect on the bearing pressure, the estimated settlement of footing is:

$$\rho = \frac{0.82 \text{ tsf}}{1.2 \text{ tsf}} = 0.68 \text{ in.}$$

From Table 5.9 (in Part 1), the adjusted settlement using 90 percent reliability, ρ', is:

$$\rho' = (1.05)(\rho) = (1.05)(0.68) = 0.71 \text{ in.}$$

6.3.4.2 D'Appolonia et al. Method

From Eq. 5.2.2 (in Part 1), the settlement of the footing, based on the recommendation of D'Appolonia et al. is: $\rho = \mu_o \mu_1$ (p B/M).

With $D_f/B = (3.0)/(9.60) = 0.31$, and $L/B = (35.0)/(9.60) = 3.65$, read $\mu_o = 0.96$ from Figure 5.3 in Part 1.

With $H_t/B = (40.0)/(9.60) = 4.17$, and $L/B = 3.65$ ft, read $\mu_1 = 1.15$ from Figure 5.3 in Part 1.

With $\overline{N} = 12$, read M = 290 tsf from the line of Normally Loaded Sand or Sand and Gravel in Figure 5.4 (Part 1).

Therefore, the settlement estimated from Eq. 5.2.2 (Part 1) is:

$$\rho = \mu_o \mu_1 \frac{p B}{M} = (0.96)(1.15)\frac{(0.82)(9.60)}{290}$$

$$= 0.03 \text{ ft} = 0.36 \text{ in.}$$

From Table 5.9 (Part 1), the adjusted settlement using 90 percent reliability, ρ', is:

$$\rho' = (2.00)(\rho) = (2.00)(0.36) = \underline{0.72 \text{ in.}}$$

6.3.4.3 Conclusions

The amounts of settlement estimated by the Terzaghi and Peck method and the D'Appolonia et al. method are nearly the same in this example. Settlement less than 1 in. can be considered tolerable in most retaining walls.

6.4 EXAMPLE 3—DESIGN OF AN ABUTMENT

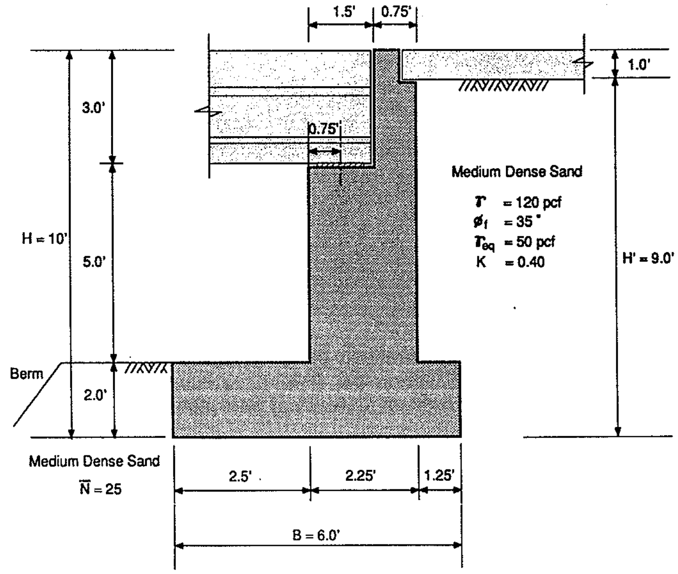
Problem Statement. Using the ASD method and the LFD method, the stability and safety for the abutment shown in Figure 6.3 is to be checked. The abutment is founded on sandy gravel with an average standard penetration test blow count of 25. The ultimate bearing capacity of the foundation soil is estimated to be 30.0 ksf.

The backfill material is medium dense sand with an angle of internal friction equal to 35 deg, and unit weight of 120 pcf. The density of the concrete is 150 pcf. The friction angle between the base and foundation soil, δ_b, is estimated to be 29 deg.

The passive pressure of the soil in front of the abutment will be ignored.

A live load surcharge equal to 2 ft of earth acts on the surface of the backfill. The weight of the approach slab is considered as dead load surcharge. (If the approach slab is designed to span across the backfill, both surcharges could be ignored.)

In this example, because the wall and foundation soil are relatively stiff, at-rest earth pressures are obtained from equivalent fluid pressures. From Table 4.6, for medium dense sand or gravel, the equivalent fluid unit weight, γ_{eq}, is 50 pcf and the horizontal earth pressure coefficient, K, is 0.40.



where $\overline{N}$ = average standard penetration test blow count
γ = unit weight of soil
ϕ_f = internal friction angle
γ_{eq} = equivalent fluid pressure
K = horizontal earth pressure coefficient

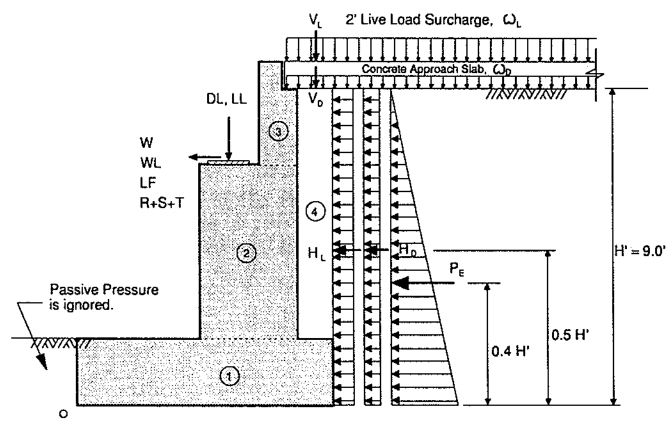
Figure 6.3. Example 3—a bridge abutment.

6.4.1 Determination of Loads and Earth Pressures

The loadings to be considered in this example are shown in Figure 6.4. Considering loadings of types D, L, E, W, WL, LF, and R+S+T, Eq. 3.4.1 can be expressed as:

$$\text{Gr N} = \gamma[\beta_D D + \beta_L L + \beta_E E + \beta_W W + \beta_{WL} WL + \beta_{LF} LF + \beta_R (R+S+T)]$$

From Table 3.2, the relevant loading combinations are Groups I, II, III, IV, V, and VI for both ASD and LFD.



Given:
DL = 7.5 k/ft (Dead Load from Bridge Superstructure)
LL = 6.0 k/ft (Live Load from Bridge Superstructure)
W = 0.20 k/ft (Wind Load on Superstructure)
WL = 0.05 k/ft (Wind Load on Live Load)
LF = 0.25 k/ft (Longitudinal Force)
R+S+T = 10 % of DL = 0.75 k/ft (Rib Shortening, Shrinkage and Temperature Load)
ω_L = (2')(0.120 pcf) = 0.24 ksf
ω_D = (1')(0.150 kcf) = 0.15 ksf
$P_E = (1/2)(\gamma_{eq})H'^2 = (1/2)(0.050)(9.0)^2 = 2.03$ k/ft (Equivalent Fluid Pressure Force)
$H_L = K(\omega_L)H' = (0.40)(0.24)(9.0) = 0.86$ k/ft (Lateral Earth Pressure Force due to Live Load Surcharge)
$H_D = K(\omega_D)H' = (0.40)(0.15)(9.0) = 0.54$ k/ft (Lateral Earth Pressure Force due to Dead Load Surcharge)
$V_L = (0.24)(1.25') = 0.30$ k/ft (Vertical Resultant of Live Load Surcharge)
$V_D = (0.15)(1.25') = 0.19$ k/ft (Vertical Resultant of Dead Load Surcharge)

Figure 6.4. Example 3—summary of loadings applied to the abutment.

6.4.1.1 Load Factors

The following load factors are multiplied by the corresponding load component and the products summed for each load combination.

A. For ASD:

Gr	D	L	Back-fill E_v	E_h	Sur-charge E_{hD}	E_{hL}	W	WL	LF	R+S+T	%*
I	1	1	1	1	1	1	0	0	0	0	100
II	1	0	1	1	1	0	1	0	0	0	125
III	1	1	1	1	1	1	0.3	1	1	0	125
IV	1	1	1	1	1	1	0	0	0	1	125
V	1	0	1	1	1	0	1	0	0	1	140
VI	1	1	1	1	1	1	0.3	1	1	1	140

*The numbers in this column represent percentages of basic unit stress, which are usually multiplied on the resistance side. However, for convenience in comparison with LFD, the loads in ASD will be divided by these percentages.

B. For LFD:

load factors = $\gamma\beta_i$

Gr	D	L	Back-fill E_v	E_h	Sur-charge E_{hD}	E_{hL}	W	WL	LF	R+S+T
I	1.3	2.17	1.3	1.69	1.69	1.69	0	0	0	0
Ia*	0.98	2.17	1.3	1.69	1.69	1.69	0	0	0	0
II	1.3	0	1.3	1.69	1.69	0	1.3	0	0	0
IIa	0.98	0	1.3	1.69	1.69	0	1.3	0	0	0
III	1.3	1.3	1.3	1.69	1.69	1.69	0.39	1.3	1.3	0
IIIa	0.98	1.3	1.3	1.69	1.69	1.69	0.39	1.3	1.3	0
IV	1.3	1.3	1.3	1.69	1.69	1.69	0	0	0	1.3
IVa	0.98	1.3	1.3	1.69	1.69	1.69	0	0	0	1.3
V	1.25	0	1.25	1.63	1.63	0	1.25	0	0	1.25
Va	0.94	0	1.25	1.63	1.63	0	1.25	0	0	1.25
VI	1.25	1.25	1.25	1.63	1.63	1.63	0.38	1.25	1.25	1.25
VIa	0.94	1.25	1.25	1.63	1.63	1.63	0.38	1.25	1.25	1.25

*The value of β_D for load combination groups I, II, or VI is 1.0 and these often control the maximum bearing pressure. However, the groups, such as Ia, IIa, or VIa, which employ $\beta_D = 0.75$ are used to check the eccentricity or sliding stability criteria.

6.4.1.2 Unfactored Loads

Unfactored vertical and horizontal loads are summarized as follows.

A. Vertical loads:

Items	V_{unf} (k/ft)	Arm about o (ft)	Moment about o (k-ft/ft)
1. (6.0)(2.0)(0.150)	1.80	3.00	5.40
2. (2.25)(5.0)(0.150)	1.69	3.63	6.13
3. (0.75)(3.0)(0.150)	0.34	4.38	1.49
4. (1.25)(7.0)(0.120)	1.05	5.38	5.65
DL (superstructure)	7.50	3.25	24.38
LL (superstructure)	6.00	3.25	19.50
V_D (due to ω_D)	0.19	5.38	1.02
V_L (due to ω_L)	0.30	5.38	1.61

B. Horizontal loads:

Items	H_{unf} (k/ft)	Arm about o (ft)	Moment about o (k-ft/ft)
P_E (earth pressure)	2.03	3.6 (0.4H')	7.31
H_D (due to ω_D)	0.54	4.5 (0.5H')	2.43
H_L (due to ω_L)	0.86	4.5 (0.5H')	3.87
W (wind on structure)	0.20	7.0	1.40
WL (wind on LL)	0.05	7.0	0.35
LF	0.25	7.0	1.75
R+S+T	0.75	7.0	5.25

6.4.2 The ASD Method

6.4.2.1 Design Loads

A. Vertical loads and moments due to vertical loads:

(a) vertical loads, V_{asd} (kip/ft)

Items *	(1) D	(2) D	(3) D	(4) E_V	DL D	LL L	V_D D	V_L L	V_{asd}		
V_{unf}	1.80	1.69	0.34	1.05	7.50	6.00	0.19	0.30	Sum	%	Sum/%
I	1.80	1.69	0.34	1.05	7.50	6.00	0.19	0.30	18.87	100	18.87
II	1.80	1.69	0.34	1.05	7.50	0	0.19	0	12.57	125	10.06
III	1.80	1.69	0.34	1.05	7.50	6.00	0.19	0.30	18.87	125	15.10
IV	1.80	1.69	0.34	1.05	7.50	6.00	0.19	0.30	18.87	125	15.10
V	1.80	1.69	0.34	1.05	7.50	0	0.19	0	12.57	140	8.98
VI	1.80	1.69	0.34	1.05	7.50	6.00	0.19	0.30	18.87	140	13.48

where: V_{unf} = unfactored vertical loads
* = notations used by AASHTO

(b) moments, M_{Vasd} (kip-ft/ft)

Items *	(1) D	(2) D	(3) D	(4) E_V	DL D	LL L	V_D D	V_L L	M_{Vasd}		
M_{unf}	5.4	6.13	1.49	5.65	24.38	19.5	1.02	1.61	Sum	%	Sum/%
I	5.4	6.13	1.49	5.65	24.38	19.5	1.02	1.61	65.18	100	65.18
II	5.4	6.13	1.49	5.65	24.38	0	1.02	0	44.07	125	35.26
III	5.4	6.13	1.49	5.65	24.38	19.5	1.02	1.61	65.18	125	52.14
IV	5.4	6.13	1.49	5.65	24.38	19.5	1.02	1.61	65.18	125	52.14
V	5.4	6.13	1.49	5.65	24.38	0	1.02	0	44.07	140	31.48
VI	5.4	6.13	1.49	5.65	24.38	19.5	1.02	1.61	65.18	140	46.56

where: M_{unf} = moment due to unfactored vertical loads
* = notations used by AASHTO

B. Horizontal loads and moments due to horizontal loads:

(a) horizontal loads, H_{asd} (kip/ft)

Items *	P_E E_h	H_D E_{hD}	H_L E_{hL}	W W	WL WL	LF LF	R+S+T R+S+T	H_{asd}		
H_{unf}	2.03	0.54	0.86	0.20	0.05	0.25	0.75	Sum	%	Sum/%
I	2.03	0.54	0.86	0	0	0	0	3.43	100	3.43
II	2.03	0.54	0	0.20	0	0	0	2.77	125	2.22
III	2.03	0.54	0.86	0.06	0.05	0.25	0	3.79	125	3.03
IV	2.03	0.54	0.86	0	0	0	0.75	4.18	125	3.34
V	2.03	0.54	0	0.20	0	0	0.75	3.52	140	2.51
VI	2.03	0.54	0.86	0.06	0.05	0.25	0.75	4.54	140	3.24

where: H_{unf} = unfactored horizontal loads
* = notations used by AASHTO

(b) moments, M_{Hasd} (kip-ft/ft)

Items	P_E	H_D	H_L	W	WL	LF	R+S+T	M_{Hasd}		
*	E_h	E_{hD}	E_{hL}	W	WL	LF	R+S+T			
M_{Hunf}	7.31	2.43	3.87	1.40	0.35	1.75	5.25	Sum	%	Sum/%
I	7.31	2.43	3.87	0	0	0	0	13.61	100	13.61
II	7.31	2.43	0	1.40	0	0	0	11.14	125	8.91
III	7.31	2.43	3.87	0.42	0.35	1.75	0	16.13	125	12.90
IV	7.31	2.43	3.87	0	0	0	5.25	18.86	125	15.09
V	7.31	2.43	0	1.40	0	0	5.25	16.39	140	11.71
VI	7.31	2.43	3.87	0.42	0.35	1.75	5.25	21.38	140	15.27

where: M_{Hunf} = moment due to unfactored horizontal loads (kip-ft/ft)

6.4.2.2 Stability and Safety Criteria

A. Eccentricity check:

Gr	V_{asd} (k/ft)	H_{asd} (k/ft)	M_{Vasd} (k-ft/ft)	M_{Hasd} (k-ft/ft)	X_o (ft)	e (ft)		e_{max} (ft)	q_t (ksf)	q_{unif} (ksf)
I	18.87	3.43	65.18	13.61	2.73	0.27	<	1.00	3.99	3.46
II	10.06	2.22	35.26	8.90	2.62	0.38	<	1.00	2.31	1.92
III	15.10	3.03	52.14	12.90	2.60	0.40	<	1.00	3.52	2.90
IV	15.10	3.34	52.14	15.09	2.45	0.55	<	1.00	3.90	3.08
V	8.98	2.51	31.48	11.71	2.20	0.80	<	1.00	2.69	2.04
VI	13.48	3.24	46.56	15.27	2.32	0.68	<	1.00	3.77	2.91

where: X_o = location of resultant = $(M_{Vasd} - M_{Hasd})/V_{asd}$
e = eccentricity = $B/2 - X_o$
$e_{max} = B/6 = 6.00/6 = 1.00$ ft
q_t = trapezoidal bearing pressure at toe (see Fig. 5.7)
q_{unif} = uniformly distributed bearing pressure at toe (see Fig. 5.7)

B. Bearing capacity check (as previously mentioned, reduction factors, R_I, due to inclined load shall be considered in bearing capacity check):

Gr	V_{asd} (k/ft)	H_{asd} (k/ft)	$H_{asd} \div V_{asd}$	D_f/B_e	R_I	q^v_{ult} (ksf)	q^i_{ult} (ksf)	$q^i_{ult} \div$ FS		q_{max} (ksf)
I	18.87	3.43	0.18	0.37	0.55	30.0	16.50	4.13	>	3.46
II	10.06	2.22	0.22	0.38	0.51	30.0	15.30	3.83	>	1.92
III	15.10	3.03	0.20	0.39	0.55	30.0	16.50	4.13	>	2.90
IV	15.10	3.34	0.22	0.41	0.51	30.0	15.30	3.83	>	3.08
V	8.98	2.51	0.27	0.46	0.40	30.0	12.00	3.00	>	2.04
VI	13.48	3.24	0.24	0.43	0.47	30.0	14.10	3.53	>	2.91

where: D_f = depth from the ground surface to the bottom of footing = 2.0 ft
B_e = effective base width $2X_o$ in this example
R_I = reduction factor due to inclined load
q^v_{ult} = vertical bearing capacity
q^i_{ult} = inclined bearing capacity = $(R_I)(q^v_{ult})$
FS = factor of safety for SPT method = 4.0
q_{max} = maximum bearing pressure due to loadings = q_{unif} in this example

C. Sliding stability check (because uniformly distributed bearing pressures are used, the effective base width, B_e, for calculation of sliding resistance is $2X_o$):

Gr	V_{asd} (k/ft)	tan δ_b	F_r (k/ft)	FS	F_r/FS (k/ft)		H_{asd}
I	18.87	0.554	10.45	1.5	6.97	>	3.43
II	10.06	0.554	5.57	1.5	3.71	>	2.22
III	15.10	0.554	8.37	1.5	5.58	>	3.03
IV	15.10	0.554	8.37	1.5	5.58	>	3.34
V	8.98	0.554	4.97	1.5	3.31	>	2.51
VI	13.48	0.554	7.47	1.5	4.98	>	3.24

where: $F_r = N_{asd} \tan \delta_b + C_a B_e$
N_{asd} = resultant force normal to base = V_{asd}
δ_b = frictional angle between base and soil = 29 deg
C_a = adhesion = 0

6.4.2.3 Conclusions for ASD

Group V produces critical results with respect to eccentricity criterion. Although Group I generates the largest bearing pressure, Group IV generates the least margin of safety in the bearing capacity check; the safety margin of Group IV is slightly smaller than that of Group I. This is because the reduction factor, R_I, of Group VI is less than that of Group I. Group V yields the most critical result in the sliding stability check.

However, this abutment satisfies all the stability and safety criteria in the ASD method. Therefore, the design of the abutment is acceptable using the ASD criteria.

6.4.3 The LFD Method

6.4.3.1 Design Loads

A. Vertical loads and moments due to vertical loads:

(a) factored vertical loads, V_u (kip/ft)

Items	(1)	(2)	(3)	(4)	DL	LL	V_D	V_L	V_U
*	D	D	D	E_V	D	L	D	L	
V_{unf}	1.80	1.69	0.34	1.05	7.50	6.00	0.19	0.30	Total
I	2.34	2.19	0.44	1.37	9.75	13.03	0.24	0.65	30.01
Ia	1.76	1.66	0.33	1.37	7.35	13.03	0.19	0.65	26.33
II	2.34	2.19	0.44	1.37	9.75	0	0.24	0	16.33
IIa	1.76	1.66	0.33	1.37	7.35	0	0.19	0	12.66
III	2.34	2.19	0.44	1.37	9.75	7.80	0.24	0.39	24.52
IIIa	1.76	1.66	0.33	1.37	7.35	7.80	0.19	0.39	20.85
IV	2.34	2.19	0.44	1.37	9.75	7.80	0.24	0.39	24.52
IVa	1.76	1.66	0.33	1.37	7.35	7.80	0.19	0.39	20.85
V	2.25	2.11	0.42	1.31	9.38	0	0.23	0	15.70
Va	1.69	1.59	0.32	1.31	7.05	0	0.18	0	12.14
VI	2.25	2.11	0.42	1.31	9.38	7.50	0.23	0.38	23.58
VIa	1.69	1.59	0.32	1.31	7.05	7.50	0.18	0.38	20.02

where: V_{unf} = unfactored vertical loads
* = notations used by AASHTO

(b) factored moments, M_{Vu} (kip-ft/ft)

Items *	(1) D	(2) D	(3) D	(4) E_V	DL D	LL L	V_D D	V_L L	M_{Vu}
M_{Vunf}	5.40	6.13	1.49	5.65	24.38	19.50	1.02	1.61	Total
I	7.02	7.97	1.94	7.35	31.69	42.32	1.33	3.49	103.11
Ia	5.29	6.01	1.46	7.35	23.89	42.32	1.00	3.49	90.81
II	7.02	7.97	1.94	7.35	31.69	0	1.33	0	57.30
IIa	5.29	6.01	1.46	7.35	23.89	0	1.00	0	45.00
III	7.02	7.97	1.94	7.35	31.69	25.35	1.33	2.09	84.74
IIIa	5.29	6.01	1.46	7.35	23.89	25.35	1.00	2.09	72.44
IV	7.02	7.97	1.94	7.35	31.69	25.35	1.33	2.09	84.74
IVa	5.29	6.01	1.46	7.35	23.89	25.35	1.00	2.09	72.44
V	6.75	7.66	1.86	7.06	30.48	0	1.28	0	55.09
Va	5.08	5.76	1.40	7.06	22.92	0	0.96	0	43.18
VI	6.75	7.66	1.86	7.06	30.48	24.38	1.28	2.01	81.48
VIa	5.08	5.76	1.40	7.06	22.92	24.38	0.96	2.01	69.57

* = notations used by AASHTO

B. Horizontal loads and moments due to horizontal loads:

(a) factored horizontal loads, H_u (kip/ft)

Items *	P_E E_h	H_D E_{hD}	H_L E_{hL}	W W	WL WL	LF LF	R+S+T R+S+T	H_u
H_{unf}	2.03	0.54	0.86	0.20	0.05	0.25	0.75	Total
I	3.43	0.91	1.45	0	0	0	0	5.79
Ia	3.43	0.91	1.45	0	0	0	0	5.79
II	3.43	0.91	0	0.26	0	0	0	4.60
IIa	3.43	0.91	0	0.26	0	0	0	4.60
III	3.43	0.91	1.45	0.08	0.07	0.33	0	6.27
IIIa	3.43	0.91	1.45	0.08	0.07	0.33	0	6.27
IV	3.43	0.91	1.45	0	0	0	0.98	6.77
IVa	3.43	0.91	1.45	0	0	0	0.98	6.77
V	3.30	0.88	0	0.25	0	0	0.94	5.37
Va	3.30	0.88	0	0.25	0	0	0.94	5.37
VI	3.30	0.88	1.40	0.08	0.06	0.31	0.94	6.97
VIa	3.30	0.88	1.40	0.08	0.06	0.31	0.94	6.97

where: H_{unf} = unfactored horizontal loads
* = notations used by AASHTO

(b) factored moments, M_{Hu} (kip-ft/ft)

Items *	P_E E_h	H_D E_{hD}	H_L E_{hL}	W W	WL WL	LF LF	R+S+T R+S+T	M_{Hu}
M_{Hunf}	7.31	2.43	3.87	1.40	0.35	1.75	5.25	Total
I	12.35	4.11	6.54	0	0	0	0	23.00
Ia	12.35	4.11	6.54	0	0	0	0	23.00
II	12.35	4.11	0	1.82	0	0	0	18.28
IIa	12.35	4.11	0	1.82	0	0	0	18.28
III	12.35	4.11	6.54	0.55	0.46	2.28	0	26.29
IIIa	12.35	4.11	6.54	0.55	0.46	2.28	0	26.29
IV	12.35	4.11	6.54	0	0	0	6.83	29.83
IVa	12.35	4.11	6.54	0	0	0	6.83	29.83
V	11.88	3.95	0	1.75	0	0	6.56	24.14
Va	11.88	3.95	0	1.75	0	0	6.56	24.14
VI	11.88	3.95	6.29	0.53	0.44	2.19	6.56	31.84
VIa	11.88	3.95	6.29	0.53	0.44	2.19	6.56	31.84

where: M_{Hunf} = moment due to unfactored horizontal loads
* = notations used by AASHTO

6.4.3.2 Stability and Safety Criteria

A. Eccentricity check:

Gr	V_u (k/ft)	H_u (k/ft)	M_{Vu} (k-ft/ft)	M_{Hu} (k-ft/ft)	X_o (ft)	e (ft)		e_{max} (ft)	q_t (ksf)	q_{unif} (ksf)
I	30.02	5.79	103.11	23.00	2.67	0.33	<	1.50	6.65	5.62
Ia	26.33	5.79	90.81	23.00	2.58	0.42	<	1.50	6.23	5.10
II	16.35	4.60	57.30	18.28	2.39	0.61	<	1.50	4.39	3.42
IIa	12.66	4.60	45.00	18.28	2.11	0.89	<	1.50	3.99	3.00
III	24.54	6.27	84.74	26.29	2.38	0.62	<	1.50	6.63	5.16
IIIa	20.85	6.27	72.44	26.29	2.21	0.79	<	1.50	6.22	4.72
IV	24.54	6.77	84.74	29.83	2.24	0.76	<	1.50	7.20	5.48
IVa	20.85	6.77	72.44	29.83	2.04	0.96	<	1.50	6.81	5.11
V	15.72	5.37	55.09	24.14	1.97	1.03	<	1.50	5.32	3.99
Va	12.14	5.37	43.18	24.14	1.57	1.43	<	1.50	5.15	3.87
VI	23.60	6.97	81.48	31.84	2.10	0.90	<	1.50	7.47	5.62
VIa	20.02	6.97	69.57	31.84	1.88	1.12	<	1.50	7.10	5.32

where: X_o = location of resultant = $(M_{Vu} - M_{Hu})/V_u$
e = eccentricity = $B/2 - X_o$
e_{max} = $B/4 = 6.00/4 = 1.50$ ft
q_t = triangular or trapezoidal bearing pressure at toe (see Fig. 5.7)
q_{unif} = uniformly distributed bearing pressure at toe (see Fig. 5.7)

B. Bearing capacity check (the reduction factors, R_I, used in the table below are assumed to be the same as those used in the ASD procedure):

Gr	R_I (from ASD)	q^v_{ult} (ksf)	q^i_{ult} (ksf)	ϕq^i_{ult} (ksf)		q_{max} (ksf)
I	0.55	30.0	16.50	7.43	>	5.62
Ia	0.55	30.0	16.50	7.43	>	5.10
II	0.51	30.0	15.30	6.89	>	3.42
IIa	0.51	30.0	15.30	6.89	>	3.00
III	0.55	30.0	16.50	7.43	>	5.16
IIIa	0.55	30.0	16.50	7.43	>	4.72
IV	0.51	30.0	15.30	6.89	>	5.48
IVa	0.51	30.0	15.30	6.89	>	5.11
V	0.40	30.0	12.00	5.40	>	3.99
Va	0.40	30.0	12.00	5.40	>	3.87
VI	0.47	30.0	14.10	6.35	>	5.62
VIa	0.47	30.0	14.10	6.35	>	5.32

where: D_f = depth from the soil surface to the bottom of footing
B_e = $2X_o$
q^v_{ult} = vertical bearing capacity
q^i_{ult} = inclined bearing capacity = $(R_I)(q^v_{ult})$
ϕq^i_{ult} = factored bearing capacity ($\phi = 0.45$)
q_{max} = maximum bearing pressure due to loadings = q_{unif} in this example

C. Sliding stability check (because the uniformly distributed bearing pressures are used, the effective base width for calculation of sliding resistance is $2X_o$):

Gr	V_u (k/ft)	$\tan \delta_b$	F_{ru} (k/ft)	ϕ_s	$\phi_s F_{ru}$ (k/ft)		H_u (k/ft)
I	30.02	0.554	16.63	0.80	13.30	>	5.79
Ia	26.33	0.554	14.59	0.80	11.67	>	5.79
II	16.35	0.554	9.06	0.80	7.25	>	4.60
IIa	12.66	0.554	7.01	0.80	5.61	>	4.60
III	24.54	0.554	13.60	0.80	10.88	>	6.27
IIIa	20.85	0.554	11.55	0.80	9.24	>	6.27
IV	24.54	0.554	13.60	0.80	10.88	>	6.77
IVa	20.85	0.554	11.55	0.80	9.24	>	6.77
V	15.72	0.554	8.71	0.80	6.97	>	5.37
Va	12.14	0.554	6.73	0.80	5.38	>	5.37
VI	23.60	0.554	13.07	0.80	10.46	>	6.97
VIa	20.02	0.554	11.09	0.80	8.87	>	6.97

where: $F_{ru} = N_u \tan \delta_b + C_a B_e$
N_u = factored resultant force normal to base = V_u
δ_b = frictional angle between base and soil = 29 deg
C_a = adhesion = 0
ϕ_s = sliding performance factor for SPT data

6.4.3.3 Conclusions for LFD

Group Va, where destabilizing effects are considered by using a β_D of 0.75, generates the largest eccentricity. Groups I and VI produce the largest bearing pressure, but Group VI has a lower bearing capacity because of a larger reduction for load inclination. Group Va also produces the least safety margin in the sliding stability check, which is the most critical design criterion in this example.

All of the stability and safety criteria are satisfied in the LFD method. Thus, using the LFD criteria, the abutment is acceptable.

6.4.4 Serviceability Limit State Check

Estimate the settlement of the abutment using the Terzaghi and Peck method and the D'Appolonia et al. method. Check tolerable movement criteria according to the engineering manual for estimating tolerable movements of bridges (Part 5).

The minimum average SPT blow count, $\overline{N}$, within the range of depth from the footing base to the depth B below the bottom of footing is 25.

6.4.4.1 Estimation of Settlement

Group I is used for estimating settlement in this example because it generates the largest bearing pressure. The effective base width, $B_e = 2X_o = 5.46$ ft and the uniform bearing pressure, p, is 3.46 ksf or 1.73 tsf.

A. Terzaghi and Peck method: From Figure 5.1 (in Part 1) read the bearing pressure corresponding to a 1-in. settlement with $B = B_e = 5.46$ ft and the minimum average SPT blow counts, $\overline{N} = 25$; thus, p1 in. = 2.9 tsf.

Assuming that the water table is below the foundation at least 2B and has no effect on the bearing pressure, the estimated settlement of the footing is:

$$\rho = \frac{1.73 \text{ tsf}}{2.9 \text{ tsf}} = 0.60 \text{ in.}$$

From Table 5.9 (in Part 1), the adjusted settlement using 90 percent reliability, ρ', is:

$$\rho' = (1.05)(\rho) = (1.05)(0.60) = \underline{0.63 \text{ in.}}$$

B. D'Appolonia et al. method: From Eq. 5.2.2 (in Part 1), the settlement of the footing based on the recommendation of D'Appolonia et al. is:

$$\rho = \mu_o \mu_1 \frac{p B}{M}$$

With $D_f/B = (2.0)/(5.46) = 0.37$ and $L/B = (30.0)/(5.46) = 5.50$, read $\mu_o = 0.95$ from Figure 5.3 (in Part 1).

With $H_t/B = (40.0)/(5.46) = 7.33$ and $L/B = 5.50$ ft, read $\mu_1 = 1.25$ from Figure 5.3 (in Part 1).

With $\overline{N} = 25$, read M = 400 tsf from the line of Normally Loaded Sand or Sand and Gravel in Figure 5.4 (Part 1).

Thus, the settlement estimated from Eq. 5.2.2 (Part 1) is:

$$\rho = \mu_o \mu_1 \frac{p B}{M} = (0.95)(1.25)\frac{(1.73)(5.46)}{400}$$
$$= 0.028 \text{ ft} = 0.34 \text{ in.}$$

From Table 5.9 (in Part 1), the adjusted settlement using 90 percent reliability, ρ', is:

$$\rho' = (2.00)(\rho) = (2.00)(0.34) = \underline{0.68 \text{ in.}}$$

6.4.4.2 Tolerable Settlement of the Abutment

Assume that the abutment supports an end of a continuous two span bridge superstructure and each span is 60 ft long. According to the criteria described in the engineering manual for estimating tolerable movements of bridges (Part 5), the tolerable angular distortion is 0.004. Therefore, the allowable differential settlement between the abutment and the center pier is: $\rho_a = (0.004)(60) = 0.24$ ft = 2.9 in.

The settlement of the center pier is estimated to be 0.5 in. However, in the event the center pier does not experience this total estimated settlement, it is conservative to assume that the differential settlement between the abutment and the center pier is 0.68 in. This amount of differential settlement is within the tolerable value of 2.9 in.

Therefore, the abutment satisfies all design criteria for both ultimate limit state and serviceability limit state.

REFERENCES

American Association of State Highway and Transportation Officials, "Guide Specifications for Seismic Design of Highway Bridges," AASHTO, Washington, D.C., 1983.

American Association of State Highway and Transportation Officials, "Standard Specifications for Highway Bridges," Fourteen Edition, AASHTO, Washington, D.C., 1989.

Barker, R.M., Duncan, J.M. and Rojiani, K.B., "Load Factor Design Criteria for Highway Structure Foundations," Interim Report for NCHRP 24-4, VPI & SU, Oct. 1988.

Bowles, J.E., "Foundation Analysis and Design," Fourth Edition, McGraw-Hill Publication Co., 1988 pp. 304-579.

Bowles, J.E., "Spread Footings," Chapter 15 in Foundation Engineering Handbook Edited by Winterkorn H. and Fang H.Y., Van Nostrand Reinhold Co., New York, 1975, pp. 481-503.

Brown, W.G., "Difficulties Associated with Predicting Depth of Freeze or Thaw," Canadian Geotechnical Journal, Vol. 1, No. 4, 1964, pp. 215-226.

Canadian Geotechnical Society, "Canadian Foundation Engineering Manual", 2nd Edition, Bitech Publishers Ltd., 1985, 460 pp.

Caquot, A. and Kerisel, J., "Tables for the Calculation of Passive Pressure, Active Pressure and Bearing Capacity of Foundations," Gauthier-Villars, Imprimeur-Libraire, Libraire du Bureau des Longitudes, de L'Ecole Polytechnique, Paris, 1948, 120 pp.

Carroll, R.G., Jr., and Murphy, J.C., "Drainage Objective: Prefabricated Drainage Composites," Geotechnical Fabrics Report, Vol 3, NO. 3, 1985, pp. 14-18.

Clausen, C.J.F. and Johansen, S., "Earth Pressures Measured Against a Section of a Basement Wall," Proceeding, 5th European Conference on SMFE, Madrid, 1972, pp. 515-516.

Clayton, C.R.I. and Milititsky, J., "Earth Pressure and Earth-Retaining Structures," Surrey University Press, Glasgow, 1986, 300 pp.

Clough, G.W. and Duncan, J.M., "Finite Element Analyses of Retaining Wall Behavior," Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 97, SM12, Dec., 1971, pp. 1657-1673.

Clough, G.W. and Duncan, J.M., "Foundation Engineering Handbook," 2nd. Edition, edited by H.Y. Fang, Van Nostrand Reinhold, New York, NY, 1991, pp. 223-235.

Danish Geotechnical Institute, "Code of Practice for Foundation Engineering", Danish Geotechnical Institute, Copenhagen, Denmark, Bulletin No. 32, 1978, p. 52.

D'Appolonia, Inc., "Recommended Specifications for the Design of Foundations, Retaining Walls and Substructures," Drafted Report for NCHRP 12-35, Prepared for Transportation Research Board, Washington, D.C., 1989.

Duncan, J.M., Clough, G.W. and Ebeling, R.M., "Behavior and Design of Gravity Earth Retaining Structures," Proceeding of Conference on Design and Performance of Earth Retaining Structures, ASCE, Cornell University, Ithaca, New York, June, 1990, pp. 251-277.

Duncan, J.M. and Tan, C. K., "Engineering Manual for Estimating Tolerable Movements of Bridges," Final Draft for NCHRP 24-4, VPI & SU, May, 1991.

Elms, D.G. and Martin, G.R., "Factors involved in the Seismic Design of Bridge Abutments," Applied Technology Council Workshop on Earthquake Resistance of Highway Bridges, ATC-6-1, Jan. 1979, pp. 230-252.

Elms, D.G. and Richards, R., "Seismic Design of Retaining Walls," Proceeding of Conference on Design and Performance of Earth Retaining Structures, ASCE, Cornell University, Ithaca, New York, June, 1990, pp. 854-871.

Geotechnical Control Office, "Guide to Retaining Wall Design," Geoguide 1, Engineering Development Department, Hong Kong. Available from: U.S. Army Engineer Waterways experiment Station, P.O. Box 631, Vicksburg, MS 39180, 1982.

Grivas, D.A. and Souflis, C., "Probabilistic Safety Analysis of Earth Retaining Structures During Earthquakes," Report No. NSF/CEE-82030, National Science Foundation, Washington, DC, July, 1982, 136 pp.

Höeg, K. and Murarka, R.P., "Probabilistic Analysis and Design of a Retaining Wall," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, GT3, March, 1974, pp. 349-366.

Holtz, W.G. and Gibbs, H.J., "Engineering Properties of Expansive Clays", Transactions, ASCE, Vol. 120, 1956.

Huntington, W.C., "Earth Pressures and Retaining Walls," John Willey and Sons, Inc. New York, 1957, 534 pp.

Jaky, J., "The Coefficient of Earth Pressure At-Rest," Journal for Society of Hungarian Architects and Engineers, Budapest, Hungary, Oct., 1944, pp. 355-358.

Kulhawy, F.H., Trautmann, C.H., Beech, J.F., O'Rourke, T.D., and McGuire, W., "Transmission Line Structure Foundations for Uplift-Compression Loading", EPRI Report EL-2870, Electric Power Research Institute, 1983.

Laursen, E.W. and Toch, A., "Scour around Bridge Piers and Abutments," Iowa Highway Research Board, Bulletin 4, 1956.

Mayne, P.W. and Kulhawy, F.H., "K_o-OCR Relationships in Soils," Journal of the Geotechnical Engineering, ASCE, Vol. 108, No. GT6, June, 1982, pp. 851-872.

Meyerhof, G.G., "The Ultimate Bearing Capacity of Foundations under Eccentric and Inclined Loads", Proc., 3rd International Conference on Soil Mechanics and Foundation Engineering, Zurich, Vol. 1, 1953, pp. 440-445.

Meyerhof, G.G., "Safety Factors in Soil Mechanics," Canadian Geotechnical Journal, Toronto, Ontario, Canada, Vol. 7, No. 4, Nov., 1970, pp. 349-355.

Meyerhof, G.G., "Safety Factors and Limit States Analysis in Geotechnical Engineering," Canadian Geotechnical Journal, Toronto, Ontario, Canada, Vol. 21, 1984, pp. 1-7.

Munfakh, G.A., "Innovative Earth Retaining Structures: Selection, Design & Performance," Proceeding of Conference on Design and Performance of Earth Retaining Structures, ASCE, Cornell University, Ithaca, New York, June, 1990, pp. 85-118.

Newman, M., "Standard Cantilever, Retaining Walls," McGraw-Hill Publication Co., 1976, 648 pp.

New Zealand Ministry of Works and Development, "Retaining Wall Design Notes," Wellington, New Zealand, July, 1979, 43 pp.

Ooi, P.S.K., Duncan, J.M., Rojiani, K.B. and Barker, R.M., "Engineering Manual for Driven Piles," Preliminary Draft for NCHRP 24-4, VPI & SU, May, 1991.

Ooi, P.S.K., Rojiani, K.B., Duncan, J.M. and Barker, R.M., "Engineering Manual for Drilled Shafts," Preliminary Draft for NCHRP 24-4, VPI & SU, May, 1991.

Peck, R.B., Hansen, W.E. and Thornburn, T.H., "Foundation Engineering," Second Edition, John Wiley and Sons Inc., NY, 1974, 514 pp.

Seed, H.B. and Whitman, R.V., "Design of Earth Retaining Structures for Dynamic Loads," ASCE Speciality Conference on Lateral Stresses in the Ground and Earth Retaining Structures, Cornell University, 1970, pp. 103-147.

Seelye, E.E., "Foundations: Design and Practice," John Wiley and Sons, Inc., NY, 1956.

Sherif, M.A., Ishibashi, I. and Lee, C.D., "Earth Pressures Against Rigid Retaining Walls," Journal of Geotechnical Engineering Division, ASCE, Vol. 108, GT5, 1982, pp. 679-695.

Schnore, A.R., "Selecting Retaining Wall Type and Specifying Proprietary Retaining Walls in NYSDOT Practice," Proceeding of Conference on Design and Performance of Earth Retaining Structures, ASCE, Cornell University, Ithaca, New York, June, 1990, pp. 119-124.

Sibley, E.A., "Backfill Adjacent to Structures," Proceedings of the Montana Conference on Soil Mechanics and Foundation Engineering, Department of Civil Engineering, Montana State University, Bozeman, MT, 1967. Available from: U.S. Army Engineer Waterways experiment Station, P.O. Box 631, Vicksburg, MS 39180.

Skempton, A.W., "The Bearing Capacity of Clays", Proc. of the Building Research Congress, London, England, Vol. 1, 1951, pp. 180-189.

Snyder, R. and Moses, F., "Load Factor Design for Substructures and Retaining Walls," FHWA Report No. 79-S0862, Nov., 1978, 153 pp.

Sowers, G.F., "Shallow Foundations," Chapter 6 in Foundation Engineering edited by Leonards, G.A., McGraw-Hill Publication Co., 1962, pp. 525-632.

Tan, C.K., Duncan, J.M., Barker, R.M. and Rojiani, K., "An Engineering Manual for Shallow Foundations," Preliminary Draft for NCHRP 24-4, VPI & SU, May, 1991.

Tschebotarioff, G.P., "Retaining Structures," Chapter 5 in Foundation Engineering edited by Leonards, G.A., McGraw-Hill Publication Co., 1962, pp. 438-524.

Terzaghi, K., "Retaining Wall Design for Fifteen-Mile Falls Dam," Eng. News Record, May, 1934, pp. 632-636.

Terzaghi, K. and Peck, R.B., "Soil Mechanics in Engineering Practice," John Wiley and Sons Inc., NY, 1967, pp. 729.

U.S. Army Corps of Engineers, "Report on Frost Penetration," Addendum No. 1, 1945-47, Corps of Engineers, US Army, New England Division, Boston, 1949.

U.S. Army Corps of Engineers, "Engineering and Design Retaining and Flood Walls," Manual EM No. 1110-2-2502, Washington, D.C., Sept., 1989.

U.S. Dept. of Navy, "NAVFAC DM7.1, Soil Mechanics," Naval Facilities Engineering Command, VA, May 1982, 348 pp.
U.S. Dept. of Navy, "NAVFAC DM7.2, Foundations and Earth Structures," Naval Facilities Engineering Command, VA, May 1982, 244 pp.

Wu, T.H., "Retaining Walls," Chapter 12 in Foundation Engineering Handbook Edited by Winterkorn H. and Fang H.Y., Van Nostrand Reinhold Co., New York, 1975, pp. 402-417.

NOTATIONS AND SYMBOLS

Symbol		
A	=	maximum expected acceleration of bedrock
b	=	width of pier
B	=	footing base width
B	=	buoyancy (Section 3.4)
B_e	=	effective footing base width
c	=	cohesion
c_a	=	adhesion
CF	=	centrifugal force
D	=	dead load
D_f	=	embedment depth for foundations
DL	=	dead load from bridge superstructure
e	=	eccentricity of the resultant
e	=	void ratio
E	=	earth pressure
E_h	=	horizontal earth load (unfactored)
E_{hD}	=	horizontal earth load due to dead load surcharge
E_{hL}	=	horizontal earth load due to live load surcharge
E_v	=	vertical earth load (unfactored)
E_{vD}	=	vertical earth load due to dead load surcharge
E_{vL}	=	vertical earth load due to live load surcharge
EQ	=	earthquake
f(g)	=	probability density function of g = R - Q
$f_R(r)$	=	probability density function of resistance
$f_Q(q)$	=	probability density function of load
F_r	=	frictional resistance at the footing base (unfactored)
F_{ru}	=	frictional resistance at the footing base (factored)

FS	=	factor of safety
g	=	32.2 ft/sec.^{2}
g	=	R - Q
$\bar{g}$	=	mean of g
H	=	height of the wall
H'	=	effective height of a wall
H_{asd}	=	horizontal load used in ASD
H_D	=	lateral earth pressure force produced by dead load surcharge
H_L	=	lateral earth pressure force produced by live load surcharge
H_t	=	thickness of compressible layer
H_u	=	factored horizontal load
H_{unf}	=	unfactored horizontal loads
i	=	the sloping angle of backfill
I	=	impact due to live load
ICE	=	ice pressure
I_p	=	plasticity index
k_v	=	vertical acceleration coefficient
k_h	=	horizontal acceleration coefficient
K	=	horizontal earth pressure coefficient
K_o	=	coefficient of at-rest earth pressure
K_{oi}	=	coefficient of at-rest pressure for sloping backfills
K_{ou}	=	coefficient of at-rest pressure for overconsolidated soils
K_a	=	coefficient of static active earth pressure
K_p	=	coefficient of static passive earth pressure
K_{ae}	=	coefficient of seismic active earth pressure
K_{pe}	=	coefficient of seismic passive earth pressure
L	=	live load
L	=	length of foundation (Section 6.3.4)
LF	=	longitudinal force from live load
LL	=	live load from bridge superstructure
L_s	=	lateral loads from the bridge superstructure
M	=	modulus of compressibility (Section 6.3.4)
M_{Hasd}	=	moment due to horizontal load (used in ASD)
M_{Hu}	=	moment due to factored horizontal load
M_{Hunf}	=	moment due to unfactored horizontal load
M_u	=	factored moment
M_{unf}	=	unfactored moment
M_{Vasd}	=	moment due to vertical load (used in ASD)
M_{Vu}	=	moment due to factored vertical load
M_{Vunf}	=	moment due to unfactored vertical load
n	=	porosity
N	=	loading group number (Section 3.4)
N	=	Standard Penetration Test (SPT) blow count
$\bar{N}$	=	average SPT blow count
N_{asd}	=	vertical resultant force used in ASD
N_u	=	vertical resultant force due to factored loads
N_{unf}	=	vertical resultant force due to unfactored loads
$N_{32^\circ F}$	=	number of days below 32° F
OCR	=	over consolidation ratio
p	=	average applied bearing pressure under service loading condition
$P1"$	=	bearing pressure corresponding to 1 inch settlement (tsf)
p_o	=	at-rest earth pressure
p_h	=	lateral earth pressure
p_w	=	hydrostatic water pressure
P_a	=	static active earth pressure force
P_E	=	equivalent earth pressure force
P_{ae}	=	seismic active earth pressure force
P_f	=	probability of failure
P_h	=	horizontal component of earth pressure force
P_{hi}	=	horizontal earth pressure force i
P_p	=	static passive earth pressure force
P_{pe}	=	seismic passive earth pressure force
P_s	=	vertical loads from bridge superstructure
P_v	=	vertical component of earth pressure force

q_{max} = maximum bearing stress due to unfactored loads

q_s = uniform surcharge pressure

q_t = triangular bearing pressure at toe

q_{ult} = ultimate unit bearing capacity

q^i_{ult} = ultimate inclined unit bearing capacity

q^v_{ult} = ultimate vertical unit bearing capacity

q_{umax} = maximum bearing stress due to factored loads

q_{unif} = uniformly distributed bearing pressure at toe

Q = load used in LRFD

$\bar{Q}$ = mean of load

Q_i = load component i

R = rib shortening (Section 3.4)

R = resistance used in LRFD

$\bar{R}$ = mean of resistance

R_I = reduction factor applied to q_{ult} due to inclined load

R_n = nominal resistance

S = shrinkage

SF = stream flow pressure

T = temperature effect

T = annual average daily temperature in Fahrenheit

V_{asd} = vertical load used in ASD

V_D = resultant of dead load surcharge

V_L = resultant of live load surcharge

V_u = factored vertical load

V_{unf} = unfactored vertical load

W = wind load on structure

W = weight of wall component or soil

W_c = weight of concrete mass of the wall

W_e = weight of soil mass behind the wall stem and above the footing up to the heel

W_s = weight of surcharge on soil surface behind wall

WL = wind load on live load

X_o = location of the resultant measured from point o at toe

y = position of horizontal resultant force measured from bottom of footing

z = depth below the soil surface of backfill

z_w = depth to ground water table

<u>Greek</u>

α = peak ground acceleration coefficient

β = coefficient used in load combination

β = safety index in reliability analysis

β = angle of wall stem

γ = load factor

γ = unit weight of soils

γ' = submerged unit weight of soils

γ_{eq} = equivalent fluid unit weight

γ_d = dry unit weight of soils (Figure 3.3)

γ_i = load factor for load component i

γ_w = unit weight of water

δ = friction angle between wall and backfill soil

δ_b = friction angle between base and base soil

Δ = movement of top of wall used in Table 4.2

Δp_h = increase in the horizontal pressure due to surcharge load

Δp_v = increase in the vertical pressure due to surcharge load

ΔP_{ae} = increased earth pressure force due to seismic load (Fig. 4.12)

θ = arctan $[k_h / (1-k_v)]$

μ_o = influence factors for immediate settlement, accounting for effect of footing embedment

μ_1 = influence factors for immediate settlement, accounting for effect of finite thickness of a compressible layer

ν	=	Poisson's ratio
ρ	=	settlement of footing
ρ_a	=	allowable differential settlement of footing
ρ'	=	adjusted settlement of footing
σ	=	normal stress in soil
σ_g	=	standard deviation of $g = R - Q$
σ_R	=	standard deviation of R
σ_Q	=	standard deviation of Q
σ_x	=	horizontal stress in soil
σ_z	=	vertical stress in soil
σ_v'	=	effective vertical stress in soil
σ_x'	=	effective horizontal stress in soil
τ	=	shear stress in soil
ϕ	=	performance factor used in LFD
ϕ'	=	drained friction angle in soil or rock
ϕ_f	=	internal friction angle in soil or rock
ϕ_s	=	performance factor for sliding
ω_D	=	uniformly distributed load due to weight of the approach slab
ω_L	=	uniformly distributed load due to live load surcharge
ω_s	=	uniformly distributed load applied to backfill surface (Figure 5.6)

Part 4—Engineering Manual for Drilled Shafts

P.S.K. Ooi, K.B. Rojiani, J.M. Duncan, R.M. Barker

CONTENTS

CHAPTER 1

INTRODUCTION

The primary function of foundations is to transmit loads to the ground safely and to avoid excessive settlements or lateral movements. Drilled shafts, which are used to support many bridges, buildings, and other structures, are especially useful where underlying layers include weak or compressible strata.

The purpose of this manual is to draw together practical procedures for the design of drilled shafts. The theoretical and empirical procedures described provide methods suitable for design of single drilled shafts and groups of drilled shafts that are subjected to vertical and horizontal loads.

The design procedures presented in this manual incorporate the concepts of load factor design, or LFD. The LFD approach provides a logical method of dealing with uncertainties of component loads, strength and behavior, and for incorporating suitable margins of safety. LFD and other procedures similar in format are being used with increasing frequency in civil engineering. Load factor design has been incorporated in the American Association of State Highway and Transportation Officials (AASHTO) specifications for the design of bridge superstructures since the mid-1970s, but not for substructure and foundation design. Therefore, bridge engineers who use LFD for the superstructure must develop two sets of loads—one for design of the superstructure and another for design of the foundations (Barker et al., 1988). The development of load factor design procedures for bridge foundations will make this duplication of effort unnecessary.

In the sections that follow, a brief description of the various methods of constructing drilled shafts is given in Chapter 2. Chapter 3 discusses the design requirements and the factors influencing the safety of drilled shaft foundations. Chapter 4 considers axial loading of shafts, and Chapter 5 presents a new approach for the design of laterally loaded drilled shafts.

CHAPTER 2

CLASSIFICATION OF DEEP FOUNDATIONS AND DRILLED SHAFTS

2.1 TYPES OF DEEP FOUNDATIONS

Deep foundations can be described as columnar elements in the soil which transfer the loads from a superstructure (such as a bridge or a building) into the soil or rock. Deep foundations must be able to support axial, horizontal and uplift loads effectively.

Deep foundations can be divided into two classes: (1) piles that are installed by driving and (2) drilled shafts or drilled piers that are installed by placing concrete in drilled holes. Figure 2.1 shows a typical drilled shaft.

This manual discusses the design aspects of drilled shafts, but does not cover drilled piles installed with continuous flight augers that are concreted as the auger is being extracted. The design of driven piles is dealt with separately in Part 2.

2.2 ADVANTAGES AND DISADVANTAGES OF DRILLED SHAFTS

The advantages of drilled shafts include the following:

1. Excavation during construction allows the bearing stratum to be inspected and tested. If the bearing stratum is inadequate, the drilled shaft can be extended to greater depths.
2. Extending drilled shafts through boulders, rocks, and hard strata is easier than driving piles through such obstacles.
3. Disturbance of foundation soils supporting nearby structures is minimal because drilled shafts are built with less displacement of the ground than is involved in driving displacement piles. The nondisplacement nature of the construction of drilled shafts minimizes heave and settlement during construction.
4. Vibration and noise associated with pile driving can be avoided using drilled shafts.
5. Changes in geometry of drilled shafts can be made readily when needed, e.g., increasing the diameter or length, or adding a bell or an underream (see Figure 2.1).
6. Because drilled shafts can support heavy loads, caps can sometimes be eliminated, resulting in lower cost.
7. Drilled shafts can be socketed into scour-resistant materials such as soft rock, whereas driven piles often cannot penetrate such materials.

Disadvantages stem from:

1. Successful drilled shaft construction is very much dependent on the skills and experience of the contractor. This can be disadvantageous if the contractor is not skilled.
2. The action of driving piles in certain soils (e.g., loose sands) results in densification of the soil, thereby increasing the skin friction. The excavation process during drilled shaft construction, however, results in stress relief and possibly expansion of the soil, which can lead to larger settlements.
3. Drilled shafts are seldom used in soft clays and are seldom constructed in soils under artesian conditions because of the

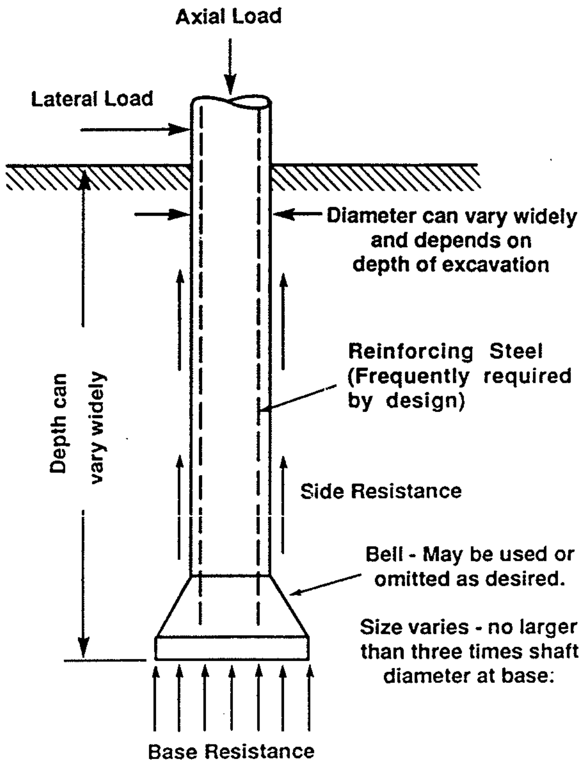


Figure 2.1. A typical drilled shaft. (From Reese and O'Neill, 1988)

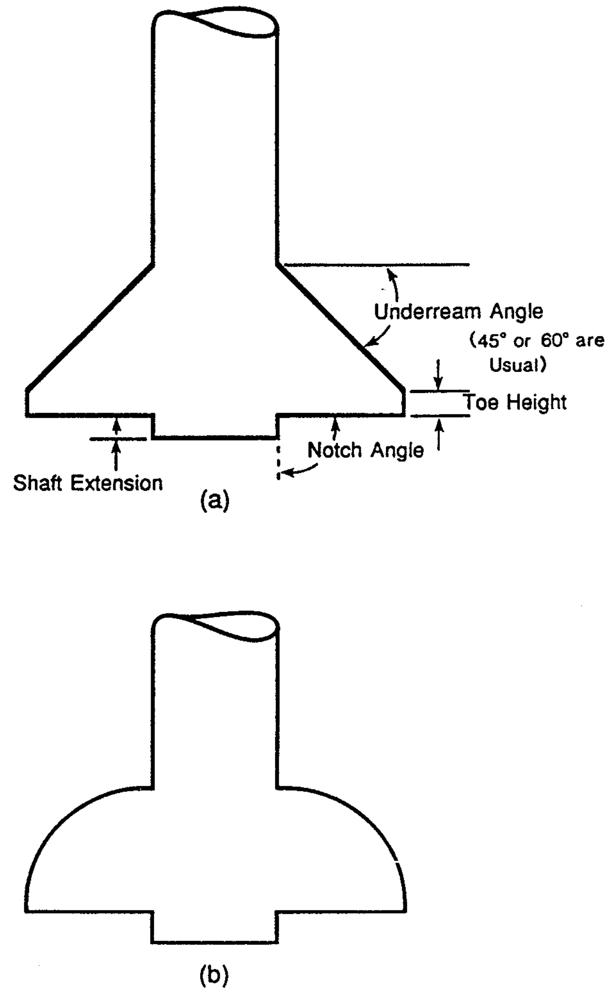


Figure 2.2. Shapes of typical underreams (a) cut with a "standard" reamer; (b) cut with a "bucket" reamer. (From Reese and O'Neill, 1988)

problem of maintaining stability of the drilled hole.

4. Drilled shafts are sometimes used individually in place of a group of driven piles. In a group of piles, failure of one pile does not necessarily result in failure of the entire foundation. However, this type of redundancy is not provided by a single drilled shaft.

5. The construction operation generally causes a reduction in the shear strength of the soil.

6. Small diameter drilled shafts cannot be constructed very successfully.

2.3 CONSTRUCTION METHODS FOR DRILLED SHAFTS

Different subsurface conditions warrant different methods of construction. Three methods commonly used include the dry, casing and wet methods. Selection of the method of construction is part of the design process, because the design is very much affected by the method of construction.

Drilled shafts are almost always installed vertically. Constructing batter drilled shafts is difficult because of problems in maintaining hole stability during excavation, installing casing and rebar cages in inclined holes, placing concrete in inclined holes, and finding suitable construction equipment.

The methods of constructing drilled shafts described in this section are the ones most commonly used in the United States, where the holes are excavated using a rotary drill. The construction of underreams is discussed first, followed by the dry, casing and wet methods of construction.

2.3.1 Underreams (or Bells)

Underreams or bells have decreased in popularity because: (1) recent advancements in the technology of drilled shafts have shown that straight sided drilled shafts can be designed effectively for most conditions; (2) construction of underreams poses several problems, especially in maintaining the stability of the excavation for the bell in soils that have low shear strengths, joints and groundwater flow; (3) the settlement necessary to mobilize the capacity of the underream sometimes exceeds the tolerable settlement of the structure; and (4) the outer portions of underreams are difficult to reinforce. As a result, they are seldom reinforced.

However, underreams are still favorable for use in stiff clays or shales, and in subsoil conditions where the bedrock is fairly near the ground surface.

The general shapes of underreams are shown in Figures 2.2(a) and 2.2(b). The maximum diameter of the underream is three times the shaft diameter. Because the outside of the bell cannot be reinforced, the toe height and underream angle can be varied to limit the tensile stresses in the bell to allowable values.

Table 2.1. Maximum net bearing pressure that can be sustained in underreams without cracking. (After Reese and O'Neill, 1988)

Underream Angle	f_c'*	
	3 ksi	4 ksi
45°	8 ksf	15 ksf
60°	16 ksf	25 ksf

These values apply to bells with toe heights of 3 in. and a base diameter equal to three times the diameter of the shaft.

* f_c' = 28 day concrete compressive strength

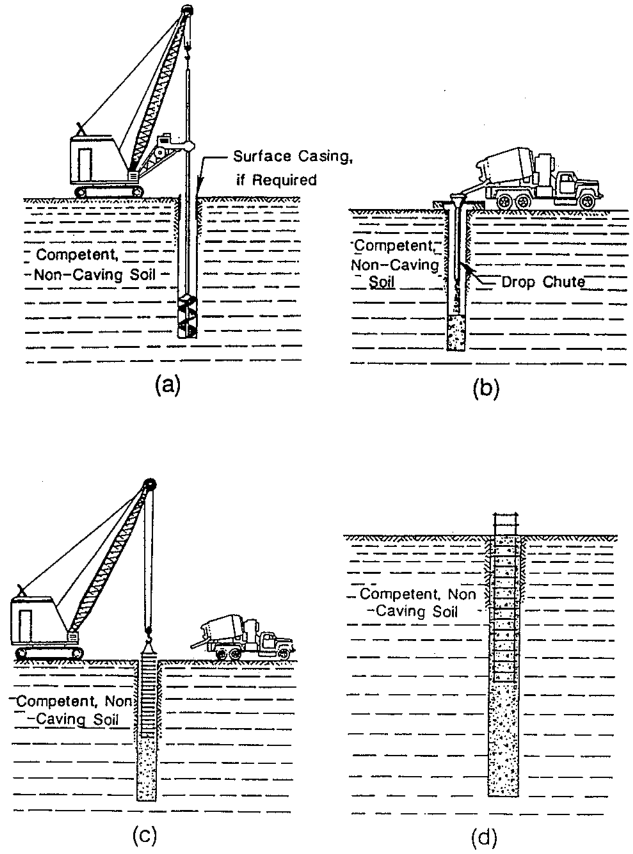


Figure 2.3. Dry method of construction: (a) initiating drilling, (b) starting concrete pour, (c) placing rebar cage, (d) completed shaft. (From Reese and O'Neill, 1988)

The shaft extension or reamer seat provides a means of centering the underream during construction. However, it may not be necessary if special underreaming tools are used. The notch angle is typically 90 deg, but may be rounded off during construction.

The maximum tensile stress in an unreinforced bell under axial loading with a given toe height will occur near the notch because of stress concentration effects. Table 2.1 shows the guidelines provided by Reese and O'Neill (1988) for the maximum bearing stress permitted in an unreinforced underream, with a toe height of 3 in. and a bell diameter equal to 3 times the shaft diameter.

Figure 2.2(b) shows an alternate shape for an underream cut with a bucket reamer. Although the construction of a "bucket-cut" underream requires more concrete and is more difficult to clean out, it has the advantage of being able to carry a larger tensile stress.

2.3.2 Dry Method of Construction

The dry method of constructing drilled shafts is applicable to soils above the groundwater table that are stable against caving when the excavation reaches its full depth; for example, homogeneous stiff clays, and sands with some cohesion or apparent cohesion. This method is also applicable to soils below the water table if the soil has a low permeability and, therefore, has a small rate of seepage into the excavation.

Figure 2.3 shows the steps involved in this method of construction. The construction procedure is as follows (Reese and O'Neill, 1988):

1. Position appropriate drilling equipment and excavate to the intended depth. Excavation for underreams can be made but is not shown in Figure 2.3(a).
2. Pour concrete in the hole using care not to strike the sides of the excavation, as shown in Figure 2.3(b).
3. Place rebar cage in hole. To prevent segregation, use a tremie or a drop chute to place the rest of the concrete (Figure 2.3(c)). The rebar cage can be omitted or used through the entire length of the drilled shaft depending on the conditions of loading. The rebar cage shown in Figure 2.3(c), for example, is placed in the upper portion of the shaft only.
4. Pour concrete until the shaft is completely constructed.

2.3.3 Casing Method of Construction

The casing method is applicable to soils that will cave or deform excessively when the hole is excavated; an example is clean sand below the water table.

Figure 2.4 shows the sequence of construction of an underream shaft in a caving stratum sandwiched between two competent strata. The procedure is as follows (Reese and O'Neill, 1988):

1. Position drilling equipment and excavate until the stratum of caving soil is encountered (Figure 2.4(a)).
2. Introduce slurry when the caving soil is reached (Figure 2.4(b)). As the excavation proceeds, the slurry should be cleaned by circulating it through screens and cyclones or by using a clean-out bucket. The slurry usually consists of a suspension of bentonite in water if the groundwater is fresh water. In salt-water environments, either attapulgite or bentonite with added deflocculants may be used in the slurry.
3. Introduce the casing through the caving soil into the impermeable stratum (Figure 2.4(c)). There should be a sufficient depth of embedment of the casing in the underlying impermeable soil to effect a seal.
4. Remove the slurry from inside the casing using a bailing bucket (Figure 2.4(d)).

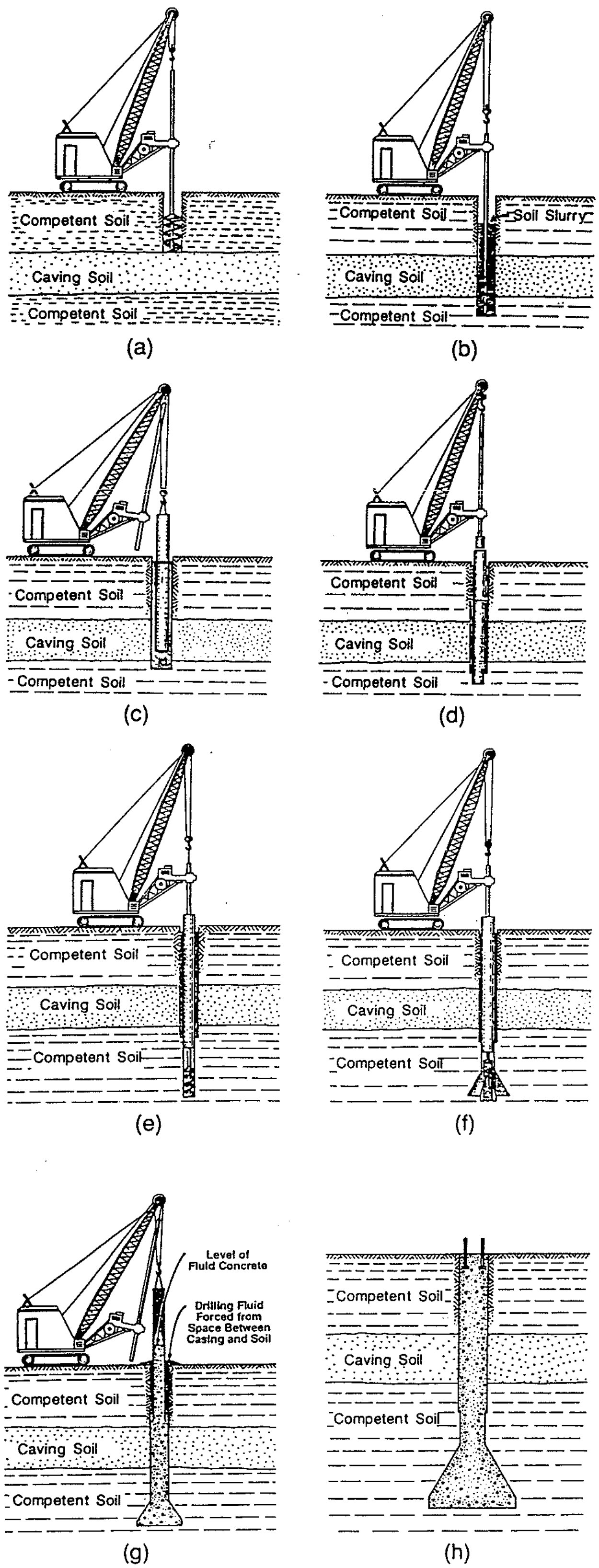


Figure 2.4. Casing method of construction: (a) initiating drilling, (b) drilling with slurry, (c) introducing casing, (d) casing sealed and slurry being removed from interior of casing, (e) drilling below casing, (f) underreaming, (g) removing casing, and (h) completed shaft. (From Reese and O'Neill, 1988)

5. Proceed with the excavation using a smaller drill that will pass through the casing (Figure 2.4(e)).

6. Introduce a belling tool to excavate the bell-shaped underream (Figure 2.4(f)).

7. After the reinforcing steel cage has been placed, fill the hole with fresh concrete and remove the casing (Figure 2.4(g)).

The seal at the bottom of the casing in the lower impermeable stratum should not be broken until the hydrostatic pressure exerted by the concrete is sufficient to lift the slurry that is trapped behind the casing. Once the seal is broken when the casing is withdrawn, concrete will flow around the base of the casing to displace the slurry in the annular space between the soil and the casing. The withdrawal of the casing is the most crucial operation for the following reasons: (a) if there is a premature set in the concrete, the concrete will move up with the casing forming a crack into which slurry can flow, (b) if the workability of the concrete is too low, the same can happen, and (c) if the casing is pulled too soon, slurry can flow into the concrete and contaminate it. (To achieve good workability of concrete, it is important that the slump is in the range of 6 to 8 in., and that the residual slump is approximately 4 in., 4 hours after mixing.)

8. Figure 2.4(h) shows the completed underream shaft.

According to Reese and O'Neill (1988), some engineers believe that the shaft resistance along the portion of the drilled pier where the casing is placed is less than the shaft resistance of piers constructed using the dry or wet methods. However, the design methods described in Section 4 of this manual assign the same shaft resistance, independent of the method of construction, unless otherwise stated. Design methods can be considered independent of construction method, provided construction specifications are written so as to exclude entrapment of slurry or other deleterious material in the completed drilled shaft, and that construction according to the specifications is assured through knowledgeable field inspection.

Figure 2.5 shows a different variation of the casing method of construction. This alternative is applicable to constructing drilled shafts in caving soils that are below the water table and that overlie an impermeable stratum. The main difference in this method from the method described earlier is that the casing is vibrated or driven into place through the sand and into the impermeable stratum. Disadvantages of this method include: (a) settlement of the soil at the ground surface due to densification, and (b) difficulty of removing the casing. This method is advantageous in that larger values of shaft resistance can be attained due to soil densification and prohibition of relaxation of ground stresses as a result of driving or vibrating the casing into place.

2.3.4 Slurry Method of Construction

The slurry (or wet) method of construction is applicable to sites with caving soils such as the ones described in Section 2.3.3. The wet method is especially useful when it is not possible to seal the casing in an impermeable stratum below the caving soil. This method is suitable for construction of drilled piers in lakes, rivers, and marine environments with the aid of barges.

Figure 2.6 shows the sequence of construction using the wet method. The procedure is as follows:

1. Position the drilling equipment and commence drilling until the caving stratum is reached. Introduce slurry and continue

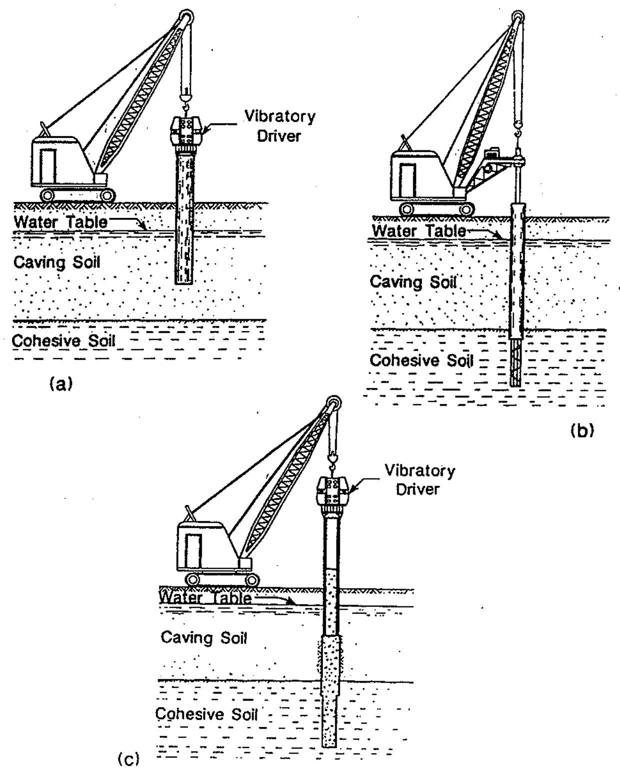


Figure 2.5. Alternative method of construction with casing: (a) installation of casing, (b) drilling ahead of casing, (c) removing casing with vibratory driver. (From Reese and O'Neill, 1988)

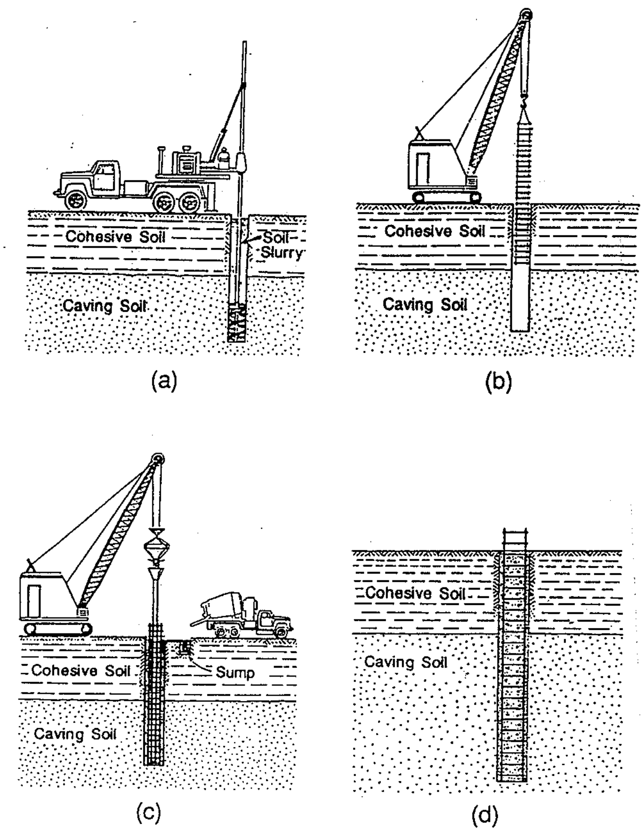


Figure 2.6. Slurry method of construction: (a) drilling to full depth with slurry, (b) placing rebar cage, (c) placing concrete, (d) completed shaft. (From Reese and O'Neill, 1988)

drilling till the full depth of the excavation is completed. The drilling tool should have a hollow stem or other suitable pressure relief device so the slurry can flow through, and a vacuum will not form beneath the drilling tool. If a vacuum forms, the hole may possibly collapse.

Another important consideration is that the slurry should be representatively sampled and tested to ensure that it meets appropriate standards. The slurry should be tested for density, viscosity, and pH. Other properties, such as sand content, shear strength, free water, and cake thickness, can be measured or related to the first three properties (Stebbins and Williams, 1986). (Note that a filter cake is formed when bentonite particles from the slurry flow into the soil and remain in the voids. The filter cake prevents loss of slurry from the excavation, and also acts to prevent caving of the excavation. The distance penetrated by the bentonite into the soil is called the cake thickness.) A thorough review of slurry specifications for constructing drilled shafts in different soil conditions is given by Reese and O'Neill (1988).

2. Place the rebar cage.

3. Pour concrete in with tremie or pump to displace the lower density slurry. Care must be taken to ensure that the tremie is positioned and maintained at the bottom of the excavation until a height of 5 ft of concrete has been poured. As more concrete is added, the tremie should be maintained at a minimum distance of 5 ft below the top of the concrete pour.

4. Figure 2.6(d) shows the completed shaft.

2.4 IMPLICATIONS OF CONSTRUCTION ON DESIGN

The design methods described in this manual assume that construction of drilled shafts is done under good quality control and supervision, and that the finished shafts have high integrity. For example, if the drilled shaft is constructed using the casing method and if the casing is removed when the concrete has set prematurely, thereby allowing soil or slurry to move into the gap created during withdrawal of the casing, the bearing capacity of the shaft will be greatly reduced.

Some possible construction problems are shown in Figure 2.7 (Reese and Wright, 1977). Table 2.2 summarizes some solutions to these construction problems.

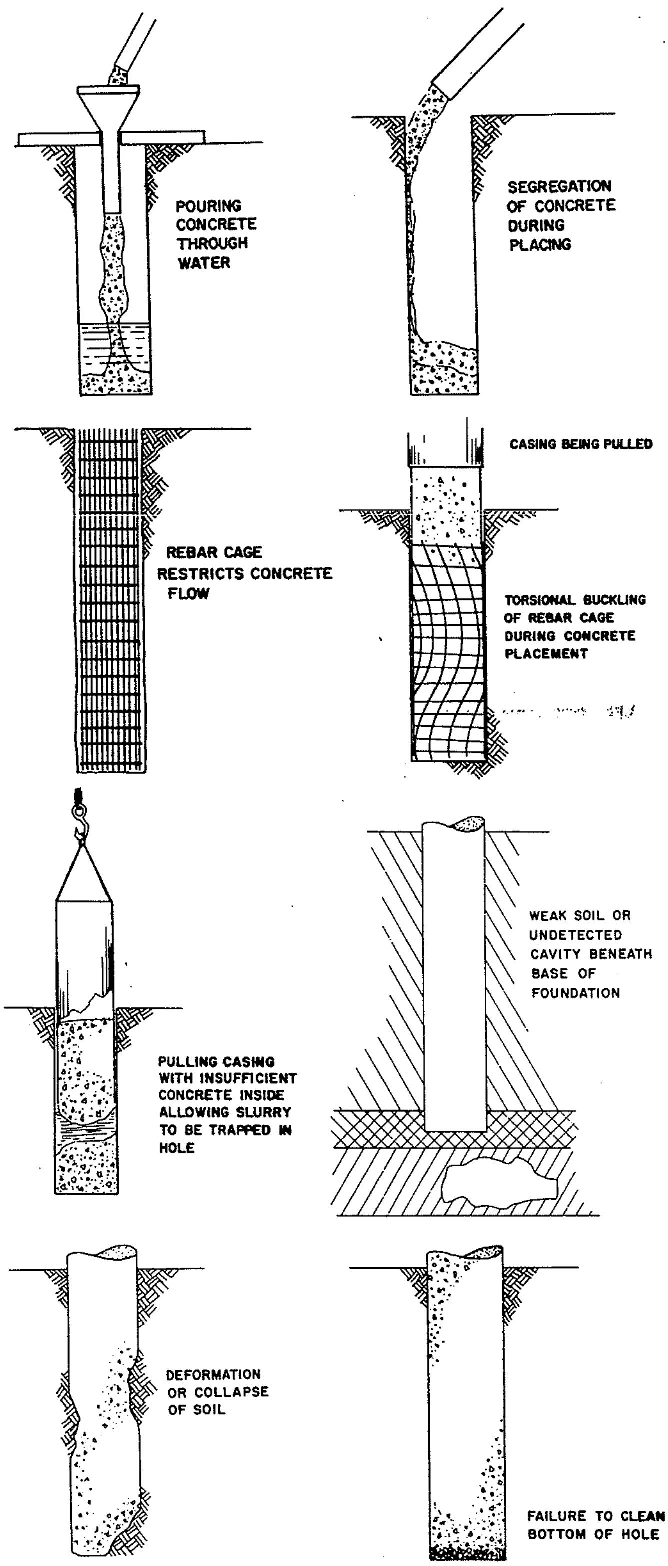


Figure 2.7. Construction deficiencies. (From Reese and Wright, 1977)

Table 2.2. Possible construction problems with drilled shafts. (After Reese and Wright, 1977)

PROBLEM	SOLUTION
Pouring concrete through water	Removal of water by bailing or use of tremie
Segregation of concrete during placing	If free-fall is employed, exercising care to see that concrete falls to final location without striking anything, or use of tremie
Restricted flow of concrete through or around rebar cage	Designing of rebar cage with adequate spacing for normal concrete (all clear spaces at least three times the size of largest aggregate) or use of special mix with small-sized coarse aggregate
Torsional buckling of rebar cage during concrete placement with casing method	Strengthening rebar cage by use of circumferential bands welded to lower portion of cage, use of concrete with improved flow characteristics, use of retarder in concrete allowing casing to be pulled very slowly
Pulling casing with insufficient concrete inside	Always having casing extending above ground surface and always having casing filled with a sufficient head of concrete with good flow characteristics before casing is pulled
Weak soil or undetected cavity beneath base of foundation	Requiring exploration to a depth of a few diameters below the bottom of the excavation
Deformation or collapse of soil	Such problems are readily detected by even the minimum of inspection
Failure to clean bottom of hole	Modern construction methods permit all but a negligible amount of loose material to be removed by construction tools; inspection can be accomplished from ground surface

CHAPTER 3

DESIGN REQUIREMENTS FOR DRILLED SHAFT FOUNDATIONS

Deep foundations must be capable of transmitting the loads to the soil without reaching a "limit state." A limit state is reached when the structure no longer fulfills its design requirements. There are two types of limit states: (1) An *ultimate limit state* corresponds to the maximum load-carrying capacity of the foundation (this may be reached through either structural or soil failure); an ultimate limit state corresponds to complete collapse. (2) A *serviceability limit state* corresponds to loss of serviceability, and occurs before collapse. A serviceability limit state involves unacceptable deformations or undesirable damage levels. This may be reached through excessive differential or total foundation settlements, excessive lateral displacements, or structural deterioration of the foundation.

3.1 LOAD FACTOR DESIGN CONCEPT

In load factor design (LFD), it is recognized that loads and resistances are probabilistic and not deterministic in nature. Different types and magnitudes of loads have varying probabilities of occurrence. In order to account for their differing probabilities of occurrence, each load component is amplified by a load factor, the value of which depends on the level of uncertainty of the load component.

The factored loads are compared to the design strengths or resistances. The design resistances are obtained by multiplying nominal values of resistances by performance factors, usually denoted as ϕ. The objective of a design is to ensure that the design resistance is greater than or equal to the sum of the factored loads, i.e.,

$$\phi R \geq \Sigma \gamma_i Q_i \tag{3.1.1}$$

where ϕ = performance factor, R = resistance corresponding to the limit state considered, Q_i = load effect due to load component i, and γ_i = load factor for load component i.

Various combinations of loads are considered in design, to ensure that the structure and foundation will have sufficient capacity to resist all of the types of loading to which it may be subjected during its life. This manual uses the load factors and load combinations described in the 1989 AASHTO specifications for the design of bridges.

3.2 LOAD FACTORS AND LOAD COMBINATIONS

Loads acting on bridge superstructures include one or more of the following: dead load, live and impact loads, thrust due to earth pressures, buoyancy, wind load, longitudinal and centrifugal forces caused by moving vehicles, earthquake loads, stream and ice flow forces, and forces induced by changes in the dimensions of the structure, such as shrinkage and temperature effects.

One difference between the loads acting on the bridge superstructure and those that act on the foundation is that impact loads are usually assumed to be fully dissipated before reaching the foundation. However, bent piers and integral abutments should be designed to carry impact loads, and these are the most common substructures in which drilled shafts are used. The load combinations and load factors for the design of the superstructure, as given in the 1989 AASHTO specifications, can be used for the design of foundations as follows:

$$\begin{aligned}\text{Total Load} = \gamma[&\beta_D D + \beta_L L + \beta_C CF + \beta_E E + \beta_B B + \\ &\beta_{SF} SF + \beta_W W + \beta_{WL} WL + \beta_{LF} LF + \\ &\beta_R(R + S + T) + \beta_{EQ} EQ + \beta_{ICE} ICE]\end{aligned} \tag{3.2.1}$$

where γ = load factor (see Tables 3.1 and 3.2), β = coefficient (see Tables 3.1 and 3.2), D = dead load, L = live load, E = earth pressure, B = buoyancy, W = wind load, WL = wind load on live load, 100 lb per linear ft; and LF = longitudinal force from live load, CF = centrifugal force, R = rib shortening, S = shrinkage, T = temperature, EQ = earthquake, SF = stream flow pressure, ICE = ice pressure.

The factored load combinations considered by AASHTO are given in Table 3.1. Each line in the table, designated by loading

Table 3.1. Table of coefficients of γ and β for ultimate limit states. (After AASHTO, 1989)

Col.No.	1	2	3	3A	4	5	6	7	8	9	10	11	12	13
		β-FACTORS												
GROUP	γ	D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE
I	1.3	β_D	1.67	0	1	β_E	1	1	0	0	0	0	0	0
IA	1.3	β_D	2.2	0	0	0	0	0	0	0	0	0	0	0
IB	1.3	β_D	0	1	1	β_E	1	1	0	0	0	0	0	0
II	1.3	β_D	0	0	0	β_E	1	1	1	0	0	0	0	0
III	1.3	β_D	1	0	1	β_E	1	1	0.3	1	1	0	0	0
IV	1.3	β_D	1	0	1	β_E	1	1	0	0	0	1	0	0
V	1.25	β_D	0	0	0	β_E	1	1	1	0	0	1	0	0
VI	1.25	β_D	1	0	1	β_E	1	1	0.3	1	1	1	0	0
VII	1.3	β_D	0	0	0	β_E	1	1	0	0	0	0	1	0
VIII	1.3	β_D	1	0	1	β_E	1	1	0	0	0	0	0	1
IX	1.2	β_D	0	0	0	β_E	1	1	1	0	0	0	0	1

$(L+I)_n$ - Live load plus impact for AASHTO Highway H or HS loading
$(L+I)_p$ - Live load plus impact consistent with the overload criteria of the operation agency.

β_E = 1.3 for lateral earth pressure for retaining walls and rigid frames.
β_E = 0.5 for lateral earth pressure when checking positive moments in rigid frames.
β_E = 1.0 for vertical earth pressure
β_D = 1.0 for flexural and tension members

For Column Design

β_D = 0.75 when checking member for minimum axial load and maximum moment or maximum eccentricity
β_D = 1.0 when checking member for maximum axial load and minimum moment

Table 3.2. Table of coefficients of γ and β for serviceability limit states. (After AASHTO, 1989)

Col.No.	1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14
		β-FACTORS													
GROUP	γ	D	$(L+I)_n$	$(L+I)_p$	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE	%
I	1	1	1	0	1	β_E	1	1	0	0	0	0	0	0	100
IA	1	1	2	0	0	0	0	0	0	0	0	0	0	0	150
IB	1	1	0	1	1	β_E	1	1	0	0	0	0	0	0	**
II	1	1	0	0	0	1	1	1	1	0	0	0	0	0	125
III	1	1	1	0	1	β_E	1	1	0.3	1	1	0	0	0	125
IV	1	1	1	0	1	β_E	1	1	0	0	0	1	0	0	125
V	1	1	0	0	0	1	1	1	1	0	0	1	0	0	140
VI	1	1	1	0	1	β_E	1	1	0.3	1	1	1	0	0	140
VII	1	1	0	0	0	1	1	1	0	0	0	0	1	0	133
VIII	1	1	1	0	1	1	1	1	0	0	0	0	0	1	140
IX	1	1	0	0	0	1	1	1	1	0	0	0	0	1	150

$(L+I)_n$ - Live load plus impact for AASHTO Highway H or HS loading
$(L+I)_p$ - Live load plus impact consistent with the overload criteria of the operation agency.

$$\text{** Percentage} = \frac{\text{Maximum Unit Stress (Operating Rating)}}{\text{Allowable Basic Unit Stress}} \times 100$$

% in Column 14 is the maximum permissible percentage of basic unit stress for load group indicated

No increase in allowable unit stresses shall be permitted for members or connections carrying wind loads only.

β_E = 1.0 for vertical and lateral loads on all structures except reinforced concrete boxes.
β_E = 1.0 and 0.5 for lateral loads on rigid frames (check both loadings to see which one governs)

group numbers I through IX, gives the values of the load factors, γ, and the coefficients, β, that govern the contributions to the total load. For example in group (load combination) I, total load $= 1.3\ (D + 1.67L_n + CF + \beta_E E + B + SF)$.

Loading groups I, II, and III usually apply to the design of the superstructures and substructures; groups IV, V, and VI apply usually to the design of arches and frames; while groups VII, VIII, and IX apply usually to the design of substructures (Heins and Firmage, 1979). Column 14 of Table 3.2 gives the percentage increase in allowable stresses permitted in the load combinations, and is mainly used in working stress design. The increase in allowable stresses accounts for the fact that the probability of the load components reaching their maximum values simultaneously varies from one load combination to another.

Although uplift load considerations on foundations caused by expansive soils are not shown in Tables 3.1 and 3.2, they nevertheless should be considered. Expansive soils are "prevalent in a belt extending from Texas northward through Oklahoma, into the upper Missouri valley" (Peck, Hanson and Thornburn, 1974). Loads caused by expansive soils will be discussed later in this chapter.

3.3 DESIGN REQUIREMENTS FOR DRILLED SHAFTS

Drilled shafts should be designed for both axial and lateral loading conditions. The two principal design considerations for drilled shafts under axial loads are ultimate load capacity and settlement. The ultimate load capacity of a drilled shaft may be governed either by the structural capacity of the drilled shaft or the bearing capacity of the soil. Drilled shafts that are subjected to lateral loads must also be safe against ultimate failure of the soil or the concrete shaft, and excessive lateral deflections.

3.3.1 Structural Capacity

Axially loaded drilled shafts may fail in compression or by buckling. Buckling may occur in long and slender shafts that extend for a portion of their lengths through water or air. Scour of the soil around the shafts could expose portions of their lengths and increase the likelihood of buckling.

A drilled shaft will fail in compression when the loads exceed the structural or soil capacity. The structural capacity of the shaft is usually greater than the ultimate soil capacity except when the shaft bears on sound rock. Nevertheless, the adequacy of the drilled shaft against structural failure must always be checked. The tensile capacity of drilled shafts should also be checked when the drilled shafts are subject to uplift loads.

Laterally loaded shafts will fail in flexure if the induced bending moment exceeds the moment capacity of the shafts. The structural capacity of the drilled shaft is dependent on both the moment and axial load. The structural adequacy of drilled shafts is checked using load-moment interaction diagrams. These are envelopes of the combinations of moment and axial load that would cause structural failure.

3.3.2 Soil Capacity

The ultimate bearing capacity of a drilled shaft is the sum of its tip and shaft capacities. During failure, the shear stress at the interface of the drilled shaft and soil reaches a limiting value. This can occur under both compressive and upward loads.

Drilled shafts in saturated clays are usually designed using total stress analyses where the undrained shear strength of the clay is used. Long-term loads will lead to an increase in the shear strength of the clay around the shaft as the clay consolidates with time. Associated with this consolidation will be some settlement of the foundation. However, there is a possibility that negative pore pressures can develop along the sides of drilled shafts in heavily overconsolidated clays or shales, leading to soil softening over time. As a result, total stress methods of analyzing drilled shafts in heavily overconsolidated clays or shales may be unconservative. In this case, Reese and O'Neill (1988) proposed using the undrained shear strength measured on a triaxial or direct shear specimen that has previously been allowed to imbibe water. The purpose of the imbibition is to approximate the long-term softening behavior.

Chapter 4 presents the α method (the total stress method of analysis) for drilled shafts in clays (Reese and O'Neill, 1988). Empirical methods based on the standard penetration test (SPT) are described for drilled shafts in sands. Drilled shafts may be used individually or in groups. The bearing capacity of drilled shafts and groups of drilled shafts is addressed in Chapter 4.

The ultimate capacity is usually not a controlling factor in the design of drilled shafts to resist lateral loads. The governing criterion in lateral load design is usually either maximum tolerable deflection or structural capacity.

3.3.3 Movement

Horizontal movements in buildings are caused by wind loads, earth pressures, and earthquakes. Horizontal movements occur at bridge abutments and piers because of lateral forces from earth pressure, wind loads, stream flow forces, braking forces of vehicles, and earthquakes. Lateral movements of buildings must be limited to prevent architectural and structural damage. Lateral movements of abutments and piers must be limited to prevent damage to bearings and expansion joints (functional and structural damage), and poor ride quality.

Excessive movements of foundations supporting bridges may lead to discontinuities in the slope of the riding surface, damage to the bridge superstructure or substructure, jamming of bearings and expansion joints, or even collapse. It is necessary in bridge design to estimate the maximum settlement and lateral movement anticipated in the foundations, and to ensure that they fall within tolerable limits. If both vertical and horizontal displacements are possible, the horizontal displacement of bridge foundations should be limited to 1 in. If vertical displacements are small, the horizontal displacement should be limited to 1.5 in. (Moulton et al., 1985).

Load tests on instrumented drilled shafts have shown that the movement required to mobilize shaft resistance in drilled shafts is smaller than that required to mobilize end-bearing. The shaft capacity in clays is fully mobilized when the settlement is less than 1 percent of the shaft diameter. The end bearing of drilled shafts in clay, however, is not mobilized until the shaft settles about 2 percent to 5 percent of its diameter. For drilled shafts in sands, the side resistance is fully mobilized at settlements less than 1 percent of its diameter. However, very large displacements are required to fully mobilize the end-bearing of drilled shafts in

sands and the tolerable settlement will usually be exceeded much before the end-bearing is fully mobilized. This is an important design consideration when the working load acting on the drilled shaft exceeds the shaft resistance. In this case, larger settlements may be required to mobilize the portion of the end-bearing that supports the load not carried by the side resistance. For design purposes, the "ultimate" end-bearing capacity of drilled shafts in sand is usually limited to the capacity mobilized at a settlement of 5 percent of the diameter of the drilled shaft.

3.4 SPACING OF DRILLED SHAFTS

Drilled shafts should be spaced far enough apart so that extraction of the casing during construction will not adversely affect their integrity. To achieve this, the minimum spacing between adjacent piers (edge-to-edge, at their closest points) should not be less than 3 ft. This minimum spacing must include proper allowances for cantilever tolerance of alignment and possible oversize of the drilled hole. Consistent with this requirement, it is desirable to minimize the spacing between adjacent piers in order to minimize the cost of the cap.

3.5 EXPANSIVE OR SWELLING SOILS

Soils which have a tendency to undergo volume change due to variations in the moisture content are considered expansive. A quantitative measure of soil expansivity is the coefficient of linear extensibility or "COLE," which gives an estimate of the vertical component of swelling of the soil. Krohn and Slosson (1980) defined low, moderate, and high expansivity as follows:

High—Generally includes soils high in clay, that are made up of a large percentage of montmorillonitic minerals. These soils have a COLE value usually greater than 6%.

Moderate—Generally includes soils containing moderate amounts of clay that also contain some montmorillonitic minerals. COLE values for these soils vary between 3% and 6%.

Low—Generally includes soils containing some clay; however, the clay consists mainly of kaolinite and/or other low shrink/swell clay minerals. These soils have COLE values generally lower than 3%.

The Waterways Experiment Station (WES) classification method provides a useful classification for swell potential in terms of the Atterberg limits and the swell suction pressure measured in a soil suction test. Table 3.3 gives the WES classifications for low, marginal and highly expansive soils.

Table 3.3. WES method of identifying potentially expansive soils. (After Reese and O'Neill, 1988)

LL	PI	Suction Pressure (tsf)	Potential Swell (%)	Potential Swell Classification
> 60	> 35	> 4	> 1.5	High
50-60	35-35	1.5-4	0.5-1.5	Marginal
< 50	< 25	< 1.5	< 0.5	Low

The depths of seasonal moisture change where expansive soils occur can be identified by the following observations: (1) depth to which jointed, slickensided and blocky soils are obtained in tube samples; (2) depth to which there is a change in color; (3) depth to which moisture content is erratic; and (4) depth to which the liquidity index is erratic. The liquidity index usually approaches a constant value in zones of stable moisture.

The following steps should be taken when the soils are suspected to be expansive (Reese and O'Neill, 1988):

1. Identify the expansive soil.
2. Estimate the depth of the expansive stratum.
3. Estimate the amount of swell.
4. Estimate the uplift force (see Reese and O'Neill, 1988).
5. Select the appropriate construction alternative. Reese and O'Neill (1988) discussed three measures to counter uplift loads due to swelling soils (Figure 3.1). Figure 3.1(a) shows the use of a permanent surface casing to separate the swelling soil from the drilled shaft. Figure 3.1(b) shows the composite use of a drilled shaft at the bottom and a steel H-pile in the expansive stratum. The steel H-pile can be coated with an asphaltic layer so that the uplift loads are not transmitted through the weak concrete. Figure 3.1(c) shows the use of reinforcement to resist the tensile stresses caused by the uplift loads.

3.6 OTHER DESIGN CONSIDERATIONS

3.6.1 Scour

Scour around bridge foundations can create a severe safety hazard. Therefore, bridge foundations should be designed to survive the effects of possible scour. Geotechnical analyses of bridge foundations should be performed assuming that the soil above the estimated scour line has been removed, and is not available to provide bearing or lateral support (FHWA, 1988).

Three possible effects of scour should be considered in design (FHWA, 1988): (1) *Aggradation and degradation*—aggradation is the deposition of stream bed material eroded from other portions of a stream, whereas degradation is the removal of stream bed material thereby lowering the bed elevation. Aggradation and degradation are long-term effects caused by natural or man-made conditions. (2) *General scour and contraction scour*—general scour and contraction scour are characterized by the removal of stream bed material across the entire width of the stream. Flow velocities increase as a result of contraction of the flow channel or change in the downstream water surface elevation. One instance when contraction scour may occur is when the approach embankment of a bridge encroaches into the channel of a stream or its flood plain. (3) *Local scour*—occurs when bed material is removed from a small portion of the width of the stream. Obstructions to flow, such as bridge piers and abutments, induce acceleration of the flow, causing vortices that wash away the bed material.

Scour is usually evaluated for a flood with a return period of about 100 years. The FHWA recommends that: (1) the top of the cap should be located below the depth of contraction scour to reduce obstruction to flow and to minimize local scour; and (2) a few long shafts should be used rather than many short piers. This results in higher safety against drilled shaft failure due to scour.

3.6.2 Deterioration

Drilled shafts may be attacked by deleterious substances in the ground, such as organic materials, acids, sulphates, and salt. Abrasion of concrete can occur if the drilled shafts are exposed to soils being moved by currents and waves, floating debris, and ice. High quality concrete and ample cover for the reinforcement provide protection against abrasion and corrosion. In an environment rich in sulphates, sulphate resisting cement should be used in the concrete mix.

3.6.3 Miscellaneous

Other considerations that may affect the choice and design of the foundation include the type of equipment available for construction, the availability of field load test equipment, and noise and vibratory restrictions in the area. For example, if a school or hospital is nearby, drilled shaft foundations may be preferred over driven piles.

3.7 DESIGN PROCEDURE FOR DRILLED SHAFT FOUNDATIONS

The design of drilled shaft foundations involves the following steps:

1. Develop a soil profile based on soil borings for the site. Include details of strength profiles, compressibility characteristics, stress history and geology of the soils, and identify the favorable and unfavorable zones in the subsoil.
2. Estimate the loads for the ultimate and the serviceability limit states. Loads due to negative skin friction should be included.
3. For stream crossings, determine the water profiles for the site and the expected depth of scour during flood.
4. Select the length of the drilled shafts.
5. Estimate the axial capacity considering both soil and structural capacity.
6. Determine the required number of drilled shafts and their spacing and locations.
7. Estimate the capacity of the single drilled shaft or the group of drilled shafts. If the capacity is not sufficient for a single shaft, increase the length or diameter of the shaft. If the capacity is not sufficient for a group of shafts, increase the number of shafts or the spacing between the shafts.
8. Check for possible punching of the drilled shaft or group of drilled shafts into any weak stratum that may be present beneath the bearing stratum.
9. Determine the tolerable settlement of the drilled shaft or group of drilled shafts and estimate its settlement. If the settlement exceeds the tolerable value for a single drilled shaft, either increase the length or the diameter of the base of the shaft. If the settlement exceeds the tolerable value for a group of drilled shafts, increase the length of the shafts or the spacing.
10. Check the uplift capacity of the drilled shaft or group of drilled shafts, if it will be subject to uplife loads.
11. Check the structural capacity of the drilled shaft or group of drilled shafts under lateral loading.
12. Determine the tolerable lateral displacement of the drilled shaft or group of drilled shafts and calculate the lateral displace-

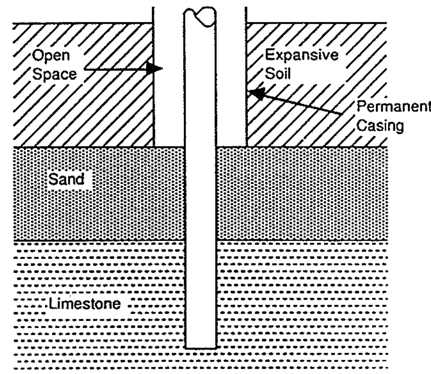


Figure 3.1(a). Use of permanent surface casing for design in expansive soils. (From Reese and O'Neill, 1988)

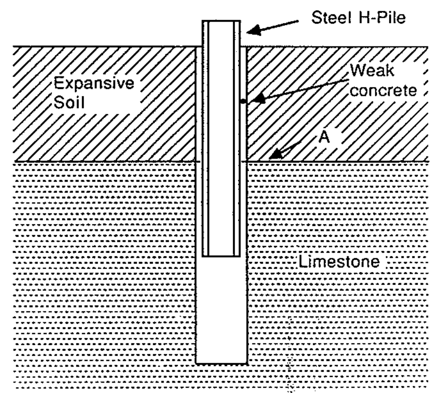


Figure 3.1(b). Raba method of design in expansive soil. (From Reese and O'Neill, 1988)

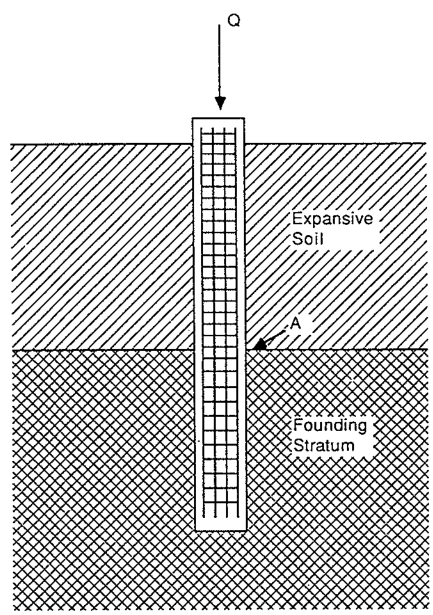


Figure 3.1(c). Use of rebar cage for design in expansive soil. (From Reese and O'Neill, 1988)

Table 3.4. Summary of ultimate and serviceability limit states that must be considered in the design of drilled shaft foundations.

DESIGN CONSIDERATION	ULTIMATE LIMIT STATE	SERVICEABILITY LIMIT STATE
Structural capacity of single drilled shafts	X	
Bearing capacity of single drilled shafts	X	
Bearing capacity of groups of drilled shafts	X	
Punching into lower weak stratum	X	
Settlement of single drilled shafts		X
Settlement of groups of drilled shafts		X
Tensile capacity of drilled shafts during uplift	X	
Uplift capacity of single drilled shafts	X	
Structural capacity of drilled shafts under lateral loading	X	
Lateral movement of single drilled shafts when subjected to lateral loads		X
Lateral movement of groups of drilled shafts when subjected to lateral loads		X

ment. If the lateral displacement is greater than the tolerable lateral displacement for a single drilled shaft, increase the diameter or the length of the shaft. If the lateral displacement is greater than the tolerable displacement for a group of shafts, increase the number of drilled shafts or increase the spacing.

13. Determine whether load tests are required to verify the design.

A summary of the ultimate and serviceability limit states that should be considered during the design stage is given in Table 3.4.

CHAPTER 4

DESIGN OF DRILLED SHAFTS FOR AXIAL LOADING

Significant advances have been made in recent years in developing improved understanding of the behavior of axially loaded drilled shafts. Design procedures based on the results of these studies are summarized in this chapter. Three limit states may be reached in drilled shafts subjected to axial loads. These are: (1) structural failure of the drilled shaft, (2) bearing capacity failure of the soil, and (3) excessive settlement. Failure of the drilled shaft or the soil is called an "ultimate limit state" (ULS). Excessive settlement, a less drastic occurrence, is called a "serviceability limit state" (SLS).

Both ultimate and serviceability limit states are addressed in this section. The structural capacity of drilled shafts is discussed first, followed by the bearing capacity of single drilled shafts and groups of drilled shafts, and finally the settlement of individual and groups of drilled shafts. The last section of this chapter focuses on the effects of negative skin friction and the uplift capacity of drilled shafts. Straight shaft and single underreamed drilled shafts are considered, but multiunderreamed shafts are beyond the scope of this manual.

4.1 STRUCTURAL CAPACITY

Axially loaded drilled shafts can fail structurally either in compression or by buckling. Buckling usually does not take place in shafts of normal dimensions constructed in soft soils (Poulos and Davis, 1980). However, buckling analyses are warranted in long and slender drilled shafts that extend for a portion of their lengths through water or air. Scour around drilled shafts increases the likelihood that they may fail by buckling, and the maximum possible depth of scour should be considered in design.

4.1.1 Axial Compression

The axial load in a drilled shaft should not exceed the factored axial structural capacity. The following criterion expresses this fact:

$$\phi_a P_n \geq \gamma_D P_D + \gamma_L P_L \tag{4.1.1}$$

where ϕ_a = performance factor for the nominal structural capacity, P_n = nominal structural capacity of the drilled shaft, P_D and P_L are the axial loads due to dead and live loads respectively, and γ_D and γ_L are the dead and live load factors. In general, for conditions where other types of loads may act on the drilled shafts, the design criterion may be expressed as:

$$\phi_a P_n \geq \Sigma \gamma_i P_i \tag{4.1.2}$$

where γ_i = load factor for the load i and P_i = axial load due to load i.

The factored nominal axial capacity of the drilled shaft can be expressed as follows:

$$\phi_a P_n = r\phi_a(0.85f_c'A_c + f_yA_y) \tag{4.1.3}$$

where $\phi_a P_n$ = factored nominal capacity of the drilled shaft; ϕ_a = capacity reduction factor, 0.75 for spiral columns and 0.70 for tied columns; r = eccentricity factor, 0.85 for spiral columns and 0.80 for tied columns; f_c' = 28-day concrete cylinder strength; f_y = yield stress of steel; A_c = cross-sectional area of concrete = $A_g - A_y$; A_g = gross cross-sectional area of the drilled shaft; and A_y = cross-sectional area of steel.

The drilled shaft carrying the maximum load in an eccentrically loaded group must be checked for structural failure. In Figure 4.1, the factored total vertical load acting on a group of drilled shafts, denoted as P_g, acts at a distance e_x and e_y from the centroid in the x and y-directions. The factored axial load on any drilled shaft, $P_{x,y}$ may be calculated from the following expression (Scott, 1980):

$$P_{x,y} = P_g\left[\frac{1}{N_{ds}} + \frac{e_x x}{\Sigma x^2} + \frac{e_y y}{\Sigma y^2}\right] \tag{4.1.4}$$

where x and y are the distances of the drilled shaft from the centroid in the x and y directions respectively; N_{ds} is the number of drilled shafts in the group.

For example, consider the group shown in Figure 4.1. If P_g = 500 tons, e_x = 4 ft, e_y = 2 ft, and the spacing is 6 ft center-to-center, the maximum load carried by a drilled shaft can be determined as follows: $\Sigma x^2 = 6\,(6)^2 = 216$; $\Sigma y^2 = 6\,(6)^2 = 216$.

The most heavily loaded drilled shaft is drilled shaft 3, in the first quadrant:

$$P_3 = 500\,[1/9 + 4(6)/216 + 2(6)/216] = 139 \text{ tons}$$

Shaft number 7 carries a tensile force:

$$P_7 = 500\,[1/9 - 4(6)/216 - 2(6)/216] = -27.8 \text{ tons}$$

Design against tensile failure is considered in Section 4.5.

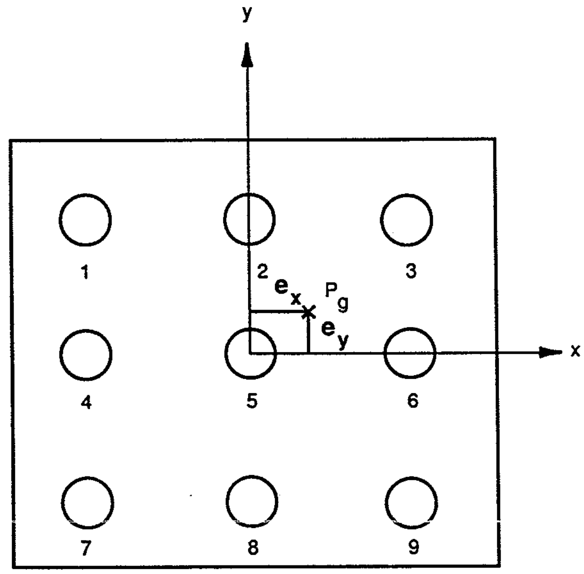


Figure 4.1. Eccentric loading on a group of drilled shafts. (From Reese and O'Neill, 1988)

4.1.2 Buckling of Partially Embedded Drilled Shafts

Drilled shafts that extend above the ground through air or water may buckle when subjected to axial loads, and the possibility of buckling failure may control their structural capacity. In order to evaluate the buckling capacity of partially embedded drilled shafts, it is necessary to determine at what depth below the ground surface should the shaft be assumed to be fixed. Davisson and Robinson (1965) have developed a method for estimating this depth to fixity.

4.1.2.1 Davisson and Robinson's Procedure

Davisson and Robinson (1965) presented solutions for the buckling loads of partially embedded piles in terms of an equivalent free standing length. The equivalent free standing length is the sum of the unsupported length of the drilled shaft above ground, and an additional length to the depth of fixity below ground. This depth to fixity is a function of the flexural stiffness of the shaft, E_pI_p, and the soil stiffness. The soil stiffness can be expressed in terms of a soil modulus (E_s, force/length^2). The soil modulus is usually considered to remain constant with depth in clays, and to vary linearly with depth in granular soils.

For long shafts, the equivalent free standing length, L_{eq}, can be written as follows:

Modulus constant with depth (clays)

$$L_{eq} = L_u + 1.4R \tag{4.1.2.1}$$

Modulus increasing linearly with depth (sands)

$$L_{eq} = L_u + 1.8T \tag{4.1.2.2}$$

where L_u = unsupported length of shaft extending above ground $R = [(E_pI_p)/E_s]^{0.25}$, in units of length; E_p = Young's modulus of drilled shaft, in units of force/length^2; I_p = moment of inertia

of drilled shaft, length[4]; E_s = soil modulus, force/length[2]; $E_s = 67S_u$ for clays (Davisson and Robinson, 1965); S_u = undrained shear strength of clays, force/length[2]; $T = [(E_pI_p)/n_h]^{0.2}$; n_h = rate of increase of soil modulus with depth, force/length[3]; $n_h = E_s/z$; and z = depth.

Davisson and Robinson's procedure applies to different boundary conditions at the top of the drilled shaft; the bottom boundary condition is assumed to be fixed against rotation and translation at the depth of fixity. Selection of appropriate boundary conditions at the top of the drilled shaft depends on the type of structure, the fixity of bearings, and the number of rows of drilled shafts along the length and width of the group. Figure 4.2 shows four possible boundary conditions at the top of the drilled shaft where it connects with the structure, and expressions for the critical buckling load in ideal columns for each case.

Based on lateral load tests of piles in sand, Alizadeh and Davisson (1970) found that n_h is strongly dependent on deflection when the lateral deflection is less than 3 percent of the pile width. At larger deflections, the value of n_h becomes almost independent of the lateral deflection.

Terzaghi (1955) recommended values of n_h that are appropriate for lateral deflections that are about 5 percent of the width of the pile or drilled shaft (Table 4.1). Reese et al. (1974) recommended using values of n_h that are between 3 and 4 times larger than Terzaghi's recommended values for constructing the initial slope of p-y curves.

For the buckling analysis of partially embedded drilled shafts, values of n_h corresponding to smaller deflections, on the order of 0.5 percent of the width or less, appear to be most appropriate. Evans (1982) showed that for lateral deflections of this magnitude, it is reasonable to use values of n_h about 3 times as high as the values recommended by Terzaghi. The two right-hand columns of Table 4.1 contain values of n_h appropriate for lateral deflections on the order of 0.5 percent of the width of the drilled shaft.

4.1.2.2 Group Effects on Buckling Loads

The effect of pile spacing on the soil modulus has been studied by Prakash (Prakash and Sharma, 1990). He found that, at spacings greater than 8 times the pile width, neighboring piles have no effect on the soil modulus or buckling capacity. However, at a spacing of 3 times the pile width, the effective soil modulus reduces to 25 percent of the value applicable to a single pile. For intermediate spacings, modulus values can be estimated by interpolation.

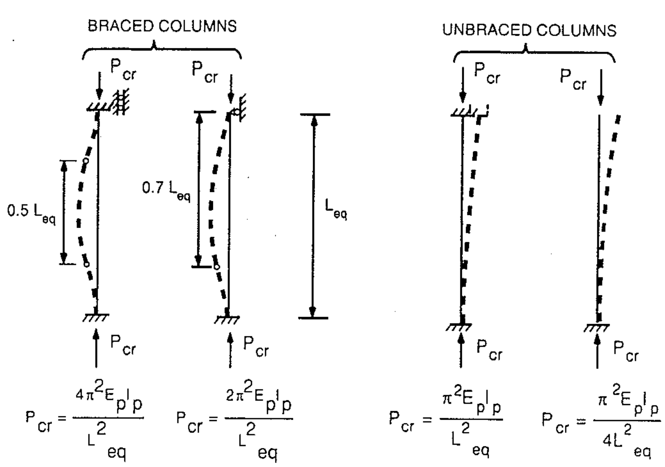


Figure 4.2. Critical buckling loads for centrally loaded columns with various end conditions.

Table 4.1. Coefficient of horizontal subgrade reaction, n_h, in lb/in.^{3}

	Terzaghi (1955)		Reese et al. (1974)	Recommended	
	Dry or Moist Sand	Submerged Sand	Submerged Sand	Dry or Moist Sand	Submerged Sand
Loose	8	5	20	30	15
Medium	24	16	60	80	40
Dense	65	39	125	200	100

4.1.2.3 Design Procedure

Buckling loads for partially embedded free standing drilled shafts can be calculated using the following steps:

1. Estimate the value of n_h (for sands) or E_s (for clays) and calculate the value of T (for sands) or R (for clays). For groups of drilled shafts, the soil modulus should be reduced to account for the effects of neighboring shafts as described earlier.
2. Calculate the equivalent length of the drilled shaft, L_{eq}, using Eqs. 4.1.2.1 or 4.1.2.2, whichever is appropriate.
3. Use the appropriate expression from Figure 4.2 to calculate the buckling load, P_{cr}. Four equations are given in Figure 4.2 for four different restraint conditions at the top of the drilled shaft.

After the ideal buckling load has been determined, the safe design load for the column, considering the effects of end moments and eccentricity of loading, can be determined using normal design procedures for columns and beam-columns.

4.2 BEARING CAPACITY

4.2.1 Presumptive Bearing Capacities of Soils and Rocks

In the absence of sufficient soil strength data to estimate drilled shaft capacities rationally, bearing capacities of drilled shafts may be estimated using presumptive bearing capacities. These values should be used only as a rough guide to possible capacities. When used in design, presumptive bearing capacities must be substantiated by load tests or rational methods of analysis based on soil data from the site.

Presumptive bearing capacities that have been published are "allowable" values, intended for use in working stress design.

4.2.2 Rational Methods of Estimating Bearing Capacities of Drilled Shafts

The ultimate bearing capacity of a drilled shaft is the sum of the side and base resistances:

$$Q_{ult} = Q_s + Q_p \quad (4.2.2.1)$$

where Q_{ult} = total ultimate bearing capacity of a drilled shaft, Q_s = ultimate load carried by side resistance = $A_s q_s$, Q_p = ultimate load carried by the base = $A_p q_p$, A_s = surface area of the sides of the shaft over its penetration depth, A_p = area of base of the drilled shaft, q_s = ultimate unit side resistance of the shaft, and q_p = ultimate unit base resistance of the shaft.

The load factor design criterion may be expressed as:

$$\phi_q Q_{ult} \geq \gamma_D P_D + \gamma_L P_L \quad (4.2.2.2)$$

where ϕ_q = the performance factor for the ultimate bearing capacity of a drilled shaft.

In general,

$$\phi_q Q_{ult} \geq \Sigma \gamma_i P_i \quad (4.2.2.3)$$

where γ_i is the load factor for load i and P_i is the axial load due to load i.

One rational method of estimating the bearing capacity of drilled shafts in compression is called the "static" approach. Static formulas are based either on classical soil mechanics theories or empirical correlations. These include total and effective stress methods and methods based on in situ tests, such as the standard penetration test (SPT). The total stress method is more suited for drilled shafts in cohesive soils, while the SPT correlation is better suited for drilled shafts in cohesionless soils. The effective stress method is applicable to drilled shafts in both cohesive and cohesionless soils.

4.2.2.1 Bearing Capacity of Drilled Shafts in Clays

The ultimate capacities of drilled shafts in clay are usually governed by the conditions at the end of construction. Therefore, drilled shafts in clays are usually designed using total stress methods. However, in some circumstances the strength of the soil can decrease with time. Conditions that may involve reduction of strength and capacity after construction include drilled piers in expansive clays that swell after installation. Another situation where the shear strength of the soil changes with time is when consolidation of the soil around a drilled shaft may result in downward movement of the soil relative to the drilled shaft, thus inducing negative skin friction on the shaft, which increases the load on the pier and reduces the magnitude of the load that the shaft can carry without excessive settlement. In these cases, effective stress analyses may be used.

1. *Total Stress Analysis.*

a. *Shaft resistance (α-method)*—The α-method relates the adhesion between the drilled shaft and the clays to the undrained shear strength of the clay. The ultimate unit skin friction, q_s, can be expressed by:

$$q_s = \alpha S_u \quad (4.2.2.4)$$

where S_u = undrained shear strength and α = adhesion factor applied to S_u. Reese and O'Neill (1988) developed a procedure for prescribing α values along the length of drilled shafts in overconsolidated clays. Table 4.2 shows the values of α recommended by Reese and O'Neill. Figure 4.3 shows the portions of the length of drilled shafts that are considered not to contribute to shaft adhesion.

Table 4.2. Recommended values of α for drilled shafts in clay. (After Reese and O'Neill, 1988)

Location Along Drilled Shaft	Undrained Shear Strength, S_u	Value of α
From ground surface to depth along drilled shaft of 5 ft*	-	0
Bottom 1 diameter of the drilled shaft or 1 stem diameter above the top of the bell	-	0
All other points along the sides of the drilled shaft	< 2 tsf	0.55
	2 - 3 tsf	0.49
	3 - 4 tsf	0.42
	4 - 5 tsf	0.38
	5 - 6 tsf	0.35
	6 - 7 tsf	0.33
	7 - 8 tsf	0.32
	8 - 9 tsf	0.31
	> 9 tsf	Treat as Rock

* The depth of 5 ft may be increased if the drilled shaft is installed in expansive clay, or if there is substantial groundline deflection from lateral loading.

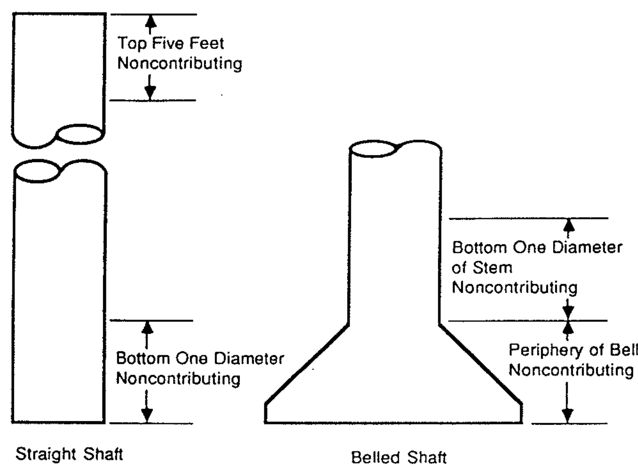


Figure 4.3. Explanation of portions of drilled shafts not considered in computing side resistance.

The value of α is zero for the top 5 ft, consistent with findings from load tests. Load test data on instrumented drilled shafts have shown load transfer to be zero at the ground surface, increasing with depth. Because the rate of increase has not been determined with much certainty, Reese and O'Neill chose to use $\alpha = 0$ in the top 5 ft.

The footnote in Table 4.2 accounts for the following. During dry weather, expansive soils shrink and move away from the shaft. A value of $\alpha = 0$ may be selected for a depth greater than 5 ft as indicated by the depth of seasonal moisture change in areas with expansive soil, and (b) during lateral loading, the clay

at the groundline may be pushed away due to lateral deflection of the shaft, especially if the loads are cyclic in nature, causing the shaft to be deflected back and forth.

The value of α is also zero for a distance of 1 diameter above the base of the shaft, because downward movement of the base can cause a tensile crack to develop in the soil near the base.

Based on load tests on drilled shafts in clay, Reese and O'Neill suggested the use of values of α, as given in Table 4.2, for the remaining portion of the drilled shaft. The value of α may be different from those given in Table 4.2 in sensitive clays. In such soils, load tests should be conducted to establish appropriate values of α.

The data used in deriving this design method does not include clays with S_u greater than 6 tsf, OCR greater than 10, or sensitivity greater than 4.

b. *End bearing*—Reese and O'Neill (1988) applied Skempton's (1951) expression for end bearing of piles in clay, to drilled shafts as follows:

$$q_p = N_c S_u \leq 40 \text{ tsf} \tag{4.2.2.5}$$

where $N_c = 6(1 + 0.2Z/D_p) \leq 9$, S_u = average undrained shear strength of clay over a depth of one to two diameters below the base, Z = distance that the shaft extends into the ground, and D_p = diameter of the base of the shaft.

The limiting value of q_p (40 tsf) is based on the largest value measured in clays and is not a theoretical limit. Higher values of q_p may be used if indicated by load test results.

N_c should be reduced by one-third (i.e., use $2N_c/3$ in computations) in soft clays to account for large displacements prior to bearing capacity failure.

If D_p exceeds 75 in., the ultimate unit end bearing capacity of drilled shafts in stiff to hard clay should be reduced to q_{pr} as follows (Reese and O'Neill, 1988):

$$q_{pr} = F_r q_p \tag{4.2.2.6}$$

where $F_r = \dfrac{2.5}{aD_p \text{ (in.)} + 2.5b} \leq 1.0$

$a = 0.0071 + 0.0021\, Z/D_p \leq 0.015$

$b = 0.45\sqrt{S_u \text{ (ksf)}}$ where $0.5 \leq b \leq 1.5$

Equation 4.2.2.6 is based on load tests of large diameter underreamed drilled shafts in clay, and q_{pr} corresponds to a base settlement of 2.5 in.

2. *Effective Stress Analysis.*

a. *Shaft resistance (β-method)*—Stas and Kulhawy (1984) proposed that the ultimate unit side resistance of drilled shafts under long-term sustained loading may be estimated as follows:

$$q_s = \frac{K}{K_o}\left[\sigma_v' K_o \tan\left\{\phi'\left(\frac{\delta}{\phi'}\right)\right\} + c'\right] \tag{4.2.2.7}$$

where K/K_o = ratio of horizontal soil stress coefficient to the in situ value; σ_v' = vertical effective stress; K_o = in situ horizontal soil stress coefficient before construction; ϕ' = effective stress angle of internal friction; c' = effective cohesion; δ/ϕ' = ratio of the angle of friction of the interface, δ, to the soil angle of internal friction, ϕ'. In an extensive series of tests, Potyondy (1961) determined the following values of δ/ϕ' (0.50 for clay/steel interfaces and 0.68 to 0.95 for clay/concrete interfaces). The lower value is for a smooth surface finish (e.g., concrete cast in steel forms) while the higher value is for a rough surface finish (e.g., concrete cast in the ground).

The ratio K/K_o depends on the method of installing the drilled shaft. Typical values range between two-thirds and one. The lower value is used for slurry construction, while the higher value is for dry construction. Casing construction below the groundwater table can be considered as an intermediate case. The disadvantage of using this method is that the construction method must be known prior to design.

b. *End bearing*—The drained ultimate unit base resistance may be approximated as follows (Kulhawy et al., 1983):

$$q_p = \sigma_v' N_q' \tag{4.2.2.8}$$

where σ_v' = vertical effective stress, and N_q' = modified bearing capacity factor obtained from Figure 4.4.

It may be seen that the value of N_q' depends on the value of rigidity index, a term which accounts for soil deformability and the variation of the bearing capacity factor with depth. Vesic (1975) defined I_r as follows:

$$I_r = \frac{E_s}{2(1 + \mu)\sigma_v' \tan\phi'} \tag{4.2.2.9}$$

where E_s = Young's modulus of the soil, μ = Poisson's ratio of the soil, σ_v' = vertical effective stress measured at a depth of

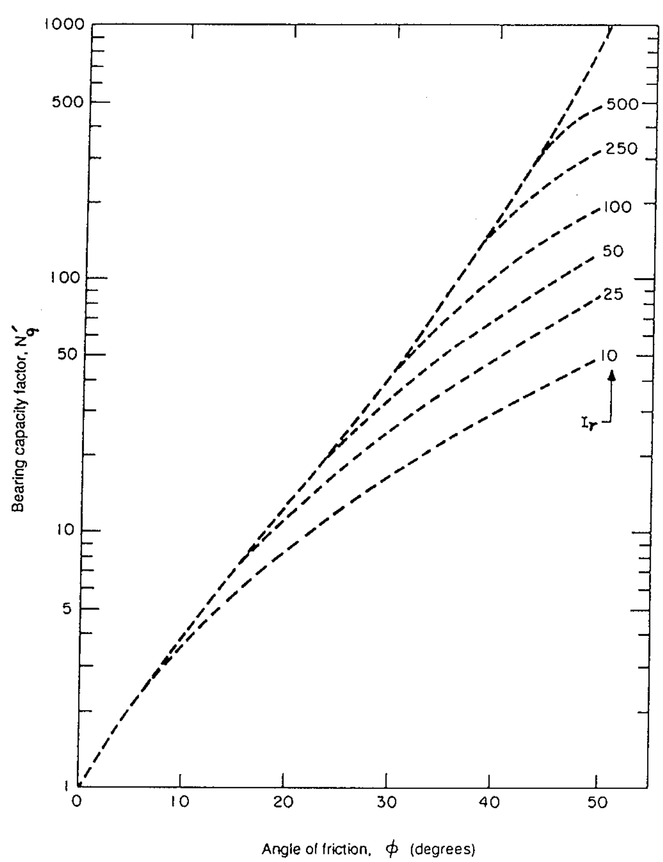


Figure 4.4. Modified N_q bearing capacity factor for deep foundations in drained loading. (After Kulhawy et al., 1983)

D/2 below the base of the drilled shaft, and ϕ' = effective friction angle of the soil.

Trofimenkov and Vorobkov approximated the rigidity index for clays as follows (Kulhawy et al., 1983):

$$I_r = 320(1 - LI)s \qquad (4.2.2.10)$$

where LI = liquidity index of the clay = $(w_n - PL)/(LL - PL)$; w_n = natural water content of the clay; PL = plastic limit of the clay; LL = liquid limit of the clay; s = stress correction factor = $2.2/(1.2 + \sigma_v/\sigma_{n1})$; σ_v = original vertical in situ stress (tsf) at a depth of half diameter below the base; and σ_{n1} = normalizing stress = 1 tsf. Equation 4.2.2.10 is valid for liquidity indices between 0 and 0.9.

4.2.2.2 Bearing Capacity of Drilled Shafts in Sands

Although many field load tests have been performed on drilled shafts in clays, very few have been performed on drilled shafts in sands. The shear strength of cohesionless soils can be characterized by an angle of internal friction, ϕ', or empirically related to values of SPT blow count, N. Methods of estimating shaft resistance and end bearing using either ϕ' or N values are presented, as follows:

1. *Shaft Resistance.* Table 4.3 summarizes 5 methods of predicting shaft resistance of bored piles in sand. Quiros and Reese (1977) and Reese and O'Neill (1988) indicate that the unit side resistance should be limited to 2 tsf, corresponding to the maximum value ever measured; Touma and Reese suggest an upper limit of 2.5 tsf. These values, however, are not theoretical limits. Higher values can be used if they are verified by load tests.

Table 4.3. Summary of procedures for estimating side resistance, q_s, of drilled shafts in sand.

REFERENCE	DESCRIPTION
Touma and Reese (1974)	$q_s = K\sigma_v'\tan\phi' < 2.5$ tsf where K = 0.7 for $D_b \le 25$ ft K = 0.6 for 25ft < $D_b \le 40$ ft K = 0.5 for $D_b > 40$ ft
Meyerhof (1976)	q_s (tsf) = $\frac{N}{100}$
Quiros and Reese (1977)	q_s (tsf) = 0.026N < 2 tsf
Reese and Wright (1977)	q_s (tsf) = $\frac{N}{34}$ for $N \le 53$ q_s (tsf) = $\frac{N - 53}{450}$ + 1.6 for $53 < N \le 100$
Reese and O'Neill (1988)	q_s (tsf) = $\beta\sigma_v' \le 2$ tsf for $0.25 \le \beta \le 1.2$ where $\beta = 1.5 - 0.135\sqrt{z}$

where N = uncorrected SPT blow count
σ_v' = vertical effective stress
ϕ' = friction angle of sand
K = load transfer factor
D_b = embedment of drilled shaft in sand bearing layer
β = load transfer coefficient

Table 4.4. Summary of procedures for estimating base resistance, q_p, of drilled shafts in sand.

REFERENCE	DESCRIPTION
Touma and Reese (1974)	Loose q_p (tsf) = 0 Medium Dense q_p (tsf) = $\frac{16}{k}$ Very Dense q_p (tsf) = $\frac{40}{k}$ k = 1 for D_p < 1.67 ft & k = 0.6D_p for $D_p \ge$ 1.67 ft. Applicable only if D_b > 10D
Meyerhof (1976)	q_p (tsf) = $\frac{2N_{corr}D_b}{15D_p} < \frac{4}{3}N_{corr}$ for sand $< N_{corr}$ for nonplastic silts
Quiros and Reese (1977)	Same as Touma and Reese (1974)
Reese and Wright (1977)	q_p (tsf) = $\frac{2}{3}$N for $N \le 60$ q_p (tsf) = 40 for N > 60
Reese and O'Neill (1988)	q_p (tsf) = 0.6N for $N \le 75$ q_p (tsf) = 45 for N > 75

where N_{corr} = SPT blow count corrected for overburden pressure
= $[0.77\log_{10}(20/\sigma_v')]N$
N = uncorrected SPT blow count
D_p = base diameter of drilled shaft in ft
D_b = embedment of drilled shaft in sand bearing layer

It may be noted that the side resistance of drilled shafts in sand can be estimated using either the friction angle (Touma and Reese, 1974) or the SPT blow count (Meyerhoff, 1976; Quiros and Reese, 1977; and Reese and Wright, 1977). Reese and O'Neill (1988) proposed a method for uncemented sands that uses a different approach in that the shaft resistance is independent of the soil friction angle and the SPT blow count. They suggested that the friction angle approaches a common value for uncemented sands because of the high shearing strains in the sand and stress relief that occur during drilling.

2. *End Bearing.* Load tests show that large settlements are required to mobilize the maximum end bearing resistance of drilled shafts in sands. Because large settlements are not tolerable in most structures, the procedures presented in Table 4.4 for calculating the ultimate unit end bearing capacity, q_p, are based on a downward movement equal to either 1 in. (Touma and Reese, 1974; and Quiros and Reese, 1977); or 5 percent of the base diameter (Reese and Wright, 1977; and Reese and O'Neill, 1988).

Reese and O'Neill (1988) recommend that for base diameters greater than 50 in., q_p should be reduced to q_{pr} as follows:

$$q_{pr} = \frac{50}{D_p} q_p \qquad (4.2.2.11)$$

where q_{pr} = reduced base resistance for $D_p > 50$ in.; D_p = diameter of the base of the shaft, in.; and q_p = ultimate unit end bearing resistance calculated using one of the methods in Table 4.4.

Meyerhof's expression for base resistance stems from the idea that the point resistance increases linearly with embedment up to a limiting depth of 10 shaft diameters; thereafter, the point resistance remains constant with depth.

3. *Performance Factors.* As discussed previously, five methods (Touma and Reese, 1974; Meyerhof, 1976; Quiros and Reese, 1977; Reese and Wright, 1977; and Reese and O'Neill, 1988) have been developed for estimating the side resistance and end bearing capacities of drilled shafts in sands and gravels. Comparison of these methods shows that they may result in widely divergent estimates of capacity for the same conditions. Unfortunately, the information available from field load tests at present is insufficient to determine which of the methods is most reliable and most generally applicable.

Because of the shortage of field data, it is not possible at present to determine with precision what values of performance factors should be used for drilled shafts in sands and gravels. Accordingly, the best procedure appears to be to estimate the capacity using all of the applicable methods, and to select the factored capacity using judgment and any available experience with similar conditions. The inherent great variability of the capacities of drilled shafts in sand logically suggests that values of performance factors for shafts in sand should be smaller than for shafts in clay.

4.2.2.3 Bearing Capacity of Drilled Shafts Socketed in Rock

Drilled shafts socketed in rock derive their axial capacities from either end bearing or side resistance, or both. The depth of the socket is typically one to three times the diameter (Canadian Geotechnical Society, 1985). The design procedure presented in this section assumes that: (1) the rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used, (2) the drilling fluid used will not form a lubricated film on the sides of the excavation, and (3) the bottom of the excavation is properly cleaned out. This is especially important if the capacity of the drilled shaft is based on the end bearing.

The design procedure proposed by Reese and O'Neill (1988) for bearing capacity of drilled shafts socketed in rock assumes that the load is carried entirely by the shaft if the computed settlement is less than 0.4 in. Conversely, loads that cause settlements greater than 0.4 in. are assumed to be carried entirely by the base of the drilled shaft. This method is conservative because loads are assumed to be carried entirely in side resistance or entirely in end bearing, and no allowance is made for the loads to be carried by a combination of side resistance and end bearing. The steps in the design procedure are as follows:

1. Estimate the settlement of the portion of the drilled shaft that is socketed in rock. This consists of two components: (a) the elastic shortening of the drilled shaft, ρ_e, which can be computed as follows:

$$\rho_e = \frac{(\Sigma P_i)H_s}{A_{soc}E_c} \qquad (4.2.2.12)$$

where H_s = depth of the socket, ΣP_i = working load at the top of the socket, A_{soc} = cross-sectional area of the socket, and E_c = Young's modulus of concrete in the socket, considering the stiffness of any steel reinforcement; and (b) settlement of the base of the drilled shaft, ρ_{base}, which can be computed as follows:

$$\rho_{base} = \frac{(\Sigma P_i)I_\rho}{D_s E_r} \qquad (4.2.2.13)$$

where I_ρ = influence coefficient obtained from Figure 4.5, D_s = diameter of the base of the drilled shaft socket, and E_r = modulus of the in situ rock, taking the joints and their spacing into account.

The Young's modulus of the in situ rock, E_r, can be estimated by:

$$E_r = K_E E_i \qquad (4.2.2.14)$$

where E_i = intact rock modulus found either by testing or by

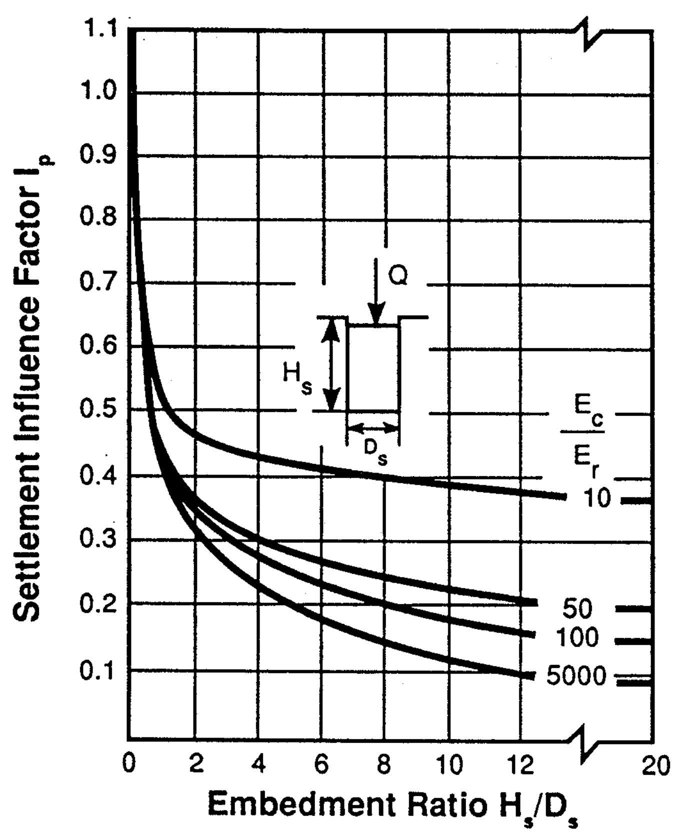


Figure 4.5. Elastic settlement influence factor as a function of embedment ratio and modular ratio. (After Donald, Sloan and Chiu, 1980, as presented by Reese and O'Neill, 1988)

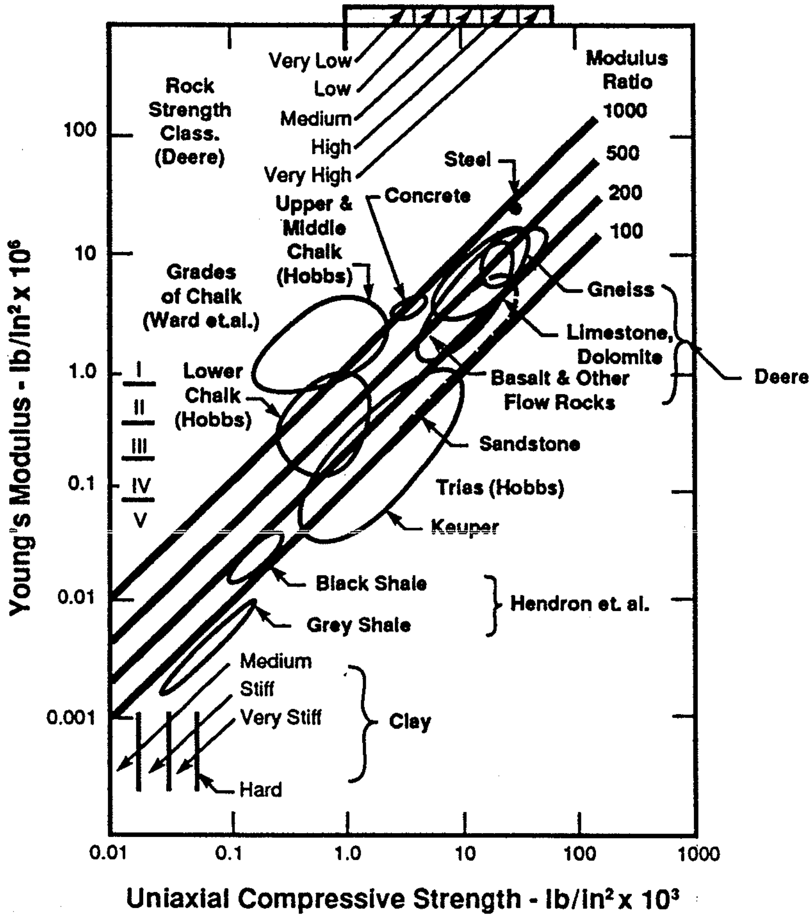


Figure 4.6. Engineering classification of intact rock. (After Deere, 1968, and Peck, 1976, as presented by Reese and O'Neill, 1988)

means of Figure 4.6; K_E = modulus modification ratio, related to the rock quality designation, RQD, as shown in Figure 4.7.

2. Calculate $\rho_e + \rho_{base}$. If the sum is less than 0.4 in., compute the ultimate capacity based on shaft resistance alone (step 3). If the sum is greater than 0.4 in., compute the ultimate capacity based on base resistance alone (step 4).

3. Estimate the side resistance of drilled shafts socketed in rock as follows: if the uniaxial compressive strength of the rock is less than or equal to 280 psi, the ultimate unit side resistance, q_s, is given by (Carter and Kulhawy, 1987):

$$q_s = 0.15 q_u \qquad (4.2.2.15)$$

where q_u is the uniaxial compressive strength of the rock. If the uniaxial compressive strength of the rock or concrete (in the drilled shaft), whichever is less, is greater than 280 psi, q_s is given by (Horvath and Kenney, 1979):

$$q_s = 2.5\sqrt{q_u} \qquad (4.2.2.16)$$

where q_s and q_u are in psi.

4. Estimate the base resistance of the drilled shaft socket from the uniaxial compression strength as follows (Canadian Geotechnical Society, 1985):

$$q_p = 3 q_u K_{sp} d \qquad (4.2.2.17)$$

where q_u = average uniaxial compression strength of the rock

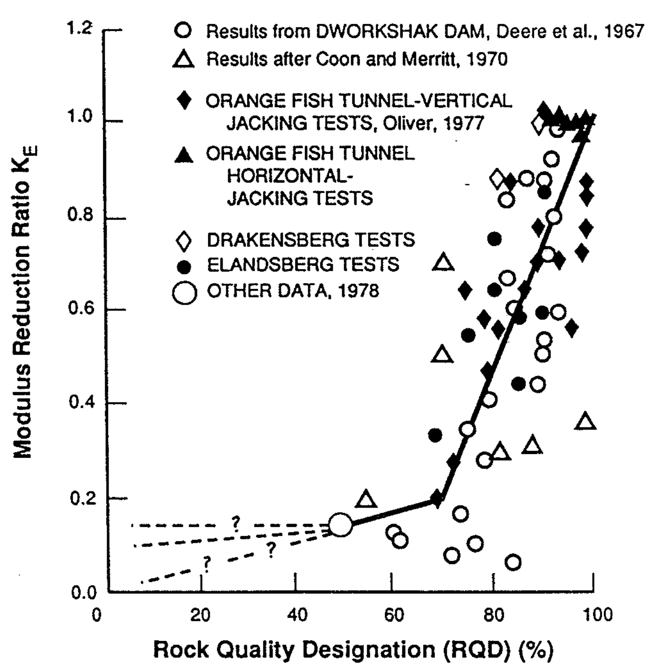


Figure 4.7. Modulus reduction ratio as a function of RQD. (After Bieniawski, 1984, as presented by Reese and O'Neill, 1988)

core, K_{sp} = dimensionless bearing capacity coefficient, and

$$K_{sp} = \frac{3 + s_d/D_s}{10[1 + 300t_d/s_d]^{0.5}} \text{ (See Figure 4.8)} \qquad (4.2.2.18)$$

in which: d = dimensionless depth factor $= 1 + 0.4H_s/D_s \leq 3.4$; s_d = spacing of discontinuities; t_d = width or thickness of discontinuities; D_s = diameter of drilled shaft socket; and H_s = depth of embedment of drilled shaft socket = 0 for drilled shafts resting of top of bedrock.

This method is not applicable to soft stratified rocks, such as shale or limestone. When the method is applicable, the rocks are usually so sound that the structural capacity will govern the design (Fellenius et al., 1989). This method is applicable only if $s_d > 1$ ft and $t_d < 0.25$ in. for unfilled discontinuities, or $t_d < 1$ in. for discontinuities filled with soil or rock debris, and $D_s > 1$ ft.

Alternatively, estimate the base resistance of drilled shafts socketed in rock using results from pressuremeter tests as follows (Canadian Geotechnical Society, 1975):

$$q_p = K_b(p_l - p_o) + \sigma_v \qquad (4.2.2.19)$$

where p_l = limit pressure determined from pressuremeter tests averaged over a distance of 2 diameters above and below the base, p_o = at rest horizontal stress measured at the base elevation, σ_v = total vertical stress at the base elevation, and K_b = dimensionless coefficient which depends on the socket diameter to socket depth ratio as follows:

H_s/D_s	0	1	2	3	5	7
K_b	0.8	2.8	3.6	4.2	4.9	5.2

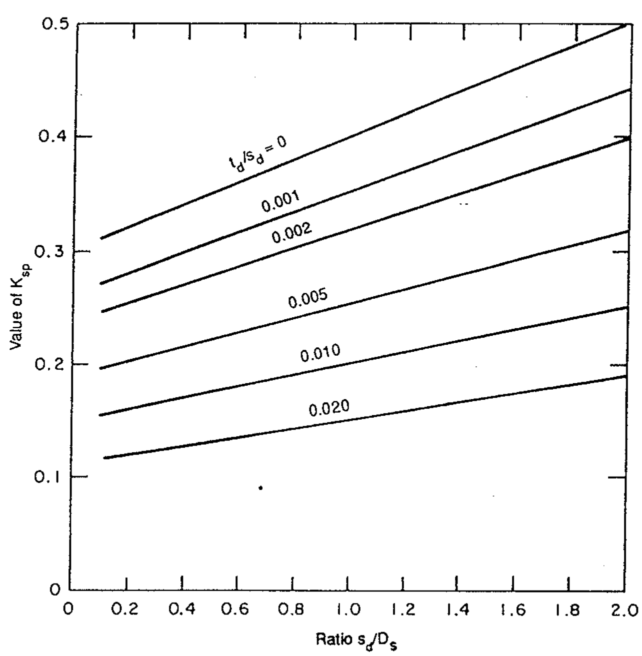


Figure 4.8. Bearing capacity coefficient, K_{sp}. (After Canadian Geotechnical Society, 1985)

4.2.2.4 Load Tests

Load tests on drilled shafts are often used to verify the ultimate load capacity estimated by means of the methods described earlier. It is desirable that a plunging failure load be obtained during a load test. However, this may be economically or physically impractical when the loads are very large. In such instances, the load test should be carried out until the base settles a distance equal to at least 5 percent of the base diameter.

Several loading procedures are given in ASTM D1143. These include the standard loading procedure, constant rate of penetration method, quick load test, constant time interval loading, and settlement control method.

The Osterberg load test apparatus was specially designed for load testing of drilled shafts. Osterberg (1984) proposed using a special flat pressure cell, installed below the tip of the shaft, that will differentiate the tip capacity contribution from the shaft capacity contribution (Figure 4.9).

4.2.2.5 Groups of Drilled Shafts

The design requirements for groups of drilled shafts are similar to those for single drilled shafts, i.e.,

$$\phi_g Q_g \geq \text{group load effect} \qquad (4.2.2.20)$$

where ϕ_g = performance factor for group capacity and Q_g = group capacity.

For groups of drilled shafts in cohesive soil, the mode of behavior depends on whether or not the cap is in contact with the ground. If the cap is in contact with the ground, groups of shafts may fail as a unit consisting of the shafts together with the block of soil contained within the shafts. The ultimate bearing capacity in this case should be taken as the minimum of the following two values: (1) the sum of the individual capacities of the drilled shafts, or (2) the bearing capacity for block failure of the group.

For a group of drilled shafts of width X, length Y, and depth Z (Figure 4.10), the bearing capacity for block failure in cohesive soils is given by:

$$Q_g = (2X + 2Y)\, Z\bar{S}_u + XYN_cS_u \qquad (4.2.2.21)$$

where $\bar{S}_u$ = average undrained shear strength along the depth of penetration of the drilled shafts, S_u = undrained shear strength below the base of the shafts, and

$$N_c = 5(1 + 0.2X/Y)(1 + 0.2Z/X) \; for\ Z/X \leq 2.5 \qquad (4.2.2.22)$$

$$N_c = 7.5(1 + 0.2X/Y) \qquad for\ Z/X > 2.5 \qquad (4.2.2.23)$$

If the cap is not in firm contact with the ground and the clay is normally consolidated or slightly overconsolidated or is sensitive, the individual capacity of the drilled shaft must be multiplied by an efficiency factor, η, where $\eta = 0.7$ for a center-to-center spacing of 3D and $\eta = 1.0$ for a spacing of 6D (Reese and O'Neill, 1988). The value of η may be linearly interpolated for intermediate spacing. The group capacity is then calculated as the minimum of: (1) the sum of the individual capacities of the drilled shaft multiplied by η, or (2) the bearing capacity for block failure as described above.

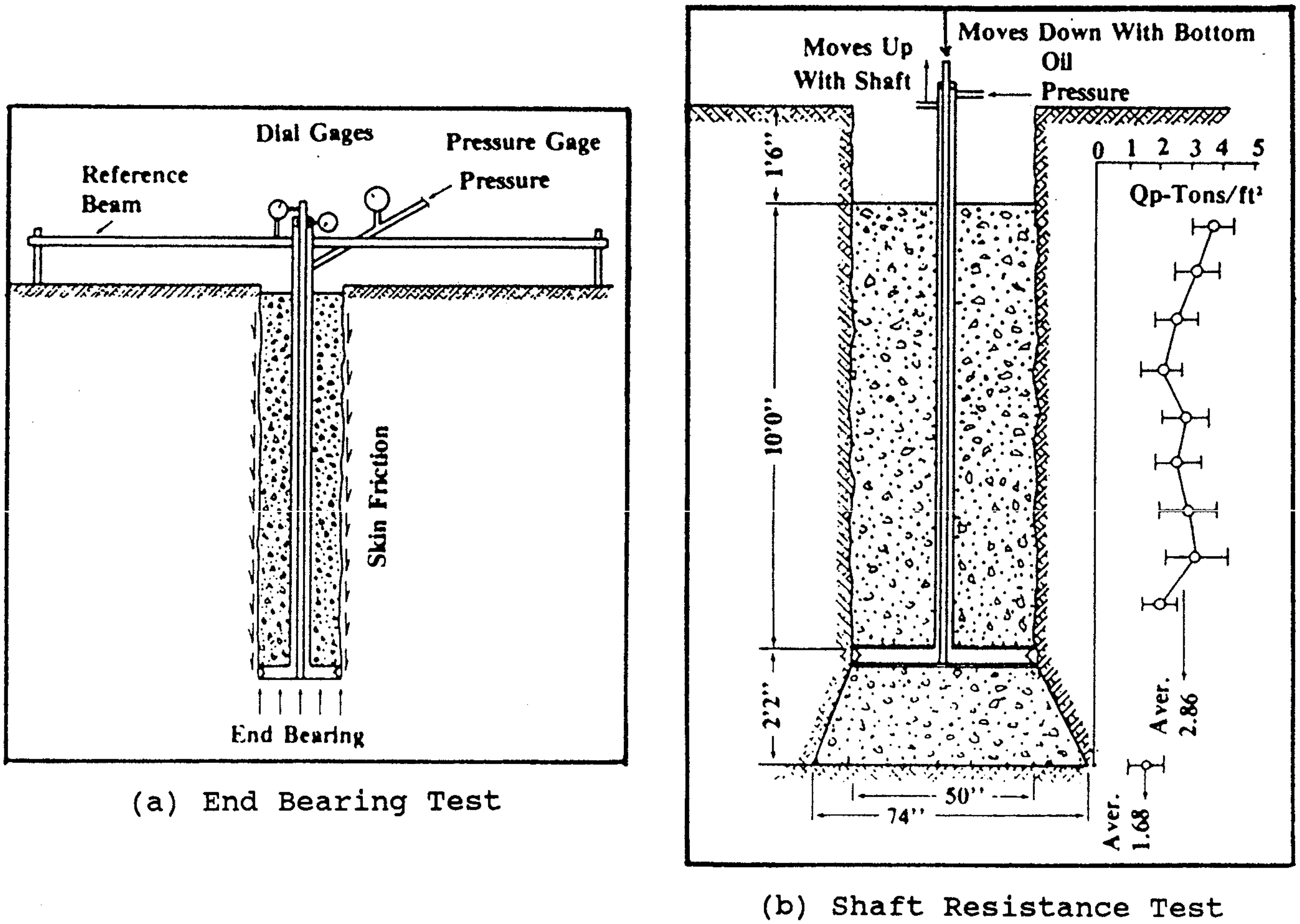


Figure 4.9. Osterberg load test apparatus for drilled shafts. (After Osterberg, 1984)

If the cap is not in firm contact with the ground, and the clay is heavily overconsolidated and insensitive, the group capacity should be estimated in a similar manner as the case where the cap is in contact with the ground.

Installation of drilled shafts in cohesionless soils results in stress relief. Therefore, the density of the sand may decrease during construction of drilled shafts. The ultimate bearing capacity of a group of drilled shafts in sand is estimated by multiplying the sum of the capacities of all the shafts in the group by a group efficiency factor. The group efficiency factor, defined as the ratio of the ultimate load capacity of the group to the sum of the ultimate capacities of the individual shafts, is 0.7 for a center-to-center spacing of three diameters and 1.0 for a spacing of six diameters (Reese and O'Neill, 1988). The efficiency factor can be interpolated for intermediate spacings. Evaluation of group capacity of drilled shafts in cohesionless soil is the same whether the cap is or is not in firm contact with the ground.

Block failure can also occur when the base of a group of shafts overlie a layer of soil very much weaker than the layer in which they terminate. The bearing capacity of the base of the equivalent pier, q_p can be computed as follows:

$$q_p = q_o + \frac{(q_l - q_o)H}{10X} \leq q_l \qquad (4.2.2.24)$$

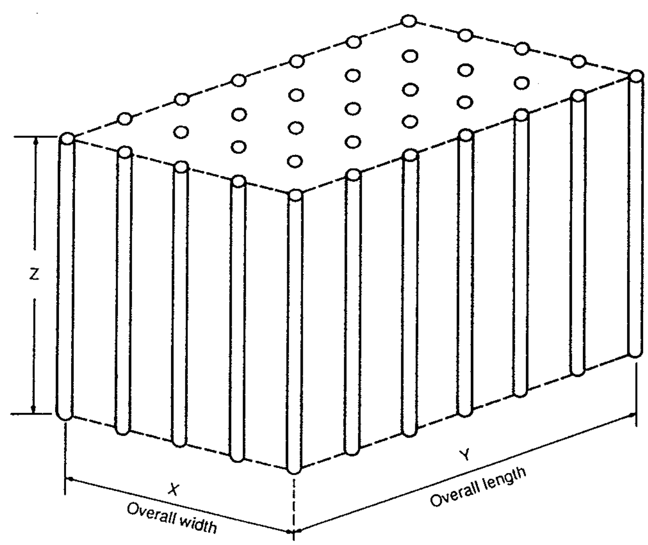


Figure 4.10. Group of drilled shafts acting as a block foundation.

where q_o = bearing capacity of base if it were at the top of the lower (weak) soil, q_l = bearing capacity of base in the upper soil in the absence of the softer lower soil, H = vertical distance

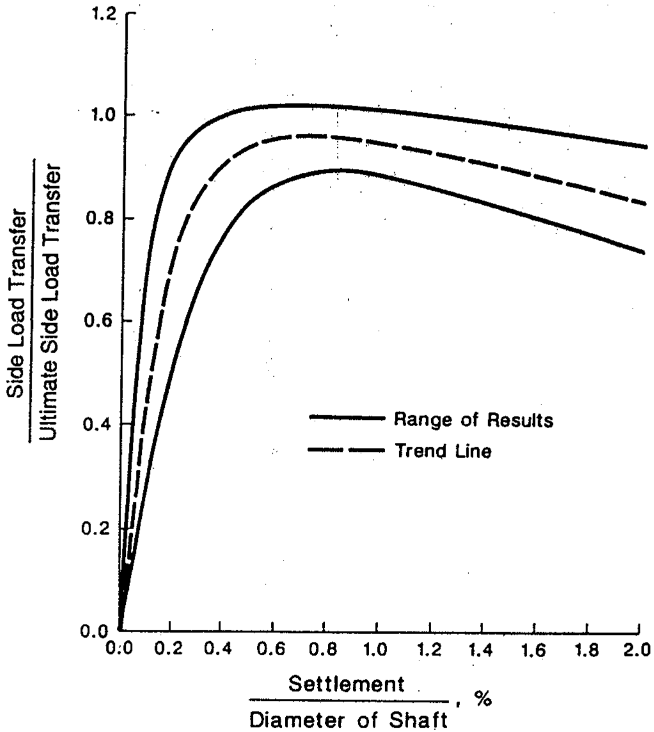


Figure 4.11. Normalized curves showing load transfer in side resistance versus settlement for drilled shafts in clay. (From Reese and O'Neill, 1988)

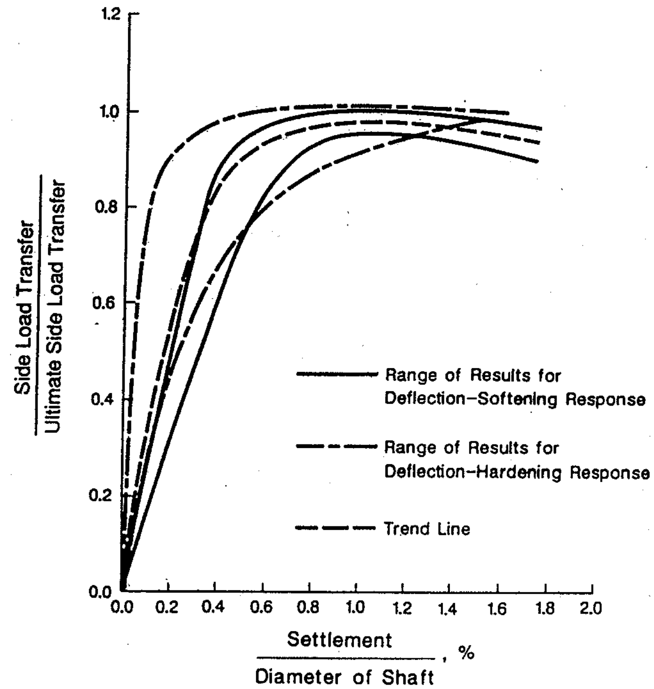


Figure 4.13. Normalized curves showing load transfer in side resistance versus settlement for drilled shafts in cohesionless soils. (From Reese and O'Neill, 1988)

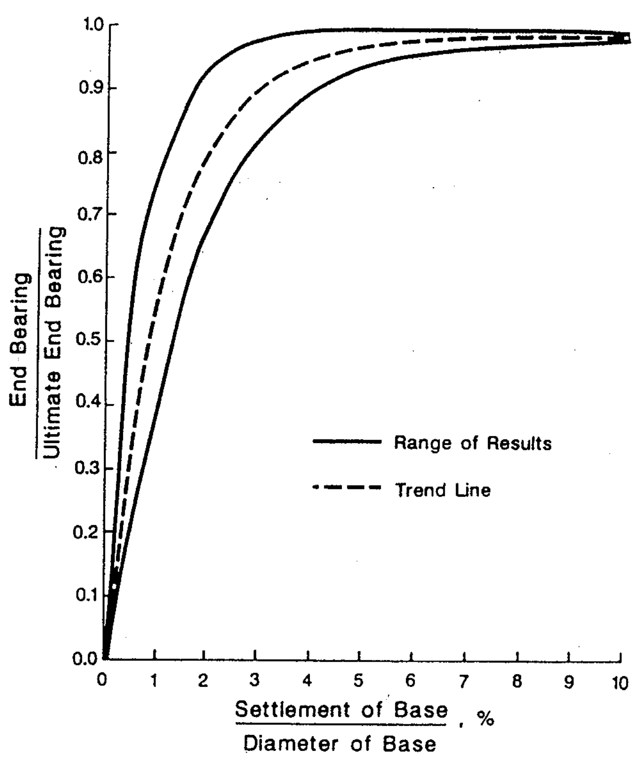


Figure 4.12. Normalized curves showing load transfer in end bearing versus settlement for drilled shafts in clay. (From Reese and O'Neill, 1988)

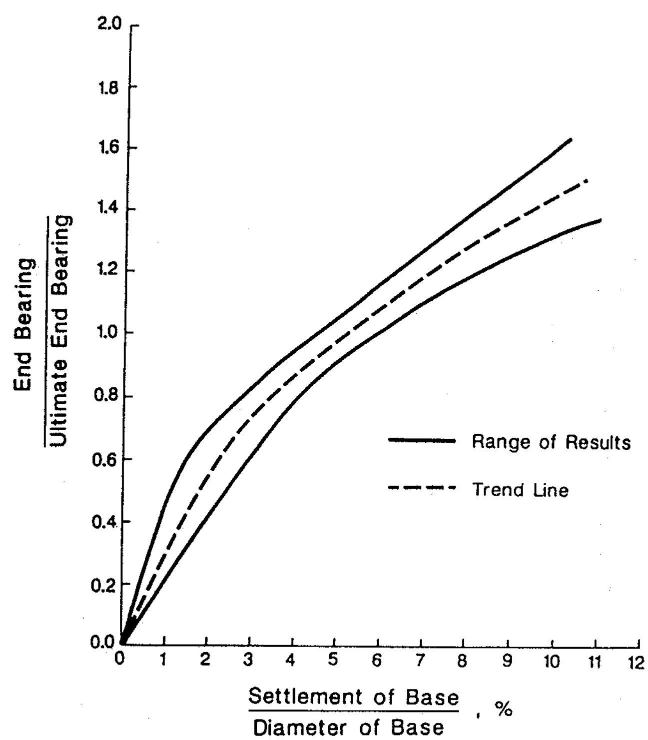


Figure 4.14. Normalized curves showing load transfer in end bearing versus settlement for drilled shafts in cohesionless soil. (From Reese and O'Neill, 1988)

from the base of the shafts in the group to the top of the weak layer, and X = width (least horizontal dimension) of the group.

The performance factors for the group capacity calculated using the sum of the individual capacities are the same as those for the single capacity of a drilled shaft. A separate performance factor must be used for the block failure mechanism.

4.3 SETTLEMENT

Drilled shafts may be installed individually or in groups. Settlement of drilled shafts installed in sand and rock are usually small, and they occur fairly rapidly. However, shafts in clay may settle over a longer period of time as they consolidate.

In estimating settlements of drilled shafts in clay, only unfactored permanent loads are considered. However, unfactored live loads must be added to the permanent loads when considering settlement of drilled shafts in granular soil.

4.3.1 Settlement of Single Drilled Shafts

Reese and O'Neill (1988) have summarized load-settlement data for drilled shafts in dimensionless form as shown in Figures 4.11 through 4.14. Figures 4.11 and 4.12 show the load-settlement curves in side resistance and in end bearing for shafts in clay. Figures 4.13 and 4.14 are similar curves for shafts in sand. These curves provide a useful guide for estimating short-term settlements of drilled shafts.

The values of the load-settlement curves in side resistance were obtained at different depths, taking into account elastic shortening of the shaft. While elastic shortening may be small in relatively short shafts, it may be quite substantial in longer shafts. The amount of elastic shortening in drilled shafts varies with depth. Reese and O'Neill (1988) have described an approximate procedure for estimating the elastic shortening of long drilled shafts.

Long-term settlements of drilled shafts in clay are not reflected in Figures 4.11 and 4.12. Consolidation settlements should be added to the short-term settlements. However, because drilled shafts are usually installed in heavily overconsolidated soils, consolidation settlements are usually small.

Settlements induced by loads in end bearing are different for shafts in sand and in clay. While drilled shafts in clay typically have a well-defined plunging load, the tip load of drilled shafts in sand continue to increase as the settlement increases beyond 5 percent of the base diameter. Q_p is typically fully mobilized at displacements of 2 to 5 percent of the base diameter for shafts in cohesive soil. For shafts in cohesionless soil, there is no well defined failure at any displacement. The ultimate unit end bearing is defined arbitrarily as the bearing pressure required to cause settlement equal to 5 percent of the pier diameter, even though this does not correspond to complete failure of the soil beneath the base of the pier.

The curves in Figures 4.11 and 4.13 also show the settlements at which the ultimate side resistance is mobilized. Q_s is typically fully mobilized at displacements of 0.2 to 0.8 percent of the shaft diameter for shafts in cohesive soil. For shafts in cohesionless soil, this value is 0.1 to 1 percent.

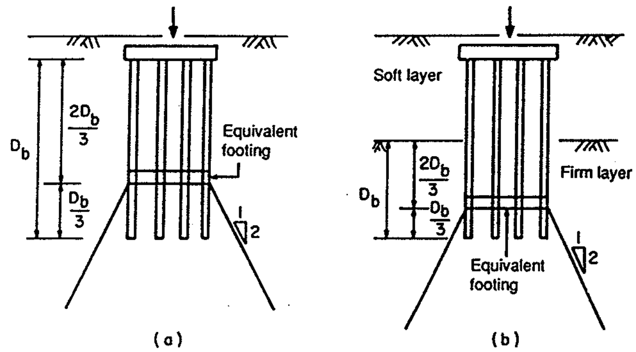


Figure 4.15. Location of equivalent footing. (After Duncan and Buchignani, 1976)

4.3.2 Group Settlement

Loads causing settlement of groups of drilled shafts are assumed to act on an equivalent footing located at two-thirds of the depth of embedment of the shafts in the layer which provides support (Duncan and Buchignani, 1976), as shown in Figure 4.15.

4.3.2.1 Cohesionless Soil

Meyerhof related the settlement of groups of piles and drilled shafts (ρ in inches) to the SPT blow count of the soil as follows:

$$\rho = \frac{2q\sqrt{X}\,I}{N_{corr}} \qquad (4.3.2.1)$$

where q = net foundation pressure (including any negative skin friction per unit area) in tsf applied in $2D_b/3$ (see Figure 4.15); X = width (smallest dimension) of group of drilled shafts, in ft; I = influence factor of the effective group embedment

$$I = 1 - D'/8X \geq 0.5 \qquad (4.3.2.2)$$

D' = effective depth = $2D_b/3$; N_{corr} = average corrected penetration resistance within the seat of settlement (approximately equal to the width of the group of drilled shafts below the equivalent footing) = $[0.77\log_{10}(20/\sigma_v')]$ N; N = measured SPT blow count; and σ_v' = effective vertical stress at the depth of the SPT measurement expressed in tsf.

The settlement of groups of drilled shafts in silty sand is estimated to be twice the value found using Eq. 4.3.2.1.

Static cone penetration tests may also be used to estimate settlement of groups of drilled shafts in cohesionless soils. Meyerhof (1976) related the settlement of groups of piles and drilled shafts to the average static cone resistance as follows:

$$\rho = \frac{qXI}{2q_c} \qquad (4.3.2.3)$$

where q_c = average static cone resistance within the seat of settlement and q, X, and I have been defined previously. The units for q, q_c and X should be consistent.

4.3.2.2 Cohesive Soil

The settlement of groups of drilled shafts in cohesive soils may occur over a considerable period of time. The long-term settlement of groups of drilled shafts in clay may be calculated using the methods employed in estimating settlement of shallow foundations. For this purpose, the load carried by a group of shafts is assumed to be transferred to the soil through an equivalent footing located at two-thirds the depth of the drilled shafts (see Figure 4.15).

The components contributing to the total settlement of a group of drilled shafts in clay are: immediate settlement, consolidation settlement, and secondary compression settlement. They can be estimated using the same procedures as used for shallow foundations.

The procedure for determining whether or not the settlement capacity of drilled shafts is sufficient is as follows:

1. Determine the tolerable settlement, ρ_{tol}.
2. Estimate the settlement of the drilled shaft or group of drilled shafts, ρ.
3. If the estimated settlement for an individual shaft exceeds the tolerable settlement, increase the length of the drilled shaft or its base diameter and repeat steps 1 and 2. If the settlement of a group of drilled shafts exceeds the tolerable value, increase the number of drilled shafts, the length of the drilled shafts, or the spacing and repeat steps 1 and 2.

4.4 NEGATIVE SKIN FRICTION

Negative skin friction is the downdrag force induced when the soil around the shafts moves downward relative to the shafts. Settlement of the soil around the shaft may occur due to: placement of fill, groundwater fluctuations, and other causes (Poulos and Davis, 1980).

Negative skin friction may be estimated using the rational methods discussed in Section 4.2.2 (the α and β methods). When using the α method, the top 5 feet and bottom one stem diameter do not contribute to the downdrag loads, and an allowance should be made for the possible increase in the undrained shear strength with time as consolidation occurs. An alternative approach would be to use the β method where the long-term conditions after consolidation should be considered. The unit negative skin friction, using the effective stress method (β method) is given by:

$$q_{sn} = \beta\sigma_v' \tag{4.4.1}$$

and the downdrag load is given by:

$$P_{sn} = q_{sn}a_sD_n \tag{4.4.2}$$

where P_{sn} = downdrag load, a_s = perimeter of the drilled shaft, and D_n = length of drilled shaft embedded in settling soil.

4.4.1 Design Considerations

Downdrag loads can increase the settlement of drilled shafts, but they rarely cause capacity problems. Settlement of drilled shafts should be checked when downdrag loads (unfactored) act together with dead loads. Temporary live loads and downdrag loads do not act together. This is because temporary live loads will compress the drilled shafts elastically and reduce the downdrag load. When the live load is removed, the drilled shafts will rebound elastically, thereby restoring the downdrag load.

If the magnitude of the downdrag load exceeds the magnitude of the live load, the structural and soil capacities should be checked for the dead load plus downdrag. The load factor for the downdrag load is the reciprocal of the performance factor for the ultimate side resistance of the drilled shafts. The following criterion expresses this fact:

$$\phi R \geq \gamma_D P_D + \frac{1}{\phi_{qs}} P_{sn} \tag{4.4.1.1}$$

where ϕ = performance factor corresponding to the limit state considered, R = resistance corresponding to the limit state considered, and ϕ_{qs} = performance factor for the ultimate skin resistance of the drilled shaft.

4.4.2 Neutral Plane

The neutral plane is defined as the elevation at which the settlement of the drilled shaft and the settlement of the soil are the same, as shown in Figure 4.16. Above the neutral plane, the soil loads the shaft in negative skin friction. Below the neutral plane, the shaft derives support from the soil. The distribution of load and resistance in a drilled shaft are shown in Figure 4.16(a). A dead load, P_D, acts at the top of the drilled shaft. With increasing depth, the load on the drilled shaft increases because of the negative skin friction. The total load acting on the drilled shaft, $P_D + P_{sn}$, increases accordingly. The drilled shaft resistance is equal to the tip capacity at the base, Q_p, and increases upwards as the side resistance, Q_s, increases. This is represented by the curve, $Q_p + Q_s$. The two curves intersect at the neutral plane. This is the location of the maximum load on the drilled shaft. The neutral plane of drilled shafts end-bearing on rock is located at the base of the drilled shafts.

4.4.3 Settlement

Figure 4.16(b) illustrates the procedure for estimating the settlement at the top of the drilled shaft. The settlement of the top of the drilled shaft is the sum of the settlement at the neutral plane and the elastic compression of the drilled shaft above the neutral plane (Figure 4.16(b)). Unfactored loads are used to estimate the settlement.

4.5 UPLIFT

Uplift of deep foundations may be caused by: swelling soils, frost heave, buoyancy, lateral loads, and upward loads. Drilled shafts subjected to uplift must be designed to withstand tensile stresses and pullout from the soil. Pullout resistance is usually adequate in long drilled shafts, but shafts end-bearing on bedrock at shallow depths may have small pullout resistance.

4.5.1 Uplift Capacity of a Single Drilled Shaft

Each drilled shaft in a group is loaded in either tension or compression. The load acting on each drilled shaft in a group may be estimated using Eq. 4.1.4.

4.5.1.1 Soil Capacity

The ultimate uplift capacity of a single drilled shaft is estimated in a manner similar to that for estimating the ultimate side resistance for drilled shafts in compression (Section 4.2.2). The design requirement for uplift is as follows:

$$\phi_u Q_s \geq P_{x,y} \tag{4.5.1.1}$$

where Q_s = ultimate uplift capacity due to shaft resistance, $P_{x,y}$ = factored tensile load effect in the drilled shaft (see Eq. 4.1.4), and ϕ_u = performance factor for uplift capacity.

The performance factors for axial compression and uplift are different because (1) the diameter and, thus, the area of the drilled shaft, decreases in tension due to the Poisson effect, thereby making uplift capacity smaller than compressive load capacity; and (2) drilled shafts in tension unload the soil. This reduces the overburden effective stress and, hence, the uplift side resistance of the drilled shaft.

The uplift capacity of a drilled shaft with an enlarged base (bell) should be calculated assuming that the bell behaves as an anchor (Reese and O'Neill, 1988). Any skin friction above the bell should be discounted. The uplift capacity of a belled drilled shaft ($Q_{s,bell}$) may be calculated as follows:

$$Q_{s,bell} = q_{s,bell} A_u \tag{4.5.1.2}$$

where $q_{s,bell}$ = unit uplift capacity of a belled drilled shaft = $N_u S_u$; A_u = annular area between the bell and the shaft = $\pi(D_p^2 - D^2)/4$; N_u = uplift bearing capacity factor; and S_u = undrained shear strength averaged over a distance of 2 bell diameters, $2D_p$, above the base. If the soil above the founding stratum is expansive, S_u should be averaged over $2D_p$ above the bottom of the base, or over the depth of penetration of the drilled shaft in the founding stratum, whichever is less. D_p = diameter of the bell and D = shaft diameter.

The value of N_u varies from 0 at $D_b/D_p = 0.75$, to 8 at $D_b/D_p = 2.5$, where D_b is the depth below the top of the founding stratum (Yazdanbod et al., 1987). As shown in Figure 4.17, the top of the founding stratum should be taken at the base of the zone of seasonal moisture change. This procedure conservatively neglects the uplift resistance contribution due to soil suction and the weight of the drilled shaft.

The uplift capacity of a drilled shaft may be verified by a load test according to ASTM D3689.

4.5.1.2 Structural Capacity

Fellenius et al. (1989) recommend that tensile loads should be carried entirely by the reinforcement, and that the tensile strength of concrete should be neglected. The design requirement is as follows:

$$\phi_t f_u A_y \geq P_{x,y} \tag{4.5.1.3}$$

where f_u = tensile strength of structural steel = yield stress of steel reinforcement, f_y; A_y = total area of steel reinforcements; and ϕ_t = performance factor for tensile capacity of steel = 0.9. In addition, the connection to the cap must be designed to transfer the tensile loads.

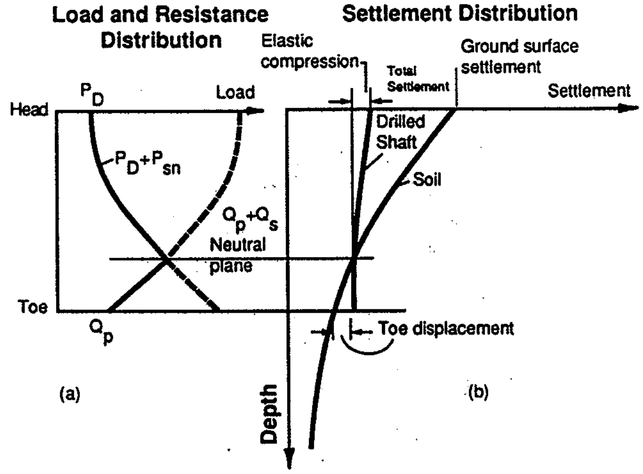


Figure 4.16. Calculation of the location of the neutral plane and the settlement of a drilled shaft or a group of shafts. (After Canadian Geotechnical Society, 1985)

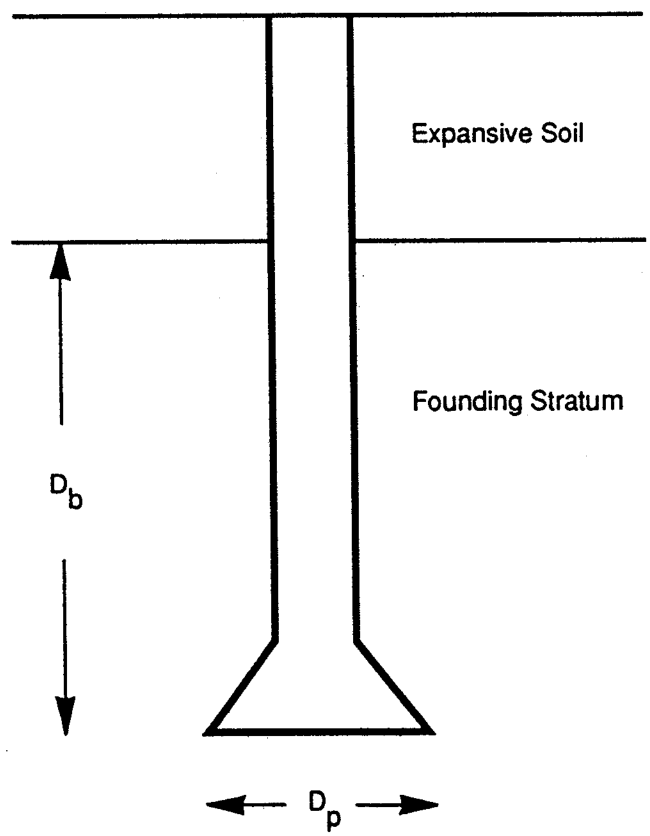


Figure 4.17. Uplift of an underreamed drilled shaft.

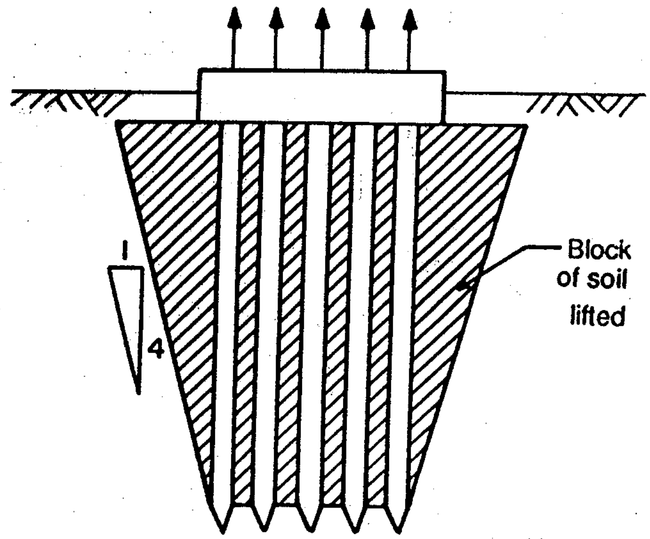


Figure 4.18(a). Uplift of group of closely spaced drilled shafts in cohesionless soils. (After Tomlinson, 1987)

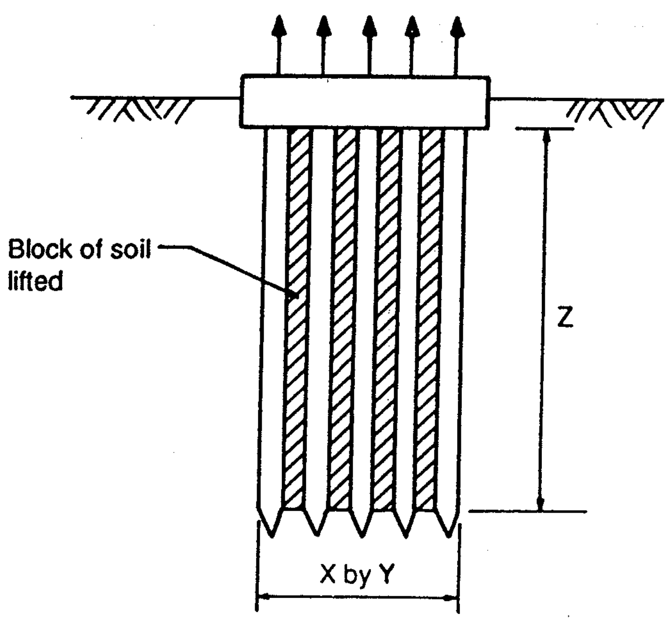


Figure 4.18(b). Uplift of group of drilled shafts in cohesive soils. (After Tomlinson, 1987)

4.5.2 Group Uplift Capacities

The ultimate uplift capacity of a group of drilled shafts should be taken as the minimum of the following two values: (1) the sum of the individual uplift capacities of the drilled shafts, or (2) the uplift capacity of the group considered as a block. The mechanism for the latter is different for drilled shafts in clays and sands.

The shaft friction of groups of drilled shafts in sands deteriorates with time if the shafts are subjected to vibratory and lateral loads. For pile groups, Tomlinson (1987) suggests that the weight of the block uplifted be estimated using a spread of load of 1 in 4 (Figure 4.18(a)) from the base of the group. Buoyant unit weights should be used for soil below the groundwater level. The same can be applied to groups of drilled shafts.

In clays, the uplift resistance of the block in undrained shear can be calculated as shown in Figure 4.18(b):

$$Q_{ug} = (2XZ + 2YZ)\bar{S}_u + W_g \qquad (4.5.2.1)$$

where Q_{ug} = ultimate uplift resistance of the group, X = width of the group, Y = length of the group, Z = depth of the block of soil below cap, $\bar{S}_u$ = average undrained shear strength along drilled shaft, and W_g = weight of the block of soil, drilled shafts and cap.

Table 4.5. Summary of performance factors for geotechnical ultimate limit states in axially loaded drilled shafts.

	METHOD/SOIL/CONDITION		PERFORMANCE FACTOR
ULTIMATE BEARING CAPACITY OF SINGLE DRILLED SHAFTS	SIDE RESISTANCE IN CLAY	α-method (Reese & O'Neill)	0.65
		β-method (Stas & Kulhawy)	0.50
	BASE RESISTANCE IN CLAY	Total Stress (Reese & O'Neill)	0.55
		Effective Stress (Stas & Kulhawy)	0.45
	SIDE RESISTANCE IN SAND	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See Discussion in Section 4.2.2
	BASE RESISTANCE IN SAND	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See Discussion in Section 4.2.2
	SIDE RESISTANCE IN ROCK	Carter & Kulhawy	0.55
		Horvath and Kenney	0.65
	BASE RESISTANCE IN ROCK	Canadian Foundation Engineering Method	0.50
		Pressuremeter Method	0.50
	SKIN FRICTION AND END BEARING	Load Test	0.80
BLOCK FAILURE	Clay		0.65
UPLIFT CAPACITY OF SINGLE DRILLED SHAFTS	CLAY	α-method (Reese & O'Neill)	0.55
		β-method (Stas & Kulhawy)	0.40
		Belled Shafts (Reese & O'Neill)	0.50
	SAND	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See Discussion in Section 4.2.2
	ROCK	Carter & Kulhawy	0.45
		Horvath & Kenney	0.55
		Load Test	0.80
GROUP UPLIFT CAPACITY		Sand	0.55
		Clay	0.55

4.6 PERFORMANCE FACTORS

Table 4.5 gives the geotechnical performance factors for the

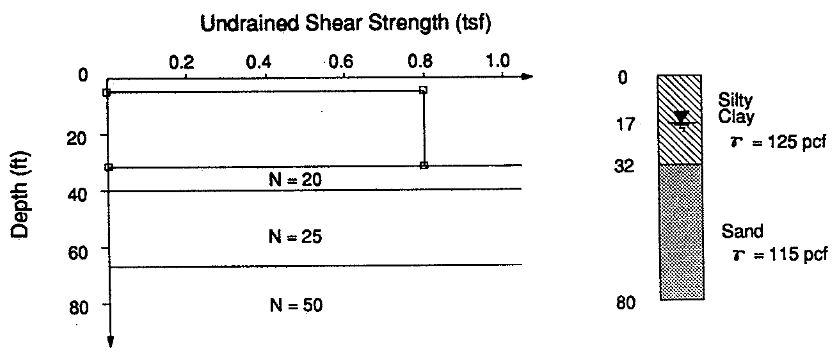
ultimate limit state. Serviceability limit state analyses should be carried out with unfactored loads and unfactored resistance.

4.7 DESIGN EXAMPLES

The design procedures discussed in the previous sections are demonstrated in the following example problems.

Example 4.1

Check the adequacy of the straight sided drilled shaft in the soil profile below to support a dead load of 100 tons and a live load of 20 tons. Diameter of the shaft = 30 in. Length of shaft = 59 ft.



1. Determine the design load on the drilled shaft

$$\gamma_D P_D + \gamma_L P_L = 1.3\ (100) + 2.17\ (20)$$
$$= 130 + 43$$
$$= 173 \text{ tons}$$

2. Estimate the axial capacity of the drilled shaft.

<u>Structural capacity</u>

From Equation 4.1-3

$$\phi_a P_n = (0.8)\ (0.7)\ [0.85\ f'_c\ A_c + f_y\ A_y]$$

Using a drilled shaft with f'c = 4 ksi; fy = 60 ksi and reinforced with 18# 10 bars.

$$A_y = (18)\ (1.27) = 22.9 \text{ in.}^2$$

$$A_g = \frac{\pi}{4}\ (30)^2 = 707 \text{ in.}^2$$

$$A_c = A_g - A_y = 707 - 22.9 = 684 \text{ in.}^2$$

$\phi_a = P_n = (0.8)(0.7)[(0.85)(4)(684) + (60)(22.9)]$

$= 2072$ kips

$= 1036$ tons > 173 tons

Bearing Capacity

Side resistance of shaft in clay

From depth 0 to 5 ft, $S_u = 0 \therefore Q_{Si} = 0$

From depth 5 ft to 32 ft, $S_u = 0.8$ tsf.

$A_{S2} = (\pi \frac{30}{12})\, 27 = 212\ ft^2$

$Q_{S2} = \alpha S_u As2 = (0.55)(0.8)(212)$

$= 93$ tons

Side resistance of shaft in sand

From depth 32 to 40 ft, N = 20 — Weighted average of

From depth 40 to 59 ft, N = 25 — $N = [(20)(8) + (25)(19)]/27 = 23.5$

From depth 32 to 59 ft, $A_S = \pi (\frac{30}{12})\, 27 = 212\ ft^2$

(i) Touma and Reese (1974)

From Table 4.3, K = 0.6 for 25 ft < $D_b \leq 40$ ft

Depth of strata (ft)	Mid-depth of strata (ft)	σ'_v (tsf)	As (ft^2)	N	ϕ' (deg)	$Q_S = K\sigma'_v \tan\phi' A_S$ (tons)
32-40	36	1.637	62.8	20	37.5°	47.3
40-59	49.5	1.992	149.2	25	38.8°	143.4

191 tons

(ii) Meyerhof (1976)

$Q_{S3} = \frac{N}{100}(212) = \frac{23.5}{100}(212)$

$= 50$ tons

(iii) Quiros and Reese (1977)

$Q_{S3} = 0.026\ N\ (212)$

$= (0.026)(23.5)(212)$

$= 130$ tons

(iv) Reese and Wright (1977)

$Q_{S3} = \frac{N}{34}(212)$

$= \frac{23.5}{34}(212)$

$= 147$ tons

(v) Reese and O'Neill (1988)

$\beta = 1.5 - 0.135\sqrt{Z}$

$= 1.5 - 0.135\sqrt{59}$

$= 0.463$

$\sigma'_v = [(125)(17) + (125-62.4)(15) + (115-62.4)\ 13.5]/2000$

$= 1.887$ tsf

$\therefore Q_{S3} = \beta\sigma'_v (212)$

$= (0.463)(1.887)(212)$

$= 185$ tons

Base resistance of shaft in sand

(i) Touma and Reese (1974)

$$Q_p = \frac{16}{0.6D} A_g$$

$$= \frac{16}{(0.6)\left(\frac{30}{12}\right)} \frac{707}{144}$$

$$= \underline{52 \text{ tons}}$$

(ii) Meyerhof (1976)

$D_b = 27$ ft $\qquad \therefore \frac{D_b}{D_p} = \frac{27}{2.5} > 10$

$\therefore$ Use limiting value of $Q_p = \frac{4}{3} N_{corr} A_g$

$$\sigma'_v = [(125)(17) + (62.6)(15) + (52.6)(27)]/2000$$

$$= 2.24 \text{ tsf}$$

$$N_{corr} = (0.77)\left[\log_{10}\left(\frac{20}{2.24}\right)\right](25)$$

$$= 18.3$$

$$\therefore Q_p = \frac{4}{3}(18.3)\left(\frac{707}{144}\right)$$

$$= \underline{120 \text{ tons}}$$

(iii) Quiros and Reese (1977)

Same as Touma and Reese

i.e. $Q_p = 52$ tons

(iv) Reese and Wright (1977)

$$Q_p = \frac{2}{3} N A_g$$

$$= \frac{2}{3}(25)\left(\frac{707}{144}\right)$$

$$= \underline{82 \text{ tons}}$$

(v) O'Neill and Reese (1988)

$$Q_p = 0.6 N \left(\frac{707}{144}\right)$$

$$= (0.6)(25)\left(\frac{707}{144}\right)$$

$$= \underline{74 \text{ tons}}$$

<u>Summary of Results</u>

	Q_{S2} (tons)	Q_{S3} (tons)	Q_p (tons)	Q_{ult} (tons)	$\phi_s Q_s + \phi_p Q_p$* (tons)	Factor of Safety $= Q_{ult}/120$
Touma & Reese	93	191	52	336	182	2.80
Meyerhof	93	50	120	263	145<173	2.19
Quiros & Reese	93	130	52	275	151<173	2.29
Reese & Wright	93	147	82	322	175	2.68
Reese & O'Neill	93	185	74	352	190	2.93

The example problem shows that three methods (Touma & Reese, Reese & Wright and Reese & O'Neill) indicate that the bearing capacity of the drilled shaft is adequate while the other two methods (Meyerhof & Quiros and Reese) indicate that the drilled shaft needs to be lenghtened or the diameter increased, in order to be safe.

* Using $\phi_{qs} = \phi_{qp} = 0.50$ for calculating capacities in sand
$\phi_{qs} = 0.65$ for the α – method

The Touma and Reese & Quiros and Reese methods for calculating base resistance are based on a settlement of 1 inch while the Reese and Wright & Reese and O'Neill's methods are based on a settlement of 5% of the base diameter (1.5 in.). This explains the lower base resistance calculated using the Touma and Reese, and Quiros and Reese methods.

3. Check settlement under working load

Since most of the capacity is derived from the side resistance and end bearing in sand, Figs. 4.13 and 4.14 will be used to estimate the settlement.

Using a modulus of 3.5×10^3 ksi for the drilled shaft and assuming that the average load in the shaft is 60% of the total applied load, the elastic shortening of the shaft is estimated as:

$$\rho_e = \frac{PZ}{AE} = \frac{(0.6)\,(120)\,(2000)\,(59)\,(12)}{(707)\,(3.5 \times 10^3)\,(1000)}$$

$$= 0.041 \text{ in.}$$

Using the trend lines of Figs. 4.13 and 4.14, the following table is obtained.

Trial Settlement	Side Resistance			End Bearing			Q_T
(in.)	ρ/D (%)	Q/Q_S	Q^* (tons)	ρ/D (%)	Q/Q_P	Q^{**} (tons)	(tons)
0.04	0.133	0.40	96	0.133	0.02	2	98
0.05	0.167	0.49	118	0.167	0.03	2	120***

* For the purpose of this example, it is assumed that the side resistance is obtained from the Reese and Wright method.

$$Q_S = 93 + 147 = 240$$

** It is assumed that the end bearing is obtained using the Reese and Wright method i.e. Q_P = 82 tons.

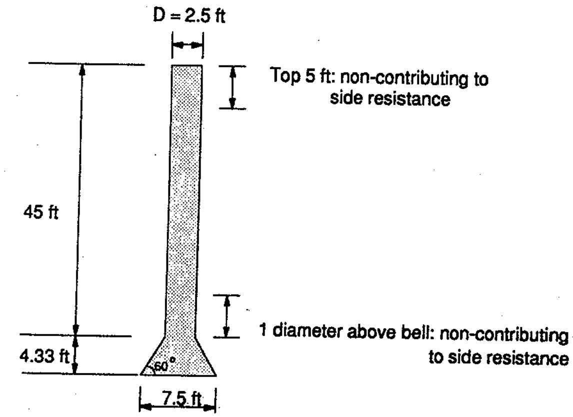
*** Corresponds exactly to working load of 120 tons.

The settlement at the top of the drilled shaft would be about 0.05 in. + elastic compression = 0.05 + 0.04 ≅ 0.1 in.

This would be acceptable under almost any conceivable circumstances.

Example 4.2

Check the adequacy of a underreamed drilled shaft in a stiff clay with a uniform strength of 2000 psf to support a dead load of 140 tons and a live load of 50 tons. The groundwater table is at a very large depth and is not considered in this example. The shaft dimensions are shown in the figure below.



1. Determine the design load on the drilled shaft

$$\gamma_D P_D + \gamma_L P_L = 1.3\,(140) + 2.17\,(50)$$

$$= 182 + 109$$

$$= 291 \text{ tons}$$

2. Estimate axial capacity of the drilled shaft

<u>Structural capacity of shaft</u>

From Equation 4.1-3

$$\phi_a P_n = (0.8)\,(0.7)\,[0.85\, f'_c\, A_c + f_y\, A_y]$$

Assume f'c = 4 ksi and fy = 60 ksi and 18 # 9 bars are used to reinforce the drilled shaft.

$$A_y = (18)\,(1.0) = 18 \text{ in.}^2$$

$$A_g = \frac{\pi}{4}\,(30)^2 = 707 \text{ in.}^2$$

$A_C = A_g - A_y = 707 - 18 = 689$ in.^{2}

$\phi_a P_n = (0.8)(0.7)[(0.85)(4)(689) + (60)(18)]$

$= 1917$ kips

$= 958$ tons > 291 tons.

Structural capacity of bell

In Table 2.1, the maximum net bearing pressure that can be sustained in a 60° underream without cracking is 25 ksf. Applying a concrete strength reduction factor of 0.85, the structural capacity of the bell is:

$= 0.85 \left(\frac{25}{2}\right) \frac{\pi}{4} (7.5)^2$

$= 469$ tons $> (291 - 162) = 129$ tons

$= Q_{S2}$ (see below)

Bearing Capacity

Shaft resistance of straight sided shaft

From depth 0 to 5 ft, $Q_{S1} = 0$

From depth 5 ft to 425 ft, $S_u = 2000$ psf $= 1$ tsf

$A_{S2} = \pi (2.5)(42.5 - 5) = 295$ ft.^{2}

$Q_{S2} = \alpha S_u A_{S2} = (0.55)(1)(295)$

$= 162$ tons

Base resistance

Since $D_p = 7.5 \times 12 = 90$ in. > 75 in., see Eq. 4.2.2.6 to calculate a reduced base resistance.

$a = 0.0071 + 0.0021 \, Z/D_p$

$= 0.0071 + 0.0021 (49.33/7.5)$

$= 0.0209 \leq 0.015$

$b = 0.45\sqrt{S_u \text{ (ksf)}}$ $\quad 0.5 \leq b \leq 1.5$

$= 0.45\sqrt{2}$

$= 0.636$

$$F_r = \frac{2.5}{(0.015)(7.5)(12) + (2.5)(0.636)}$$

$= 0.85$

$q_{pr} = 0.85 \, q_p$

$q_p = N_c S_u \quad \leq 40$ tsf

$N_c = 6 [1 + 0.2 \, Z/D_p] \leq 9$

$N_c = 6 [1 + (0.2)(49.33/7.5)]$

$= 13.9 > 9$

$\therefore N_c = 9$

$q_p = (9)(1)$

$= 9$ tsf

$Q_p = (0.85)(9)\left(\frac{\pi}{4}\right)(7.5)^2$

$= 338$ tons

The factored ultimate bearing capacity

$= \phi_{qs} Q_s + \phi_{qp} Q_p$

$= (0.65)(162) + (0.55)(338)$

$= 105 + 186$

$= 291$ tons ≥ 291 tons

3. Check settlement under working load

From Fig. 4.11, the maximum side resistance is reached at a settlement of 0.6% of the diameter

$$= \frac{0.6}{100}(30) = 0.18 \text{ in.}$$

The full side resistance of 162 tons can be assumed to be fully mobilized.

The remainder of the working load (190 - 162) = 28 tons will be carried by the base.

$$\frac{\text{End Bearing}}{\text{Ultimate End Bearing}} = \frac{28}{338} = 0.07 \text{ or } 7\%$$

From Fig. 4.12, $\frac{\text{Settlement of Base}}{\text{Diameter of Base}} = 0.2\ \%$

$$\therefore \text{Settlement of Base} = \frac{0.2}{100}(90)$$

$$= 0.18 \text{ in.}$$

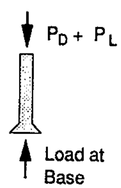
Therefore the full side resistance will be mobilized but only 7% of the base resistance will be mobilized.

The elastic compression of the shaft assuming that the bell is incompressible can be found as follows:

$$\rho_c = \frac{PZ}{AE} \qquad \text{where } P = \frac{P_D + P_L + \text{Load carried by base}}{2}$$

$$= \frac{190 + 28}{2}$$

$$= 109 \text{ tons}$$



$$\therefore \rho_c = \frac{(109)(2000)(45)(12)}{(707)(3.5 \times 10^6)}$$

$$= 0.048$$

$$\therefore \text{Settlement at top of drilled shaft} = 0.18 + 0.048$$

$$= 0.23 \text{ in.}$$

Example 4.3

Check the adequacy of a 30 in. diameter drilled shaft socketed 10 ft into basalt to support a dead load of 100 tons and a live load of 60 tons. The compressive strength of the basalt is 1.0 ksi and the RQD is 70%.

1. Determine the design load on the drilled shaft

$$\gamma_D P_D + \gamma_L P_L = 1.3\,(100) + 2.17\,(60)$$

$$= 130 + 130$$

$$= 260 \text{ tons}$$

2. Estimate the axial capacity of the drilled shaft

<u>Structural Capacity</u>

From Equation 4.1-3

$$\phi_a P_n = (0.8)(0.7)[0.85 f'_c A_c + f_y A_y]$$

The drilled shaft is reinforced with 8 # 11 bars with a yield stress fy = 60 ksi. The compressive strength of the concrete $f'_c = 4$ ksi;

$$A_y = (8)(1.56) = 12.5 \text{ in.}^2$$

$$A_g = \frac{\pi}{4}(30)^2 = 707 \text{ in.}^2$$

$$A_c = A_g - A_y = 707 - 12.5 = 695 \text{ in.}^2$$

$$\phi_a P_n = (0.8)(0.7)[(0.85)(4)(695) + (60)(12.5)]$$

$$= 1743 \text{ kips}$$

$$= 872 \text{ tons} > 260 \text{ tons}$$

<u>Bearing Capacity</u>

From Fig. 4.6, the Young's modulus of the intact basalt is 5×10^2 ksi and the modulus ratio is 500. From Fig. 4.7, the modulus of the in situ rock mass (RQD = 70%) is 20% of the intact modulus. ∴ In situ modulus of rock, $E_r = (0.2)(5 \times 10^2) = 100$ ksi.

In order to determine whether the bearing capacity of the drilled shaft is derived from shaft resistance or base resistance, calculate $\rho_e + \rho_{base}$ to see if it is greater than or less than 0.4 in.

ρ_e:

From Equation 4.2.2-12,

$$\rho_e = \frac{(\Sigma P_i)\, H_s}{A_{soc}\, E_c}$$

$$= \frac{(160)\,(2000)\,(10)\,(12)}{(707)\; 3.5 \times 10^6}$$

$$= 0.016 \text{ in.}$$

ρ_{base}:

From Equation 4.2.2-13,

$$\rho_{base} = \frac{(\Sigma P_i)\, I_s}{D_s\, E_r}$$

From Fig. 4.5, $I_p = 0.32$ for $\frac{H_s}{D_s} = \frac{10}{2.5} = 4$ and

$$\frac{E_c}{E_r} = \frac{3.5 \times 10^6}{100 \times 10^3} = 35$$

$$\therefore\ \rho_{base} = \frac{(160)\,(2000)\,(0.32)}{(30)\; 100 \times 10^3}$$

$$= 0.034 \text{ in.}$$

$$\therefore\ \rho_e + \rho_{base} = 0.016 + 0.034$$

$$= 0.05 \text{ in.} < 0.4 \text{ in.}$$

$\therefore$ Drilled shaft capacity is derived mainly from side resistance.

Since the uniaxial compressive strength of the concrete is > 280 psi, use Equation 4.2.2-16.

$$q_s = 2.5\sqrt{q_u}$$

$$= 2.5\sqrt{1000}$$

$$= 79.1 \text{ psi}$$

$$Q_S = q_S\, A_S$$

$$= (79.1)\,(\pi)\,(30)\,(10)\,(12)/(2000)$$

$$= 447 \text{ tons}$$

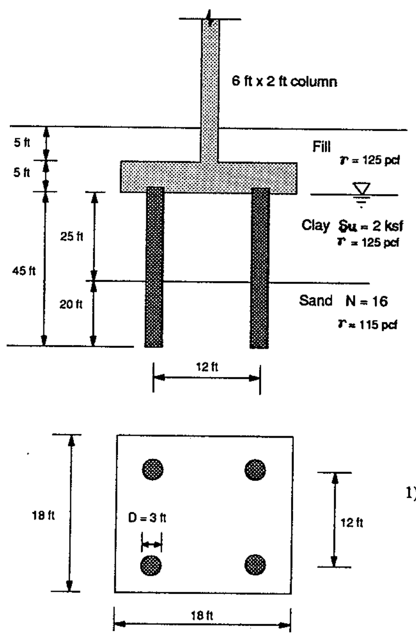
Factored capacity $= \phi_{qs}\, Q_S$

$$= (0.65)\,(447)$$

$$= 291 \text{ tons} > 260 \text{ tons}$$

Example 4.4

Check the adequacy of the group of drilled shafts shown below to support a dead load of 90 tons and a live load of 30 tons. Consider the effects of negative skin friction.



1) Determine the design load on the group of shafts

Weight of cap = 5(18)(18)(150)/2000 = 122 tons

Weight of soil above cap = $5[(18)^2 - (6)(2)]125/2000$

= 98 tons

Dead load due to cap and soil = 122 + 98

= 220 tons

$$\therefore \gamma_D P_D + \gamma_L P_L = 1.3\,(220 + 90) + 2.17\,(30)$$

$$= 403 + 65$$
$$= 468 \text{ tons}$$

2. Estimate axial capacity of the drilled shaft

<u>Structural capacity</u>

From Equation 4.1-3

$$\phi_a P_n = (0.8)\,(0.7)\,[0.85\, f'_c\, A_c + f_y\, A_y]$$

Using a drilled shaft with $f'_c = 4$ ksi and $f_y = 60$ ksi with 15 # 10 bars.

$$A_y = (15)\,(1.27) = 19.1 \text{ in.}^2$$

$$A_g = \frac{\pi}{4}\,(36)^2 = 1018 \text{ in.}^2$$

$$A_c = A_g - A_y = 1018 - 19.1 = 999 \text{ in.}^2$$

$$\therefore \phi_a P_n = (0.8)\,(0.7)\,[(0.85)\,(4)\,(999) + (60)\,(19.1)]$$

$$= 2544 \text{ kips}$$

$$= 1272 \text{ tons} < 468/4 \text{ or } 117 \text{ tons/shaft}$$

<u>Bearing capacity of a single drilled shaft</u>

Side resistance of shaft in clay

From depth 0 to 5 ft, $Q_{S1} = 0$ (Depth 0 refers to top of shaft)

From depth 5 ft to 25 ft, $S_u = 2$ ksf = 1 tsf

$$Q_{S2} = \alpha S_u\, A_{S2} = (0.55)\,(1)\,(\pi)\,(3)\,(20)$$

$$= 104 \text{ tons}$$

Side resistance of shaft in sand

From depth 25 ft to 45 ft, using Reese and Wright's procedure* for illustrative purpose in this example:

$$A_{S3} = \frac{N}{34}\,A_{S3} = \frac{16}{34}\,(\pi)\,(3)\,(20)$$

$$= 89 \text{ tons}$$

* As discussed in Section 4.2.2, designer should use all 5 methods and apply judgment to select factored capacity.

End bearing of shaft in sand

$$Q_p = \frac{2}{3}\ N\ A_p \qquad \text{(Reese and Wright, 1977)}$$

$$= \frac{2}{3}\ (16)\ (1018)/144$$

$$= 75 \text{ tons}$$

Total factored capacity of a single drilled shaft

Factored capacity = (0.65) (104) + (0.5) (89) + (0.5) (75)

= 150 tons

Assumed ϕ – factors for side and tip resistance using Reese & Wright's method.

Group capacity

The capacity of a group of drilled shafts in sand is the sum of the individual shaft capacities multiplied by an efficiency factor, η. The value of η is 0.7 for a spacing of 3D and 1 for a spacing of 6D.

$\therefore\ \eta = 0.8$ for a spacing of 4D

Factored group capacity = $\eta\ N_{ds}\ \phi_q\ Q_{ult}$

= (0.8) (4) (150)

= 480 tons > 468 tons

3. Estimate settlement of group of drilled shafts in sand stratum (bearing layer)

From Equation 4.3.2-1,

$$\rho = \frac{2q\sqrt{X}\ I}{N_{corr}}$$

$$q = \frac{90 + 220 + 30}{(15)^2}$$

$$= 1.51 \text{ tsf}$$

$$I = 1 - \frac{D'}{8X} \geq 0.5$$

$$= 1 - \frac{(\frac{2}{3})\ (20)}{(8)\ (15)}$$

$$= \frac{8}{9}$$

$$\sigma'_v = [(125 - 62.4) + 20\ (115 - 62.4)]/2000 \doteq 1.31 \text{ tsf.}$$

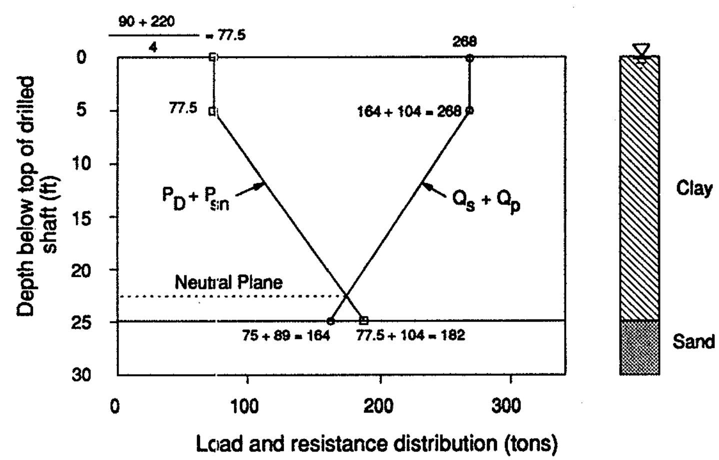
$$N_{corr} = [0.77 \log_{10} (20/1.31)]\ 16$$

$$= 14.6$$

$$\therefore\ \rho = \frac{(2)\ (1.51)\ \sqrt{15}\ (\frac{8}{9})}{14.6}$$

$$\approx 0.7 \text{ in.}$$

4. Check the effect of the downdrag load.



The load at the neutral plane is approximately 172 tons. Since this is greater than the applied load per shaft of $\frac{220 + 90 + 30}{4} = 85$ tons, settlement, bearing capacity and structural capacity should be checked for downdrag loading.

(i) Determine the design load due to the dead load and downdrag load

$$\gamma_D P_D + \frac{1}{\phi_n} P_{sn} = (1.3)(220 + 90) + \frac{1}{0.65}(172 - 77.5)\,4$$

$$= 403 + 582$$

$= 985$ tons for the group or 246 tons/shaft

(ii) Structural capacity

The structural capacity of 1261 tons per shaft is adequate to support the load per shaft of 246 tons.

(iii) Bearing capacity

The factored bearing capacity of 480 tons for the group is inadequate to support the load effect of 984 tons due to the dead load and downdrag load. Either more shafts, larger shafts or wider spacings should be used.

(iv) Settlement

$$q = \frac{(4)(246)}{(15)(15)} = 4.37 \text{ tsf}$$

$$\therefore \rho = \frac{4.37}{1.51}(0.7)$$

$$= 2 \text{ in.}$$

CHAPTER 5

DESIGN OF DRILLED SHAFTS FOR LATERAL LOADING

Lateral loads on drilled shaft foundations arise because of wind, earthquake, water pressures, earth pressures, and live loads. Drilled shaft foundations must be designed to withstand such forces without failing (i.e., without reaching the ultimate limit state), and without deflecting excessively (i.e., without reaching the serviceability limit state).

Batter piles are frequently used to resist lateral loads. However, constructing batter drilled shafts is difficult. Construction problems include maintaining hole stability during excavation, installing casing and rebar cages in inclined holes, concrete placement in inclined holes, and availability of suitable construction equipment. Because of these difficulties, batter drilled shafts are used infrequently.

The governing criterion in the design of laterally loaded drilled shafts is almost always the maximum tolerable deflection or the structural capacity of the shaft itself. Ultimate soil failure does not control the design because mobilizing the ultimate lateral capacity of the soil requires such large displacements that this is not a realistic possibility.

In designing vertical drilled shafts to resist lateral loads, both lateral deflection and structural capacity should be considered. Procedures for addressing these issues are described in the following sections.

5.1 LATERAL DEFLECTION

One of the design objectives is to ensure that the lateral deflection of the drilled shaft or group of drilled shafts does not exceed the tolerable limit. The lateral deflection of a group of drilled shafts can be related to the lateral deflection of a single drilled shaft. Procedures for estimating lateral deflections of single drilled shafts and groups of drilled shafts are described in the following sections.

5.1.1. Deflection of Single Drilled Shafts

The behavior of deep foundations under lateral load can be analyzed using the following methods (Poulos and Davis, 1980): (1) elastic analysis, (2) subgrade reaction analysis, and (3) p-y analysis. Elastic analyses and subgrade reaction analyses are based on the assumption that the soil behaves as a linear material; and p-y analyses model nonlinear behavior, but require the use of computer programs and involve considerable engineering time.

The condition of restraint against rotation at the top of a drilled shaft has a strong effect on the magnitude of its lateral deflection under load. Drilled shafts that are embedded in reinforced concrete caps are effectively restrained from rotation at the top, and they deflect laterally with negligible rotation at the top of the shaft. On the other hand, some drilled shafts are connected directly to the structure without a cap; in which case, they are free to rotate and translate at the top. The lateral deflection of a fixed-head shaft is one-fourth as large as the deflection of a free-head shaft subjected to the same load.

The procedure for analyzing lateral deflections of single drilled shafts described in this manual is the one developed by Evans and Duncan (1982). The method models nonlinear behavior of the soil, but does not require computer analyses.

5.1.1.1 Evans and Duncan Procedure

Evans and Duncan (1982) related lateral deflections of deep foundations to the lateral loads using what they called a characteristic load, P_c. The characteristic load, P_c, embodies the important properties of the drilled shaft (diameter, stiffness) and the soil (strength, stiffness) that determine the way that the drilled shaft and soil respond to lateral loads. The larger the value of P_c, the greater is the capacity of the drilled shaft to carry lateral loads, and the smaller is its deflection under a given lateral load.

The Evans and Duncan procedure is applicable to drilled shafts with a length-to-diameter ratio of 10 or greater for shafts in firm soils and 15 or greater for shafts in soft soils.

5.1.1.2 Fixed-Head Drilled Shafts

The procedures and charts discussed in this section are for fixed-head drilled shafts. Charts in dimensionless form were developed for sand and clay (Figures 5.1 and 5.2). These charts show the variation of P_s/P_c with Y_s/D. P_s is the unfactored lateral load, Y_s is the shaft displacement, and D is the diameter of the drilled shaft. These charts model the same nonlinear behavior of soil as the p-y method of analysis. The procedure for

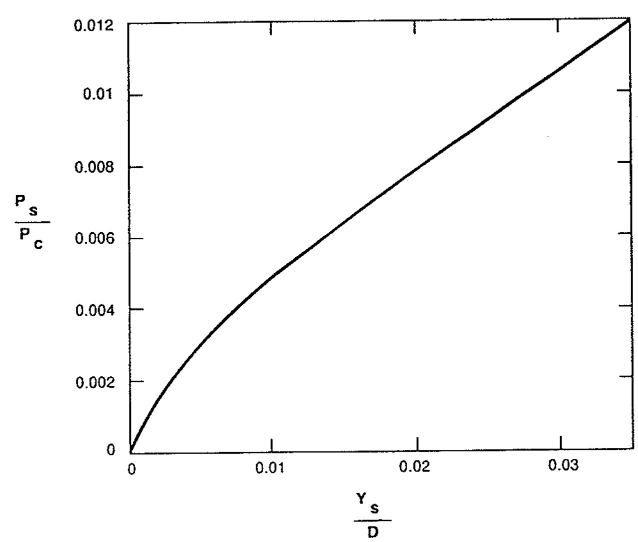


Figure 5.1. Lateral load versus deflection for fixed-head drilled shafts in sand. (After Evans and Duncan, 1982)

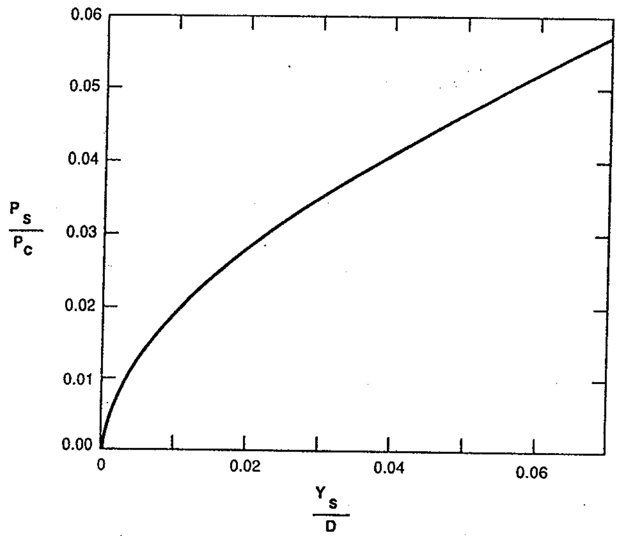


Figure 5.2. Lateral load versus deflection for fixed-head drilled shafts in clay. (After Evans and Duncan, 1982)

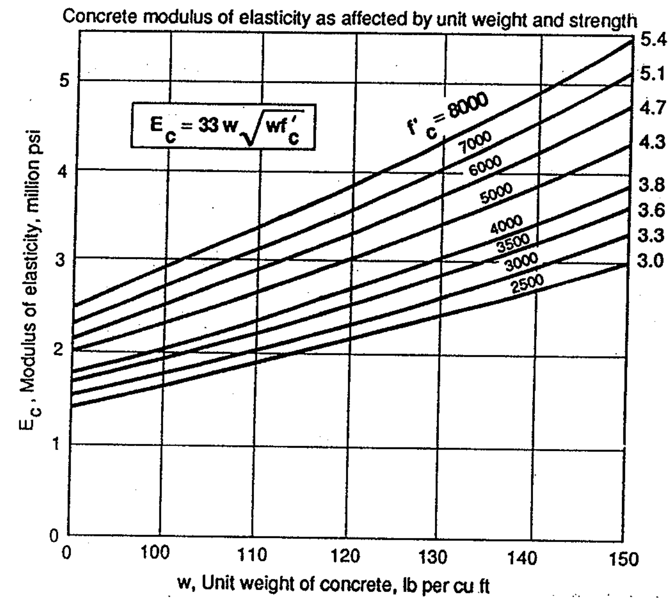


Figure 5.3. Modulus of elasticity of concrete. (After PCI, 1985)

determining the lateral deflection of a drilled shaft, using Figures 5.1 and 5.2, is as follows:

1. Select the diameter, D, the concrete modulus, E_c, and the steel reinforcement for the drilled shaft.

The quantities needed for analysis are the flexural stiffness of the drilled pier, E_pI_p and R_I, the ratio of the moment of inertia of the shaft to the moment of inertia of a solid, unreinforced, circular section. The moment of inertia of the shaft can be calculated considering the separate contribution of the concrete and the steel. The Young's modulus of the shaft, E_p, is conveniently taken as being equal to the Young's modulus of concrete, E_c, which can be related to the concrete compressive strength and density, as shown in Figure 5.3. The modulus of steel can be taken as 29×10^6 psi. Figure 5.4 shows an example for calculating E_pI_p and R_I for a drilled shaft.

2. Estimate the average undrained shear strength, S_u, for clays, or the average angle of internal friction, ϕ', for sands.

The behavior of the soil close to the ground surface is most important with regard to lateral loads. The properties (S_u for clays, ϕ' and unit weight, γ', for sands) should be averaged over a depth extending about eight shaft diameters below the top of the drilled shaft. Buoyant unit weights for sands are used below the water table.

3. Determine the characteristic load, P_c, which is defined by the following equations:

For clay

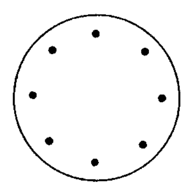
$$P_c = 7.34\, D^2 (E_pR_I)(S_u/E_pR_I)^{0.683} \qquad (5.1.1.1)$$

For sand

$$P_c = 1.57\, D^2 (E_pR_I)(\gamma' D\phi' K_p/E_pR_I)^{0.57} \qquad (5.1.1.2)$$

where R_I = moment of inertia ratio = I_p/I_{solid} (see Table 5.1 for values of R_I for drilled shafts of various diameters and percentages of steel area); I_{solid} = moment of inertia of a solid circular cross section = $\pi D^4/64$.

Thus, $E_pR_I = (E_pI_p)/(\pi D^4/64)$; K_p = Rankine passive earth



An 18" diameter drilled shaft is reinforced with 8#5 bars. Compute the moment of inertia, I_p, and the moment of inertia ratio, R_I, for the drilled shaft with E_c = 3500 ksi, E_s = 29 000 ksi and cover, c = 3 in.

D = shaft diameter = 18 in.
d_s = steel bar diameter = 5/8 in.

Moment of inertia of gross cross-section

$$I_c = \frac{\pi D^4}{64} - 8\frac{\pi d_s^4}{64} - \left(2 + \frac{4}{(\sqrt{2})^2}\right)\frac{\pi d_s^2}{4}\left[\frac{D}{2} - c - \frac{d_s}{2}\right]^2$$

I_c = 5 113 in^4

Moment of inertia of steel

$$I_s = 8\frac{\pi d_s^4}{64} + \left(2 + \frac{4}{(\sqrt{2})^2}\right)\frac{\pi d_s^2}{4}\left[\frac{D}{2} - c - \frac{d_s}{2}\right]^2$$

I_s = 40 in^4

$E_pI_p = E_cI_c + E_sI_s$

E_pI_p = 3500(5113) + 29000(40)

E_pI_p = 1.91 X 10^7 kip-in^2

For the equivalent homogeneous section, use $E_p = E_c$

$$\text{Therefore, } I_p = \frac{E_cI_c + E_sI_s}{E_c} = \frac{1.91 \text{ X } 10^7}{3500}$$

I_p = 5444 in^4

Moment of inertia ratio

$$R_I = \frac{I_p}{I_{solid}} = \frac{5444(64)}{\pi(18)^4}$$

R_I = 1.06 for A_s/A_g = 0.01

Figure 5.4. Example for calculating an equivalent I_p and R_I using $E_p = E_c$.

pressure coefficient $= \tan^2 (45° + \phi'/2)$; and ϕ' = angle of internal friction for sand, in degrees.

4. Calculate the value of the load ratio P_s/P_c.
5. Use Figure 5.1 for shafts in sand or Figure 5.2 for shafts in clay to determine the value of Y_s/D.
6. Calculate $Y_s = D\ (Y_s/D)$.

This procedure has been used to develop lateral load-deflection curves for some commonly used drilled shaft sections. Charts for drilled shafts of 18-in., 24-in., 30-in., and 36-in. diameters, with percentages of reinforcement equal to 1 percent, 2 percent, 4 percent and 8 percent, constructed in sand and clay, are shown in Figures 5.5 through 5.12. For these drilled shafts and soil conditions, deflections can be estimated directly using the charts. For example, a lateral load of 25 kip acting on an 18-in. drilled shaft with 4 percent steel reinforcement, constructed in clay with an undrained shear strength of 2 ksf, will result in a lateral deflection of about 0.1 in. (Figure 5.11).

For sands, charts were developed for friction angles of 30 deg, 35 deg, and 40 deg. The water table was assumed to be at or above the ground surface. For intermediate values of friction angle between those shown in the charts, deflections may be estimated by interpolation.

For clays, the load-deflection curves were developed for undrained shear strengths of 1, 2, and 4 ksf. Deflections for intermediate values of undrained shear strengths can be estimated by interpolation.

Table 5.1. R_I values for drilled shafts with E_c = 3500 ksi, E_s = 29000 ksi, and c = 3 in.

	DIAMETER OF DRILLED SHAFT			
A_s/A_g	18 in.	24 in.	30 in	36 in.
0.01	1.06	1.07	1.09	1.09
0.02	1.11	1.14	1.16	1.18
0.04	1.21	1.27	1.31	1.34
0.08	1.38	1.50	1.58	1.63

where A_s = area of steel and

A_g = gross cross-sectional area of drilled shaft

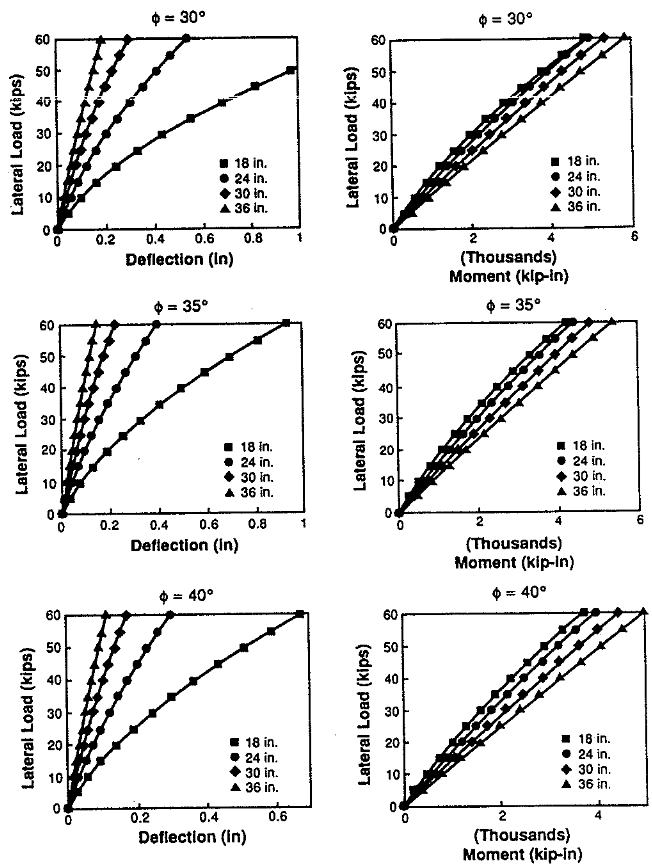


Figure 5.5. Load versus deflection and load versus moment for drilled shafts (A_s/A_g = 1%) in sand.

5.1.1.3 Free-Head Drilled Shafts

A lateral load, P_s, acting at a distance, e, above the ground, can be resolved into two components as shown in Figure 5.13—a lateral load with the same magnitude, P_s, acting at the groundline, plus a bending moment, M_e, equal to the lateral load multiplied by the eccentricity (i.e., $M_e = P_s e$). The lateral displacement of a free-head drilled shaft can be estimated using nonlinear superposition of the deflection caused by the lateral load, Y_{sP}, and the deflection caused by the bending moment, Y_{sM}.

The component of the lateral displacement, Y_{sP}, due to the groundline lateral load, can be estimated using Figures 5.14 and 5.15. The procedure is the same as described in connection with Figures 5.1 and 5.2.

The component of the lateral displacement, Y_{sM}, due to the bending moment, can be estimated as follows:

1. Calculate the bending moment $M_e = P_s e$.
2. Determine the characteristic moment, M_c, which is defined by the following equations:

For clay

$$M_c = 3.86\ D^3\ (E_p R_I)\ (S_u/E_p R_I)^{0.46} \qquad (5.1.1.3)$$

For sand

$$M_c = 1.33\ D^3\ (E_p R_I)\ (\gamma' D \phi' K_p/E_p R_I)^{0.4} \qquad (5.1.1.4)$$

where R_I, K_p, and ϕ' are as defined previously.

3. Calculate the ratio M_e/M_c.
4. Use Figure 5.16 for shafts in sand and Figure 5.17 for shafts in clay to determine the value of Y_{sM}/D.
5. Calculate $Y_{sM} = D(Y_{sM}/D)$.

Knowing Y_{sP} and Y_{sM}, the total lateral deflection of a free-head drilled shaft can then be estimated using nonlinear superposition as follows (Evans and Duncan, 1982):

1. Using Y_{sM} and Figure 5.14 for shafts in sand, or Figure 5.15 for shafts in clay, calculate P_M as shown in Figure 5.18(b). P_M is the equivalent lateral load that would cause the deflection Y_{sM}.
2. Using Y_{sP} and Figure 5.16 for shafts in sand and Figure 5.17 for shafts in clay, calculate M_P as shown in Figure 5.18(e). M_P is the equivalent moment that would cause the deflection Y_{sP}.

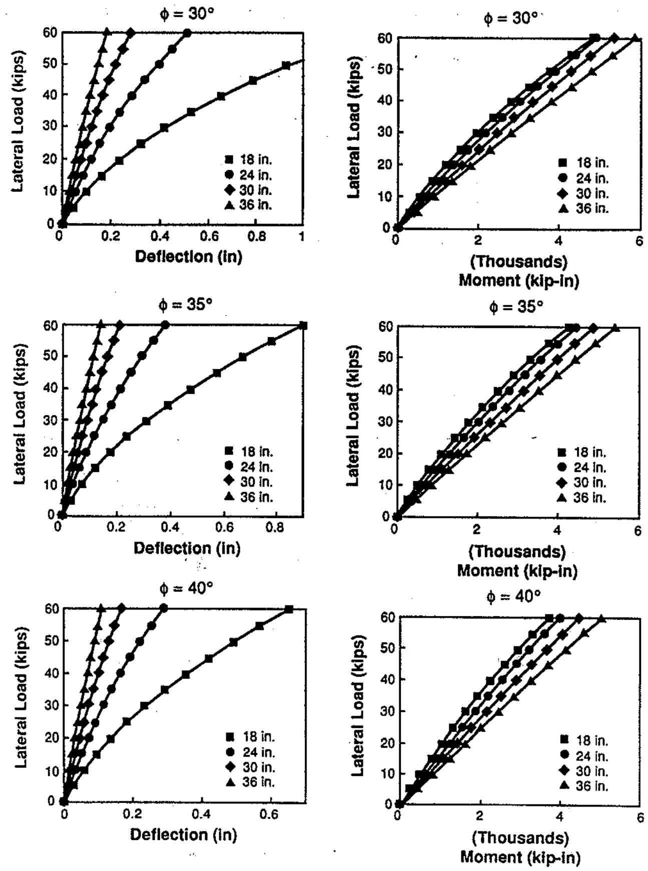


Figure 5.6. Load versus deflection and load versus moment for drilled shafts ($A_s/A_g = 2\%$) in sand.

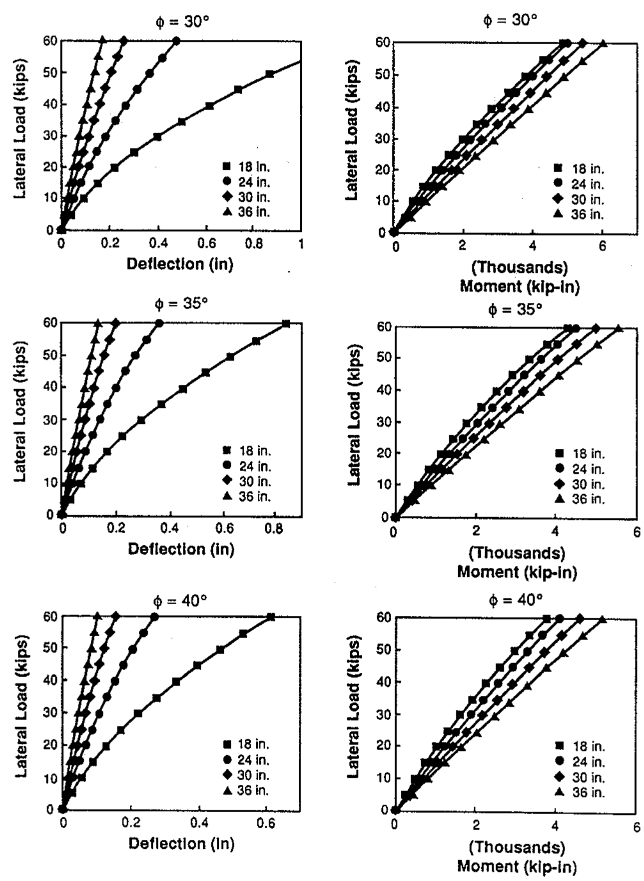


Figure 5.7. Load versus deflection and load versus moment for drilled shafts ($A_s/A_g = 4\%$) in sand.

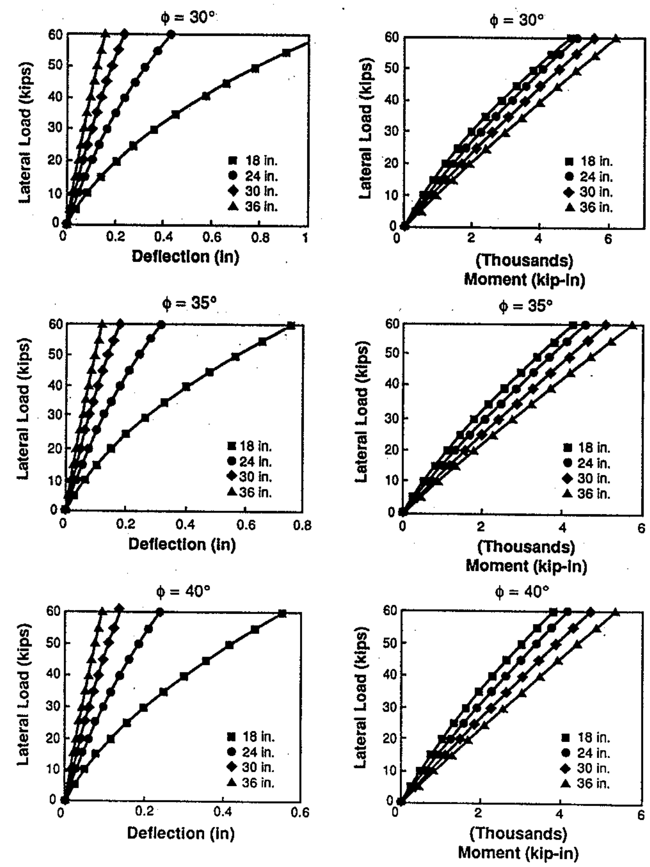


Figure 5.8. Load versus deflection and load versus moment for drilled shafts ($A_s/A_g = 8\%$) in sand.

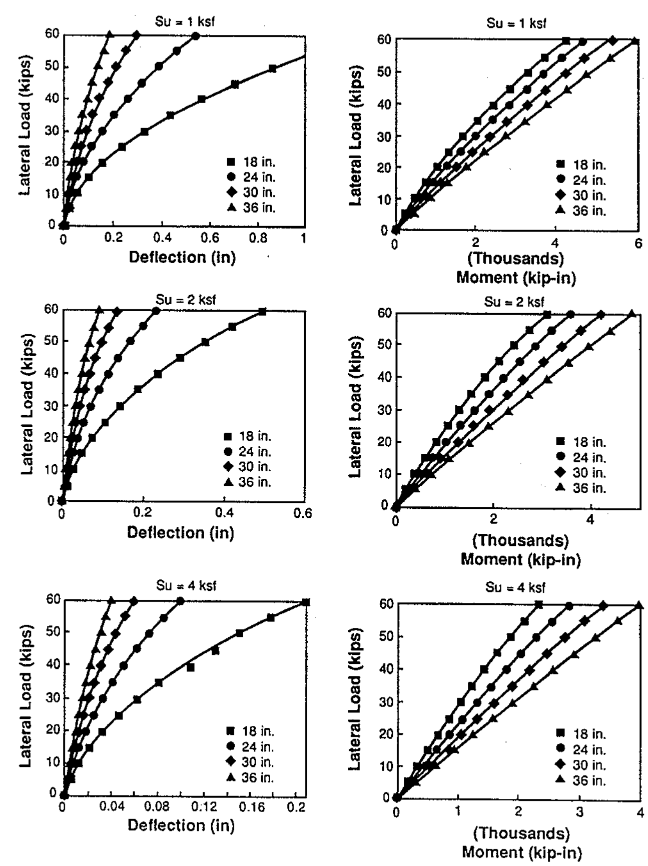


Figure 5.9. Load versus deflection and load versus moment for drilled shafts ($A_s/A_g = 1\%$) in clay.

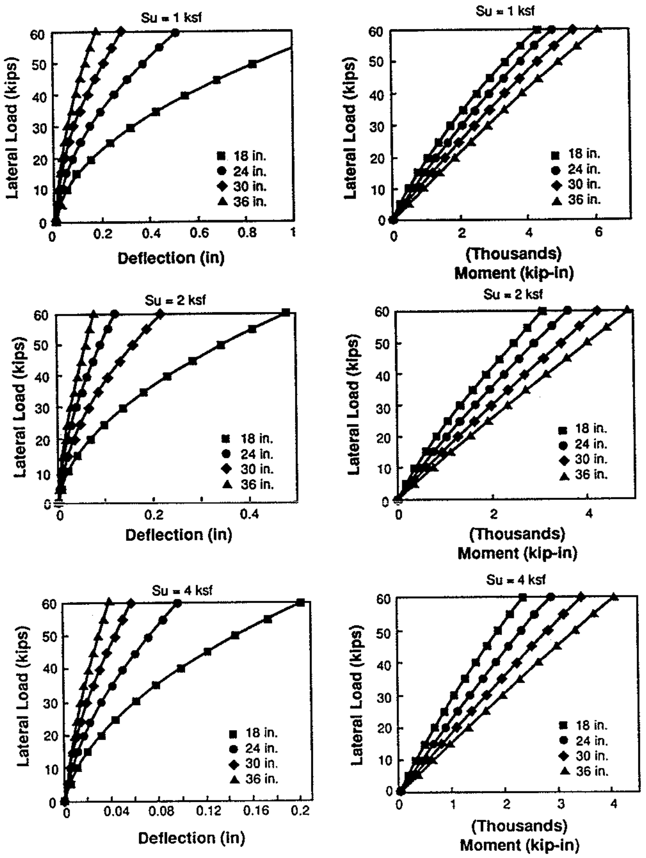


Figure 5.10. Load versus deflection and load versus moment for drilled shafts ($A_s/A_g = 2\%$) in clay.

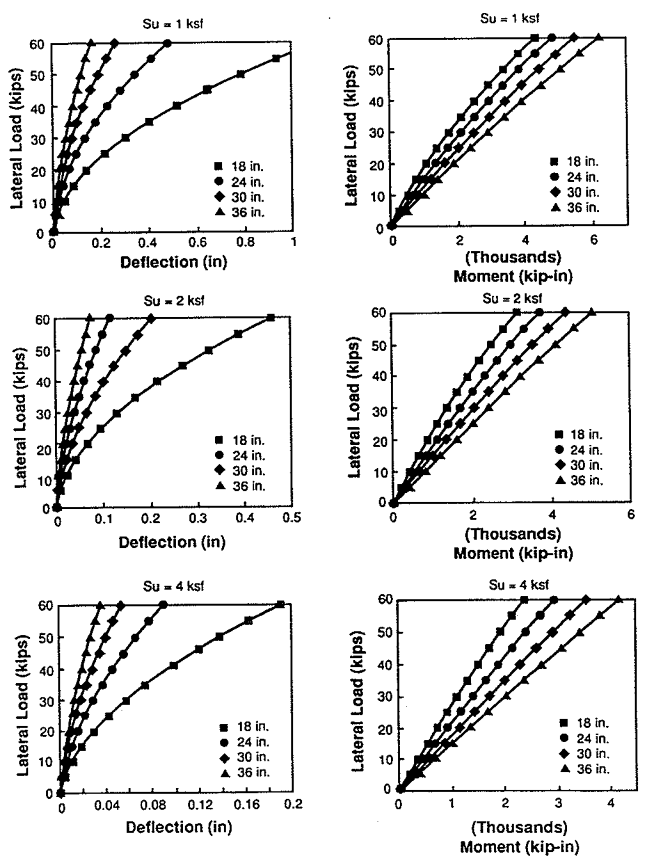


Figure 5.11. Load versus deflection and load versus moment for drilled shafts ($A_s/A_g = 4\%$) in clay.

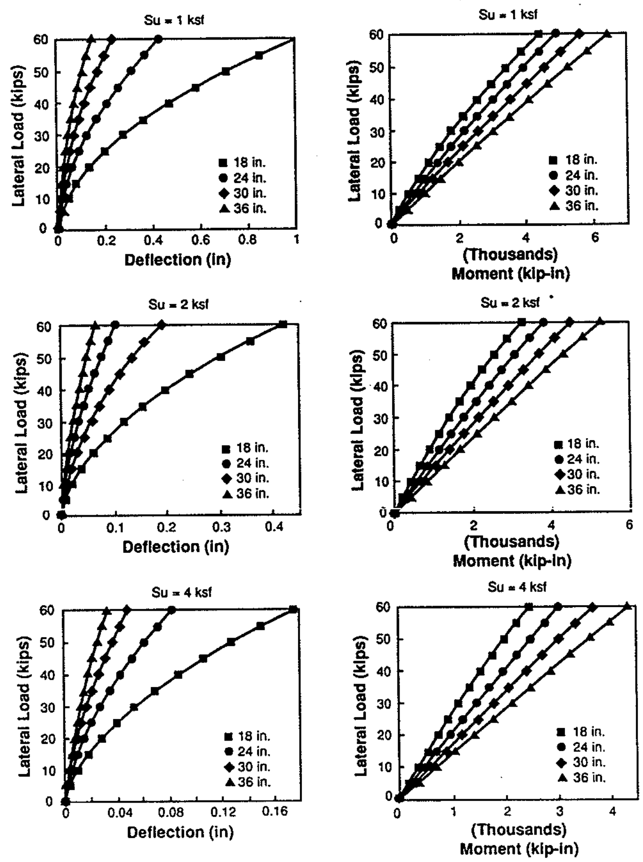


Figure 5.12. Load versus deflection and load versus moment for drilled shafts ($A_s/A_g = 8\%$) in clay.

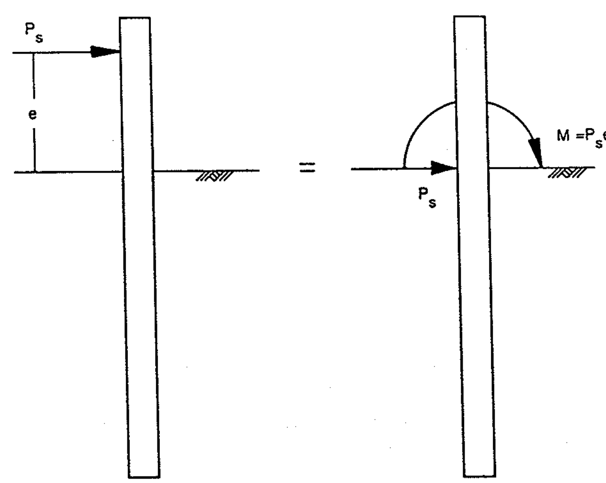


Figure 5.13. Resolution of eccentric load into a lateral load acting on the groundline and a moment.

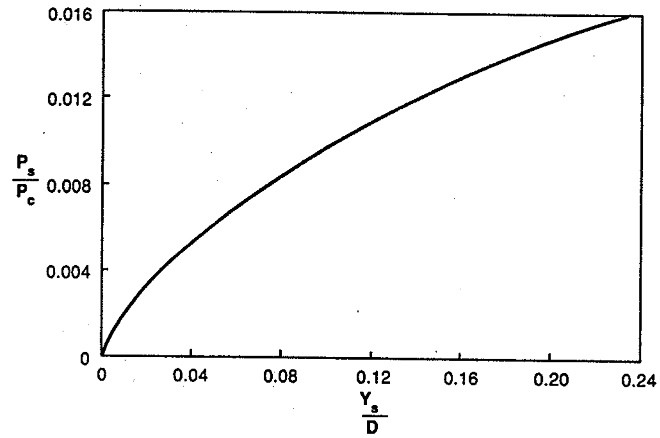


Figure 5.14. Load deformation curves for free-head drilled shafts in sand. (After Evans and Duncan, 1982)

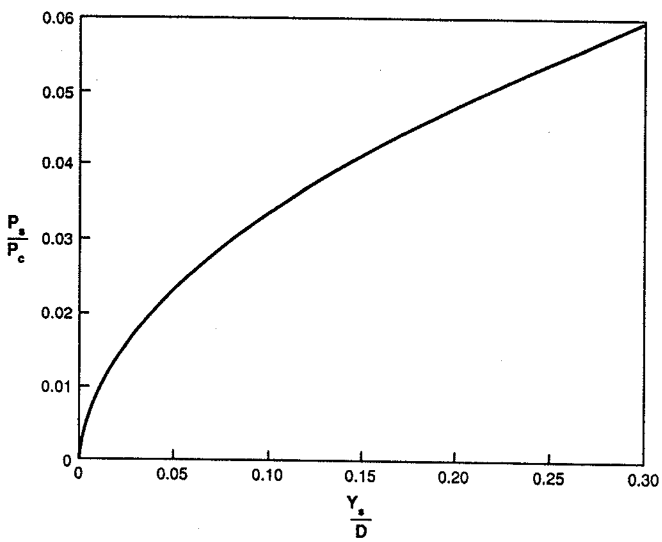


Figure 5.15. Load deformation curves for free-head drilled shafts in clay. (After Evans and Duncan, 1982)

3. Determine the deflection Y_{sPM} caused by the lateral load $(P_s + P_M)$ as shown in Figure 5.18(c). Y_{sPM} is the deflection caused by the sum of the real load plus the equivalent load.

4. Determine the deflection Y_{sMP} caused by the moment $(M_s + M_p)$ as shown in Figure 5.18(f). Y_{sMP} is the deflection caused by the sum of the real moment plus the equivalent moment.

5. Estimate the total deflection, Y_s, using the equation $Y_s = 0.5(Y_{sPM} + Y_{sMP})$.

5.1.2 Deflection of Groups of Drilled Shafts

A group of drilled shafts will deflect more than a single drilled shaft subjected to the same lateral load per shaft. This is due to interaction effects whereby deflection of each drilled shaft in a group causes deflection of the surrounding soil and, thereby, increases the deflections of neighboring shafts. However, where a single row of shafts is constructed side by side and loading is normal to a line containing the shaft heads, group action need not be considered unless the shafts are closer than three diameters center-to-center. Group action must be considered when the lateral loads act in line with the single row of shafts.

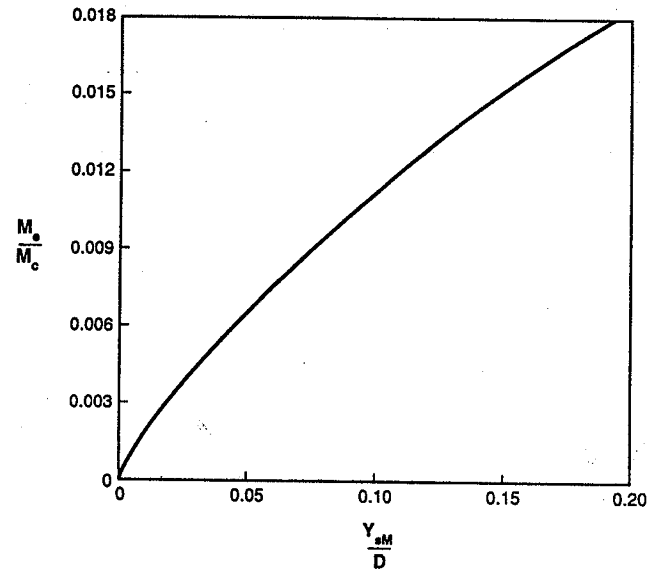


Figure 5.16. Moment deformation curves for free-head drilled shafts in sand. (After Evans and Duncan, 1982)

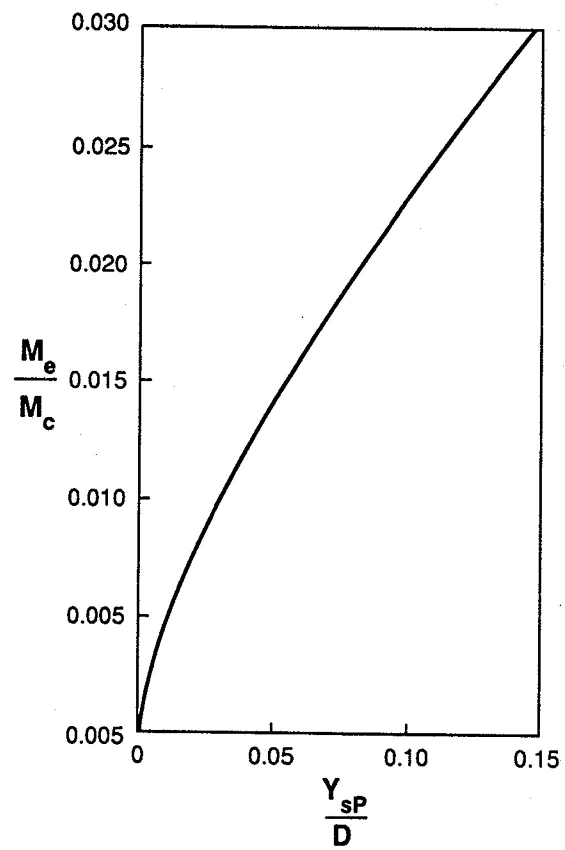


Figure 5.17. Moment deformation curves for free-head drilled shafts in clay. (After Evans and Duncan, 1982)

5.1.2.1 Groups of Fixed-Head Drilled Shafts

The lateral deflection, Y_g, of a group of fixed-head drilled

shafts may be estimated using the following semiempirical equation:

$$Y_g = \frac{A + N_{ds}}{B\sqrt{\dfrac{S}{D} + \dfrac{P_s}{CP_N}}} Y_s \qquad (5.1.2.1)$$

where Y_s = lateral displacement of a single fixed-head drilled shaft subjected to a lateral load P_s; N_{ds} = number of drilled shafts in group; S = average spacing of drilled shafts; D = diameter of drilled shaft; P_s = average lateral load per drilled shaft = P_{Yg}/N_{ds}; P_{Yg} = lateral load on the group of drilled shafts; and

$$P_N = K_p \gamma D^3 \textit{ for sand} \qquad (5.1.2.2)$$

$$P_N = S_u D^2 \quad \textit{for clay} \qquad 5.1.2.3)$$

where γ = *total* unit weight of sand; K_p = passive earth pressure coefficient = $\tan^2 (45° + \phi'/2)$; ϕ' = average angle of internal friction of sand within the upper 8 shaft diameters; S_u = average undrained shear strength of clay within the upper 8 shaft diameters; A = 16 for clay, = 9 for sand; B = 5.5 for clay, = 3 for sand; and C = 3 for clay, = 16 for sand.

The foregoing equation was developed through a parametric study of a large number of groups of deep foundations using the theory proposed by Focht and Koch (1973). It was developed for uniformly spaced drilled shafts, but can be used for groups with nonuniform spacing if the average shaft spacing is used in the calculations.

A computer program for calculating the lateral displacement of groups of drilled shafts using the theory of Focht and Koch has been developed by the writers and was used to perform the parametric study.

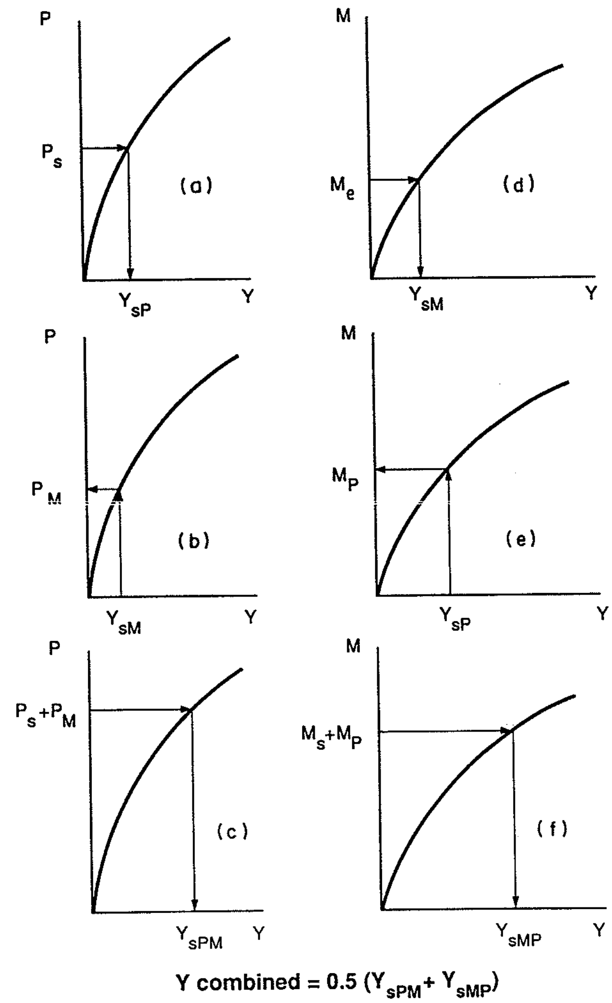


Figure 5.18. Nonlinear superposition. (After Evans and Duncan, 1982)

5.1.2.2 Groups of Free-Head Drilled Shafts

As with single free-head drilled shafts, a load acting above the groundline on a group of free-head drilled shafts will give rise to two components of lateral deflection: (1) the component due to a groundline lateral load, and (2) the component due to the moment effect of the eccentric load. The lateral deflection of a group of free-head drilled shafts may be estimated using the Focht and Koch (1973) procedure.

If the lateral displacement of a group of drilled shafts, Y_g, is greater than the tolerable value, the diameter of the shafts, the number of shafts, or the spacing of the shafts should be increased until Y_g is less than the tolerable value.

5.2 BENDING MOMENTS

5.2.1 Estimation of Bending Moment in a Single Drilled Shaft

5.2.1.1 Fixed-Head Drilled Shafts

Evans and Duncan (1982) developed a simple procedure for estimating the maximum bending moment induced in single drilled shafts, M_s, due to a lateral load at the top of the shaft. They developed the design charts shown in Figures 5.19 and 5.20 for fixed-head shafts in sand and clay. These charts show

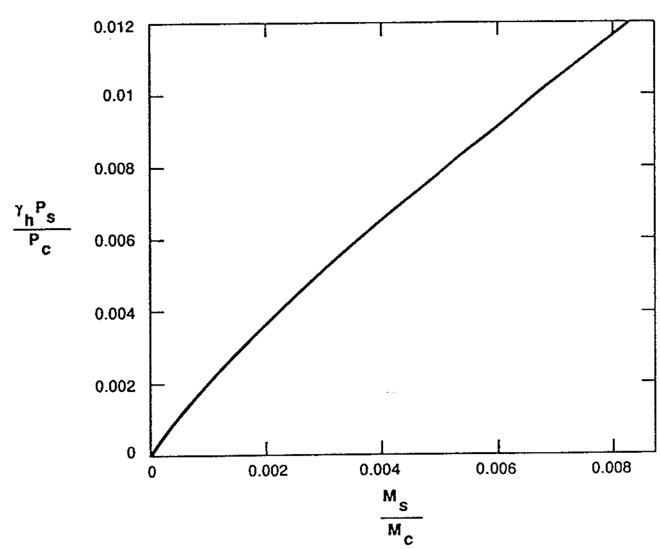


Figure 5.19. Lateral load versus moment for fixed-head drilled shafts in sand. (After Evans and Duncan, 1982)

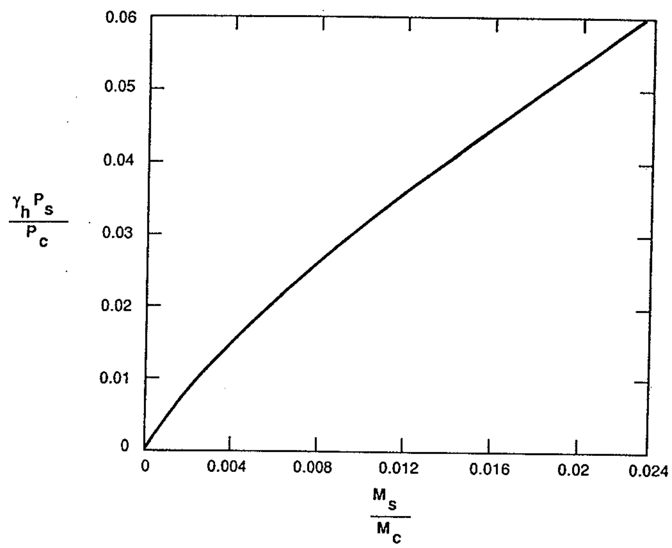


Figure 5.20. Lateral load versus moment for fixed-head drilled shafts in clay. (After Evans and Duncan, 1982)

the variation of M_s/M_c with P_s/P_c, where M_s = maximum moment in a single drilled shaft and M_c = characteristic moment.

Using these charts, the bending moment in a laterally loaded drilled shaft can be estimated as follows:

1. Select a drilled shaft section of diameter D, Young's modulus E_p, and moment of inertia I_p.
2. Estimate the average undrained shear strength, S_u, for clays, or the average angle of internal friction, ϕ', for sands. The behavior is governed by the soil close to the ground surface. The properties (S_u for clays, ϕ' and unit weight, γ', for sands) should be averaged over a depth extending about eight shaft diameters below the elevation of the top of the drilled shaft. Buoyant unit weights are used below the water table.
3. Determine the characteristic load, P_c, using Eq. 5.1.1.1 for clay or Eq. 5.1.1.2 for sand.
4. Calculate the factored lateral load, $\gamma_h P_s$, and the value of the load ratio, $\gamma_h P_s/P_c$; γ_h is the load factor for lateral loads.
5. Use Figure 5.19 for fixed-head shafts in sand and Figure 5.20 for fixed-head shafts in clay to determine the value of M_s/M_c.
6. Determine the characteristic moment, M_c, which is defined by the following equations:

For clay

$$M_c = 3.86\, D^3\, (E_p R_I)(S_u/E_p R_I)^{0.46} \tag{5.2.1.1}$$

For sand

$$M_c = 1.33\, D^3\, (E_p R_I)(\gamma' D \phi' K_p/E_p R_I)^{0.4} \tag{5.2.1.2}$$

where R_I, K_p, and ϕ' are as defined previously.

7. Calculate $M_s = M_c(M_s/M_c)$.

This procedure has been used to develop lateral load-moment curves for some commonly used fixed-head drilled shaft sections. Charts for drilled shafts of 18-in., 24-in., 30-in., and 36-in. diameters, with percentages of steel reinforcement equal to 1 percent, 2 percent, 4 percent, and 8 percent, constructed in sand and clay are shown on the right-hand sides of Figures 5.5 through 5.12. For these drilled shafts and soil conditions, bending moments can be estimated directly using the charts. For example, a lateral load of 50 kip acting on a 36-in. diameter drilled shaft with 1 percent steel reinforcement, constructed in sand with a friction angle of 40 deg, will induce a maximum bending moment of 4200 kip-in. (Figure 5.5).

5.2.1.2 Free-Head Drilled Shafts

The maximum moment in a free-head drilled shaft occurs at some depth below ground. The maximum moment is due in part to the horizontal load at ground level and in part to the moment at the ground level. The magnitude of the maximum moment is needed for design, and in some cases it may be necessary also to know the depth below ground at which the maximum moment occurs. These quantities cannot be determined directly using the procedure developed by Evans and Duncan, but they can be calculated using the theory described by Matlock and Reese (1961), based on the value of groundline deflection calculated using the Evans and Duncan procedure. The technique for estimating the groundline deflection is explained in Section 5.1.1. When the groundline deflection, Y_s, has been determined, the magnitude of the maximum moment and its depth below ground are estimated as follows:

1. Calculate the lateral deflection of the free-head drilled shaft, Y_s, using the procedure described in Section 5.1.1. If the lateral load acts eccentrically above the groundline, the nonlinear superposition procedure should be used to calculate Y_s.
2. Calculate the characteristic length, T, of the drilled shaft by solving the following equation for T:

$$Y_s = \frac{2.435 P_s}{E_p I_p} T^3 + \frac{1.623 M_e}{E_p I_p} T^2 \tag{5.2.1.3}$$

where Y_s, E_p, I_p, and P_s are as defined previously and T is the characteristic length.

3. Calculate the maximum bending moment in the drilled shaft using the following expression:

$$M = k_M M_e \tag{5.2.1.4}$$

where M_e is the bending moment at the groundline ($M_e = P_s e$), P_s is the lateral load, e is the eccentricity of the lateral load above the groundline, k_M is a moment multiplier which is a function of T/e. The value of k_M can be calculated as follows:

$$k_M = 1 + 0.756(T/e) \tag{5.2.1.5}$$

where T is the characteristic length defined in Eq. 5.2.1.3. The approximate location of the maximum bending moment can be estimated using Table 5.2.

5.2.2 Estimation of Maximum Bending Moments in Groups of Drilled Shafts

As discussed previously, the deflection of any drilled shaft in a group causes deflection of the surrounding soil and drilled

shafts, thus leading to larger deflection for the group than for single drilled shafts subjected to the same load per shaft. The bending moment in a drilled shaft within a group is also larger than that in a single drilled shaft subjected to the same loading. This is because the interaction effects, by causing more deflection, also increase the bending moment in the shafts.

5.2.2.1 Groups of Fixed-Head Drilled Shafts

Brown et al. (1987 and 1988) found that the maximum bending moment in a group of free-head piles occurs in the leading row (or front row) of piles. However, current theories on lateral loading of groups of piles or drilled shafts are not able to predict this behavior. A semiempirical procedure that provides a reasonable approximation of the maximum bending moment in the leading row of a group of piles or drilled shafts has been developed using the theory described by Focht and Koch (1973) and has been confirmed by comparison with field load tests. The increase in moment due to group interaction was studied for a large number of cases by first estimating the group deflection using the theory of Focht and Koch (1973), and then "softening" the soil (reducing S_u for clays or ϕ' for sands) until the single drilled shaft deflection (calculated using the Evans and Duncan approach) matched the lateral deflection of the group. Through this study, the following empirical equation was developed. The equation relates the maximum bending moment of the most severely loaded drilled shaft in the group to the maximum bending moment in a single drilled shaft:

$$M_g = [Y_g/Y_s]^n\, M_s \qquad (5.2.2.1)$$

where M_s = maximum bending moment in a single fixed-head drilled shaft subjected to a lateral load P_s, calculated using the procedure of Section 5.2.1; M_g = maximum bending moment in a drilled shaft within a group; Y_s = lateral deflection of a single fixed-head drilled shaft subjected to a lateral load, P_s (the value of Y_s is estimated using the procedure discussed in Section 5.1.1); Y_g = lateral group deflection estimated using Eq. 5.1.2.1; and

$$n = \frac{\gamma_h P_s}{150 P_N} + 0.25 \text{ for clay} \qquad (5.2.2.2)$$

$$n = \frac{\gamma_h P_s}{300 P_N} + 0.3 \text{ for sand} \qquad (5.2.2.3)$$

P_N is as defined previously in Eqs. 5.1.2.2 and 5.1.2.3, and γ_h is the load factor for the lateral load.

5.2.2.2 Groups of Free-Head Drilled Shafts

When subjected to lateral loads, the maximum bending moment in a group of free-head drilled shafts will also occur in the leading row of drilled shafts. The procedure for softening the soil and matching the deflection of a single free-head shaft with that of a group of shafts calculated using Focht and Koch's (1973) procedure can be used to estimate the maximum bending moment in the leading row of shafts. If the lateral load on the group of shafts acts eccentrically above the groundline, the analysis must consider the deflection components due to both the moment and the lateral load acting at the groundline.

Table 5.2. Approximate location of the occurrence of the maximum bending moment in free-head drilled shafts.

T/e	z/T
0.0	0.0
0.1	0.4
0.2	0.5
0.3	0.6
0.4	0.7
0.5	0.8
0.8	0.9
1.6	1.0
3.0	1.2
14.0	1.4

5.2.3 Structural Capacity of Drilled Shafts Subjected to Axial Loads and Bending

The structural capacity of a drilled shaft is dependent on both moment and axial load. An axial load-moment interaction diagram is an envelope of the combinations of moment and axial load that would cause failure in the drilled shaft.

Normalized load-moment interaction diagrams for drilled shafts are shown in Figures 5.21 and 5.22 for $f_y/f_c' = 10$ and $f_y/f_c' = 15$ respectively. The factored axial load, $\Sigma\gamma_i P_i$, has been normalized by dividing by the factored nominal axial capacity, $\phi_a P_n$. Similarly, the factored bending moment, $\gamma_m M$, has been normalized by dividing by the factored nominal moment capacity $\phi_m M_n$. The γ-factors account for uncertainties in the loads and moments, and the ϕ-factors account for uncertainties in structural capacity. The factored structural capacity of drilled shafts can be estimated as follows:

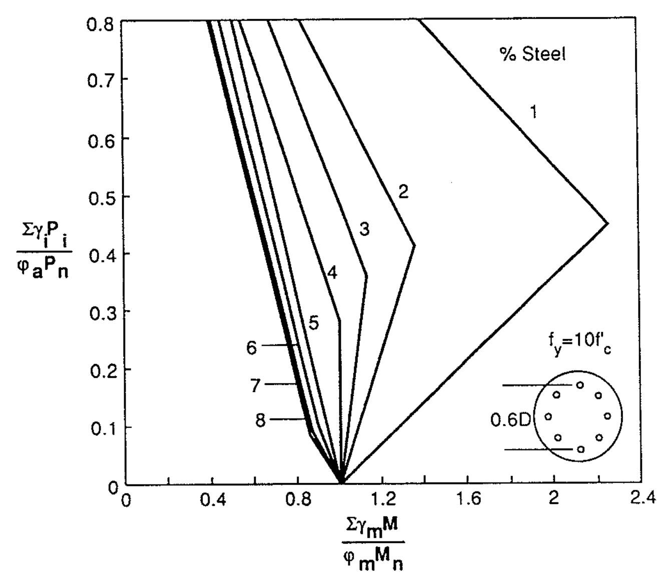


Figure 5.21. Normalized load-moment interaction curves for drilled shafts, $f_y = 10f_c'$.

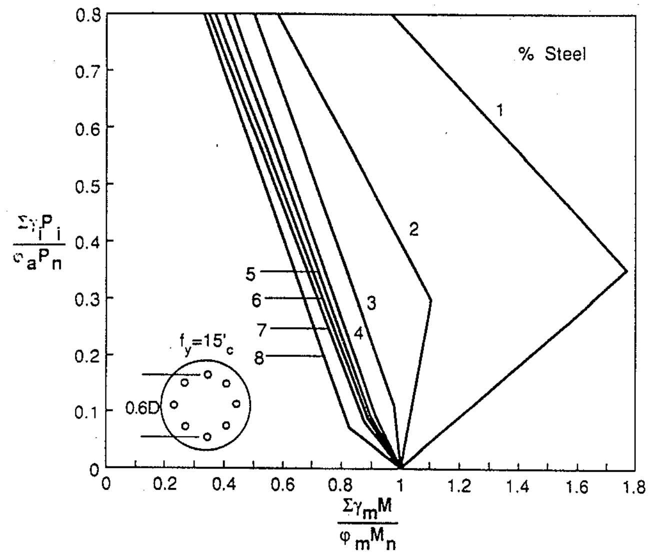
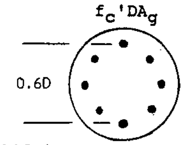


Figure 5.22. Normalized load-moment interaction curves for drilled shafts, $f_y = 15f_c'$.

Table 5.3. Nominal moment capacity, M_n, for drilled shafts.



RATIO OF AREA OF STEEL TO GROSS CROSS-SECTIONAL AREA	$\frac{M_n}{f_c' D A_g}$ $f_y = 10f_c'$	$f_y = 15f_c'$
0.01	0.037	0.050
0.02	0.067	0.092
0.03	0.088	0.119
0.04	0.107	0.147
0.05	0.126	0.172
0.06	0.144	0.197
0.07	0.161	0.208
0.08	0.176	0.244

f_c' = 28 day concrete cylinder strength
f_y = yield stress of steel
D = diameter of drilled shaft
A_g = gross cross-sectional area of concrete

$$\phi_a P_n = \phi_a(0.85f_c'A_c + f_y A_y) \quad (5.2.3.1)$$

where $\phi_a P_n$ = factored nominal axial capacity of the drilled shaft; ϕ_a = capacity reduction factor, 0.75 for spiral columns and 0.70 for tied columns; f_c' = 28-day concrete cylinder strength; f_y = yield stress of steel; A_c = cross-sectional area of concrete; and A_y = cross-sectional area of steel.

Note that it is possible to develop load-moment interaction charts similar to those of Figures 5.21 and 5.22 for values of f_y/f_c' other than 10 or 15, and cage diameters other than 0.6D.

The procedure for checking the structural adequacy of drilled shafts using the normalized load-moment interaction curve is as follows:

1. Estimate the axial load per drilled shaft and calculate the combined axial load effect, $\Sigma\gamma_i P_i$.
2. Determine the factored nominal axial structural capacity of the drilled shaft, $\phi_a P_n$ using Eq. 5.2.3.1.
3. Calculate the factored design bending moment, $\gamma_m M$, in the drilled shaft using factored loads.
4. Estimate the nominal structural moment capacity of the pile, M_n from Table 5.3.
5. Calculate $\phi_m M_n$ where ϕ_m = 0.9 for reinforced concrete.
6. Determine the ratios $\Sigma\gamma_i P_i/\phi_a P_n$ and $\Sigma\gamma_m M/\phi_m M_n$, and locate a point at these coordinate values on the normalized load-moment interaction diagram. If the point falls on or close to the interaction curve and inside the area enveloped by the interaction curve and the two axes, the drilled shaft chosen is adequate. If it falls outside this region, a larger section is needed. Steps 2 through 7 should be repeated until the point falls inside and close to the interaction curve. If the point falls inside the region but far away from the interaction curve (e.g., near the origin), the drilled shaft chosen has more capacity than required. Steps 2 to 7 can be repeated for smaller drilled shaft sections to achieve greater design economy.

5.3 DESIGN EXAMPLES

The design procedures discussed earlier in this chapter are demonstrated in the following example problems.

Example 5.1

Using the charts developed by Evans and Duncan (1982), determine the lateral deflection of a 24 in. diameter free-head drilled shaft subjected to a lateral stream flow force of 30 kips acting at a distance of 8 ft above the groundline. The soil is a silty clay with an average undrained shear strength of 2 ksf along the top 16 ft. The drilled shaft is reinforced with 8 # 7 bars. The Young's modulus of the concrete is 3.5 x 10^3 ksi. Check the structural adequacy of the section.

1. Calculate the percentage of steel

$$A_y = [\frac{\pi}{4} (\frac{7}{8})^2] = 4.81 \text{ in.}^2$$

$$A_g = \frac{\pi}{4} (24)^2 = 452 \text{ in.}^2$$

$$\% \text{ steel} = \frac{4.81}{452} \times 100 = 1.06\%$$

2. Estimate the lateral deflection assuming the load acts along the groundline

From Table 5.1, $R_I = 1.09 \therefore E_p R_I = 3.5 \times 10^6 \times 1.09 = 3815$ ksi

Using Eq. 5.1.1-1 and assuming negligible scour

$$P_c = 7.34D^2 (E_p R_I) (S_u/E_p R_I)^{0.683}$$

$$= 7.34 (24)^2 (3815) (\frac{2}{144} \frac{1}{3815})^{0.683}$$

$$= 3111 \text{ kips}$$

$$\frac{P_s}{P_c} = \frac{30}{3111} = 0.0096$$

From Fig. 5.15, $Y_{sp}/D = 0.011$

$$Y_S = (0.011)(24) = 0.26 \text{ in.}$$

3. Estimate the lateral deflection due to the moment component of the load.

$$M_e = P_S e = (30)(8) = 240 \text{ kip-ft} = 2880 \text{ kip-in.}$$

Using Equation 5.1.1-3

$$M_C = 3.86 D^3 (E_p R_I) (S_u/E_p R_I)^{0.46}$$

$$= (3.86)(24)^3 (3815) (\frac{2}{144} \frac{1}{3815})^{0.46}$$

$$= 641\ 000 \text{ kip-in.}$$

$$\therefore \frac{M_e}{M_c} = 0.0045$$

From Fig. 5.17,

$$\frac{Y_{SM}}{D} = 0.01$$

$$Y_{SM} = (0.01)(24)$$

$$= 0.24 \text{ in.}$$

4. Apply on-linear superposition to calculate lateral deflection of drilled shaft

From Fig. 5.15,

$$\frac{P_M}{P_C} = 0.0095 \text{ for } \frac{Y_{SM}}{D} = 0.01$$

$$P_M = (0.0095)(3111) = 29.6 \text{ kips.}$$

From Fig. 5.17,

$$\frac{M_P}{M_C} = 0.0048 \quad \text{for } \frac{Y_{SP}}{D} = 0.011$$

$$M_P = (0.0048)(641000) = 3077 \text{ kip-in.}$$

$$(P_S + P_M) = (30 + 29.6) = 59.6 \text{ kips}$$

$$\frac{P_S + P_M}{P_C} = \frac{59.6}{3111} = 0.019$$

From Fig. 5.15,

$$\frac{Y_{SPM}}{D} = 0.035$$

$$Y_{SPM} = (0.035)\ (24) = 0.84 \text{ in.}$$

$$(M_e + M_p) = (2880 + 3077) = 5957 \text{ kip-in.}$$

$$\frac{M_e + M_p}{M_c} = \frac{5957}{641000} = 0.0093$$

From Fig. 5.17,

$$\frac{Y_{SMP}}{D} = 0.027$$

$$Y_{SMP} = (0.027)\ (24) = 0.65 \text{ in.}$$

$$\therefore\ Y_S = \frac{1}{2}(Y_{SPM} + Y_{SMP})$$

$$= \frac{1}{2}(0.84 + 0.65)$$

$$= 0.75 \text{ in.}$$

5. Calculate maximum bending moment in the drilled shaft

$$I_p = \frac{\pi D^4}{64} R_I = \frac{\pi (24)^4}{64} 1.09$$

$$= 17752 \text{ in.}^4$$

Using Equation 5.2.1-3 and applying a load factor $\gamma_{SF} = 1.3$ to the lateral stream force and moment component of the stream force, according to AASHTO's load combination I,

$$\frac{(2.435)\ (1.3)\ (30)}{(3.5 \times 10^3)\ (17752)} T^3 + \frac{(1.623)\ (1.3)\ (2880)}{(3.5 \times 10^3)\ (17752)} T^2 = 0.75$$

$$\frac{T^3}{654300} + \frac{T^2}{10220} - 0.75 = 0$$

By trial and error, $T \approx 62$ in.

$$T/e = \frac{62}{(8)\ (12)} = 0.646$$

$\therefore$ From Equation 5.2.1-5,

$$k_M = 0.756\ (0.646) + 1$$

$$= 1.49$$

$\therefore$ From Equation 5.2.1-4, the maximum bending moment in the drilled shaft

$$= k_M M_e$$

$$= (1.49)\ (2880)$$

$$= 4290 \text{ kip-in.}$$

6. Calculate moment capacity of drilled shaft.

Assuming $f'_c = 4$ ksi and $f_y = 60$ ksi, $f_y/f'_c = 15$

From Table 5.3,

$$\frac{M_n}{f'_c\, DA_g} \approx 0.053 \text{ for 1.06\% stee}$$

$$M_n = (0.053)\ (4)\ (24)\ (452)$$

$$= 2300 \text{ kip-in.}$$

$$\phi_m M_n = (0.9)\ (2300)$$

$$= 2070 \text{ kip-in.}$$

$$\therefore\ \frac{\gamma_m M}{\phi_m M_n} = \frac{4290}{2070} \qquad \text{taking } \gamma_m = 1$$

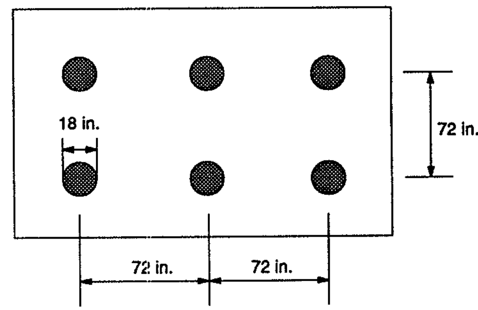
$$= 2.07$$

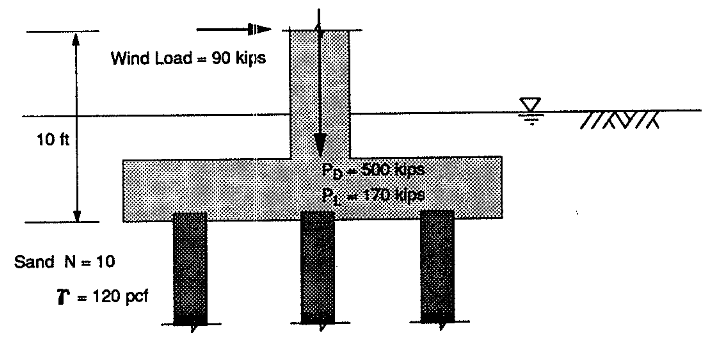
This point will not plot inside the load-moment interaction diagram (Fig. 5.22) for drilled shafts with 1% reinforcement. Thus either increase the % of steel or the diameter of the drilled shaft or both.**

** Note: Buckling of the drilled shaft should also be checked in accordance to Section 4.1.2 for free-head shafts with portions of the shaft unsupported laterally above ground.

Example 5.2

Estimate the lateral deflection of the group of fixed-head drilled shafts shown below and check the structural adequacy of the shafts when subjected to a lateral wind load of 90 kips. acting at a distance of 10 ft above the top of the shafts. The drilled shaft is reinforced with 8 # 7 bars.





(1) Calculate the percentage of steel

$$A_y = 8\left[\frac{\pi}{4}\left(\frac{7}{8}\right)^2\right] = 4.81 \text{ in.}^2$$

$$A_g = \frac{\pi}{4}(18)^2 = 254 \text{ in.}^2$$

$$\% \text{ steel} = \frac{4.81}{254} \times 100 = 1.89\%$$

(2) Estimate the lateral deflection of a single drilled shaft

Load per drilled shaft = $\frac{90}{6}$ = 15 kips.

An SPT blow count of 10 corresponds to a ϕ' of 35° (based on correlation by Meyerhof (1956))

Using Fig. 5.5, $\phi' = 35°$, D = 18", $A_y/A_g = 1\%$, $Y_S = 0.13$ in.

Using Fig. 5.6, $\phi' = 35°$, D = 18", $A_y/A_g = 2\%$, $Y_S = 0.12$ in.

(3) Estimate the group deflection

Using Equation 5.1.2-1

$$Y_g = \frac{9 + N_{ds}}{3\sqrt{\frac{S}{D} + \frac{P_S}{16P_N}}} Y_S$$

$$P_N = K_p \gamma D^3$$

$$= \frac{1 + \sin 35°}{1 - \sin 35°} \; \frac{120}{(1000)\,(12)^3} (18)^3$$

$$= 1.49$$

$$\therefore Y_g = \frac{9 + 6}{3\sqrt{4 + \frac{15}{(16)\,(1.49)}}} \; 0.12$$

$$= (2.32)\,(0.12)$$

$$= 0.28 \text{ in.}$$

(4) Calculate location of resultant force acting on cap.

$$\Sigma \gamma_i P_i = (1.3)\,(500) + (2.17)\,(170)$$

$$= 650 + 369$$

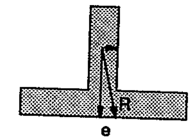
$$= 1019 \text{ kips.}$$

$$\gamma_w P_w = (1.3)\,(90)$$

$$= 117 \text{ kips}$$

Eccentricity of R, e = (117) (10)/1019

= 1.15 ft.



(5) Calculate axial load in most heavily loaded shaft.

The most heavily loaded shaft supports a factored axial load of

$$P_{x,y} = P_g \left[\frac{1}{N_{ds}} + \frac{e_x x}{\Sigma x^2} + \frac{e_y y}{\Sigma y^2}\right] \qquad \text{Eq. 4.1-4}$$

$$= (1019)\left[\frac{1}{6} + \frac{(1.15)\,(6)}{4\,(6)^2} + 0\right]$$

= 219 kips/shaft

(6) Estimate the maximum bending moment in a single drilled shaft

$P_S = 15$ kips, $\phi' = 35°$

Using Fig. 5.5, M = 780 kip-in. for $A_y/A_g = 1\%$

Using Fig. 5.6, M = 780 kip-in. for $A_y/A_g = 2\%$

$\therefore M_S = 780$ kip-in.

(7) Estimate the maximum bending moment in the most heavily loaded shaft.

Using Equation 5.2.2-1

$Y_g/Y_s = 2.32$

$$n = \frac{\gamma_h P_S}{300 P_N} + 0.3$$

$$= \frac{(1.3)(15)}{(300)(1.49)} + 0.3$$

$$= 0.344$$

$$M_g = (2.32)^{0.34} M_S$$

$$= (1.33)(780) = 1037 \text{ kip-in.}$$

(8) Check structural adequacy using load-moment interaction diagram

Factored structural axial capacity excluding the eccentricity factor (Equation 4.1-3)

$$\phi_a P_n = 0.7\,[0.85\, f'_c A_c + f_y A_y]$$

$$= 0.7\,[(0.85)(4)(254 - 4.81) + (60)(4.81)]$$

$$= 795 \text{ kips/shaft}$$

Moment capacity for a drilled shaft (Table 5.3) with $f_y/f'_C = 15$

$$\frac{M_n}{f'_c D A g} = 0.0874$$

$$M_n = (0.0874)(4)(18)(254)$$

$$= 1598 \text{ kip-in.}$$

$$\frac{P_{x,y}}{\phi_a P_n} = \frac{219}{795} = 0.28$$

$$\frac{M_g}{\phi_m M_n} = \frac{1037}{(0.9)(1598)} = \frac{1037}{1438} = 0.72$$

The point (0.72, 0.28) plots inside the load moment interaction diagram of Fig. 5.22.
∴ The structural capacity is more than adequate. The size of the drilled shafts could be reduced to achieve a more economical design.

REFERENCES

1. Alizadeh M., and Davisson M.T., "Lateral Load Tests on Piles - Arkansas River Project", ASCE, JSMFED, Vol. 96, No. 9, 1970, pp. 1583-1604.

2. American Association of State Highway and Transportation Officials, "Standard Specifications for Highway Bridges", Fourteenth Edition, 1989.

3. Barker R.M., Duncan J.M. and Rojiani K.B., "Load Factor Design Criteria for Highway Structure Foundations", Interim Rept. for NCHRP, VPI&SU, Oct. 1988, 140 pp.

4. Bienawski Z.T., "Rock Mechanics Design in Mining and Tunneling, A.A. Balkema: Rotterdam/Boston, 1984, 272 pp.

5. Brown D.A., Reese L.C. and O'Neill M.W., "Cyclic Lateral Loading of a Large Scale Pile Group", ASCE, JGED, Vol. 113, Np. 11, November, 1987, pp. 1326-1343.

6. Brown D.A., Morrison C. and Reese L.C., "Lateral Load Behavior of Pile Group in Sand", ASCE, JGED, Vol. 114, No. 11, November, 1988, pp. 1261-1276.

7. Canadian Geotechnical Society, "Canadian Foundation Engineering Manual", 2nd Edition, Bitech Publishers Ltd., 1985, 460 pp.

8. Carter J.P. and Kulhawy F.H., "Analysis and Design of Foundations Socketed into Rock", EPRI Report No. EL-5918, New York, 1988, 158 pp.

9. Davisson M.T., and Robinson K.E., "Bending and Buckling of Partially Embedded Piles." Proc. 6th Int. Conf. Soil Mechanics and Foundation Engineering, Montreal, Canada, 1965, pp. 243-246.

10. Deere D.V., "Geological Considerations", Chapter 1 in Rock Mechanics in Engineering Practice by K.G. Stagg and O. C. Zienkiewicz, John Wiley and Sons, Inc., New York, 1968, pp 1-20.

11. Donald I.B., Sloan S.W. and Chiu H.K., "Theoretical Analysis of Rock Socketed Piles", Proc. Int. Conf. on Structural Foundations on Rock, Sydney, Balkema: Rotterdam, 1980.

12. Duncan J.M. and Buchignani A.L., "An Engineering Manual for Settlement Studies", Geotechnical Engineering Report, Univ. of Calif. Berkeley, 1976, 94 pp.

13. Evans Jr. L.T. and Duncan J.M., "Simplified Analysis of Laterally Loaded Piles", UC Berkeley Rept. No. UCB/GT/82-04, July, 1982, 245 pp.

14. Fellenius B.H., Samson L and Tavenas F., "Geotechnical Guidelines - Pile Design", Public Works Canada, Marine Works Sector, Ottawa Ontario K1A OM2, Canada, August 1989.

15. FHWA, "Interim Procedures for Evaluating Scour at Bridges", FHWA Publication to be incorporated in "Hydraulic Engineering Circular No. 18 (HEC-18), "Scour at Bridges" which is scheduled for Publication in 1989, Sept. 1988, 62 pp.

16. Focht J.A. and Koch K.J., "Rational Analysis of the Lateral Performance of Offshore Pile Groups", Proc. of the Fifth Offshore Technology Conference, Houston, Texas, Vol. 2, Paper OTC 1896, 1973, pp. 701-708.

17. Heins C.P. and Firmage D.A., "Design of Modern Steel Highway Bridges", John Wiley and Sons, New York, 1979, 463 pp.

18. Horvath R.G. and Kenney T.C., "Shaft Resistance of Rock Socketed Drilled Piers", Proc., Symposium on Deep Foundations, ASCE, Atlanta, Georgia, 1979, pp. 182-214.

19. Krohn J.P. and Slosson J.E., "Assessment of Expansive Soils in the United States", Proc. Fourth Int. Conf. on Expansive Soils, ASCE, Vol. 1, Denver, Colorado, June 1980, pp. 596-608.

20. Kulhawy F.H., Trautmann C.H., Beech J.F., O'Rourke T.D. and McGuire W, "Transmission Line Structure Foundations for Uplift-Compression Loading", EPRI Rept. EL-2870, Electric Power Research Institute, 1983.

21. Matlock H. and Reese L.C., "Foundation Analysis of Offshore Pile Supported Structures", Proc. Fifth Int. Conf. on Soil Mech. and Found. Eng., Vol. 2, 1961, pp. 91-97.

22. Meyerhof G.G., "Penetration Tests and Bearing Capacity of Cohesionless Soils", ASCE, JSMFED, Vol. 82, SM1, January 1956, pp. 866-1 to 866-19.

23. Meyerhof G.G., "Bearing Capacity and Settlement of Pile Foundations", ASCE JGED, Vol. 102, No. GT3, March 1976, pp 196-228.

24. Moulton L.K., GangaRao H.V.S. and Halverson G.T., "Tolerable Movement Criteria for Highway Bridges", FHWA/RD-85/107, Federal Highway Administration, Washington D.C., 1985, 118 pp.

25. Osterberg J.O., "A New Simplified Method for Load Testing Drilled Shafts", Foundation Drilling, Vol. XXIII, No. 6, pp. 9-11.

26. Peck R.B., Hanson W.E. and Thornburn T.H., "Foundation Engineering, Second Edition, John Wiley and Sons, Inc., New York, 1974.

27. Peck R.B., "Rock Foundations for Structures", Proc. ASCE Specialty Conf. on Rock Engineering for Foundations and Slopes, Boulder, Colorado, 1976.

28. Potyondy J.G., "Skin Friction between Various Soils and Construction Materials", Geotechnique, Vol. XI, No. 4, 1961, pp. 339-353.

29. Poulos H.G. and Davis E.H., "Pile Foundation Design and Analysis", John Wiley and Sons, 1980, 397 pp.

30. Prakash S. and Sharma H.D. "Pile Foundations in Engineering Practice", John Wiley & Sons, Inc., 1990, 734 pp.

31. Prestressed Concrete Institute, "PCI Design Handbook - Precast and Prestressed Concrete", 3rd Edition, Prestressed Concrete Institute, 20 North Wacker Dr., Chicago Il 60606, 1985.

32. Quiros G.W. and Reese L.C., "Design Procedures for Axially Loaded Drilled Shafts", Research Rept. 176-5F, Project 3-5-72-176, Center for Highway Research, Univ. of Texas, Austin, December 1977, 156 pp.

33. Reese L.C., Cox W.R., and Koop F.D., "Analysis of Laterally Loaded Piles in Sand", 6th Annual Offshore Technology Conf., Paper No. OTC 2080, 1974.

34. Reese L.C. and O'Neill M.W., "Drilled Shafts: Construction Procedures and Design Methods", FHWA Publication No. FHWA-HI-88-042 or ADSC Publication No. ADSC-TL-4, August 1988, 564 pp.

35. Reese L.C. and Wright S.J., "Drilled Shaft Manual-Construction Procedures and Design for Axial Loading", Vol. 1, U.S. Dept. of Transportation, Implementation Division, HDV-22, Implementation Package 77-21, July 1977, 140 pp.

36. Scott C.R., "An Introduction to Soil Mechanics and Foundations", Third Edition, Applied Science and Publishers Ltd., 1980, 406 pp.

37. Skempton A.W., "The Bearing Capacity of Clays", Proc. of the Building Research Congress, London, England, Vol. 1, 1951, pp. 180-189.

38. Stas C.V. and Kulhawy F.H., "Critical Evaluation of Design Methods for Foundations Under Axial Uplift and Compression Loading", EPRI Report No. EL-3771, November, 1984, 198 pp.

39. Stebbins E.E. and Williams R.C., "Wet-Hole Drilled Shaft Construction", Proc. Drilled Foundation Seminar, Auburn University, February, 1986.

40. Terzaghi K., "Evaluation of coefficient of subgrade reaction", Geotechnique, No. 5, 1955, pp. 297-326.

41. Tomlinson M.J., "Pile Design and Construction Practice", Viewpoint Publication, 1987, 415 pp.

42. Touma F.T. and Reese L.C., "Behavior of Bored Piles in Sand", ASCE JGED, Vol. 100, No. GT 7, July 1974, pp. 749-761.

43. Vesic A.S., "Bearing Capacity of Shallow Foundations", Chapter 3 in Foundation Engineering Handbook, Ed. by Winterkorn H. and Fang H.Y., Van Nostrand Reinhold Co., New York, 1975, pp. 121-147.

44. Yazdanbod A., Sheikh S.A. and O'Neill M.W., "Uplift of Shallow Underream in Jointed Clay", Proc. ASCE, Foundations for Transmission Line Towers, Ed. by Briaud J.L., Atlantic City, New Jersey, April 1987, pp. 110-127.

NOTATIONS AND SYMBOLS

Symbol	Represents
<u>ENGLISH</u>	
a	Factor for calculating the reduction factor for end bearing of drilled shafts with large bases
a_s	Perimeter of drilled shaft
A	Constant used in the equation for predicting lateral displacement of groups of drilled shafts
A_c	Cross-sectional area of concrete
A_g	Gross cross-sectional area of drilled shaft
A_p	Area of base of drilled shaft
A_s	Surface area of the sides of a drilled shaft over its penetration depth (in Chapter 4)
A_s	Area of steel (in Chapter 5)
A_{soc}	Cross-sectional area of drilled shaft socket
A_u	Annular area between the bell and the shaft
b	Factor for calculating the reduction factor for end bearing of drilled shafts with large bases
B	Constant used in the equation for predicting lateral displacement of groups of drilled shafts
c	Cover
c'	Effective cohesion
C	Constant used in the equation for predicting lateral displacement of groups of drilled shafts
COLE	Coefficient of linear extensibility
d	Dimensionless depth factor for estimating tip capacity of drilled shafts socketed in rock
d_s	Diameter of steel bars

Symbol	Represents
D	Diameter of Drilled Shaft
D'	Effective depth of group of drilled shafts
D_b	Depth of embedment of drilled shaft into a bearing stratum
D_n	Depth of drilled shaft embedded in settling soil and subject to downdrag loading
D_p	Diameter of the base of a drilled shaft
D_s	Diameter of drilled shaft socketed into rock
e	Eccentricity of lateral load above ground
e_x	Eccentricity of load on a group of drilled shafts in the x-direction
e_y	Eccentricity of load on a group of drillled shafts in the y-direction
E_c	Young's modulus of concrete
E_i	Intact rock modulus
E_r	Modulus of the in situ rock
E_s	Soil modulus
E_p	Young's modulus of drilled shaft
f_c'	28 day concrete cylinder strength
f_u	Tensile strength of steel
f_y	Yield stress of steel
F_r	Reduction factor for end bearing of drilled shafts with large bases
H	Distance between the tip of a drilled shaft and a weaker underlying soil layer
H_s	Depth of embedment of drilled shaft socketed into rock
I	Influence factor for the effective embedment of a group of drilled shafts
I_c	Moment of inertia of concrete
I_p	Moment of inertia of a drilled shaft
I_r	Rigidity index of soil
I_s	Moment of inertia of steel
I_{solid}	Moment of inertia of a solid circular cross-section
I_ρ	Influence coefficient for settlement of drilled shafts socketed into rock
k	Reduction factor for the tip capacity of drilled shafts (in sand) with a base diameter greater than 20 in. so as to limit the shaft settlement to 1 in.
k_M	Moment multiliplier
K	Coefficient of lateral earth pressure
K	Load transfer factor in Table 4.3
K_b	Dimensionless end bearing capacity coefficient
K_E	Modulus modification ratio related to the RQD
K_o	In situ horizontal soil stress coefficient
K_p	Rankine passive earth pressure coefficient
K_{sp}	Dimensionless bearing capacity coefficient
L_{eq}	Equivalent free standing length of a partially embedded drilled shaft
L_u	Unsupported length of a drilled shaft extending above ground in a partially embedded shaft
LI	Liquidity index
LL	Liquid limit
M_c	Characteristic moment

Symbol	Represents
M_e	Bending moment caused by lateral load acting above groundline
M_g	Maximum bending moment in a drilled shaft within a group of drilled shafts
M_n	Nominal structural moment capacity of a drilled shaft
M_p	Equivalent moment that would cause a lateral deflection equal to Y_{SP}
M_s	Maximum bending moment in a single fixed-head drilled shaft
n	Exponent used in estimating the maximum bending moment in a drilled shaft within a group of drilled shafts
n_h	Coefficient of horizontal subgrade reaction
N	SPT blow count
N_c	Bearing capacity factor
N_{corr}	Corrected SPT-N value
N_{ds}	Number of drilled shafts in a group
N_q'	Modified bearing capacity factor
N_u	Uplift bearing capacity factor
Pl	Limit pressure determined from pressuremeter tests
P_o	At rest horizontal stress measured from pressuremeter tests
P_c	Characteristic load
P_{cr}	Critical buckling load
P_D	Dead load
P_g	Factored total axial load acting on a pile group
P_i	Axial load due to load i
P_L	Live load
P_n	Nominal structural axial capacity of a drilled shaft
P_M	Equivalent lateral load that would cause a lateral displacement equal to Y_{SM}
P_N	Normalizing load for estimating lateral deflections and maximum bending moments in laterally loaded groups of drilled shafts
P_s	Lateral load per drilled shaft (unfactored)
P_{sn}	Downdrag load (unfactored)
$P_{x,y}$	Factored axial load acting on a drilled shfat in a group of shafts. The drilled shaft has coordinates (x,y) with respect to the centroidal origin in the group
P_{Yg}	Lateral load on a group of drilled shafts (unfactored)
PI	Plasticity index
PL	Plastic limit
q	Net foundation pressure applied at $2D_b/3$
q_c	Cone penetration resistance
q_l	Limiting point resistance in upper stratum
q_o	Limiting point resistance in lower stratum
q_p	Ultimate unit end bearing of a drilled shaft
q_{pr}	Reduced unit end bearing for drilled shafts with large bases
q_s	Ultimate unit side resistance of drilled shaft
$q_{s\ bell}$	Unit uplift capacity of a belled drilled shaft
q_{sn}	Unit downdrag load
q_u	Uniaxial compressive strength of rock

Symbol	Represents
Q_g	Ultimate capacity of a group of drilled shafts
Q_i	Load effect due to load component i
Q_p	Ultimate load carried by base of drilled shaft
Q_s	Ultimate load carried by side resistance
$Q_{s\ bell}$	Uplift capacity of a belled drilled shaft
Q_{ug}	Ultimate uplift resistance of a group of drilled shafts
Q_{ult}	Total ultimate bearing capacity
r	Eccentricity factor for calculating factored structural axial capacity of columns
R	Resistance corresponding to the limit state considered
R	Characteristic length of soil-drilled shaft system in clays
R_I	Moment of inertia ratio
RQD	Rock quality designation
s	Stress correction factor
s_d	Spacing of discontinuities
S	Average spacing of drilled shafts
S_u	Undrained shear strength
$\overline{S}_u$	Average undrained shear strength along drilled shaft
t_d	Width of discontinuities
T	Characteristic length of soil-drilled shaft system in sands
w_n	Natural water content of soil
W_g	Weight of block of soil, drilled shafts and cap

Symbol	Represents
x	Distance of the drilled shaft from the centroid of the cap in the x-direction
X	Width of group of drilled shafts
y	Distance of the drilled shaft from the centroid of the cap in the y-direction
Y	Length of group of drilled shafts
Y_g	Lateral displacement of a laterally loaded group of drilled shafts
Y_s	Total lateral displacement of a single drilled shaft
Y_{sM}	Lateral displacement component due to the bending moment M_e
Y_{sP}	Lateral displacement component due to a lateral load P_s
Y_{sMP}	Lateral displacement caused by the moment $(M_s + M_p)$
Y_{sPM}	Lateral displacement caused by the lateral load $(P_s + P_M)$
Y_{tol}	Tolerable lateral displacement
z	Depth below ground surface
Z	Total embedded length of a drilled shaft
GREEK	
α	Adhesion factor applied to S_u
β	Coefficient relating the vertical effective stress and the unit skin friction of a drilled shaft when used in Section 4
β	Load factor coefficient for a load component when used in Section 3
γ	Total unit weight of soil when used in Sections 4 and 5
γ	Load factor when used in Section 3

Symbol	Represents
γ'	Effective unit weight of soil
γ_D	Load factor for dead load
γ_h	Load factor for lateral load
γ_i	Load factor for load component i
γ_L	Load factor for live load
γ_m	Moment factor
δ	Angle of shearing resistance between soil and drilled shaft
η	Efficiency factor for the capacity of a group of drilled shafts
μ	Poisson's ratio of soil
ρ	Settlement of a group of drilled shafts
ρ_e	Elastic shortening
ρ_{base}	Settlement of the base of the drilled shaft
ρ_{tol}	Tolerable settlement
σ_v	Total vertical stress
σ_v'	Vertical effective stress
σ_{n1}	Normalizing stress
ϕ	Performance factor
ϕ'	Angle of internal friction of soil
ϕ_a	Performance factor for the nominal axial structural capacity of a drilled shaft
ϕ_g	Performance factor for the bearing capacity of a group of shafts failing as a unit consisting of the shafts and the block of soil contained within the shafts
ϕ_m	Performance factor for the ultimate structural moment capacity of a drilled shaft

Symbol	Represents
ϕ_q	Performance factor for the total ultimate bearing capacity of a drilled shaft
ϕ_{qs}	Performance factor for the ultimate side capacity of a drilled shaft
ϕ_{qp}	Performance factor for the ultimate end bearing capacity of a drilled shaft
ϕ_t	Performance factor for the tensile strength of steel
ϕ_u	Performance factor for the uplift capacity of a single drilled shaft

Part 5—Engineering Manual for Estimating Tolerable Movements of Bridges

J. M. DUNCAN AND C. K. TAN

CONTENTS

CHAPTER 1

INTRODUCTION

Foundations of bridges should be designed so that their settlements and horizontal movements will not cause excessive or intolerable damage. It is important to understand the factors that control the magnitudes of movements that can be tolerated in bridges, and to design bridge foundations so that their movements do not exceed these limits. This manual describes the problems that result from settlements and horizontal movements of bridges, discusses the state of practice with regard to estimating the magnitudes of movements that bridges can tolerate, and gives examples to illustrate how tolerable movement limits can be estimated for various types of bridges.

Moulton et al. (1985) made an investigation of criteria for tolerable movements of highway bridges. Based on a review of the literature, it was concluded that "until recently there was virtually nothing of a specific nature in the literature with respect to the tolerable movement of bridges." They also found that "the design practice of most agencies does not routinely involve the consideration of tolerable bridge movements."

Moulton et al. (1985) also summarized the results from a survey of bridge movements and the types of problems that are caused by movements of bridge foundations and abutments. The data they collected showed that bridge movements and resulting problems are widespread. Their study showed clearly that explicit consideration of the tolerance of bridges to withstand settlements and horizontal movements is desirable, and that such consideration should logically be a part of every bridge design study.

Within the realm of load factor design of bridges, reaching the upper limit of tolerable movement is defined as a "serviceability limit state," and consideration of this limit state is a specific design requirement. Whether the design of a bridge is based on the load factor design approach or on the working stress design approach, it is logical that the likely magnitudes of movements, and the ability of the bridge superstructure to withstand the movements without intolerable consequences, should be considered as part of the design process.

In principle, the effects of movements on bridges can be evaluated in two ways: (1) through structural analysis to examine the consequences of support movements on the stresses in the bridge superstructure, and (2) through studies of the behavior of real bridges, as exemplified by the studies performed by Moulton and his co-workers, to determine the upper limits of the magnitudes of movement that are considered tolerable in practice. Both of these procedures are discussed in the following chapters.

In practice, conventional structural analyses frequently result in excessively conservative criteria with regard to the magnitudes of movements that bridges can tolerate. Consequently, in most cases, field surveys of the behavior of real bridges provide the most reliable means of establishing upper limits for tolerable movements.

An effective method of establishing the magnitude of the settlement that can be tolerated by a bridge is to (1) use the results of field surveys to estimate the amount of angular distortion that can be tolerated by the bridge, (2) use this limiting value of

angular distortion to estimate the upper limits of tolerable settlements of the bridge, and (3) use the results of field studies to estimate the amount of horizontal movement that can be tolerated by the bridge.

Chapters 2 through 4 of this manual are concerned with the following topics: the types of damage caused by foundation and abutment movements; the various components of bridge foundation and abutment settlements; criteria for tolerable settlements and horizontal movements of bridges; the use of structural analysis techniques to evaluate the possible consequences of foundation movements; and examples of the use of the information to estimate tolerable movements for bridges.

CHAPTER 2

THE NATURE OF BRIDGE FOUNDATION MOVEMENTS AND THE PROBLEMS THEY CAUSE

As a basis for establishing tolerable movement criteria for bridges, it is useful to examine the types of problems that develop in bridges when foundation movements become excessive.

2.1 TYPES OF PROBLEMS CAUSED BY MOVEMENTS OF BRIDGE ABUTMENTS AND FOUNDATIONS

Uneven movements of bridge abutments and foundations can affect the comfort of bridge users, the appearance of the bridge, the structural integrity and ability of the bridge to support loads, and even the safety of the bridge. The severity of consequences increases with the magnitude of the movements that the bridge undergoes as its supports settle and shift laterally. It is useful to review the various types of problems that have been caused by foundation and abutment movements, in order to define more precisely the specific nature of these problems.

Moulton et al. (1985) noted that even completely uniform settlements that would cause no distortion of the bridge superstructure can reduce clearance at overpasses. Differential settlements between bridge decks and approach slabs result in the familiar "bump at the end of the bridge" (Wahls, 1990). Similar problems arise where one span of a bridge settles differently from an adjacent span, producing the condition that Keene (1978) described as "faulting at expansion joints."

Emanuel (1978) and Grover (1978) pointed out that differential settlements of bridge supports may result in increased support reactions and increased internal stresses in the bridge superstructure. Grover also suggested that uneven settlements can reduce the riding quality of the bridge, and may affect the safety of users. The results of the study conducted by Moulton et al. (1985), however, indicate that rider discomfort is not likely to control the magnitude of tolerable settlements. They found from a survey of 314 bridges in the United States and Canada that "foundation movements would become intolerable for some other reason before reaching a magnitude that would create intolerable rider discomfort." It thus appears that if movements are within a tolerable range with regard to structural distress for the bridge superstructure, they will also be acceptable with respect to user comfort and safe vehicle operation.

There seems to be general agreement among investigators who have studied the performance of bridges that horizontal movements cause more severe and widespread problems than do settlements (Keene, 1978; Walkinshaw, 1978; Bozozuk, 1978; Moulton et al., 1978; and Wahls, 1990). The types of problems that arise as a result of differential horizontal movements between bridge decks and abutments, or between adjacent spans of bridges, include (1) shearing of anchor bolts; (2) excessive opening of expansion joints; (3) reduced effectiveness of expansion joints when clearance is reduced; (4) complete closing of expansion joints and jamming of bridge decks into abutments or adjacent spans; (5) shifting of abutments when expansion joints jam; (6) severe damage to abutment walls, approach slabs or bridge decks due to excessive loads when expansion joints jam; (7) distortion and damage to bearings devices; (8) excessive tilting of rockers; and (9) damage to rails curbs, sidewalks, and parapets (Keene, 1978; Walkinshaw, 1978; Emanuel, 1978; Grover, 1978; Keene, 1978; Moulton et al., 1985; and Yokel, 1990).

Unfortunately, while there are well-established and fairly simple techniques for estimating settlements, similar procedures for estimating horizontal movements have not been developed. Designers must estimate the possible amount of horizontal movement that may occur in a bridge using experience and judgment, or they must employ the relatively complex and time-consuming finite element method of analysis. It may be noted that a similar situation exists with respect to buildings: empirical evidence shows that horizontal movements tend to be more damaging than settlements, but there are no simple techniques for estimating the magnitudes of horizontal movements of building foundations.

2.2 COMPONENTS OF BRIDGE SETTLEMENTS

The settlements of bridges can be divided into three components—uniform settlement, tilt (or rotation), and nonuniform settlement. These are illustrated in Figure 2.1

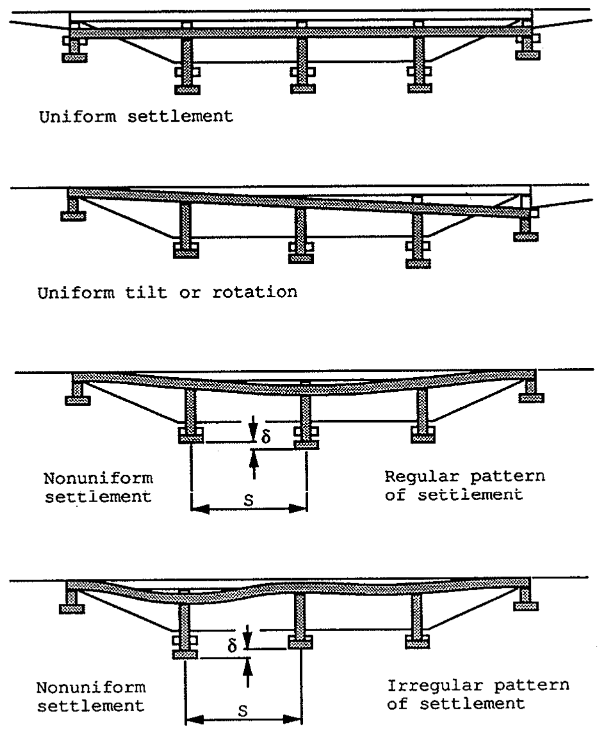
Uniform settlement corresponds to the condition in which each of the foundations beneath a bridge settles the same amount. Because settlements are never precisely uniform, this is only a hypothetical possibility. This type of settlement does form a

component of more complex settlement patterns, however. This component of settlement does not cause distortion of the bridge superstructure (Stermac, 1978; Yokel, 1990). However, excessive settlement in this mode can cause problems at the juncture between approach slabs and bridge decks (what Wahls (1990) called "the bump at the end of the bridge"), and they can also cause problems with drainage at the end of the bridge and with clearance at underpasses.

Uniform tilt, or rotation, corresponds to settlements that vary linearly along the length of the bridge. This mode of settlement is most likely for bridges with very stiff superstructures, and is probably only a realistic possibility for single-span bridges. Although this type of settlement does not cause distortion in the superstructure, it could result in the same approach slab problems, drainage problems, and clearance problems, as mentioned in the previous paragraph, and it may involve distortion where the bridge superstructure connects to substructure or foundation elements that do not rotate with the superstructure.

Nonuniform settlements (differential settlements) result in deformation of the superstructure if the superstructure is continuous over three or more foundations. Two different types of nonuniform settlement are shown in Figure 2.1. In one case, the pattern of settlement is regular (increasing toward the center from both ends); and in the other, it is irregular (the settlements vary erratically along the length of the bridge). Both types of nonuniform settlement result in distortion of the superstructures of continuous span bridges. Given equal magnitudes of settlement, an irregular pattern of settlements would be expected to cause greater distortion in the superstructure, because the differential settlement between adjacent foundations would be likely to be greater.

Nonuniform settlements can cause problems with bumps at junctures with approach slabs, problems with bumps at junctures between adjacent spans, problems with drainage, problems with clearance at underpasses, and various types of problems that arise through excessive support reactions, large internal stresses, and consequent distress in the superstructure.



A = Angular Distortion

$$A = \frac{\text{Difference in Settlement Between Foundations}}{\text{Distance Between Foundations}} = \frac{\delta}{S}$$

Figure 2.1. Components of settlement and angular distortion in bridges.

2.3 CRITERIA FOR TOLERABLE SETTLEMENTS OF BRIDGES

Because of the complexity of settlement patterns, no single measure of settlement or distortion affords a perfect indicator of whether or not a given amount of settlement will cause damage to a bridge.

Various investigators (Walkinshaw, 1978; Bozozuk, 1978; Grover, 1978; and Wahls, 1990) have suggested upper limits for the magnitudes of settlement to serve as criteria for the boundary between tolerable and intolerable performance. It is important to note that settlements may be considered "tolerable" even though they may be large enough to cause some amount of damage to a bridge. Bozozuk (1978) divided settlements and horizontal movements into three categories: (1) those that were *not harmful,* (2) those that were *harmful but tolerable,* and (3) those that were *harmful and intolerable.* Similarly, Moulton et al. (1985) used the following definition of intolerable movement: "Movement is not tolerable if damage requires costly maintenance and/or repairs and a more expensive construction to avoid this would have been preferable."

Thus, the definition of intolerable is somewhat subjective, and varies, depending on factors other than the physical condition of the bridge. Factors such as the cost and practical problems involved in repair and maintenance enter the evaluation of whether a given degree of damage in a particular bridge is tolerable or intolerable. Considering the fact that such subjective evaluations are unavoidable, it can be understood how various investigators have arrived at differing recommendations regarding the limiting amounts of tolerable movements for bridges.

Another important consideration is the fact that the study conducted by Moulton et al. (1985) involved more data, and more thorough analysis of the data, than had any previous study. It thus seems appropriate to give greatest weight to criteria developed through the work of Moulton and his co-workers, because this represents what is believed to be the best of the rather meager information available with regard to tolerable settlements of bridges, and it presents considerably more data than were available for recommendations made prior to 1985.

Criteria for settlements of bridges, expressed in terms of the magnitude of the settlement, are given in Table 2.1. It can be seen that they are reasonably consistent. The smallest value listed (2 in. or 51 mm) was suggested by Bozozuk as not harmful on the basis of his survey of the performance of actual bridges. Walkinshaw (1978) suggested 2.5 in. (63 mm) as the upper limit of tolerable settlements based on ride quality, and suggested that larger values would be acceptable based on considerations of structural distress. Grover (1978), Bozozuk (1978), and Wahls (1990) all recommend 4 in. (102 mm) as the amount of settlement that would cause some damage, but would still be tolerable.

Table 2.1. Settlement criteria for bridges expressed in terms of settlement magnitude.

Settlement Magnitude (mm)	Basis for recommendation	Recommended by
51	Not harmful	Bozozuk (1978)
63	Ride quality	Walkinshaw (1978)
>63	Structural distress	Walkinshaw (1978)
102	Ride quality and structural distress	Grover (1978)
102	Harmful but tolerable	Bozozuk (1978)
>102	Usually intolerable	Wahls (1990)

The criteria in Table 2.1 are stated without regard to the type or size of bridge. Stermac (1978), in discussing Bozozuk's recommendations, made the following observation: "To lump all bridges together amounts to saying that all bridges are the same and behave in the same way. This, of course, is not the case."

Although the criteria listed in Table 2.1 may be oversimplified, they nevertheless have some value, simply because they are based on experience with real bridges. While criteria based on angular distortion may be fundamentally more sound, the limiting values of settlement given in Table 2.1 provide a very simple and sensible rough guide as to the magnitudes of tolerable settlements of bridges.

Criteria for settlements of bridges, expressed in terms of angular distortion, are given in Table 2.2. Angular distortion is defined, as shown in Figure 2.1, by the following equation:

$$A = \frac{\delta}{S} \tag{2.3.1}$$

in which A = angular distortion, dimensionless; δ = differential settlement, or difference in settlement, between the foundations at the two ends of a span, expressed in units of length; and S = span length, expressed in the same units of length as δ.

Moulton et al. (1978) found that angular distortion provides a good basis for establishing tolerable movement magnitudes for bridges, and they suggested the criteria given in Table 2.2. Other investigators (Yokel, 1990; Wahls, 1990) have since also suggested that the limits of angular distortion given in Table 2.2 provide reasonable criteria for tolerable settlements of bridges. Their recommendations were based on review of the studies performed by Moulton et al. (1985).

The values of angular distortion given in Table 2.2 are based on data for 56 simple span bridges and 119 continuous span bridges. The portions of these data that were most critical in establishing the criteria are summarized in Table 2.3.

The data summarized in Table 2.3 show that 96 percent of 119 continuous span bridges underwent values of angular distortion as large as 0.004 without suffering damage that was considered to be intolerable. As the upper end of the range of angular distortion values increased to 0.005, the percentage of acceptable incidents dropped to 92 percent. Thus, the limit of tolerable angular distortions recommended by Moulton et al. (A = 0.004 for continuous span bridges) can be seen to be based on a 96 percent rate of acceptability among the bridges included in their study. Among all bridges, the rate of acceptability would be higher, because the data base included only bridges that had moved, and excluded those that had not moved.

Table 2.2. Settlement criteria for bridges expressed in terms of angular distortion.

Angular Distortion δ/S	Basis for recommendation	Recommended by
0.004	Tolerable for multiple-span bridges	Moulton, et al. (1985)
0.005	Tolerable for single-span bridges	Moulton, et al. (1985)

Table 2.3. Data used by Moulton et al. (1985) to establish criteria for magnitudes of angular distortion.

Value of angular distortion	Percent of 119 continuous span bridges for which this amount of angular distortion was considered to be tolerable	Percent of 56 simple span bridges for which this amount of angular distortion was considered to be tolerable *
0.000 to 0.001	100 %	98 % (100 %)
0.001 to 0.002	97 %	98 % (100 %)
0.002 to 0.003	97 %	98 % (100 %)
0.003 to 0.004	96 %	98 % (100 %)
0.004 to 0.005	92 %	98 % (100 %)
0.005 to 0.006	88 %	96 % (98 %)
0.006 to 0.008	85 %	93 % (95 %)

* Values in parentheses exclude one bridge, for which a value of angular distortion < 0.001 was associated with "intolerable" behavior. This bridge was excluded for the purpose of calculating the values in parentheses because it is considered likely that, in this particular case, the intolerable behavior was not the result of the very small angular distortion that was experienced by the bridge.

For simple span bridges, 56 cases were available to serve as a basis for establishing angular distortion criteria. Moulton et al. recommended a value of A = 0.005 for use as the upper limit of tolerable angular distortions, which corresponds to a 98 percent rate of acceptability among the bridges they studied. Examination of the data indicates that this criterion may be overly conservative. Note that for one bridge, a value of A < 0.001 was associated with intolerable behavior. It seems logical to exclude this bridge from consideration because no other bridge with A < 0.005 suffered intolerable damage, and this bridge clearly represents an aberrant case. When this one bridge is excluded, the percentages of bridges exhibiting acceptable behavior change to the values given in parentheses in Table 2.3. Considering the values shown in parentheses, 95 percent of the bridges underwent values of angular distortion as large as 0.008 without suffering damage that was considered to be intolerable.

Based on this reexamination of the data compiled by Moulton et al., it seems appropriate to revise the recommendations made by Moulton and his co-workers and to adopt the following values of angular distortion as the upper limits of acceptable behavior:

For continuous span bridges, A $\leq$ 0.004 is acceptable
For simple span bridges, a $\leq$ 0.008 is acceptable

It seems logical that simple span bridges can withstand much greater magnitudes of angular distortion than continuous span bridges without suffering intolerable levels of damage, and the data in Table 2.3 support this point of view. The authors believe that the criteria above represent a sound interpretation of the data compiled by Moulton et al. (1985), and that they provide a reasonable basis for establishing limits of acceptable differential settlements for bridges.

2.4 CRITERIA FOR TOLERABLE HORIZONTAL MOVEMENTS OF BRIDGES

A number of investigators have suggested criteria for tolerable horizontal movements based on observations of the field performance of bridges, as shown in Table 2.4. These values may be considered in two categories: not harmful, and harmful but tolerable. Bozozuk (1978) suggests that horizontal movements less than 1 in. (25 mm) are not harmful, and that horizontal movements up to 2 in. (51 mm) are harmful but tolerable.

The data presented by Moulton et al. (1985) showed that horizontal movements tended to be more severely damaging when they were accompanied by settlements than when they were not. They found that horizontal movements less than 1 in. (25 mm) were almost always reported as being tolerable, while horizontal movements greater than 2 in. (51 mm) were quite likely to be considered to be intolerable. On this basis they recommended that horizontal movements be limited to 1.5 in. (38 mm).

Similarly, Walkinshaw (1978) and Wahls (1990) suggest 51 mm as the magnitude of horizontal movement corresponding to intolerable performance.

In summary, it appears that the value of tolerable horizontal movement suggested by Moulton et al., 1.5 in. (38 mm), is a reasonable value. Although this criterion is very simple, and likely to be oversimplified, the value for the limit of tolerable horizontal movements of bridges provides some guidance for design.

An issue that is closely related to the amount of horizontal movement that bridges can tolerate is the amount of movement at abutments necessary to reduce the earth pressure on the abutment to the design value. This issue is discussed in a companion engineering manual for retaining walls and abutments (Part 3) and in geotechnical engineering texts and manuals. It is well known that if an abutment or retaining wall does not move, the earth pressure that acts on it is the "at-rest" value. If the abutment moves away from the backfill, the earth pressure will be reduced below the at-rest value. If the abutment moves far enough, the earth pressure can be reduced to a minimum value, called the active earth pressure.

Table 2.4. Horizontal movement criteria for bridges expressed in terms of movement magnitude.

Horizontal Movement mm	Basis for recommendation	Recommended by
25	Not harmful	Bozozuk (1978)
38	Tolerable in most cases	Moulton, et al. (1985)
51	Structural Distress	Walkinshaw (1978)
51	Harmful but tolerable	Bozozuk (1978)
51	Usually intolerable	Wahls (1990)

If the backfill is a clean sand or gravel, the earth pressure can be reduced to active by horizontal movement of the abutment away from the backfill that is about 0.004 times the height of the abutment wall. This is about 0.5 in. for a 10-ft high abutment wall, 1.0 in. for a 20-ft high wall, or 1.5 in. for a 30-ft high wall. (In metric units these correspond to 12 mm for a 3-m high wall, 25 mm for a 6-m high wall, and 38 mm for a 9-m high wall). These values are for sand or gravel backfill in a loose condition. The movements required to reduce the earth pressures to their active values in dense sands or gravels are smaller than 0.004 times the wall height.

If the backfill is a clay, or if it contains enough clay or plastic silt so that it is cohesive and has plasticity, the movement required to reduce the earth pressure to the active value is larger than it is for sands or gravels. Furthermore, such cohesive soils exhibit creep. If a wall moves enough to reduce the pressure to the active value, and then stops moving, the earth pressures will begin to increase from the active value. Continual movement will occur if the active earth pressure is used for design of walls backfilled with cohesive soil. When cohesive soils are used for backfill, it is prudent to base the design on at-rest, or near at-rest pressures.

Considering these facts, it is clear that the selection of earth pressure for design of an abutment wall carries with it certain implications with regard to movement of the wall. If active earth pressures are used for the design of a wall, it should be considered how much the wall will need to move to develop those pressures. Design of the expansion joints should allow for sufficient movement to reduce the earth pressures to their design values and still perform properly under all temperature conditions.

CHAPTER 3

USE OF STRUCTURAL ANALYSIS TO EVALUATE THE CONSEQUENCES OF SETTLEMENT

It seems logical that structural analysis methods can be used to evaluate the effects of settlements on bridges, as suggested by Moulton et al. (1985) and Yokel (1990). In principle, the shear forces and bending moments caused by settlements can be added to those due to imposed loads, and the resulting moments and shear forces can then be compared with the flexural and shear

capacities of the structure to determine if the settlements are tolerable. It has been found, however, that this procedure can result in excessively conservative results, indicating that very large overstress would be caused by rather small settlements, even though experience indicated such settlements could be tolerated with little or no distress.

One such study was performed by Meyerhof (1947), who investigated the effects of settlements on buildings. He analyzed the effect of foundation settlement on a five-story, three-bay reinforced concrete building. He found that 0.315 in. of settlement at one of the columns (corresponding to an angular distortion of 0.001 in bays that were 25 ft long) corresponded to a calculated increase of 74 percent in the moment in the most severely loaded beam in the building. This magnitude of increase in bending moment would certainly be expected to cause structural damage. However, field data compiled by Skempton and MacDonald (1956), Polshin and Tokar (1957), and Grant et al. (1974), indicate that framed building structures can withstand twice this amount of angular distortion without suffering even minor cracking or any other form of damage. It is thus clear that real frame buildings are able to withstand considerably greater differential settlements than would be inferred from the results of structural analyses.

Conventional structural analyses also tend to result in overly conservative settlement criteria for bridges. This can be illustrated through a simplified analysis of the amount of tolerable distortion of a reinforced concrete bridge deck. Consider a case where the center support of a two-span bridge with a continuous reinforced concrete deck settles by an amount δ relative to the ends of the deck. The bending stress induced in a simple rectangular reinforced concrete bridge deck by this settlement can be expressed as

$$f_{max} = \frac{\delta}{S}\left\{\frac{3E}{2}\,\frac{t}{S}\right\} \tag{3.1}$$

in which f_{max} = maximum bending stress induced by settlement, in same units as E; δ = differential settlement between the center pier and the abutment, in same units as S; E = Young's modulus, in stress units; t = thickness of deck, in same units as S; and S = span length, in length units.

To prevent cracking, the maximum tensile bending stress should not exceed the tensile strength of the concrete, f_t. The value of δ/S corresponding to $f_{max} = f_t$ can be expressed as:

$$\frac{\delta}{S} = \frac{2 f_t\, S}{3\, E\, t} \tag{3.2}$$

The quantities on the right-hand side of this expression can be estimated using values appropriate for concrete with compressive strength, f_c' equal to 4000 psi and reinforcing bar yield strength of 60,000 psi. These values are $f_t = 475$ psi, $E = 3.6 \times 10^6$ psi, and S/t = 24. For this combination of values, the value of δ/S calculated using Eq. 3.2 is 0.0021. Thus, this simplified analysis of the consequences of differential settlement indicates that cracking would occur in the reinforced concrete bridge deck if the angular distortion, δ/S, reached a value of 0.0021. In order to prevent this cracking, it might be concluded that the angular distortion should be limited to a value less than or equal to 0.0021.

The survey of damage to bridges caused by settlements reported by Moulton et al. (1985) shows clearly that continuous-span bridges can withstand larger values of δ/S without intolerable damage. The results of the Moulton et al. survey indicate that angular distortions of 0.004 are tolerable for continuous-span bridges, of either steel or concrete. This value of angular distortion is nearly double the value inferred from the simplified structural analysis on which Eq. 3.1 is based.

Thus, for both bridges and buildings, field experience appears to indicate that real structures can tolerate considerably greater magnitudes of angular distortion than would be inferred from the results of conventional structural analysis. The reasons for this include the fact that building materials like concrete (especially concrete while it is curing) are able to undergo a considerable amount of stress relaxation when subjected to deformations; and under conditions of very slowly imposed deformations, the effective value of the Young's modulus of concrete is considerably lower than the value for rapid loading. Thus, inferences about the stresses induced by deformations that are based on analyses that ignore the beneficial effects of stress relaxation, and the consequences of the rate of deformation, will be overly conservative for these materials. Another important factor is that part of the settlement of real structures occurs as they are built (Keene, 1978; Yokel, 1990). Therefore, the portions of the structures that are constructed last do not experience all of the settlement, and cannot be damaged by it.

It seems preferable in most cases to use field studies as the basis for tolerable movement criteria, rather than structural analyses. If, in unusual cases it is necessary to use structural analyses as the basis for evaluating maximum tolerable movements, the analyses should include consideration of important factors such as stress relaxation, creep, shrinkage, the rate of deformation, and the construction sequence.

CHAPTER 4

EXAMPLES

The procedures for evaluating tolerable settlements described in the previous sections are illustrated in the following pages by three examples.

4.1 EXAMPLE 1—CONTINUOUS-SPAN BRIDGE

Example 1 (Figure 4.1) concerns a continuous-span bridge with four foundations—two abutment walls supported on footings, and two piers supported on piles. The lengths of the end spans are 82 ft (25 m), and the length of the center span is 150 ft (45 m). The objective of the example is to illustrate how the information contained in the preceding sections can be used to establish tolerable movement magnitudes for the bridge.

One approach is to use the criteria with regard to tolerable angular distortions established by Moulton et al. (1985) to determine the upper limit of angular distortion for the bridge, and to use this value to establish the maximum tolerable settlement. For continuous-span bridges, of either steel or concrete, Moulton et al. recommended that the maximum tolerable value of angular distortion is about 0.004.

When the maximum tolerable angular distortion is used as the basis for establishing a value of maximum tolerable settlement, the shortest span controls because the tolerable differential settlement is equal to the tolerable angular distortion multiplied by the span length. In this case the end spans, with lengths of 82 ft (25 m), control. For a value of angular distortion equal to 0.004, and a span length of 82 ft (25 m), the maximum tolerable differential settlement would be (0.004) * (82 ft), which is equal to 4 in. (0.1 m, or 100 mm). This value, 4 in., is the estimated maximum tolerable differential settlement over the length of the end spans of the bridge.

Differential settlements can be calculated rationally by calculating the difference between the settlements at the two locations, in this case the abutment and the pile foundation. The problem with this approach is that settlement calculations inevitably involve some uncertainty, and the uncertainty of the estimated value of differential settlement is larger than the uncertainty in the estimated values of settlement for either the abutment or the pile foundation. As a result, estimating values of differential settlement simply as the difference between the calculated settlements for the two ends of the span involves considerable margin for inaccuracy. Most importantly, the real differential settlement might exceed the estimated value, even if the settlements are estimated with a reasonable degree of conservatism. This is because, if one of the foundations settles *less* than estimated (while the other settles the estimated amount), the differential settlement will be larger than the difference between the two values of calculated settlement.

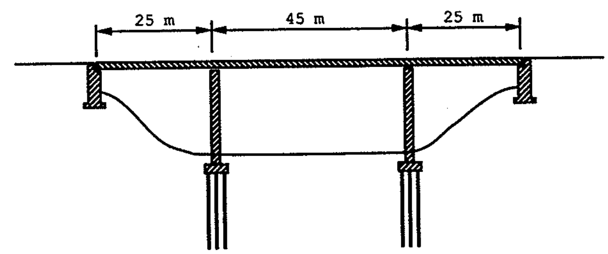
To estimate an upper limit of differential settlement, it is necessary to consider the possibility that one of the foundations might settle less than the calculated amount, while the other foundation settles by the calculated amount. If the values of settlement are estimated conservatively, the actual settlement of any foundation should not be appreciably more than the calculated value.

A simple procedure that is reasonable for many conditions is to use conventional settlement calculation methods to make conservative estimates of the settlements of all of the foundations. Such methods are described in the companion engineering manual for shallow foundations (Part 1) and in foundation engineering texts and manuals. The maximum likely value of differential settlement can then be estimated based on these two pessimistic assumptions: (1) the settlement of any foundation could be as large as the value calculated using conservative procedures; and (2) at the same time, the settlement of the adjacent foundation could be zero.

Use of these conservative assumptions would result in an estimated maximum possible differential settlement equal to the largest settlement calculated for the foundation at either end of any span. This conservative procedure is the one implied in Figure 4.1, where maximum tolerable settlement values are inferred directly from values of maximum tolerable angular distortion.

Another approach to estimating maximum tolerable settlements for the bridge is to use the values summarized in Table 2.1. These would indicate that settlements of about 4 inches (100 mm) would be expected to cause some damage, but would be tolerable. In this particular case, where the bridge has continuous spans, and the shortest span length is 82 ft (25 m), the values summarized in Table 2.1 lead to the same end result as does the use of angular distortion and the previously outlined considerations of the relationship between total and differential settlements.

The maximum tolerable horizontal movements can be estimated using the information in Table 2.4. This indicates that the maximum tolerable horizontal movement would be about 38 mm 1.5 in. (38 mm).



Tolerable angular distortion: Deck is continuous over four supports. Based on recommendations of Moulton, et al. (1985) and information summarized in Section 2.3 of this report, the tolerable angular distortion value would be 0.004.

Tolerable settlement: The shortest span lengths are 25 m. The tolerable differential settlement corresponding to angular distortion of 0.004 would be 100 mm (4 inches).

An alternative approach to estimating tolerable settlements is to use the information summarized in Table 2.1. This indicates that settlements of 100 mm (4 inches) would be expected to cause some damage, but would be tolerable in the sense defined by Moulton, et al. (1985).

Thus it would be expected that, if the maximum settlement of any of the four foundations of the bridge was 100 mm (4 inches) or less, the settlement of the bridge would be considered tolerable.

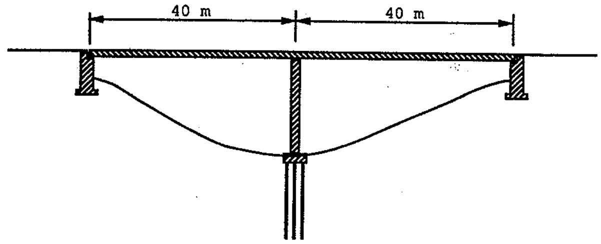
Based on the information summarized in Table 2.4 the maximum tolerable horizontal movements for the bridge would be about 38 mm (1.5 inches).

Figure 4.1. Example 1—tolerable settlement and angular distortion for a continuous-span bridge with minimum span lengths of 82 ft (25 m).

4.2 EXAMPLE 2—TWO-SPAN BRIDGE

Example 2 (Figure 4.2) concerns a two-span bridge continuous over three foundations—two abutment walls and a center pier. Based on the recommendations of Moulton et al. (1985), which are summarized in Table 2.2, the maximum tolerable angular distortion would be 0.004. With span lengths of 130 ft (40 m), this amount of angular distortion corresponds to a differential settlement between the abutment and the center pier of 6.3 in. (160 mm). The information summarized in Table 2.1 indicates that the maximum tolerable settlement for this (or any other) bridge would be about 4 in. (100 mm).

In this case, because the spans are longer than 82 ft (25 m), the criterion based on angular distortion results in a larger estimate of the tolerable settlement than the values given in Table



Tolerable angular distortion: Deck is continuous over three supports. Based on recommendations of Moulton, et al. (1985) and information summarized in Section 2.3 of this report, the tolerable angular distortion value would be 0.004.

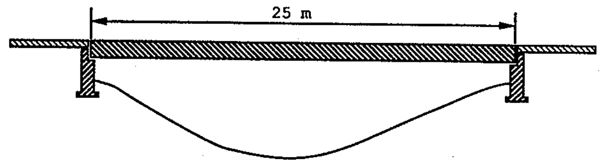
Tolerable settlement: The span lengths are 40 m. The tolerable differential settlement corresponding to angular distortion of 0.004 would be 160 mm (6.3 inches).

An alternative approach to estimating tolerable settlements is to use the information summarized in Table 2.1. This indicates that settlements of 100 mm (4 inches) would be expected to cause some damage, but would be tolerable in the sense defined by Moulton, et al. (1985).

Thus it would be expected that, if the maximum settlement of any of the four foundations of the bridge was 100 mm (4 inches) or less, the settlement of the bridge would be considered tolerable by both sets of criteria. If the maximum settlement of any of the three foundations of the bridge was 160 mm (6.3 inches), the settlement of the bridge would be considered tolerable based on the criteria established by Moulton, et al. (1985), but not by the older criteria in Table 2.1.

Based on the information summarized in Table 2.4 the maximum tolerable horizontal movements for the bridge would be about 38 mm (1.5 inches).

Figure 4.2. Example 2—tolerable settlement and angular distortion for a continuous-span bridge with span lengths of 130 ft (40 m).



Tolerable angular distortion: Deck is a single span spanning between two supports. Based on the interpretation of the data presented by Moulton, et al. (1985), as summarized in Section 2.3 of this report, the tolerable angular distortion value would be 0.008.

Tolerable settlement: The span length is 25 m. The tolerable differential settlement corresponding to angular distortion of 0.008 would be 200 mm (7.9 inches).

An alternative approach to estimating tolerable settlements is to use the information summarized in Table 2.1. This indicates that settlements of 100 mm (4 inches) would be expected to cause some damage, but would be tolerable in the sense defined by Moulton, et al. (1985).

Thus it would be expected that, if the maximum settlement of either abutment of the bridge was 100 mm (4 inches) or less, the settlement of the bridge would be considered tolerable by both criteria. If the maximum settlement of either of the abutments was 200 mm (7.9 inches), the settlement of the bridge would be considered tolerable based on the criteria described in Section 2.3, but not by the older criteria in Table 2.1.

Based on the information summarized in Table 2.4 the maximum tolerable horizontal movements for the bridge would be about 38 mm (1.5 inches).

Figure 4.3. Example 3—tolerable settlement and angular distortion for a single-span bridge with a span length of 82 ft (25 m).

2.1. In this case the designer must rely on judgment and experience to determine what is the most reasonable value of tolerable settlement. It seems likely that the two values, established as indicated in Figure 4.2, would provide reasonable bounds for estimating a sound value of tolerable settlement for the bridge.

The tolerable horizontal movement for the bridge would be estimated to be about 1.5 in. (38 mm), using the information given in Table 2.4.

4.3 EXAMPLE 3—SINGLE-SPAN BRIDGE

Example 3 (Figure 4.3) is a single-span bridge with a span length of 82 ft (25 m). As discussed in Section 2.3 of this report, the data collected by Moulton et al. (1985) support the conclusion that in single-span bridges values of angular distortion as large as 0.008 are tolerable. For a span length of 82 ft (25 m), this value of angular distortion corresponds to a differential settlement between the ends of the span of 8 in. (200 mm). This value is twice as large as the values indicated in Table 2.1. In this regard it may be noted that a number of the bridges surveyed by Bozozuk (1978), which were used to establish the values shown in Table 2.1, actually settled more than 8 in. without intolerable damage. In fact, some settled more than a foot without intolerable damage. Thus, although 8 in. (200 mm) does appear to be a large amount of settlement to be considered tolerable, there is precedent for bridges undergoing even larger settlements without suffering intolerable damage. As noted with regard to Example 2, a designer would need to call upon judgment and experience to select a value of tolerable settlement in cases where the various criteria that have been suggested do not agree.

Table 2.4 indicates that the tolerable horizontal movement for this bridge, as for any other bridge, is about 1.5 in. (38 mm).

REFERENCES

Bozozuk, M. (1978), "Bridge Foundations Move", Transportation Research Record 678, Tolerable Movements of Bridge Foundations, Sand Drains, K-Test, Slopes, and Culverts, Transportation Research Board, Washington, D. C., 1978, pp.17 - 21.

Emanuel, J. H. (1978), Discussion of "Survey of Bridge Movements in the Western United States", by J. L. Walkinshaw, Transportation Research Record 678, Tolerable Movements of Bridge Foundations, Sand Drains, K-Test, Slopes, and Culverts, Transportation Research Board, Washington, D. C., 1978, pp.11 - 12.

Grant, R., Christian, J. T., and Vanmarcke, E. H. (1974), "Differential Settlement of Buildings", Journal of Geotechnical Engineering Division, ASCE, Vol. 100, No. 9, pp 973 - 991.

Grover, R. A. (1978), "Movements of Bridge Abutments and Settlements of Approach Slabs in Ohio", Transportation Research Record 678; Tolerable Movements of Bridge Foundations, Sand Drains, K-Test, Slopes, and Culverts, Transportation Research Board, Washington, D. C., 1978, pp.12 - 17.

Keene, P. (1978), "Tolerable Movements of Bridge Foundations", Transportation Research Record 678, Tolerable Movements of Bridge Foundations, Sand Drains, K-Test, Slopes, and Culverts, Transportation Research Board, Washington, D. C., 1978, pp.1 - 6.

Kim, S. G. , Barker, R. M. Duncan, J. M., and Rojiani, K. B. (1991),"Engineering Manual for Retaining Walls and Abutments," NCHRP Project 24-4 Report, Virginia Polytechnic Institute and State University, Blacksburg, VA, 121 pages.

Meyerhof, G. G. (1947), "The Analysis of Building Frames", The Structural Engineer, Vol. 25, No. 9, Sept., pp 369 -409.

Moulton, L. K., GangaRao, H. V. S., and Halvorsen, G. T. (1985), Tolerable Movement Criteria for Highway Bridges, Report No. FHWA/RD-85/107, Federal Highway Administration, Washington, D. C., 118 pp.

Polshin, D. E., and Tokar, R. A. (1957), "Maximum Allowable Non-uniform Settlement of Structures", Proc., 4th International Conference of Soil Mechanics and Foundation Engineering, London, Vol. 1, pp 402 - 405.

Skempton, A. W., and MacDonald, D. H. (1956), "The Allowable Settlement of Buildings", Proc., Institute of Civil Engineers, London, England, Part III, Vol. 5, pp 724 - 784.

Stermac, A. G. (1978), Discussion of "Bridge Foundations Move", by M. Bozozuk, Transportation Research Record 678, Tolerable Movements of Bridge Foundations, Sand Drains, K-Test, Slopes, and Culverts, Transportation Research Board, Washington, D. C., 1978, pp.17 - 21.

Tan C. K., Duncan, J.M., Rojiani, K. B., and Barker, R. M. (1991), "Engineering Manual for Shallow Foundations," NCHRP Project 24-4 Report, Virginia Polytechnic Institute and State University, Blacksburg, VA, 170 pages.

Wahls, H. E. (1981), "Tolerable Settlement of Buildings", Journal of Geotechnical Engineering Division, ASCE, Vol. 107, No. 11, pp 1489 - 1504.

Wahls, H. E. (1990), "Design and Construction of Bridge Approaches", National Cooperative Highway Research Program Synthesis of Highway Practice 159, Transportation Research Board, National Research Council, Washington, D. C., 45 pages.

Walkinshaw, J. L. (1978), "Survey of Bridge Movements in the Western United States", Transportation Research Record 678, Tolerable Movements of Bridge Foundations, Sand Drains, K-Test, Slopes, and Culverts, Transportation Research Board, Washington, D. C., 1978, pp.6 - 11.

Yokel, F. Y. (1990), "Proposed Design Criteria for Shallow Bridge Foundations", Report of the U. S. Department of Commerce, National Institute of Standards and Technology, National Engineering Laboratory Center for Building Technology, Gaithersburg, MD, 20899, 50 pages.

NOTATIONS AND SYMBOLS

Symbol	Definition
A	angular distortion = δ/S (dimensionless)
E	Young's modulus (stress units)
f_c'	compressive strength of concrete (stress units)
f_{max}	maximum bending stress induced by settlement (stress units)
f_t	tensile strength of concrete (stress units)
S	span length (length units)
t	thickness of bridge deck (length units)
δ	differential settlement, or difference in settlement, between the foundations at opposite ends of a span (length units)

Part 6—Recommended Load Factor Design Specifications and Commentary

R. M. BARKER, J. M. DUNCAN, R. B. ROJIANI, P. S. K. OOI, C. K. TAN, AND S. G. KIM

CONTENTS

Appendix B "Specifications" and Appendix C "Commentary" of the agency final report (refer to "Synopsis of the Research") have been reproduced in Part 6 as submitted by the research agency. It is worth noting that the format does not conform with that used in the previous Parts of this Volume. Unique treatment has been employed in these appendixes because of the requirements dictated by the NCHRP Project 24-4 tasks 6 and 8—namely, "development of . . . load factor design criteria and commentary in a format suitable for consideration by the AASHTO Subcommittee on Bridges and Structures . . . and identification of other sections of the AASHTO *Standard Specifications for Highway Bridges* that may be affected by the proposed changes in the foundation design criteria." The original appendix page designations have also been retained to preserve the accuracy of cross references.

APPENDIX B

SPECIFICATIONS

NOTE: The format used in Appendix B for Sections 4, 5 and 7 is to first present a complete Table of Contents for each section and then give the recommended Load Factor Design specifications. The articles listed in the Table of Contents on Allowable Stress Design have been prepared under NCHRP Project 12-35 (D'Appolonia, 1989). Where a commentary on a Load Factor Design specification has been prepared, it is denoted by (C) after the title of the article. The text of the commentary is given in Appendix C. Finally, articles in other sections of the AASHTO Specifications for Highway Bridges (1989), that require revision to accommodate Load Factor Design are given.

TABLE OF CONTENTS

SECTION 4
FOUNDATIONS

PART A

GENERAL REQUIREMENTS AND MATERIALS

PART B
SERVICE LOAD DESIGN METHOD
ALLOWABLE STRESS DESIGN

TABLE OF CONTENTS
SECTION 4 - FOUNDATIONS
CONTINUED

TABLE OF CONTENTS
SECTION 4 - FOUNDATIONS
CONTINUED

NCHRP 12-35

TABLE OF CONTENTS
SECTION 4 - FOUNDATIONS
CONTINUED

NCHRP 12-35

NCHRP 12-35

232

NCHRP 12-35

PART 6

PART C
STRENGTH DESIGN METHOD
LOAD FACTOR DESIGN

TABLE OF CONTENTS
SECTION 4 - FOUNDATIONS
CONTINUED

TABLE OF CONTENTS
SECTION 4 - FOUNDATIONS
CONTINUED

SECTION 4

FOUNDATIONS

PART A

GENERAL REQUIREMENTS AND MATERIALS

4.1 GENERAL (Revised Article 4.1)

Foundations shall be designed to support all live and dead loads, and earth and water pressure loadings in accordance with the general principles specified in this section. The design shall be made either with reference to service loads and allowable stresses as provided in SERVICE LOAD DESIGN or, alternatively, with reference to load factors, and factored strength as provided in STRENGTH DESIGN.

4.2 FOUNDATION TYPE AND CAPACITY

4.2.1 SELECTION OF FOUNDATION TYPE (C)

Selection of foundation type shall be based on an assessment of the magnitude and direction of loading, depth to suitable bearing materials, evidence of previous flooding, potential for liquefaction, undermining or scour, and ease and cost of construction.

4.2.2 FOUNDATION CAPACITY (Revised Article 4.2.2)

Foundations shall be designed to provide adequate structural capacity, adequate foundation bearing capacity with acceptable settlements, and acceptable overall stability of slopes adjacent to the foundations. The tolerable level of structural deformation is controlled by the type and span of the superstructure.

4.2.2.1 Bearing Capacity (Revised Article 4.2.2.1)

The bearing capacity of foundations may be estimated using procedures described in Articles 4.4, 4.5 or 4.6 for service load design and Articles 4.11, 4.12 or 4.13 for strength design, or other generally accepted theories. Such theories are based on soil and rock parameters measured by in situ and/or laboratory tests. The bearing capacity may also be determined using load tests.

4.2.2.2. Settlement (Revised Article 4.2.2.2)

The settlement of foundations may be determined using procedures described in Articles 4.4, 4.5 or 4.6 for service load design and Articles 4.11, 4.12 or 4.13 for strength design, or other generally accepted methodologies. Such methods are based on soil and rock parameters measured directly or inferred from the results of in situ and/or laboratory test.

4.2.2.3 Overall Stability

The overall stability of slopes in the vicinity of foundations shall be considered as part of the design of foundations.

4.2.3 SOIL, ROCK AND OTHER PROBLEM CONDITIONS

(Content of Article 4.2.3 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

4.3 SUBSURFACE EXPLORATION AND TESTING PROGRAMS (C)

(Content of Article 4.3 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

PART B

SERVICE LOAD DESIGN METHOD

ALLOWABLE STRESS DESIGN

4.4 SPREAD FOOTINGS

(Content of Article 4.4 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989)

4.5 DRIVEN PILES

(Content of Article 4.5 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989)

4.6 DRILLED SHAFTS

(Content of Article 4.6 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989)

4.7 REFERENCES

(Content of Article 4.7 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989)

PART C

STRENGTH DESIGN METHOD

LOAD FACTOR DESIGN

4.8 SCOPE (C)

Provisions of this section shall apply for the design of spread footings, driven piles, and drilled shaft foundations.

4.9 DEFINITIONS

Batter Pile- A pile driven at an angle inclined to the vertical to provide higher resistance to lateral loads.

Combination End-Bearing and Friction Pile - Pile that derives its capacities from the contributions of both end bearing developed at the pile tip and resistance mobilized along the embedded shaft.

Deep Foundation - A foundation which derives its support by transferring loads to soil or rock at some depth below the structure by end bearing, by adhesion or friction or both.

Design Load - All applicable loads and forces or their related internal moments and forces used to proportion a foundation. In load factor design, design load refers to nominal loads multiplied by appropriate load factors.

Design Strength - The maximum load-carrying capacity of the foundation, as defined by a particular limit state. In load factor design, design strength is computed as the product of the nominal resistance and the appropriate performance factor.

Drilled Shaft - A deep foundation unit, wholly or partly embedded in the ground, constructed by placing fresh concrete in a drilled hole with or without steel reinforcement. Drilled shafts derive their capacities from the surrounding soil and/or from the soil or rock strata below their tips. Drilled shafts are also commonly referred to as caissons, drilled caissons, bored piles or drilled piers.

End-Bearing Pile - A pile whose support capacity is derived principally from the resistance of the foundation material on which the pile tip rests.

Factored Load - Load, multiplied by appropriate load factors, used to proportion a foundation in load factor design.

Friction Pile - A pile whose support capacity is derived principally from soil resistance mobilized along the side of the embedded pile.

Limit State - A limiting condition in which the foundation and/or the structure it supports are deemed to be unsafe (i.e., strength limit state), or to be no longer fully useful for their intended function (i.e., serviceability limit state).

Load Effect - The force in a foundation system (e.g., axial force, sliding force, bending moment, etc.) due to the applied loads.

Load Factor - A factor used to modify a nominal load effect, which accounts for the uncertainties associated with the determination and variability of the load effect.

Load Factor Design - A design method in which safety provisions are incorporated by separately accounting for uncertainties relative to load and resistance.

Nominal Load - A typical value or a code-specified value for a load.

Nominal Resistance - The analytically estimated load-carrying capacity of a foundation calculated using nominal dimensions and material properties, and established soil mechanics principles.

Performance Factor - A factor used to modify a nominal resistance, which accounts for the uncertainties associated with the determination of the nominal resistance and the variability of the actual capacity.

Pile - A relatively slender deep foundation unit, wholly or partly embedded in the ground, installed by driving, drilling, augering, jetting or otherwise, and which derives its capacity from the surrounding soil and/or from the soil or rock strata below its tip.

Piping - Progressive erosion of soil by seeping water, producing an open pipe through the soil, through which water flows in an uncontrolled and dangerous manner.

Shallow Foundation - A foundation which derives its support by transferring load directly to the soil or rock at shallow depth. If a single slab covers the supporting stratum beneath the entire area of the superstructure, the foundation is known as a combined footing. If various parts of the structure are supported individually, the individual supports are known as spread footings, and the foundation is called a footing foundation.

4.10 LIMIT STATES, LOAD FACTORS AND RESISTANCE FACTORS

4.10.1 GENERAL

All relevant limit states shall be considered in the design to ensure an adequate degree of safety and serviceability.

4.10.2 SERVICEABILITY LIMIT STATES

Service limit states for foundation design shall include:

- settlements, and
- lateral displacements.

The limit state for settlement shall be based upon rideability and economy. The cost of limiting foundation movements shall be compared to the cost of designing the superstructure so that it can tolerate larger movements, or of correcting the consequences of movements through maintenance, to determine minimum lifetime cost. More stringent criteria may be established by the owner.

4.10.3 STRENGTH LIMIT STATES

Strength limit states for foundation design shall include:

- bearing resistance failure,
- excessive loss of contact,
- sliding at the base of footing,
- loss of overall stability, and
- structural capacity.

Foundations shall be proportioned such that the factored resistance is not less than the effects of factored loads specified in Section 3.

4.10.4 STRENGTH REQUIREMENT (C)

Foundations shall be proportioned by the methods specified in Articles 4.11 through 4.13 so that their design strengths are at least equal to the required strengths.

The required strength is the combined effect of the factored loads for each applicable load combination stipulated in Article 3.22. The design strength is calculated for each applicable limit state as the nominal resistance, R_n, multiplied by an appropriate performance (or resistance) factor, ϕ. Methods for calculating nominal resistance are provided in Articles 4.11 through 4.13, and values of performance factors are given in Article 4.10.6.

4.10.5 LOAD COMBINATIONS AND LOAD FACTORS

Foundations shall be proportioned to withstand safely all load combinations stipulated in Article 3.22 which are applicable to the particular site or foundation type. With the exception of the portions of concrete or steel piles that are above the ground line and are rigidly connected to the superstructure as in rigid frame or continuous structures, impact forces shall not be considered in foundation design (see Article 3.8.1).

Values of γ and β coefficients for load factor design, as given in Table 3.22.1A, shall apply to strength limit state considerations; while those for service load design (also given in Table 3.22.1A) shall apply to serviceability considerations.

4.10.6 PERFORMANCE FACTORS (C)

Values of performance factors for different types of foundation systems at strength limit states shall be as specified in Tables 4.10.6-1, 4.10.6-2, and 4.10.6-3, unless regionally specific values are available.

If methods other than those given in Tables 4.10.6-1, 4.10.6-2, and 4.10.6-3 are used to estimate the soil capacity, the performance factors chosen shall provide the same reliability as those given in these tables.

TABLE 4.10.6-1: Performance Factors for Strength Limit States for Shallow Foundations

Type of Limit State	Performance Factor
1. Bearing Capacity	
a. Sand	
- Semi-empirical Procedure using SPT data	0.45
- Semi-empirical Procedure using CPT data	0.55
- Rational Method --	
using ϕ_f estimated from SPT data	0.35
using ϕ_f estimated from CPT data	0.45
b. Clay	
- Semi-empirical Procedure using CPT data	0.50
- Rational Method	
using shear strength measured in lab tests	0.60
using shear strength measured in field vane tests	0.60
using shear strength estimated from CPT data	0.50
c. Rock	
- Semi-empirical Procedure (Carter and Kulhawy)	0.60
2. Sliding	
Sliding on clay is controlled by the strength of the clay when the clay shear strength is less than 0.5 times the normal stress, and is controlled by the normal stress when the clay shear strength is greater than 0.5 times the normal stress.	
a. Precast concrete placed on sand	
using ϕ_f estimated from SPT data	0.90
using ϕ_f estimated from CPT data	0.90
b. Concrete case in place on sand	
using ϕ_f estimated from SPT data	0.80
using ϕ_f estimated from CPT data	0.80
c. Clay (where shear strength is less than 0.5 times normal pressure)	
using shear strength measured in lab tests	0.85
using shear strength measured in field tests	0.85
using shear strength estimated from CPT data	0.80
d. Clay (where the strength is greater than 0.5 times normal pressure)	0.85

where ϕ_f = frictional angle of sand,
SPT = Standard Penetration Test,
CPT = Cone Penetration Test.

TABLE 4.10.6-2: Performance Factors for Geotechnical Strength Limit States in Axially Loaded Piles

	METHOD/SOIL/CONDITION		PERFORMANCE FACTOR
ULTIMATE BEARING CAPACITY OF SINGLE PILES	SKIN FRICTION	α-method	0.70
		β-method	0.50
		λ-method	0.55
	END BEARING	Clay (Skempton, 1951)	0.70
		Sand (Kulhawy, 1983)	
		ϕ_f from CPT	0.45
		ϕ_f from SPT	0.35
		Rock (Canadian Geotech. Society, 1985)	0.50
	SKIN FRICTION AND END BEARING	SPT-method	0.45
		CPT-method	0.55
		Load Test	0.80
		Pile Driving Analyzer	0.70
BLOCK FAILURE	Clay		0.65
UPLIFT CAPACITY OF SINGLE PILES	α-method		0.60
	β-method		0.40
	λ-method		0.45
	SPT-method		0.35
	CPT-method		0.45
	Load Test		0.80
GROUP UPLIFT CAPACITY	Sand		0.55
	Clay		0.55

TABLE 4.10.6-3: Performance Factors for Geotechnical Strength Limit States in Axially Loaded Drilled Shafts

	METHOD/SOIL/CONDITION		PERFORMANCE FACTOR
ULTIMATE BEARING CAPACITY OF SINGLE DRILLED SHAFTS	SIDE RESISTANCE IN CLAY	α-method (Reese & O'Neill)	0.65
	BASE RESISTANCE IN CLAY	Total Stress (Reese & O'Neill)	0.55
	SIDE RESISTANCE IN SAND	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See Discussion in Article 4.13.3.3.3
	BASE RESISTANCE IN SAND	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See Discussion in Article 4.13.3.3.3
	SIDE RESISTANCE IN ROCK	Carter & Kulhawy	0.55
		Horvath and Kenney	0.65
	BASE RESISTANCE IN ROCK	Canadian Geotechnical Society	0.50
		Pressuremeter Method (Canadian Geotechnical Society)	0.50
	SIDE RESISTANCE AND END BEARING	Load Test	0.80
BLOCK FAILURE	Clay		0.65

Table 4.10.6-3 Continued

	METHOD/SOIL/CONDITION		PERFORMANCE FACTOR
UPLIFT CAPACITY OF SINGLE DRILLED SHAFTS	CLAY	α-method (Reese & O'Neill)	0.55
		Belled Shafts (Reese & O'Neill)	0.50
	SAND	1) Touma & Reese 2) Meyerhof 3) Quiros & Reese 4) Reese & Wright 5) Reese & O'Neill	See Discussion in Section 4.13.3.3.3
	ROCK	Carter & Kulhawy	0.45
		Horvath & Kenney	0.55
		Load Test	0.80
GROUP UPLIFT CAPACITY		Sand	0.55
		Clay	0.55

4.11 SPREAD FOOTINGS

4.11.1 GENERAL CONSIDERATIONS

4.11.1.1 General (C)

Provisions of this Article shall apply to design of isolated footings and, where applicable, to combined footings. Special attention shall be given to footings on fill.

Footings shall be designed to keep the soil pressure as nearly uniform as practicable. The distribution of soil pressure shall be consistent with properties of the soil and the structure, and with established principles of soil mechanics.

4.11.1.2 Depth (Revised Article 4.4.5.1)

The depth of footings shall be determined with respect to the character of the foundation materials and the possibility of undermining. Footings at stream crossings shall be founded at depth below the maximum anticipated depth of scour as specified in Article 4.11.1.3.

Footings not exposed to the action of stream current shall be founded on a firm foundation and below frost level.

Consideration shall be given to the use of either a geotextile or graded granular filter layer to reduce susceptibility to piping in rip rap or abutment backfill.

4.11.1.3 Scour Protection (Revised Article 4.4.5.2) (C)

Footings supported on soil or degradable rock strata shall be embedded below the maximum computed scour depth or protected with a scour countermeasure. Footings supported on massive, competent rock formations which are highly resistant to scour shall be placed directly on the cleaned rock surface. Where required, additional lateral resistance shall be provided by drilling and grouting steel dowels into the rock surface rather than blasting to embed the footing below the rock surface.

4.11.1.4 Frost Action (C)

In region where freezing of the ground occurs during the winter months, footings shall be founded below the maximum

depth of frost penetration in order to prevent damage from frost heave.

4.11.1.5 Anchorage (same as Article 4.4.6) (C)

Footings which are founded on inclined smooth solid rock surfaces and which are not restrained by an overburden of resistant material shall be effectively anchored by means of rock anchors, rock bolts, dowels, keys or other suitable means. Shallow keying of large footing areas shall be avoided where blasting is required for rock removal.

4.11.1.6 Groundwater (C)

Footings shall be designed for the highest anticipated position of the groundwater table.

The influence of groundwater table on bearing capacity of soils or rocks, and settlements of the structure shall be considered. In cases where seepage forces are present, they should also be included in the analyses.

4.11.1.7 Uplift

Where foundations may be subjected to uplift forces, they shall be investigated both for resistance to pullout and for their structural strength.

4.11.1.8 Deterioration (C)

Deterioration of the concrete in a foundation by sulfate chloride and acid attack should be investigated. Laboratory testing of soil and groundwater samples for sulfates, chloride and pH should be sufficient to assess deterioration potential. When chemical wastes are suspected, a more thorough chemical analysis of soil and groundwater samples should be considered.

4.11.1.9 Nearby Structures (C)

In cases where foundations are placed adjacent existing structures, the influence of the existing structures on the behavior of the foundation, and the effect of the foundation on the existing structures, shall be investigated.

4.11.2 NOTATIONS

<u>Symbols</u>

B = footing width (in length units)

B' = reduced effective footing width (see Article 4.11.4.1.5) (in length units)

c = soil cohesion (in units of force/length^2)

C_{w1}, C_{w2} = correction factors for groundwater effect (dimensionless)

D_f = depth to footing base (in length units)

D_w = depth to groundwater table (in length units)

E_m = elastic modulus of rock masses (in units of force/length^2)

i = type of load

L' = reduced effective length (see Article 4.11.4.1.5) (in length units)

L_i = load type i

$\bar{N}$ = average value of standard penetration test blow count (dimensionless)

N_m, N_{cm}, N_{qm} = modified bearing capacity factors used in analytic theory (dimensionless)

q_c = cone resistance (in units of force/length^2)

q_{ult} = ultimate bearing capacity (in units of force/length^2)

R_I = reduction factor due to the effect of load inclination (dimensionless)

R_n = nominal resistance

RQD = rock quality designation

s = span length (in length units)

s_u = undrained shear strength of soil (in units of force/length^2)

<u>Greek</u>

β_i = load factor coefficient for load type i (see Article C 4.10.4)

γ = load factor (see Article C 4.10.4)

γ = total (moist) unit weight of soil (see Article C 4.11.4.1.1)

δ = differential settlement between adjacent footings

ϕ = performance factor

ϕ_f = friction angle of soil

4.11.3 MOVEMENT UNDER SERVICEABILITY LIMIT STATES

4.11.3.1 General (C)

Movement of foundations in both vertical settlement and lateral displacement directions shall be investigated at service limit states.

Lateral displacement of a structure shall be evaluated when:

- horizontal or inclined loads are present,
- the foundation is placed on embankment slope,
- possibility of loss of foundation support through erosion or scour exists, or
- bearing strata are significantly inclined.

4.11.3.2 Loads

Immediate settlement shall be determined using the service load combinations given in Table 3.22.1A. Time-dependent settlement shall be determined using only the permanent loads.

Settlements and horizontal movements caused by embankment loadings behind bridge abutments should be investigated.

In seismically active areas, consideration shall be given to the potential settlements of footings on sand resulting from ground motions induced by earthquake loadings. For guidance in design, refer to AASHTO Guide Specifications for Seismic Design of Highway Bridges (1983).

4.11.3.3 Movement Criteria (Revised Article 4.4.7.2.5) (C)

Vertical and horizontal movement criteria for footings shall be developed consistent with the function and type of structure, anticipated service life, and consequences of unacceptable movements on structure performance. The tolerable movement criteria shall be established by empirical procedures or structural analyses.

The maximum angular distortion (δ/s) between adjacent foundations shall be limited to 0.008 for simple span bridges and 0.004 for continuous span bridges. These δ/s limits shall not be applicable to rigid frame structures. Rigid frames shall be designed for anticipated differential settlements based on the results of special analyses.

4.11.3.4 Settlement Analyses (C)

Foundation settlements shall be estimated using deformation analyses based on the results of laboratory or in situ testing. The soil parameters used in the analyses shall be chosen to reflect the loading history of the ground, the construction sequence and the effect of soil layering.

Both total and differential settlements, including time effect, shall be considered.

4.11.3.4.1 Settlement of Footings on Cohesionless Soils (C)

Estimates of settlement of cohesionless soils shall make allowance for the fact that settlements in these soils can be highly erratic.

No method should be considered capable of predicting settlements of footings on sand with precision.

Settlements of footings on cohesionless soils may be estimated using empirical procedures or elastic theory.

4.11.3.4.2 Settlement of Footings on Cohesive Soils (C)

For foundations on cohesive soils, both immediate and consolidation settlements shall be investigated. If the footing width is small relative to the thickness of a compressible soil, the effect of three-dimensional loading shall be considered. In highly plastic and organic clay, secondary settlements are significant and shall be included in the analysis.

4.11.3.4.3 Settlements of Footings on Rock (C)

The magnitude of consolidation and secondary settlements in rock masses containing soft seams shall be estimated by applying procedures discussed in Article 4.11.3.4.2.

4.11.4 SAFETY AGAINST SOIL FAILURE

4.11.4.1 Bearing Capacity of Foundation Soils

Several methods may be used to calculate ultimate bearing capacity of foundation soils. The calculated value of ultimate bearing capacity shall be multiplied by an appropriate performance factor, as given in Article 4.10.6, to determine the factored bearing capacity.

Footings are considered to be adequate against soil failure if the factored bearing capacity exceeds the effect of design loads.

4.11.4.1.1 Theoretical Estimation (C)

The bearing capacity should be estimated using accepted soil mechanics theories based on measured soil parameters. The soil parameter used in the analysis shall be representative of the soil shear strength under the considered loading and subsurface conditions.

4.11.4.1.2 Semi-empirical Procedures (C)

The bearing capacity of foundation soils may be estimated from the results of in situ tests or by observing foundations on similar soils. The use of a particular in situ test and the interpretation of the results shall take local experience into consideration. The following in situ tests may be used:

- Standard penetration test (SPT),
- Cone penetrometer test (CPT), and
- Pressuremeter test.

4.11.4.1.3 Plate Loading Test (C)

Bearing capacity may be determined by load tests providing that adequate subsurface explorations have been made to determine the soil profile below the foundation.

The bearing capacity determined from a load test may be extrapolated to adjacent footings where the subsurface profile is similar.

Plate load test shall be performed in accordance with the procedures specified in ASTM Standard D1194-87 or AASHTO Standard T235-74.

4.11.4.1.4 Presumptive Values (Revised Article 4.4.4.1)

Presumptive values for allowable bearing pressures on soil and rock, given in Table 4.11.4.1.4-1, shall be used only for guidance, preliminary design or design of temporary structures. The use of presumptive values shall be based on the results of subsurface exploration to identify soil and rock conditions. All values used for design shall be confirmed by field and/or laboratory testing.

The values given in Table 4.11.4.1.4-1 are applicable directly for working stress procedures. When these values are used for preliminary design, all load factors shall be taken as unity.

4.11.4.1.5 Effect of Load Eccentricity (Revised Article 4.4.7.1.1.1) (C)

For loads eccentric to the centroid of the footing, a reduced effective footing area (B′ x L′) shall be used in design. The reduced effective area is always concentrically loaded, so that the design bearing pressure on the reduced effective area is always uniform.

Footings under eccentric loads shall be designed to ensure that: (1) the product of the bearing capacity and an appropriate performance factor exceeds the effect of vertical design loads, and (2) eccentricity of loading, evaluated based on factored loads, is less than 1/4 of the footing dimension in any direction for footings on soils.

For structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact pressure distribution based on factored loads shall be used.

4.11.4.1.6 Effect of Groundwater Table (C)

Ultimate bearing capacity shall be determined based on the highest anticipated position of groundwater level at the

Table 4.11.4.1.4-1: Presumptive Allowable Bearing Pressures for Spread Footing Foundations (Modified after U.S. Department of the Navy, 1982)

Type of Bearing Material	Consistency in Place	Allowable Bearing Pressure (tsf) Ordinary Range	Allowable Bearing Pressure (tsf) Recommended Value for Use
Massive crystalline igneous and metamorphic rock: graphite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	60 to 100	80
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	30 to 40	35
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	15 to 25	20
Weathered or broken bedrock of any kind exept highly argillaceous rock (shale)	Medium hard rock	8 to 12	10
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	8 to 12	10
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC,SC)	Very dense	8 to 12	10
Gravel, gravel-sand mixtures, boulder-gravel mixtures (GW,GP, SW, SP)	Very dense Medium dense to dense Loose	6 to 10 4 to 7 2 to 6	7 5 3
Coarse to medium sand, sand with little gravel. (SW,SP)	Very dense Medium dense to dense Loose	4 to 6 2 to 4 1 to 3	4 3 1.5
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	3 to 5 2 to 4 1 to 2	3 2.5 1.5

Table 4.11.4.1.4-1 (continued)

Type of Bearing Material	Consistency in Place	Allowable Bearing Pressure (tsf) Ordinary Range	Allowable Bearing Pressure (tsf) Recommended Value for use
Fine sand, silty or clayey medium to fine sand (SP,SM,SC)	Very dense medium dense to dense Loose	3 to 5 2 to 4 1 to 2	3 2.5 1.5
Homogeneous inorganic clay, sandy or silty clay (CL,CH)	Very stiff to hard Medium stiff to stiff Soft	3 to 6 1 to 3 0.5 to 1	4 2 0.5
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML,MH)	Very stiff to hard Medium stiff to stiff Soft	2 to 4 1 to 3 0.5 to 1	3 1.5 0.5

footing location. In cases where the groundwater table is at depth less than 1.5 times the footing width below the bottom of the footing, reduction of bearing capacity, as a result of submergence effect, shall be considered.

4.11.4.2 Bearing Capacity of Foundations on Rock

The bearing capacity of footings on rock shall consider the presence, orientation and condition of discontinuities, weathering profiles and other similar profiles as they apply at a particular site, and the degree to which they shall be incorporated in the design.

For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and RQD may be applicable. Competent rock shall be defined as a rock mass with discontinuities that are tight or open not wider than one-eighth inch. For footings on less competent rock, more detailed investigations and analyses shall be performed to account for the effects of weathering, and the presence and condition of discontinuities.

Footings on rocks are considered to be adequate against bearing capacity failure if the product of the ultimate bearing capacity determined using procedures described in Articles 4.11.4.2.1 through 4.11.4.2.3 and an appropriate performance factor exceeds the effect of design loads.

4.11.4.2.1 Semi-empirical Procedures (C)

Bearing capacity of foundations on rock may be determined using empirical correlation with RQD, or other systems for evaluating rock mass quality, such as the Geomechanic Rock Mass Rating (RMR) system, or Norwegian Geotechnical Institute (NGI) Rock Mass Classification System. The use of these semi-empirical procedures shall take local experience into consideration.

4.11.4.2.2 Analytic Method (C)

The ultimate bearing capacity of foundations on rock shall be determined using established rock mechanics principles based on the rock mass strength parameters. The influence of discontinuities on the failure mode shall also be considered.

4.11.4.2.3 Load Test

Where appropriate, load tests may be performed to determine the bearing capacity of foundations on rock.

4.11.4.2.4 Presumptive Bearing Values

For simple structures on good quality rock masses, values of presumptive bearing pressure given in Table 4.11.4.2.4-1 may be used for preliminary design. The use of presumptive values shall be based on the results of subsurface exploration to identify rock conditions. All values used in design shall be confirmed by field and/or laboratory testing. The values given in Table 4.11.4.2.4-1 are directly applicable to working stress procedure, i.e., all the load factors shall be taken as unity.

4.11.4.2.5 Effect of Load Eccentricity

If the eccentricity of loading on a footing is less than 1/6 of the footing width, a trapezoidal bearing pressure shall be used in evaluating the bearing capacity. If the eccentricity is between 1/6 and 1/4 of the footing width, a triangular bearing pressure shall be used. The maximum bearing pressure shall not exceed the product of the ultimate bearing capacity multiplied by a suitable performance factor. The eccentricity of loading evaluated using factored loads shall not exceed 3/8 (37.5%) of the footing dimensions in any direction.

4.11.4.3 Failure by Sliding (C)

Failure by sliding shall be investigated for footings that support inclined loads and/or are founded on slopes.

For foundations on clay soils, possible presence of a shrinkage gap between the soil and the foundation shall be considered. If passive resistance is included as part of the shear resistance required for resisting sliding, consideration shall also be given to possible future removal of the soil in front of the foundation.

4.11.4.4 Loss of Overall Stability (Revised Article 4.4.9) (C)

The overall stability of footings, slopes and foundation soil or rock, shall be evaluated for footings located on or

Table 4.11.4.2.4-1: Presumptive Bearing Pressures (tsf) for Foundations on Rock (After Putnam, 1981)

Code	Year [1]	Bedrock [2]	Sound Foliated Rock	Sound Sedimentary Rock	Soft Rock [3]	Soft Shale	Broken Shale
Baltimore	1962	100	35	---	10	---	(4)
BOCA	1970	100	40	25	10	4	1.5
Boston	1970	100	50	10	10	---	(4)
Chicago	1970	100	100	---	---	---	---
Cleveland	1951/1969	---	---	25	---	---	---
Dallas	1968	$.2q_u$ (5)	$2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
Detroit	1956	100	100	9600	12	12	---
Indiana	1967	$.2q_u$	$2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
Kansas City	1961/1969	$.2q_u$	$2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
Los Angeles	1970	10	4	3	1	1	1
New York City	1970	60	60	60	8	---	---
New York State	---	100	40	15	---	---	---
Ohio	1970	100	40	15	10	4	---
Philadelphia	1969	50	15	10-15	8	---	---
Pittsburgh	1959/1969	25	25	25	8	8	---
Richmond	1968	100	40	25	10	4	1.5
St. Louis	1960/1970	100	40	25	10	1.5	1.5
San Francisco	1969	3-5	3-5	3-5	---	---	---
Uniform Building Code	1970	$.2q_u$	$2q_u$	$.2q_u$	$.2q_u$	$.2q_u$	$.2q_u$
NBC Canada	1970	---	---	100	---	---	---
New South Wales, Australia	1974	---	---	33	13	4.5	---

Note: 1 - Year of code or original year and date of revision
2 - Massive crystalline bedrock
3 - Soft and broken rock, not including shale
4 - Allowable bearing pressure to be determined by appropriate city official
5 - q_u = unconfined compressive strength

near a slope using applicable factored load combinations in Article 3.22 and a performance factor of 0.75.

4.11.5 STRUCTURAL CAPACITY

The structural design of footings shall comply to the provisions given in Article 4.4.11 and Article 8.16.

4.11.6 CONSTRUCTION CONSIDERATIONS FOR SHALLOW FOUNDATIONS

4.11.6.1 General

The ground conditions should be monitored closely during construction to determine whether or not the ground conditions are as foreseen and to enable prompt intervention, if necessary. The control investigation should be performed and interpreted by experienced and qualified engineers. Records of the control investigations should be kept as part of the final project data, among other things, to permit a later assessment of the foundation in connection with rehabilitation, change of neighboring structures, etc.

4.11.6.2 Excavation Monitoring

Prior to concreting footings or placing backfill, an excavation shall be free of debris and excessive water.

Monitoring by an experienced and trained person should always include a thorough examination of the sides and bottom of the excavation, with the possible addition of pits or borings to evaluate the geological conditions.

The assumptions made during the design of the foundations regarding strength, density, and groundwater conditions should be verified during construction, by visual inspection.

4.11.6.3 Compaction Monitoring

Compaction shall be carried out in a manner so that the fill material within the section under inspection is as close as practicable to uniform. The layering and compaction of the fill material should be systematic everywhere, with the same thickness of layer and number of passes with the compaction equipment used as for the inspected fill. The control measurements should be undertaken in the form of random samples.

4.12 DRIVEN PILES

4.12.1 GENERAL

The provisions of the specifications in Articles 4.5.1 through 4.5.21 with the exception of Article 4.5.6, shall apply to strength design (load factor design) of driven piles. Article 4.5.6 covers the allowable stress design of piles and shall be replaced by the articles in this section for load factor design of driven piles, unless otherwise stated.

4.12.2 NOTATIONS

a_s	= pile perimeter
A_p	= area of pile tip
A_s	= surface area of shaft of pile
CPT	= cone penetration test
d	= dimensionless depth factor for estimating tip capacity of piles in rock
D	= pile width or diameter
D′	= effective depth of pile group
D_b	= depth of embedment of pile into a bearing stratum
D_s	= diameter of socket
e_x	= eccentricity of load in the x-direction
e_y	= eccentricity of load in the y-direction
E_p	= Young's modulus of pile or drilled shaft
E_s	= soil modulus
f_s	= sleeve friction measured from a CPT at point considered
H	= distance between pile tip and a weaker underlying soil layer
H_s	= depth of embedment of pile socketed into rock
I	= influence factor for the effective group embedment
I_p	= moment of inertia of a pile
K	= coefficient of lateral earth pressure
K_c	= correction factor for sleeve friction in clay
K_s	= correction factor for sleeve friction in sand
K_{sp}	= dimensionless bearing capacity coefficient
L_f	= depth to point considered when measuring sleeve friction
n_h	= rate of increase of soil modulus with depth
N	= Standard Penetration Test (SPT) blow count
$\overline{N}$	= average uncorrected SPT blow count along pile shaft
N_{corr}	= average SPT-N value corrected for effect of overburden
N_{pile}	= number of piles in a pile group
OCR	= overconsolidation ratio

P_D	= unfactored dead load
P_g	= factored total axial load acting on a pile group
$P_{x,y}$	= factored axial load acting on a pile in a pile group; the pile has coordinates (X,Y) with respect to the centroidal origin in the pile group
PI	= plasticity index
q	= net foundation pressure
q_c	= static cone resistance
q_1	= limiting tip resistance
q_o	= limiting tip resistance in lower stratum
q_p	= ultimate unit tip resistance
q_s	= ultimate unit side resistance
q_u	= average uniaxial compressive strength of rock cores
q_{ult}	= ultimate bearing capacity
Q_p	= ultimate load carried by tip of pile
Q_s	= ultimate load carried by shaft of pile
Q_{ug}	= ultimate uplift resistance of a pile group or a group of drilled shafts
Q_{ult}	= ultimate bearing capacity
R	= characteristic length of soil-pile system in cohesive soils
s_d	= spacing of discontinuities
S	= average spacing of piles
S_u	= undrained shear strength
SPT	= Standard Penetration Test
$\overline{S}_u$	= average undrained shear strength along pile shaft
t_d	= width of discontinuities
T	= characteristic length of soil-pile system in cohesionless soils
W_g	= weight of block of soil, piles and pile cap
x	= distance of the centroid of the pile from the centroid of the pile cap in the x-direction
X	= width of smallest dimension of pile group
y	= distance of the centroid of the pile from the centroid of the pile cap in the y-direction
Y	= length of pile group or group of drilled shafts
Z	= total embedded pile length
α	= adhesion factor applied to S_u
β	= coefficient relating the vertical effective stress and the unit skin friction of a pile or drilled shaft
γ'	= effective unit weight of soil
δ	= angle of shearing resistance between soil and pile
λ	= empirical coefficient relating the passive lateral earth pressure and the unit skin friction of a pile
η	= pile group efficiency factor
ρ	= settlement
ρ_{tol}	= tolerable settlement
σ_h'	= horizontal effective stress
σ_v'	= vertical effective stress
τ_{av}	= average shear stress along side of pile
ϕ	= performance factor

ϕg = performance factor for the bearing capacity of a pile group failing as a unit consisting of the piles and the block of soil contained within the piles
ϕq = performance factor for the total ultimate bearing capacity of a pile
ϕqs = performance factor for the ultimate shaft capacity of a pile
ϕqp = performance factor for the ultimate tip capacity of a pile
ϕu = Performance factor for the uplift capacity of a single pile
ϕug = performance factor for the uplift capacity of pile groups

4.12.3 SELECTION OF DESIGN PILE CAPACITY

Piles shall be designed to have adequate bearing and structural capacity, under tolerable settlements and tolerable lateral displacements.

The supporting capacity of piles shall be determined by static analysis methods based on soil-structure interaction. Capacity may be verified with pile load test results, use of wave equation analysis, use of the dynamic pile analyzer or, less preferably, use of dynamic formulas.

4.12.3.1 Factors Affecting Axial Capacity

See Article 4.5.6.1.1. The following sub-articles shall supplement Article 4.5.6.1.1.

4.12.3.1.1 Pile Penetration

Piling used to penetrate a soft or loose upper stratum overlying a hard or firm stratum, shall penetrate the hard or firm stratum by a sufficient distance to limit lateral and vertical movement of the piles, as well as to attain sufficient vertical bearing capacity.

4.12.3.1.2 Groundwater Table And Buoyancy (C)

Ultimate bearing capacity shall be determined using the groundwater level consistent with that used to calculate load effects. For drained loading, the effect of hydrostatic pressure shall be considered in the design.

4.12.3.1.3 Effect Of Settling Ground And Downdrag Forces (C)

Possible development of downdrag loads on piles shall be considered where sites are underlain by compressible clays, silts or peats, especially where fill has recently been placed on the earlier surface, or where the groundwater is substantially lowered. Downdrag loads shall be considered as a load when the bearing capacity and settlement of pile foundations are investigated. Downdrag loads shall not be combined with transient loads.

The downdrag loads may be calculated, as specified in Article 4.12.3.3.2 with the direction of the skin friction forces reversed. The factored downdrag loads shall be added to the factored veritcal dead load appied to the deep foundation in the assessment of bearing capacity. The effect of reduced overburden pressure caused by the downdrag shall be considered in calculating the bearing capacity of the foundation.

The downdrag loads shall be added to the vertical dead load applied to the deep foundation in the assessment of settlement at service limit states.

4.12.3.1.4 Uplift

Pile foundations designed to resist uplift forces should be checked both for resistance to pullout and for structural capacity to carry tensile stresses. Uplift forces can be caused by lateral loads, buoyancy effects and expansive soils.

4.12.3.2 Movement Under Serviceability Limit State

4.12.3.2.1 General

For purposes of calculating the settlements of pile groups, loads shall be assumed to act on an equivalent footing located at two thirds of the depth of embedment of the piles into the layer which provide support as shown in Figure 4.12.3.2.1-1.

Service loads for evaluating foundation settlement shall include both the unfactored dead and live loads for piles in cohesionless soils and only the unfactored dead load for piles in cohesive soils.

Service loads for evaluating lateral displacement of foundations shall include all lateral loads in each of the load combinations as given in Article 3.22.

4.12.3.2.2 Tolerable Movement

Tolerable axial and lateral movements for driven pile foundations shall be developed consistent with the function and type of structure, fixity of bearings, anticipated service life and consequences of unacceptable displacements on performance of the structure.

Tolerable settlement criteria for foundations shall be developed considering the maximum angular distortion according to Article 4.11.3.3.

Tolerable horizontal displacement criteria shall be developed considering the potential effects of combined vertical and horizontal movement. Where combined horizontal and vertical displacements are possible, horizontal movement shall be limited to 1.0 in. or less. Where vertical displacements are small, horizontal displacements shall be limited to 2.0 in. or less (Moulton et al., 1985). If estimated or actual movements exceed these levels, special analysis and/or measures shall be considered.

4.12.3.2.3 Settlement

The settlement of a pile foundation shall not exceed the tolerable settlement, as selected according to Article 4.12.3.2.2.

4.12.3.2.3a Cohesive Soil

Procedures used for shallow foundations shall be used to estimate the settlement of a pile group, using the equivalent footing location shown in Figure 4.12.3.2.1-1.

4.12.3.2.3b Cohesionless Soil (C)

The settlement of pile groups in cohesionless soils can be estimated using results of in situ tests, and the equivalent footing location shown in Figure 4.12.3.2.1-1.

4.12.3.2.4 Lateral Displacement (C)

The lateral displacement of a pile foundation shall not exceed the tolerable lateral displacement, as selected according to Article 4.12.3.2.2.

The lateral displacement of pile groups shall be estimated using procedures that consider soil-structure interaction.

4.12.3.3 Resistance at Strength Limit States

The strength limit states that shall be considered include:

- bearing capacity of piles,
- uplift capacity of piles,
- punching of piles in strong soil into a weaker layer, and
- structural capacity of the piles.

4.12.3.3.1 Axial Loading Of Piles (C)

Preference shall be given to a design process based upon static analyses in combination with either field monitoring during driving or load tests. Load test results may be extrapolated to adjacent substructures with similar subsurface conditions. The ultimate bearing capacity of piles may be estimated using analytic methods or in situ test methods.

4.12.3.3.2 Analytic Estimates Of Pile Capacity (C)

Analytic methods may be used to estimate the ultimate bearing capacity of piles in cohesive and cohesionless soils. Both total and effective stress methods may be used provided the appropriate soil strength parameters are evaluated. The performance factors for skin friction and tip resistance, estimated using three analytic methods, shall be as provided in Table 4.10.6-2. If another analytic method is used, application of performance factors presented in Table 4.10.6-2 may not be appropriate.

4.12.3.3.3 Pile Capacity Estimates Based On In Situ Tests (C)

In situ test methods may be used to estimate the ultimate axial capacity of piles. The performance factors for the ultimate skin friction and ultimate tip resistance, estimated using in situ methods, shall be as provided in Table 4.10.6-2.

4.12.3.3.4 Piles Bearing On Rock (C)

For piles driven to weak rocks such as shales and mudstones or poor quality weathered rocks, the ultimate tip capacity shall be estimated using semi-empirical methods. The performance factor for the ultimate tip resistance of piles bearing on rock shall be as provided in Table 4.10.6-2.

4.12.3.3.5 Pile Load Test (C)

The load test method specified in ASTM D1143-81 may be used to verify the pile capacity. Tensile load testing of piles shall be done in accordance with ASTM D3689-83 Lateral load testing of piles shall be done in accordance with ASTM D3966-81. The performance factor for the axial compressive capacity, axial uplift capacity and lateral capacity obtained from pile load tests shall be as provided in Table 4.10.6-2.

4.12.3.3.6 Presumptive End Bearing Capacities

Presumptive values for allowable bearing pressures given in Table 4.11.4.1.4-1 on soil and rock shall be used only for guidance, preliminary design or design of temporary structures. The use of presumptive values shall be based on the results of subsurface exploration to identify soil and rock conditions. All values used for design shall be confirmed by field and/or laboratory testing.

4.12.3.3.7 Uplift

Uplift shall be considered when the force effects calculated based on the appropriate strength limit state load combinations are tensile.

When piles are subjected to uplift, they should be investigated for both resistance to pullout and structural ability to resist tension.

4.12.3.3.7a Single Pile Uplift Capacity (C)

The ultimate uplift capacity of a single pile shall be estimated in a manner similar to that for estimating the skin friction resistance of piles in compression in Article 4.12.3.3.2 for piles in cohesive soils and 4.12.3.3.3 for piles in cohesionless soils. Performance factors for the uplift capacity of single piles shall be as provided in Table 4.10.6-2.

4.12.3.3.7b Pile Group Uplift Capacity (C)

The ultimate uplift capacity of a pile group shall be estimated as the lesser of the sum of the individual pile uplift capacities, or the uplift capacity of the pile group considered as a block. The block mechanism for cohesionless soil shall be taken as provided in Figure C4.12.3.3.7b-1 and for cohesive soils as given in Figure C4.12.3.3.7b-2. Buoyant unit weights shall be used for soil below the groundwater level.

The performance factor for the group uplift capacity calculated as the sum of the individual pile capacities shall be the same as those for the uplift capacity of single piles as given in Table 4.10.6-2. The performance factor for the uplift capacity of the pile group considered as a block shall be as provided in Table 4.10.6-2 for pile groups in clay and in sand.

4.12.3.3.8 Lateral Load (C)

For piles subjected to lateral loads, the pile heads shall be fixed into the pile cap. Any disturbed soil or voids created from the driving of the piles shall be replaced with compacted granular material.

The effects of soil-structure or rock-structure interaction between the piles and ground, including the number and spacing or the piles in the group, shall be accounted for in the design of laterally loaded piles.

4.12.3.3.9 Batter Pile (C)

The bearing capacity of a pile group containing batter piles may be estimated by treating the batter piles as vertical piles.

4.12.3.3.10 Group Capacity

4.12.3.3.10a Cohesive Soil (C)

If the cap is not in firm contact with the ground, and if the soil at the surface is soft, the individual capacity of each pile shall be multiplied by an efficiency factor η, where $\eta = 0.7$ for a center-to-center spacing of three diameters and $\eta = 1.0$ for a center-to-center spacing of six diameters. For intermediate spacings, the value of η may be determined by linear interpolation.

If the cap is not in firm contact with the ground and if the soil is stiff, then no reduction in efficiency shall be required.

If the cap is in firm contact with the ground, then no reduction in efficiency shall be required.

The group capacity shall be the lesser of:

- the sum of the modified individual capacities of each pile in the group or
- the capacity of an equivalent pier consisting of the piles and a block of soil within the area bounded by the piles.

For the equivalent pier, the full shear strength of soil shall be used to determine the skin friction resistance, the total base area of the equivalent pier shall be used to determine the end bearing resistance, and the additional capacity of the cap shall be ignored.

The performance factor for the capacity of an equivalent pier or block failure shall be as provided in Table 4.10.6-2. The performance factors for the group capacity calculated using the sum of the individual pile capacities, are the same as those for the single pile capacity as given in Table 4.10.6-2.

4.12.3.3.10b Cohesionless Soil (C)

The ultimate bearing capacity of pile groups in cohesionless soil shall be the sum of the capacities of all the piles in the group. The efficiency factor, η, shall be 1.0 where the pile cap is, or is not in contact with the ground. The performance factor is the same as those for single pile capacities as given in Table 4.10.6-2.

4.12.3.3.10c Pile Group in Strong Soil Overlying a Weak or Compressible Soil (C)

If a pile group is embedded in a strong soil deposit overlying a weaker deposit, consideration shall be given to the potential for a punching failure of the pile tips into the weaker soil stratum. If the underlying soil stratum consists of a weaker compressible soil, consideration shall be given to the potential for large settlements in that weaker layer.

4.12.3.3.11 Dynamic/Seismic Design

Refer to Guide Specifications for Seismic Design of Highway Bridges (1983) and Lam and Martin (1986a, 1986b) for guidance regarding the design of driven piles subjected to dynamic and seismic loads.

4.12.4 STRUCTURAL DESIGN

The structural design of driven piles shall be in accordance with the provisions of Articles 4.5.7, which was developed for allowable stress design procedures. To use load factor design procedures for the structural design of driven piles, the load factor design procedures for reinforced concrete, prestressed concrete and steel in Sections 8, 9 and 10, respectively, shall be used in place of the allowable stress design procedures.

4.12.4.1 Buckling Of Piles (C)

Stability of piles shall be considered when the piles extend through water or air for a portion of their lengths.

4.12.5 CONSTRUCTION CONSIDERATIONS

Foundation design shall not be uncoupled from construction considerations. Factors such as pile driving, pile splicing and pile inspection shall be done in accordance with the provisions of this specification and Division II.

4.13 DRILLED SHAFTS

4.13.1 GENERAL

The provisions of the specifications in Articles 4.6.1 through 4.6.7 with the exception of Article 4.6.5, shall apply to the strength design (load factor design) of drilled shafts. Article 4.6.5 covers the allowable stress design of drilled shafts, and shall be replaced by the articles in this section for load factor design of drilled shafts, unless otherwise stated.

The provisions of Article 4.13 shall apply to the design of drilled shafts, but not drilled piles installed with continuous flight augers that are concreted as the auger is being extracted.

4.13.2 NOTATIONS

a = parameter used for calculating F_r
A_p = area of base of drilled shaft
A_s = surface area of a drilled pier
A_{soc} = cross-sectional area of socket
A_u = annular space between bell and shaft
b = perimeter used for calculating F_r
CPT = cone penetration test
d = dimensionless depth factor for estimating tip capacity of drilled shafts in rock
D = diameter of drilled shaft
D_b = embedment of drilled shaft in layer that provides support
D_p = diameter of base of a drilled shaft
D_s = diameter of a drilled shaft socket in rock
E_c = Young's modulus of concrete
E_i = intact rock modulus
E_p = Young's modulus of a drilled shaft
E_r = modulus of the in situ rock mass
E_s = soil modulus
F_r = reduction factor for tip resistance of large diameter drilled shaft
H_s = depth of embedment of drilled shaft socketed into rock
I_p = moment of inertia of a drilled shaft
I_ρ = influence coefficient (see Figure C4.13.3.3.4-1)
I_π = influence coefficient for settlement of drilled shafts socketed in rock
k = factor that reduces the tip capacity for shafts with a base diameter larger than 20 inches so as to limit the shaft settlement to 1 inch
K = coefficient of lateral earth pressure or load transfer factor

K_b = dimensionless bearing capacity coefficient for drilled shafts socketed in rock using pressuremeter results
K_E = modulus modification ratio
K_{sp} = dimensionless bearing capacity coefficient (see Figure C4.13.3.3.4-4)
LL = liquid limit of soil
N = uncorrected Standard Penetration Test (SPT) blow count
N_c = bearing capacity factor
N_{corr} = corrected SPT-N value
N_u = uplift bearing capacity factor
p_l = limit pressure determined from pressuremeter tests within 2D above and below base of shaft
p_o = at rest horizontal stress measured at the base of drilled shaft
P_D = unfactored dead load
PL = plastic limit of soil
q_p = ultimate unit tip resistance
q_{pr} = reduced ultimate unit tip resistance of drilled shafts
q_s = ultimate unit side resistance
$q_{s\ bell}$ = unit uplift capacity of a belled drilled shaft
q_u = uniaxial compressive strength of rock core
q_{ult} = ultimate bearing capacity
Q_p = ultimate load carried by tip of drilled shaft
Q_s = ultimate load carried by side of drilled shaft
Q_{SR} = ultimate side resistance of drilled shafts socketed in rock
Q_{ult} = total ultimate bearing capacity
R = characteristic length of soil-drilled shaft system in cohesive soils
RQD = Rock Quality Designation
s_d = spacing of discontinuities
SPT = Standard Penetration Test
S_u = undrained shear strength
t_d = width of discontinuities
T = characteristic length of soil-drilled shaft system in cohesionless soils
z = depth below ground surface
Z = total embedded length of drilled shaft

<u>Greek</u>

α = adhesion factor applied to S_u
β = coefficient relating the vertical effective stress and the unit skin friction of a drilled shaft
γ' = effective unit weight of soil
δ = angle of shearing resistance between soil and pile
η = pile group efficiency factor
ρ_{base} = settlement of the base of the drilled shaft

ρ_e = elastic shortening of drilled shaft
ρ_{tol} = tolerable settlement
σ_v' = vertical effective stress
σ_v = total vertical stress
ΣP_i = working load at top of socket
ϕ = performance factor
ϕ' or ϕ_f = angle of internal friction of soil
ϕ_q = performance factor for the total ultimate bearing capacity of a drilled shaft
ϕ_{qs} = performance factor for the ultimate shaft capacity of a drilled shaft
ϕ_{qp} = performance factor for the ultimate tip capacity of a drilled shaft

4.13.3 GEOTECHNICAL DESIGN (C)

Drilled shafts shall be designed to have adequate bearing and structural capacities under tolerable settlements and tolerable lateral movements.

The supporting capacity of drilled shafts shall be estimated by static analysis methods (analytical methods based on soil-structure interaction). Capacity may be verified with load test results.

The method of construction may affect the drilled shaft capacity and shall be considered as part of the design process. Drilled shafts may be constructed using the dry, casing or wet method of construction, or a combination of methods. In every case, hole excavation, concrete placement and all other aspects shall be performed in conformance with the provisions of this specification and Division II.

4.13.3.1 Factors Affecting Axial Capacity

See Article 4.6.5.2 for drilled shafts in soil and Article 4.6.5.3.3 for drilled shafts in rock. The following sub-articles shall supplement Articles 4.6.5.2 and 4.6.5.3.3.

4.13.3.1.1 Downdrag Loads (C)

Downdrag loads shall be evaluated, where appropriate, as indicated in Article 4.12.3.1.3.

4.13.3.1.2 Uplift (C)

The provisions of Article 4.12.3.1.4 shall apply as applicable.

Shafts designed for and constructed in expansive soil shall extend for a sufficient depth into moisture-stable soils to provide adequate anchorage to resist uplift. Sufficient clearance shall be provided between the ground surface and underside of caps or beams connecting shafts to preclude the application of uplift loads at the shaft/cap connection due to swelling ground conditions. Uplift capacity of straight sided drilled shafts shall rely only on side resistance in conformance with Article 4.13.3.3.2 for drilled shafts in cohesive soils, and Article 4.13.3.3.3 for drilled shafts in cohesionless soils. If the shaft has an enlarged base, Q_s shall be determined in conformance with Article 4.13.3.3.6.

4.13.3.2 Movement Under Serviceability Limit State

4.13.3.2.1 General (C)

The provisions of Article 4.12.3.2.1 shall apply as applicable.

In estimating settlements of drilled shafts in clay, only unfactored permanent loads shall be considered. However unfactored live loads must be added to the permanent loads when estimating settlement of shafts in granular soil.

4.13.3.2.2 Tolerable Movement

The provisions of Article 4.12.3.2.2 shall apply as applicable.

4.13.3.2.3 Settlement

The settlement of a drilled shaft foundation involving either single drilled shafts or groups of drilled shafts shall not exceed the tolerable settlement as selected according to Article 4.13.3.2.2

4.13.3.2.3a Settlement of Single Drilled Shafts (C)

The settlement of single drilled shafts shall be estimated considering short-term settlement, consolidation settlement (if

constructed in cohesive soils), and axial compression of the drilled shaft.

4.13.3.2.3b Group Settlement

The settlement of groups of drilled shafts shall be estimated using the same procedures as described for pile groups, Article 4.12.3.2.3.

- Cohesive Soil, See Article 4.12.3.2.3a

- Cohesionless Soil, See Article 4.12.3.2.3b

4.13.3.2.4 Lateral Displacement (C)

The provisions of Article 4.12.3.2.4 shall apply as applicable.

4.13.3.3 Resistance at Strength Limit States (C)

The strength limit states that must be considered include: 1) bearing capacity of drilled shafts, 2) uplift capacity of drilled shafts, and 3) punching of drilled shafts bearing in strong soil into a weaker layer below.

4.13.3.3.1 Axial Loading Of Drilled Shafts

The provisions of Article 4.12.3.3.1 shall apply as applicable.

4.13.3.3.2 Analytic Estimates Of Drilled Shaft Capacity In Cohesive Soils

Analytic (rational) methods may be used to estimate the ultimate bearing capacity of drilled shafts in cohesive soils. The performance factors for side resistance and tip resistance for three analytic methods shall be as provided in Table 4.10.6-3. If another analytic method is used, application of the performance factors in Table 4.10.6-3 may not be appropriate.

4.13.3.3.3 Estimation Of Drilled-Shaft Capacity In Cohesionless Soils (C)

The ultimate bearing capacity of drilled shafts in cohesionless soils shall be estimated using applicable methods,

and the factored capacity selected using judgment, and any available experience with similar conditions.

4.13.3.3.4 Axial Capacity In Rock (C)

In determining the axial capacity of drilled shafts with rock sockets, the side resistance from overlying soil deposits shall be ignored.

If the rock is degradable, consideration of special construction procedures, larger socket dimensions, or reduced socket capacities shall be considered.

The performance factors for drilled shafts socketed in rock shall be as provided in Table 4.10.6-3.

4.13.3.3.5 Load Test (C)

Where necessary, a full scale load test or tests shall be conducted on a drilled shaft or shafts to confirm response to load. Load tests shall be conducted using shafts constructed in a manner and of dimensions and materials identical to those planned for the production shafts.

Load tests shall be conducted following prescribed written procedures which have been developed from accepted standards and modified, as appropriate, for the conditions at the site. Standard pile load testing procedures developed by the American Society for Testing and Materials as specified in Article 4.12.3.3.5 may be modified for testing drilled shafts.

The performance factor for axial compressive capacity, axial uplift capacity and lateral capacity obtained from load tests shall be as provided in Table 4.10.6-3.

4.13.3.3.6 Uplift Capacity

Uplift shall be considered when (i) upward loads act on the drilled shafts and (ii) swelling or expansive soils act on the drilled shafts. Drilled shafts subjected to uplift forces shall be investigated, both for resistance to pullout and for their structural strength.

4.13.3.3.6a Uplift Capacity of a Single Drilled Shaft (C)

The uplift capacity of a single straight sided drilled shaft shall be estimated in a manner similar to that for estimating the

ultimate side resistance for drilled shafts in compression (Articles 4.13.3.3.2, 4.13.3.3.3 and 4.13.3.3.4).

The uplift capacity of a belled shaft shall be estimated neglecting the side resistance above the bell, and assuming that the bell behaves as an anchor.

The performance factor for the uplift capacity of drilled shafts shall be as provided in Table 4.10.6-3.

4.13.3.3.6b Group Uplift Capacity (C)

See Article 4.12.3.3.7b. The performance factors for uplift capacity of groups of drilled shafts shall be the same as those for pile groups as given in Table 4.10.6-3.

4.13.3.3.7 Lateral Load (C)

The design of laterally loaded drilled shafts is usually governed by lateral movement criteria (Article 4.13.3.2) or structural failure of the drilled shaft. The design of laterally loaded drilled shafts shall account for the effects of interaction between the shaft and ground, including the number of piers in the group.

4.13.3.3.8 Group Capacity (C)

Possible reduction in capacity from group effects shall be considered.

4.13.3.3.8a Cohesive Soil (C)

The provisions of Article 4.12.3.3.10a shall apply. The performance factor for the group capacity of an equivalent pier or block failure shall be as provided in Table 4.10.6-2 for both cases of the cap being in contact, and not in contact with the ground. The performance factors for the group capacity calculated using the sum of the individual drilled shaft capacities are the same as those for the single drilled shaft capacities.

4.13.3.3.8b Cohesionless Soil (C)

Evaluation of group capacity of shafts in cohesionless soil shall consider the spacing between adjacent shafts. Regardless of cap contact with the ground, the individual capacity of each shaft shall be reduced by a factor η for an isolated shaft, where η = 0.67 for a center-to-center (CTC) spacing of three diameters

and η = 1.0 for a center-to-center spacing of eight diameters. For intermediate spacings, the value of η may be determined by linear interpolation.

See Article 4.13.3.3.3 for a discussion on the selection of performance factors for drilled shaft capacities in cohesionless soils.

4.13.3.3.8c Group in Strong Soil Overlying Weaker Compressible Soil

The provisions of Article 4.12.3.3.10c shall apply as applicable.

4.13.3.3.9 Dynamic/Seismic Design

Refer to Guide Specifications for Seismic Design of Highway Bridges (1983) and Lam and Martin (1986a, 1986b) for guidance regarding the design of drilled shafts subjected to dynamic and seismic loads.

4.13.4 STRUCTURAL DESIGN

The structural design of drilled shafts shall be in accordance with the provisions of Article 4.6.6, which was developed for allowable stress design procedures. In order to use load factor design procedures for the structural design of drilled shafts, the load factor design procedures in Section 8 for reinforced concrete shall be used in place of the allowable stress design procedures.

4.13.4.1 Buckling Of Drilled Shafts (C)

Stability of drilled shafts shall be considered when the shafts extend through water or air for a portion of their length.

4.14 REFERENCES

American Association of State Highway and Transportation Officials (1982), AASHTO Materials, Part II - Test, 13th Edition, Washington, D.C.

American Association of State Highway and Transportation Officials (1988), Manual on Subsurface Investigations, Washington, D.C., 391 pp.

American Concrete Institute (1982), ACI's Guide to Durable Concrete, ACI Guide No. 201.2R-77, Detroit, 37 pp.

American Concrete Institute (1987), "Building Code Requirements for Reinforced Concrete", Manual of Concrete Practice, Part 3, (ACI 318-83).

American Society of Testing and Materials (1990), Annual Book of Standards, Vol. 04.08: Soil and Rock, Building Stone, Geotextoles, ASTM, Philadelphia, 1189 pp.

Baguelin, F., Jezequel, J. F., and Shields, D. H. (1978), The Pressuremeter and Foundation Engineering, Trans Tech Publications, Clausthal, 617 pp.

Barker, R. M., Duncan, J. M., Rojiani, R. B., Ooi, P., Tan, C. K., and Kim, S. G. (1990), Load Factor Design Criteria for Highway Structure Foundations, Preliminary Draft Final Report for NCHRP Project 24-4, Transportation Research Board, National Research Council.

Bienawski Z.T. (1984), "Rock Mechanics Design in Mining and Tunneling, A.A. Balkema: Rotterdam/Boston, 272 pp.

Briaud, J. L. (1990), The Pressuremeter, The Netherlands, A. A. Balkeema, in press, 314 pp.

Canadian Geotechnical Society (1985), "Canadian Foundation Engineering Manual", 2nd Edition, Bitech Publishers Ltd., 460 pp.

Carter, J. P. and Kulhawy, F. H. (1988), Analysis and Design of Drilled Shaft Foundations Socketed into Rock, Report No. EL-5918, Empire State Electric Engineering Research Corporation and Electric Power Research Institute, 158 pp.

Davisson M.T., and Robinson K.E. (1965), "Bending and Buckling of Partially Embedded Piles." Proc. 6th Int. Conf. S.M. and F.E., Montreal, Canada, pp. 243-246.

Deere D.V. (1968), "Geological Considerations", Chapter 1 in Rock Mechanics in Engineering Practice by K.G. Stagg and O. C. Zienkiewicz, John Wiley and Sons, Inc., New York, pp 1-20.

Department of the Army, "Design of Pile Foundations", Publication No. EM 1110-2-2906, U.S. Army Corps of Engineers, Washington D.C. 20314-1000, in press.

Donald I.B., Sloan S.W. and Chiu H.K. (1980), "Theoretical Analysis of Rock Socketed Piles", Proc. Int. Conf. on Structural Foundations on Rock, Sydney, Balkema: Rotterdam.

Duncan J.M. and Buchignani A.L. (1976), "An Engineering Manual for Settlement Studies", Geotechnical Engineering Report, Univ. of Calif. Berkeley, 94 pp.

Esrig M.E. and Kirby R.C. (1979), "Advances in General Effective Stress Method for the Prediction of Axial Capacity for Driven Piles in Clay", 11th Annual Offshore Technology Conference, Houston, pp. 437-449.

Evans Jr. L.T. and Duncan J.M. (1982), "Simplified Analysis of Laterally Loaded Piles", UC Berkeley Rept. No. UCB/GT/82-04, July, 245 pp.

Fellenius B.H., Samson L and Tavenas F. (1989), "Geotechnical Guidelines - Pile Design", Public Works Canada, Marine Works Sector, Ottawa Ontario K1A OM2, Canada.

Focht J.A. and Koch K.J. (1973), "Rational Analysis of the Lateral Performance of Offshore Pile Groups", Proc. of the 5th Offshore Technology Conference, Houston, Texas, Vol. 2, Paper OTC 1896, pp. 701-708.

Gifford, D. G., Kraemer, J. R., Wheeler, J. R., and McKown, A. F. (1987), Spread Footings for Highway Bridges, Report No. FHWA/RD-86/185, Federal Highway Administration, McLean, Virginia, 229 pp.

Goble G.G., Tomko J.J., Rausche F. and Green P.M. (1968), "Dynamic Studies on the Bearing Capacity of Piles", Phase II, Report No. 31, Vols. I and II, Division of Solid Mechanics, Structures and Mechanical Designs, Case Western Reserve University, Cleveland, Ohio.

Goble G.G., Likins G.E. and Rausche F. (1975), "Bearing Capacity of Piles from Dynamic Measurements", Final Report, Department of Civil Engineering, Case Western Reserve University, Cleveland, Ohio.

Goble G.G. and Rausche F. (1976), "Wave Equation Analysis of Pile Driving - WEAP Program", Vols. I, II, III and IV, U.S. Department of Transportation, Federal Highway Administration, Implementation Division, Office of Research and Development (Update, 1981).

Goble G.G. and Rausche F. (1986), "Wave Equation Analysis of Pile Foundations - WEAP86 Program", Vols. I and II, U.S. Department of Transportation, Federal Highway Administration, Office of Implementation, Office of Research and Development, McLean, VA.

Goodman, R. E. (1989), Introduction to Rock Mechanics, Second Edition, John Wiley and Sons, New York, 562 pp.

Hirsch T.J., Carr L. and Lowery L.L. (1976), "Pile Driving Analysis - Wave Equation User's Manual, TTI Program, Vols. 1-4, Report FHWA-IP-76-13.4, U.S. Department of Transportation, Federal Highway Administration, Office of Research and Development, McLean, VA.

Horvath R.G. and Kenney T.C. (1979), "Shaft Resistance of Rock Socketed Drilled Piers", Proc., Symposium on Deep Foundations, ASCE, Atlanta, Georgia, pp. 182-214.

Hrennikoff A. (1950), "Analysis of Pile Foundations with Batter Piles", Transactions ASCE, CXV.

Janbu, N. (1963), "Soil Compressibility as Determined By Oedometer and Triaxial Tests", Proc., European Conference of Soil Mechanics and Foundatin Engineering, Vol I, Wiesbaden.

Janbu, N. (1967), "Settlement Calculations Based on Tangent Modulus Concept", Bulletin No. 2, Soil Mechanics and Foundation Engineering Series, The Technical University of Norway, Trondheim, 57 pp.

Kay, J. N. and Avalle, D. L. (1982), "Application of Screw-Plate to Stiff Clays", Proc., Journal of Geotechnical Engineering Division, ASCE, Vol. 108, No. GT 1, pp 145 - 154.

Kulhawy, F. H. and Goodman, R. E. (1987), "Foundations in Rock", Chapter 5 in Ground Engineering Reference Manual, Edited by F. G. Bell, Butterworths Publishing Co.

Kulhawy F.H., Trautmann C.H., Beech J.F., O'Rourke T.D. and McGuire W (1983), "Transmission Line Structure Foundations for Uplift-Compression Loading", EPRI Rept. EL-2870, Electric Power Research Institute.

Lam I.P. and Martin G.R. (1986a), "Seismic Design of Highway Bridge Foundations, Vol. II, Design Procedures and Guidelines", FHWA/RD-86/102, Federal Highway Administration, 181 pp.

Lam I.P. and Martin G.R. (1986b), "Seismic Design of Highway Bridges, Vol. III, Example Problems and Sensitivity Studies, FHWA/RD-86/103, Federal Highway Administration, 155 pp.

Marchetti, S. (1980), "In Situ Tests by Flat Dilatometer", Proc., Journal of Geotechnical Engineering Division, ASCE, Vol. 106, No. GT3, pp 299 - 321.

Meyerhof, G. G. (1956), "Penetration Tests and Bearing Capacity of Cohesionless Soils", Proc., Journal of Soil Mechanics and Foundation Engineering, ASCE, Vol. 82, No. SM1, pp 1 - 11.

Meyerhof G.G. (1976), "Bearing Capacity and Settlement of Pile Foundations", ASCE JGED, Vol. 102, No. GT3, pp 196-228.

Moulton, L. K., GangaRao, H. V. S., and Halvorsen, G. T. (1985), Tolerable Movement Criteria for Highway Bridges, Report No. FHWA/RD-85/107, Federal Highway Administration, U.S. Department of Transportation, Washington, D. C., 109 pp.

NAVFAC DM7.2 (1982) Foundations and Earth Structures", Dept. of the Navy, Naval Facilities Engineering Command.

Nottingham L. and Schmertmann J. (1975), "An Investigation of Pile Capacity Design Procedures", Final Report D629 to Florida Dept. of Transportation from Dept. of Civil Engineering, Univ. of Florida, 159 pp.

O'Neill M.W., Ghazzaly O.I. and Ha H.B. (1977), "Analysis of Three-Dimensional Pile Groups with Non-Linear Soil Response and Pile-Soil-Pile Interaction", 9th Annual Offshore Technology Conference, Houston, pp. 245-256.

O'Neill M.W. and Tsai C.N. (1984), "An Investigation of Soil Nonlinearity and Pile-Soil Pile Interaction in Pile Group Analysis", Research Rept. No. UHUC 84-9, Dept. of Civil Eng., Univ. of Houston, prepared for US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Ooi P.S.K., Duncan J.M., Rojiani K.B., and Barker R.M. (1991), "Engineering Manual for Driven Piles", Prepared for NCHRP Project 24-4, Charles E. Via, Jr. Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA 24061.

Ooi P.S.K., Rojiani K.B., Duncan J.M. and Barker R.M. (1991), "Engineering Manual for Drilled Shafts", Prepared for NCHRP Project 24-4, Charles E. Via, Jr. Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA 24061.

Osterberg J.O. (1984), "A New Simplified Method for Load Testing Drilled Shafts", Foundation Drilling, Vol. XXIII, No. 6, pp. 9-11.

Peck, R. B. (1976), "Rock Foundations for Structures", Proc. ASCE Speciality Conference on Rock Engineering for Foundations and Slopes, Boulder, Colorado.

Peck, R. B., Hanson, W. E., and Thornburn, T. H. (1974), Foundation Engineering, Second Edition, John Wiley and Sons, New York, 514 pp.

Prakash S. and Sharma H.D. (1990), "Pile Foundations in Engineering Practice", John Wiley & Sons, Inc., 734 pp.

Quiros G.W. and Reese L.C. (1977), "Design Procedures for Axially Loaded Drilled Shafts", Research Rept. 176-5F, Project 3-5-72-176, Center for Highway Research, Univ. of Texas, Austin, 156 pp.

Rausche F. (1970), "Soil Response from Dynamic Analysis and Measurements on Piles", Ph.D. Thesis, Case Western Reserve University, Cleveland, Ohio.

Rausche F., Goble G.G. and Likens G.E. (1985), "Dynamic Determination of Pile Capacity", Journal of Geotechnical Engineering Division, ASCE, Vol. 111, No. GT#, pp. 367-383.

Reese L.C. and O'Neill M.W. (1988), "Drilled Shafts: Construction Procedures and Design Methods", FHWA Publication No. FHWA-HI-88-042 or ADSC Publication No. ADSC-TL-4, 564 pp.

Reese L.C. and Wright S.J. (1977), "Drilled Shaft Manual-Construction Procedures and Design for Axial Loading", Vol. 1, U.S. Dept. of Transportation, Implementation Division, HDV-22, Implementation Package 77-21, 140 pp.

Schmertmann, J. H. (1978), Guidelines for Cone Penetration Test - Performance and Design, FHWA-TS-78-209, U.S. Department of Transportation, Federal Highway Administration, Washington, D. C, 145 pp.

Skempton A.W. (1951), "The Bearing Capacity of Clays", Proc. of the Building Research Congress, London, England, Vol. 1, pp. 180-189.

Sowers, G. F. (1979), Introductory Soil Mechanics and Foundations: Geotechnical Engineering, McMillan Publishing Co., New York, 621 pp.

Tan, C. K., Duncan, J. M., Barker, R. M., and Rojiani, K. B. (1991), Engineering Manual for Shallow Foundations, Prepared for Project NCHRP 24-4, Transportation Research Board.

Tomlinson, M. J. (1987), Foundation Design and Construction, Fifth Edition, Longman Scientific and Technical, England, 842 pp.

Tomlinson M.J. (1987), "Pile Design and Construction Practice", Viewpoint Publication, 415 pp.

Touma F.T. and Reese L.C. (1974), "Behavior of Bored Piles in Sand", ASCE JGED, Vol. 100, No. GT 7, pp. 749-761.

U.S. Department of Navy (1982), Soil Mechanics - Design Manual 7.1, NAVFAC DM - 7.1, Naval Facilities Engineering Command, Alexandria, Virginia, 348 pp.

Vijayvergiya V.N. and Focht J.A. Jr. (1972), "A New Way to Predict the Capacity of Piles in Clay", 4th Annual Offshore Technology Conference, Houston, Vol. 2, pp. 865-874.

SECTION 5
RETAINING WALLS

PART A
GENERAL REQUIREMENTS AND MATERIALS

NCHRP 12-35

PART B
SERVICE LOAD DESIGN
ALLOWABLE STRESS DESIGN

TABLE OF CONTENTS
SECTION 5 - RETAINING WALLS
CONTINUED

NCHRP 12-35

TABLE OF CONTENTS
SECTION 5 - RETAINING WALLS
CONTINUED

PART C
STRENGTH DESIGN METHOD
LOAD FACTOR DESIGN

SECTION 5

RETAINING WALLS

PART A

GENERAL REQUIREMENTS AND MATERIALS

5.1 GENERAL

Retaining walls shall be designed to withstand lateral earth and water pressures, including any live and dead load surcharge, the self weight of the wall, temperature and shrinkage effects, and earthquake loads in accordance with the general principles specified in this section.

Retaining walls shall be designed for a service life based on consideration of the potential long-term effects of corrosion, seepage, stray currents and other potentially deleterious environmental factors on each of the material components comprising the wall. For most applications, permanent retaining walls should be designed for a minimum service life of 75 to 100 years. Retaining walls for temporary applications are typically designed for a service life of 36 months or less.

5.2 WALL TYPE AND CAPACITY

(Content of Article 5.2 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

5.3 SUBSURFACE EXPLORATION AND TESTING PROGRAMS

(Content of Article 5.3 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

5.4 NOTATIONS

(Content of Article 5.4 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

PART B

SERVICE LOAD DESIGN METHOD

ALLOWABLE STRESS DESIGN

5.5 RIGID GRAVITY AND SEMI-GRAVITY WALL DESIGN

(Content of Article 5.5 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

5.6 NONGRAVITY CANTILEVERED WALL DESIGN

(Content of Article 5.6 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

5.7 ANCHORED WALL DESIGN

(Content of Article 5.7 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

5.8 MECHANICALLY STABILIZED EARTH WALL DESIGN

(Content of Article 5.8 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

5.9 PREFABRICATED MODULAR WALL DESIGN

(Content of Article 5.9 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

5.10 REFERENCES

(Content of Article 5.10 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

PART C

STRENGTH DESIGN METHOD

LOAD FACTOR DESIGN

5.11 SCOPE

The provisions of this Part shall apply for the design of rigid gravity and semi-rigid gravity walls, and nongravity cantilevered walls.

The probabilistic LFD basis of these specifications which produces an inter-related combination of load, load factor, and statistical reliability shall be considered when selecting procedures for calculating resistance. The procedures used in developing values of performance factors contained in this Part are summarized in Appendix A of the Final Report for NCHRP Project 24-4 (Barker, et al., 1991). Other methods may be used if the statistical nature of the factors given above are considered, and are approved by the owner.

5.12 DEFINITIONS

Only terms relating to retaining walls are provided in this Section. Definitions for terms relating to foundation types and LFD design are given in Article 4.8.

Cantilever Walls- Walls that resist the forces exerted on them by flexural strength. These walls consist of a concrete wall stem, a concrete slab, and possibly a shear key.

Gravity Walls- Massive stone or concrete masonry walls which depend primarily on their weights to maintain stability. Only a nominal amount of steel is placed near the exposed faces of these walls to prevent surface cracking due to temperature changes.

Retaining Walls- Structures that provide lateral support for a mass of soil and that owe their stability primarily to their own weights and to the weights of any soils located directly above its base.

Semi-gravity Walls- These walls are somewhat more slender than gravity walls and require reinforcement consisting of vertical bars along the inner face and dowels continuing into the footing.

5.13 NOTATIONS

F_r = sliding resistance
H = height of retaining wall
H_f = factored horizontal load
K = coefficient of earth pressure
K_o = coefficient of earth pressure at rest
N = factored bearing pressure resultant
p = lateral earth pressure
P_a = active earth load
P_h = lateral earth load
P_v = vertical earth load
q_f = factored bearing capacity
q_{max} = maximum bearing pressure calculated using factored loads
q_s = surcharge loading
q_{ult} = ultimate bearing capacity
R_I = reduction factor due to load inclination effect
R_n = nominal resistance
V_f = factored vertical load
y = distance to the point of action for lateral earth pressure

<u>Greek</u>

β = load factor coefficient (see Article 5.14.4)
β_E = load factor coefficient for earth pressure
γ = load factor (See Article 5.14.4)
γ_{eq} = equivalent fluid pressure
δ = angle of shearing resistance between wall and soil
Δ = wall displacement
ϕ = performance factor

5.14 LIMIT STATES, LOAD FACTORS AND RESISTANCE FACTORS

All relevant limit states shall be considered in the design to ensure an adequate degree of safety and serviceability.

5.14.1 SERVICEABILITY LIMIT STATES (C)

Design of rigid gravity and semi-gravity walls, and nongravity cantilever walls shall consider the following serviceability limit states:

- excessive movements of retaining walls and their foundations,
- excessive vibrations caused by dynamic loadings, and
- deterioration of element(s) of retaining structures.

The limit state for settlement shall be based upon rideability and economy. The cost of limiting foundation movements shall be compared to the cost of designing the superstructure so that it can tolerate larger movements, or of correcting the consequences of movements through maintenance, to determine minimum lifetime cost. More stringent criteria may be established by the owner.

5.14.2 STRENGTH LIMIT STATES

Design of rigid gravity and semi-gravity walls, and nongravity cantilever walls shall be checked against the strength limit states of:

- bearing capacity failure ,
- lateral sliding,
- excessive loss of base contact,
- overall instability, and
- structural failure.

The limit state which governs the design depends on:

- type and function of retaining structure,
- earth pressures exerted on the wall by the retained backfill,
- geometry of the ground and the structure,
- strength of the ground,
- ground deformability,
- groundwater, and
- swelling pressure in clay backfills.

5.14.3 STRENGTH REQUIREMENT (C)

Retaining walls and their foundations shall be proportioned by the methods specified in Article 5.15 so that their design strength exceeds the required strength.

The required strength is the combined effect of factored loads for each applicable load combination stipulated in Article 3.22. The design strength is calculated for each applicable limit state as the nominal resistance, R_n, multiplied by an appropriate performance (or resistance) factor, ϕ. Procedures for calculating nominal resistance are provided in Article 5.15, and values of performance factors are given in Article 5.14.5.

5.14.4 LOAD COMBINATIONS AND LOAD FACTORS

Retaining structures and their foundations shall be proportioned to withstand safely all load combinations stipulated in Article 3.22 which are applicable to the particular site or wall/foundation type. Impact forces shall not be included in retaining wall design (refer to Article 3.8).

Values of γ and β coefficients for load factor design, as given in Table 3.22.1A, shall apply to strength limit state considerations; while those for service load design (also given in Table 3.22.1A) shall apply to serviceability considerations.

5.14.5 PERFORMANCE FACTORS (C)

Values of performance factors for geotechnical design of foundations are given in Tables 4.10.6-1 through 4.10.6-3, while those for structural design are provided in Article 8.16.1.2.2.

If methods other than those given in Tables 4.10.6-1 through 4.10.6-3 are used to estimate the soil capacity, the performance factors chosen shall provide the same reliability as those given in Tables 4.10.6-1 through 4.10.6-3.

5.15 GRAVITY AND SEMI-GRAVITY WALL DESIGN, AND CANTILEVER WALL DESIGN

5.15.1 EARTH PRESSURE DUE TO BACKFILL (C)

The provisions of Article 5.5.2 and 5.6.2 shall also apply to the load factor design of rigid gravity and semi-gravity walls, and nongravity cantilevered walls respectively; with the exception that the loads shall be factored according to the bottom half of Table 3.22.1A when checking wall stability against bearing capacity, sliding and overturning. Vertical earth pressure due to the dead load of the backfill shall have an overall load factor, $\gamma\beta_E$, of 1.0γ.

Lateral earth pressures on walls backfilled with cohesionless soils shall be designed using effective stresses. Walls backfilled with cohesive soils shall be designed using equivalent fluid pressures. The backfill, whether cohesionless or cohesive, shall be well drained, so that no water pressures act on the wall, and no significant pore pressures act in the backfill. The load factor for lateral earth pressures calculated using equivalent fluid pressures shall be the same as those calculated using effective stresses ($\gamma\beta_E = 1.3\gamma$).

The γ and β_E coefficients specified for earth pressure in Table 3.22.1A are applicable directly to active or at rest earth pressures. The resistance due to passive earth pressure in front of the wall shall be neglected unless the wall extends well below the depth of frost penetration, scour or other types of disturbance. Where passive pressure is assumed to provide resistance, the performance factor (ϕ) shall be taken as 0.6.

5.15.2 EARTH PRESSURE DUE TO SURCHARGE

In the design of retaining walls and abutments where traffic can come within a horizontal distance from the top of the wall equal to one half the wall height, the lateral earth pressure shall be increased by a live load surcharge pressure equal to not less than 2 feet of earth (Article 3.20.3). Impact loads shall not be included in the design of abutments (Article 3.8.1). Vertical earth pressure induced by live load surcharge and dead load surcharge shall have overall load factors of 1.67γ and 1.3γ, respectively. Lateral earth pressure induced by live load and dead load surcharge shall have an overall load factor of 1.3γ

Where heavy static and dynamic compaction equipment is used within a distance of one half the wall height behind the wall, the effect of additional earth pressure that may be induced by compaction shall be taken into account. The load factor for compaction-induced earth pressures shall be the same as for lateral earth pressures ($\gamma\beta_E = 1.3\gamma$).

5.15.3 WATER PRESSURE AND DRAINAGE

The provisions of Articles 5.5.3 and 5.6.3 shall also apply to the load factor design of rigid gravity and semi-gravity walls, and nongravity cantilevered walls respectively.

The backfill, whether cohesive or cohesionless, shall be well drained so that no water pressures act on the wall and no significant pore pressures act in the backfill. If a thorough drainage system is not provided to dewater the failure wedge, or if its adequate performance cannot be guaranteed, walls shall be designed to resist the maximum anticipated water pressure. For walls backfilled with cohesionless soils, the lateral earth pressure shall be calculated using buoyant unit weights below the groundwater level and multiplied by the load factor for lateral earth pressure. The wall shall be designed for these factored lateral earth pressures ($\gamma\beta_E$) plus factored hydrostatic water pressure (1.0γ).

In the case of an undrained analysis of cohesive backfills, the lateral earth pressure shall be calculated using equivalent fluid pressure, which inherently includes water pressure effects. The calculated lateral earth pressure shall then be multiplied by 1.3γ.

If the groundwater levels differ on opposite sides of the wall, the effects of seepage on wall stability and the potential for piping shall be considered. Pore pressures behind the wall can be determined by flow net procedures or various analytical methods, and shall be added to the effective horizontal stresses when calculating total lateral earth pressures on the wall. The effective lateral earth pressure shall be multiplied by $\gamma\beta_E$ (obtained from Table 3.22.1A) and the hydrostatic pressure shall be factored by 1.0γ, when designing the wall.

5.15.4 SEISMIC PRESSURE

The provisions of Article 5.6.4 shall apply to the load factor design of walls when considering earthquakes loads.

5.15.5 MOVEMENT UNDER SERVICEABILITY LIMIT STATES

The movement of wall foundation support systems shall be estimated using procedures described in Article 4.11.3, 4.12.3.2.2, or 4.13.3.2.2, for walls supported on spread footings, driven piles, or drilled shafts respectively. Such methods are based on soil and rock parameters measured directly or inferred from the results of in situ and/or laboratory tests.

Tolerable movement criteria for retaining walls shall be developed based on the function and type of wall, anticipated service life, and consequence of unacceptable movements. Tolerable movement criteria shall be established in accordance with Articles 4.11.3.5 , 4.12.3.2.3, and 4.13.3.2.3.

5.15.6 SAFETY AGAINST SOIL FAILURE

Gravity and semi-gravity walls, and cantilever walls shall be dimensioned to ensure stability against bearing capacity failure, overturning and sliding. Where a wall is supported by clayey foundation, safety against deep-seated foundation failure shall also be investigated. Stability criteria for walls with respect to various modes of failure shall be as shown in Figures 5.15.6-1 through 5.15.6-3.

5.15.6.1 Bearing Capacity Failure

The safety against bearing capacity failure shall be investigated: (1) by using factored soil pressures which are uniformly distributed over the effective base area, if the wall is supported by a soil foundation (see Figures 5.15.6-1 and 5.15.6-2); or (2) by using factored soil pressures which vary linearly over the effective base area, if the wall is supported by a rock foundation (see Figure 5.15.6-3).

Retaining walls and their foundations are considered to be adequate against bearing capacity failure if the factored bearing capacity (taking into consideration the effect of load inclination) exceeds the maximum soil pressure (q_{max}) determined using factored loads. Methods for calculating factored bearing capacity are provided in Article 4.11.4 for walls founded on spread footings, and in Articles 4.12.3.3 and 4.13.3.3 for walls supported on driven piles or drilled shafts, respectively.

5.15.6.2 Sliding

Where the retaining wall is founded on a spread footing, safety against sliding shall be investigated using the procedures specified in Article 4.11.4.3.

5.15.6.3 Overturning (C)

The safety against overturning shall be ensured by limiting the location of the factored bearing pressure resultant (N) on the wall base. For walls supported by soil foundations, location of the factored bearing pressure resultant on the base of the wall foundation shall be within the middle half of the base. For walls supported by rock foundations, location of the factored bearing pressure resultant on the base of the wall foundation shall be within the middle three quarters of the base.

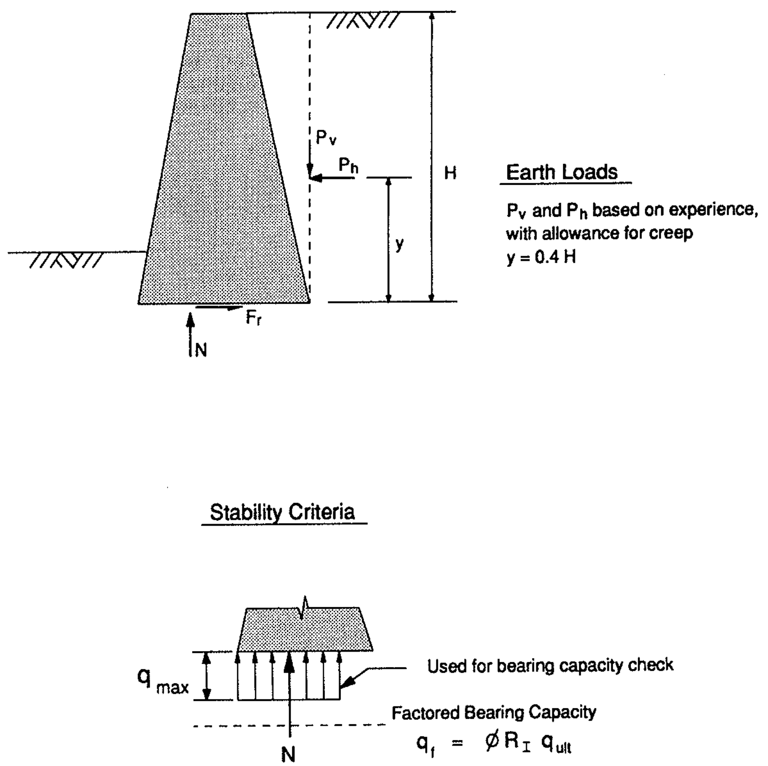


Figure 5.15.6-1: Earth Loads and Stability Criteria for Walls with Clayey Soils in the Backfill or Foundation (After Duncan et al., 1990)

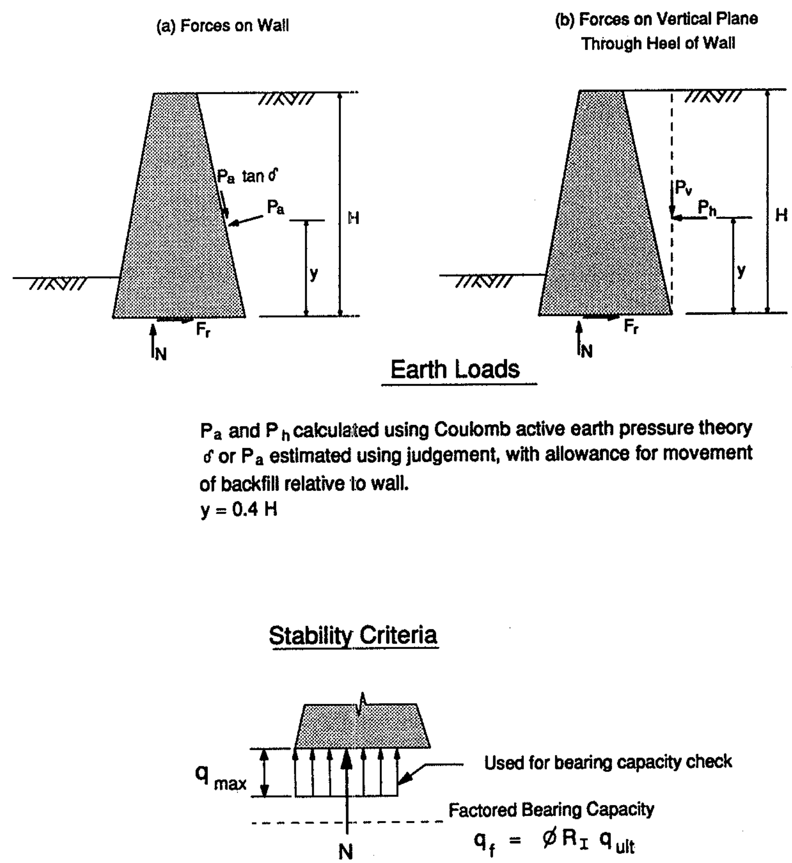


Figure 5.15.6-2: Earth Loads and Stability Criteria for Walls with Granular Backfills and Foundations on Sand or Gravel (After Duncan et al., 1990)

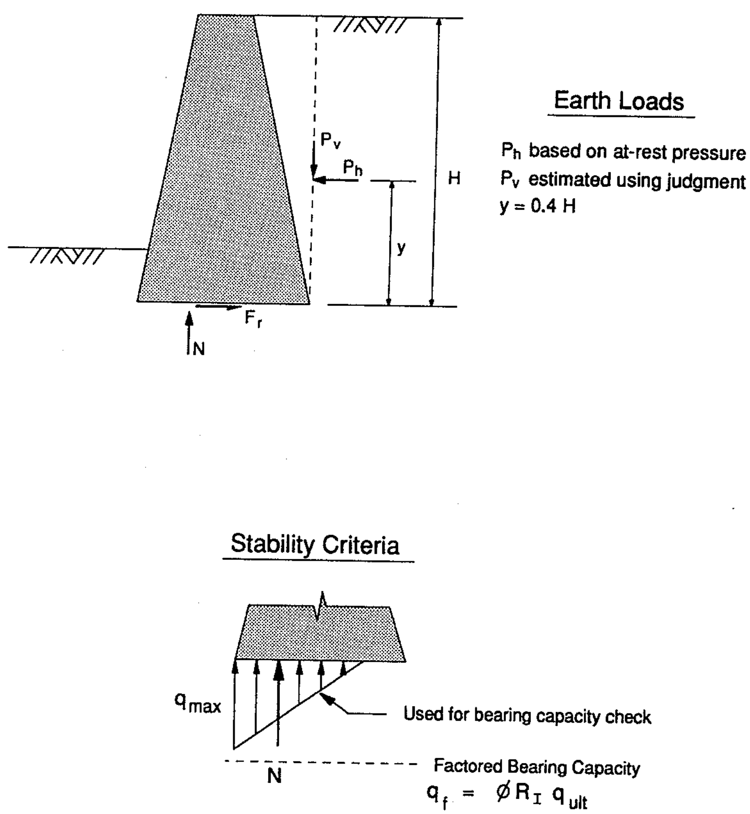


Figure 5.15.6-3: **Earth Loads and Stability Criteria for Walls with Granular Backfills and Foundations on Rock (After Duncan et al., 1990)**

5.15.6.4 Overall Stability (Revised Article 5.2.2.3) (C)

The overall stability of slopes in the vicinity of walls shall be considered.

The overall stability of the retaining wall, retained slope, and foundation soil or rock shall be evaluated for all walls using limiting equilibrium methods of analysis. The Modified Bishop, simplified Janbu or Spence methods of analysis may be used. Special exploration, testing and analyses may be required for bridge abutments or retaining walls constructed over soft deposits where consolidation and/or lateral flow of the soft soil could result in unacceptable long-term settlements or horizontal movements.

5.15.7 SAFETY AGAINST STRUCTURAL FAILURE

The structural design of individual wall elements and wall foundations shall comply to the requirements given in Section 8.

In the structural design of a footing on soil and rock at ultimate limit states, a linear contact pressure distribution determined using factored loads, as shown in Figure 5.15.7-1, shall be considered. The maximum pressure for structural design may be greater than the factored bearing capacity.

5.15.7.1 Base of Footing Slabs

See Article 5.5.6.1.

5.15.7.2 Wall Stems

See Article 5.5.6.2.

5.15.7.3 Counterforts and Buttresses

See Article 5.5.6.3.

5.15.7.4 Reinforcement

See Article 5.5.6.4.

5.15.7.5 Expansion and Contraction Joints

See Article 5.5.6.5.

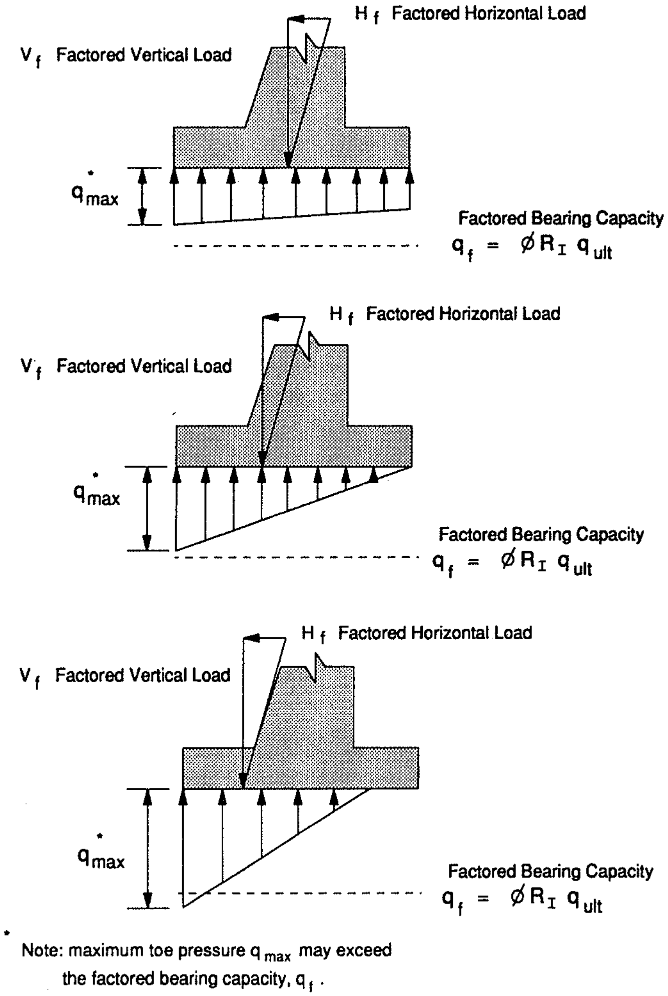


Figure 5.15.7-1: Contact Pressure Distribution for Structural Design of Footings on Soil and Rock at Strength Limit States

5.15.8 BACKFILL

Where possible, the backfill material behind all retaining walls shall be free draining, nonexpansive, noncorrosive and shall be drained by weep-holes and French drains placed at suitable intervals and elevations. In counterfort walls, there shall be at least one drain for each pocket formed by the counterforts. Silts and clays shall, if possible, be avoided for use as backfill.

5.16 REFERENCES

Barker R.M., Duncan J.M., Rojiani K.B., Ooi P.S.K., Tan C.K. and Kim S.G. (1990), "Load Factor Design Criteria for Highway Structure Foundations", Preliminary Draft Final Report for NCHRP Project 24-4, Transportation Research Board National Research Council.

Clough G.W. and Duncan J.M. (1991), "Earth Pressures", Chapter 6 of the 2nd Edition of the Foundation Engineering Handbook.

Duncan J.M., Clough G.W. and Eberling R. (1990), "Behavior and Design of Gravity Earth Retaining Structures," Proc., Symposium on Design and Performance of Earth Retaining Structures, Geotechnical Special Publication No. 25, ASCE, pp. 251-277.

Kim S.G., Barker R.M., Duncan J.M. and Rojiani K.B. (1991), "Engineering Manual for Abutments and Retaining Walls", Charles E. Via Department of Civil Engineering, Virginia Polytechnic Institute and State University.

SECTION 7
SUBSTRUCTURES

PART A
GENERAL REQUIREMENTS AND MATERIALS

NCHRP 12-35

PART B
SERVICE LOAD DESIGN METHOD
ALLOWABLE STRESS DESIGN

TABLE OF CONTENTS
SECTION 7 - SUBSTRUCTURES
CONTINUED

PART C
STRENGTH DESIGN METHOD
LOAD FACTOR DESIGN

NCHRP 24-4

SECTION 7

SUBSTRUCTURES

PART A

GENERAL REQUIREMENTS AND MATERIALS

7.1 GENERAL

7.1.1 DEFINITION

A substructure is any structural, load-supporting component generally referred to by the terms abutment, pier, retaining wall, foundation or other similar terminology.

7.1.2 LOADS

Where appropriate, piers and abutments shall be designed to withstand dead load, erection loads, live loads on the roadway, wind loads on the superstructure, forces due to stream currents, floating ice and drift, temperature and shrinkage effects, lateral earth and water pressures, scour and collision and earthquake loadings.

7.1.3 SETTLEMENT

The anticipated settlement of piers and abutments should be estimated by appropriate analysis, and the effects of differential settlement shall be accounted for in the design of the superstructure.

7.1.4 FOUNDATION AND RETAINING WALL DESIGN

Refer to Section 4 for the design of spread footing, driven pile and drilled shaft foundations and Section 5 for the design of retaining walls.

7.2 NOTATIONS

(Content of Article 7.2 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

PART B

SERVICE LOAD DESIGN METHOD

ALLOWABLE STRESS DESIGN

7.3 PIERS

(Content of Article 7.3 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

7.4 TUBULAR PIERS

(Content of Article 7.4 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

7.5 ABUTMENTS

(Content of Article 7.5 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

7.6 REFERENCES

(Content of Article 7.6 from Final Report of NCHRP Project 12-35, D'Appolonia, 1989.)

PART C

STRENGTH DESIGN METHOD

LOAD FACTOR DESIGN

7.7 GENERAL

The provisions of Article 7.1 through 7.6 shall apply to the load factor design of abutments with the exception that: (1) Article 7.5.2 on loading shall be replaced by the articles for loads, earth pressures and water pressures in Sections 5.14 and 5.15 for retaining walls, and (2) Article 7.5.2.1 shall be replaced by the articles for stability in Sections 5.14 and 5.15. Abutments shall be designed to withstand earth pressures, water pressures and other loads similar to the design of retaining walls.

PART 6

OTHER ARTICLES IN AASHTO 1989

1. Add the phrase in bold shown below into Article 3.2.2:

   3.2.2 Members shall be proportioned **either with reference to service loads and allowable stresses as provided in Service Load Design (Allowable Stress Design) or, alternatively, with reference to load factors and factored strength as provided in Strength Design (Load Factor Design).**

2. Add the phrase in bold shown below into Article 3.20.1:

   3.20.1 Structures which retain fills shall be proportioned to withstand pressure as given by Rankine's formula, **or by other expressions given in Section 5**; provided, however, that no structure shall be designed for less than an equivalent fluid weight (mass) of 30 pounds per cubic foot.

3. Replace existing Article 3.22.3 with the following:

   3.22.3 For load factor design, the gamma and beta factors given in Table 3.22.1A **shall be used for designing structural members and foundations by the load factor concept.**

4. Change the last article reference in Article 8.16.6.6.1(a) from Article 4.4.7.2 to Article 4.4.11.3.2.

APPENDIX C

COMMENTARY

SECTION 4 - FOUNDATIONS

SECTION 5 - RETAINING WALLS

NOTE: The format used in Appendix C for Sections 4 and 5 is to first present a Table of Contents for each section and then provide commentary for the recommended Load Factor Design specifications. The articles listed in the Table of Contents on Allowable Stress Design have been prepared under NCHRP Project 12-35 (D'Appolonia, 1989). Where a commentary on a Load Factor Design specification has been prepared, it is indicated by the letter C before the number and title of the corresponding specification.

TABLE OF CONTENTS

SECTION 4
FOUNDATIONS

PART C
STRENGTH DESIGN METHOD

LOAD FACTOR DESIGN

COMMENTARY

NCHRP 24-4
Page

NCHRP 24-4
Page

PART C
STRENGTH DESIGN METHOD

LOAD FACTOR DESIGN
COMMENTARY

C 4.8 SCOPE

The probabilistic LFD basis of these specifications which produces an inter-related combination of load, load factor, and statistical reliability should be considered when selecting procedures for calculating resistance not specified herein. The procedures used in developing the values of performance factors contained in this Section are summarized in Appendix A of the Final Report, NCHRP Project 24-4 (Barker, et al., 1991). Other methods may be used if the statistical nature of the factors given above are considered through consistent use of reliability theory, and are approved by the Owner.

C 4.10.4 STRENGTH REQUIREMENT

The basic requirement of safety in LFD format can be expressed by the following formula:

$$\phi R_n \geq \text{effect of } \gamma \Sigma \beta_i L_i \qquad \text{(C4.10.4-1)}$$

where

- ϕ = performance factor, see Tables 4.10.6-1 through 4.10.6-3;
- R_n = nominal resistance of foundation;
- ϕR_n = design strength (or factored resistance);
- γ = load factor, see Table 3.22.1A;
- β_i = coefficient for load type i, see Table 3.22.1A;
- i = type of load, such as dead load, live load, etc.;
- L_i = load type i;
- $\gamma \Sigma \beta_i L_i$ = required strength (or required resistance).

The formula expresses the notion that even in the highly unlikely situation where the load-carrying capacity of the foundation is very low, and, at the same time, the loads are very high, the capacity of the foundation should still be large enough to support the loads.

The load factors γ and performance factors ϕ account for the fact that loads, load effects (e.g., the computed pressures and sliding forces exerting on foundations), and the resistance can be determined only to imperfect degrees of accuracy. Load factors which often have values larger than unity account for the uncertainties in loads and their probability of occurrence. Performance factors, on the

other hand, which typically have values less than unity, account for such things as soil variabilities and model uncertainties.

C 4.10.6 PERFORMANCE FACTORS

Where statistical information was available, reliability theory, tempered in some cases with judgment, was used to derive the values of performance factors given in Tables 4.10.6-1 through 4.10.6-3. In cases where there was insufficient information for calibration using reliability theory, values of performance factors were chosen based on judgment, so that the design was consistent with that using ASD procedures. Details are provided in Appendix A of the final report for Project 24-4 (Barker, et al., 1991).

In deriving the values of performance factors given in Tables 4.10.6-1 through 4.10.6.3, the target reliability indices were chosen as 2.0 to 2.5, 2.5 to 3.5, and 3.5, respectively, for driven piles, drilled shafts and spread footings.

C 4.11.1.1 General

Problems with insufficient bearing and/or excessive settlements in fill can be significant, particularly if poor (e.g., soft, wet, frozen or nondurable) material is used or material is not properly compacted. Settlement of improperly placed or compacted fill around piers can cause substantial increases in footing loads resulting from the downward drag or friction force exerted on the pier by the settling fill (i.e., negative skin friction). Even properly placed and compacted backfill undergoes some amount of settlement depending on the material type, moisture conditions, method of placement and method and degree of compaction.

C 4.11.1.3 Scour Protection (Revised Article C4.4.5.2)

In cases where footings are founded on rocks, special attention must be paid to the effect of blasting. Blasting of highly resistant competent rock formations typically results in fracturing of the rock to some depth below the final rock surface. Because blasting would likely reduce the resistance to scour within the rock zone immediately below the footing base, blasting is not recommended.

C 4.11.1.4 Frost Action

For frost action to occur, it is generally accepted that three basic conditions must exist:

- the presence of frost susceptible soil,
- the availability of water, and
- cooling conditions that cause the soil and water to freeze.

Considerable differences of frost penetration can exist throughout the U.S. and, in some instances, even locally. Where frost protection is marginal or deficient (for example, in the case where a roadway grade adjacent to an existing structure is lowered), consideration should be given to the use of insulation to improve frost protection.

C 4.11.1.5 Anchorage (Same as Article C 4.4.6)

Blasting operations have a high probability of overbreak and/or fragmentation of the bearing rock below the footing level. Accordingly, positive anchorage is required between the rock and footing such as provided by rock anchors, bolts or dowels.

C 4.11.1.6 Groundwater

Evaluation of seepage forces and hydraulic gradients is essential in the design of foundation excavations extending below the groundwater table. Upward seepage forces in the bottom of excavations can result in piping in dense granular soil or heaving in loose granular soil which may cause bottom instability. These problems can be controlled by adequate dewatering, typically using wells or well points. Dewatering of excavations in loose granular soils can cause settlement of the surrounding ground. If adjacent structures may be damaged by such settlement or if the cost of dewatering is high, seepage cut off methods, such as sheet piling or slurry walls, may be practical or necessary.

C 4.11.1.8 Deterioration

Measures that may be used to protect concrete foundations from attack by aggressive agents include:

- Use of special materials (e.g., sulifate resistant cement and epoxy-coated steel reinforcement),
- frequent maintenance, and
- conservative design that deliberately disregard portion of the foundation material.

The choice depends, among other factors, on the severity of the problem, the decay rate and the cost.

Special types of cements may be used to reduce deterioration. ACI's Guide to Durable Concrete (1982) lists the appropriate types of cements for various types of sulfate exposure.

C 4.11.1.9 Nearby Structures

The possibility of a new foundation imposing additional loads on an existing foundation (or vice versa) should be evaluated, depending upon the proximity of the nearby structure and the relative positions between the foundation and the existing structures. A new foundation may be positioned as shown in Figure C4.11.1.9 to avoid the possibility of imposing additional loads on an existing foundation, if the footings are founded on firm soil or rock.

C 4.11.3.1 General

Elastic deformation occurs quickly and is usually small. In design, it is normally neglected. Changes in volume associated with a reduction in the water content of the subsoil is called consolidation and can be estimated and measured. Consolidation settlement occurs in all soils. In cohesionless soils, the consolidation occurs quickly and is normally not distinguishable from the elastic deformation. In cohesive soils, such as clays, the consolidation can take a considerable length of time.

Various loads may have significant effects on the magnitude of settlements or lateral displacements of the soils. The following factors should be considered in the estimation of settlements:

- the ratio of sustained load to total load,
- the duration of sustained loads and
- the time interval over which settlement or lateral displacement occurs.

The consolidation settlements in cohesive soils are time-dependent, and consequently, transient loads have negligible effect. However, in cohesionless soils where the permeability is sufficiently high, elastic deformation of the supporting soil due to transient load can take place. Since deformation in cohesionless soils often takes place during construction while the loads are being applied, such deformation can be accommodated by the structure to an extent, depending on the type and construction method.

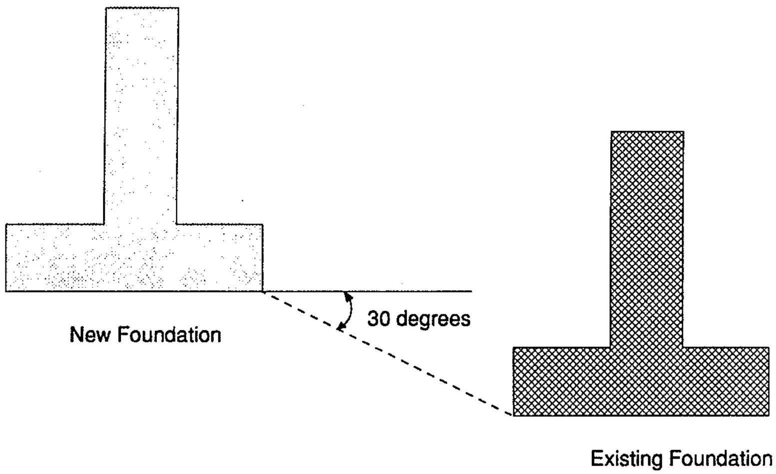


Figure C4.11.1.9: Recommended Location of a New Foundation on Firm soil or Rock

C 4.11.3.2 Loads

Deformation in cohesionless (or granular) soils often occurs as soon as loads are applied. As a consequence, settlements due to transient loads may be significant in cohesionless soils, and they should be included in settlement analyses. Consolidation settlements in cohesive soils, on the other hand, are time-dependent. Consequently, consolidation settlements due to transient loads are usually negligible, and are often disregarded in settlement analyses.

C 4.11.3.3 Movement Criteria (Revised Article C 4.4.7.2.5)

Past experience has shown that bridges can, and often do, accommodate more settlement than traditionally allowed or anticipated in design. This accommodation is accompanied by creep, relaxation and redistribution of force effects. Some studies [Moulton, et al., 1985; Duncan and Tan, 1991] have been made to synthesize apparent response, which studies indicate that angular distortions between adjacent foundations greater than 0.008 in simple-spans and 0.004 in continuous spans should not be permitted in settlement criteria. Lesser angular distortion may be appropriate after consideration of:

- cost of mitigation through increased foundations, realignment or overbuilding,
- rideability,
- aesthetics, and
- safety.

C 4.11.3.4 Settlement Analyses

Both in situ and laboratory tests can provide useful information pertaining to the load-deformation behavior of the foundation soils. These test methods include:

In situ test methods:
- cone penetrometer (Schmertman, 1978)
- pressuremeter (Briaud, 1990)
- dilatometer (Marchetti, 1980)
- screw plate (Kay and Avalle, 1982)
- plate load (ASTM, 1990)

Laboratory test methods:
- direct shear (ASTM, 1990)
- unconfined and triaxial compression (ASTM, 1990)
- consolidation (ASTM, 1990)

C 4.11.3.4.1 Settlement of Footings on Cohesionless Soils

Settlements of cohesionless soils occurs essentially as rapidly as the foundation is loaded. The immediate settlement may be estimated by several established methods.

Details for these procedures can be found in many text books and engineering manuals (e.g., Terzaghi and Peck, 1968; Sowers, 1979; NAVFAC, 1982; Gifford, et al., 1987; Tomlinson, 1989; Tan, et al., 1991).

C 4.11.3.4.2 Settlement of Footings on Cohesive Soils

In practice, footings are most likely founded on overconsolidated clays. Settlements of footings founded on these less compressible clays can be estimated using elastic theory, Baguelin, et al. (1978), or the tangent modulus method, Janbu (1963, 1967). Settlements of footings on overconsolidated clays usually occur fairly rapidly, and it is reasonable to assume that they take place as rapidly as the loads are applied.

C 4.11.3.4.3 Settlments of Footings on Rock

Where the foundations are subjected to a very large load, or where settlement tolerance may be small, settlements of footings on rock may be estimated using elastic theory. The stiffness of the rock mass should be used in such analyses.

The accuracy with which settlements can be estimated by using elastic theory is dependent on the accuracy of the estimated rock mass modulus (E_m). In some cases the value of E_m can be estimated through empirical correlation with the value of modulus of elasticity for the intact rock between joints. For unusual or poor rock mass conditions, it may be necessary to determine the modulus from in situ tests, such as plate loading and pressuremeter tests.

If bearing resistance of footings on rock is estimated using the semi-empirical procedure developed by Peck, et al. (1974), settlement of the footing is expected to be less than 0.5 inch.

C 4.11.4.1.1 Theoretical Estimation

Ultimate bearing capacity of saturated clay is related to its undrained shear strength by the following equation:

$$q_{ult} = cN_{cm} + \gamma D_f N_{qm} \qquad (C4.11.4.1.1\text{-}1)$$

where $c = S_u$ = undrained shear strength;
N_{cm}, N_{qm} = modified bearing capacity factors which are functions of footing shape, embedment depth and load inclination, see Tan, et al. (1991) or other foundation engineering textbooks or manuals for details;
γ = total unit weight of clay;
D_f = footing depth.

For cohesionless soils (such as sands or gravels), and conditions where the groundwater table is at depth greater than 1.5 times the footing width below footing base, ultimate bearing capacity may be determined from the expression given below:

$$q_{ult} = 0.5\ \gamma B\ N_{\gamma m} + \gamma D_f\ N_{qm} \qquad (C4.11.4.1.1\text{-}2)$$

where γ = total unit weight of sand or gravel;
$N_{\gamma m}$, N_{qm} = modified bearing capacity factors which are functions of soil compressibility, footing shape, embedment depth and load inclination, see Tan, et al. (1991) or other foundation engineering textbooks or manuals for details;
B = footing width;
D_f = footing depth.

When the position of groundwater table is at a higher level (less than 1.5 times the footing width below footing base), ultimate bearing capacity of soils would be smaller than that computed using Equation C4.11.4.1.1-2. The effect of groundwater is described in Article 4.11.4.1.6.

The reliability of bearing capacity estimates depends on the accuracy with which the soil parameters (undrained shear strength or friction angle) are determined. Consequently, values of performance factors vary with the means by which the soil strength is determined, as indicated in Table 4.10.6-1.

C 4.11.4.1.2 Semi-empirical Procedures

Because of difficulties in obtaining undisturbed sand samples, ultimate bearing capacity of footings on sand are best estimated using semi-empirical procedures. The ultimate bearing capacity in sand can be determined based on Standard Penetration Test (SPT) results by the following (modified after Meyerhof, 1956):

$$q_{ult} = \frac{\bar{N}B}{10}\left(C_{w1} + C_{w2}\frac{D_f}{B}\right) R_I \qquad (C4.11.4.1.2\text{-}1)$$

where $\bar{N}$ = average value of SPT blow count;
q_{ult} = ultimate bearing capacity (t/ft^2);
B = footing width (ft);
C_{w1}, C_{w2} = correction factors for groundwater effect (dimensionless);
D_f = footing embedment depth (ft);
R_I = reduction factor due to the effect of load inclination (dimensionless).

Cone penetration resistance can also be used to estimate bearing capacity directly. The ultimate bearing capacity should be determined by the following empirical formula which is based on cone penetration test (CPT) results (modified after Meyerhof, 1956):

$$q_{ult} = \frac{q_c}{40} B \left(C_{w1} + C_{w2} \frac{D_f}{B}\right) R_I \qquad (C4.11.4.1.2\text{-}2)$$

where q_c = cone resistance (t/ft^2), B = footing width (ft), and other terms are as defined previously.

The ultimate bearing capacity calculated using Equation C4.11.4.1.2-1 or C4.11.4.1.2-2 should be multiplied by an appropriate performance factor given in Table 4.10.6-1 to determine the factored bearing capacity.

C 4.11.4.1.3 Plate Loading Test

Load tests have a limited depth of influence and may not disclose long-term consolidation of foundation soils.

C 4.11.4.1.5 Effect of Load Eccentricity

As shown in Figure C4.11.4.1.5-1, the reduced dimensions for a footing can be determined as:

$$B' = 2y \qquad (C4.11.4.1.5\text{-}1)$$

$$L' = 2x \qquad (C4.11.4.1.5\text{-}2)$$

For footings that are not rectangular, such as the circular footing at the bottom of Figure C4.11.4.1.5-1 the effective area can be estimated using simple approximations and judgement.

For purposes of structural design, it is usually assumed that the bearing pressure varies linearly across the bottom of the footing. This assumption results in the

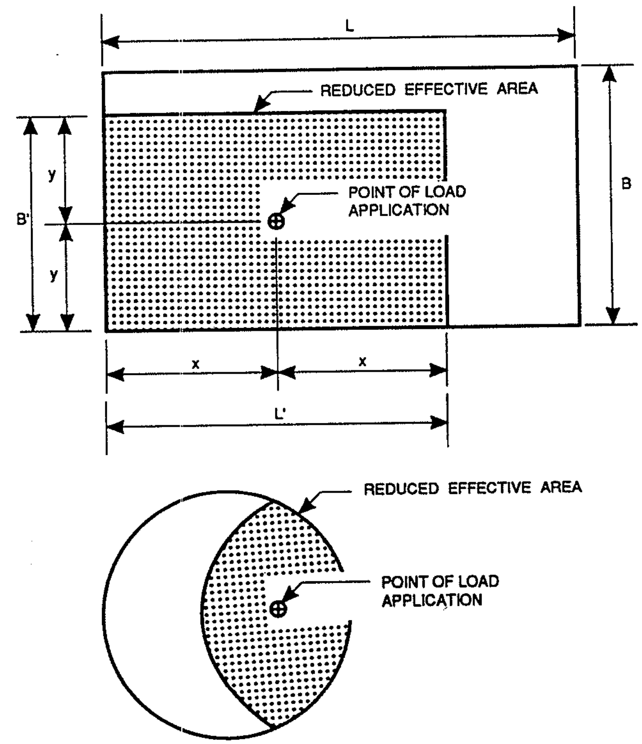


Figure C4.11.4.1.5-1: **Reduced Effective Areas for Eccentrically Loaded Footings**

slightly conservative triangular or trapezoidal contact pressure distribution.

C 4.11.4.1.6 Effect of Groundwater Table

The position of the groundwater table can significantly influence the bearing capacity of soils through its effect on shear strength and unit weight of the foundation soils. In general, the submergence of soils will reduce the effective shear strength of granular materials as well as the long-term (or drained) strength of clayey soils. Moreover, the unit weights of submerged soils are about half of those for the same soils under dry conditions. Thus, submergence effect may lead to a significant reduction in bearing capacity, and it is essential that the bearing capacity analyses be carried out assuming the highest possible water table expected within the service life of the structure.

Influence of groundwater is incorporated in the semi-empirical procedures given in Article C 4.11.4.1.2 through the use of the factors C_{w1} and C_{w2}. Similarly, the effect of groundwater can also be included in analytic theory (Equation C4.11.4.1.1-1) as follows:

For cohesionless soils

$$q_{ult} = 0.5\ \gamma B\ C_{w1} N_{\gamma m} + \gamma C_{w2}\ D_f N_{qm} \qquad \text{(C4.11.4.1.6-1)}$$

If the depth of groundwater table (D_w) is greater or equal to 1.5 times the footing width plus the footing depth (D_f), $C_{w1} = C_{w2} = 1.0$. For $D_w = D_f$, $C_{w1} = 0.5$ and $C_{w2} = 1.0$; for $D_w = 0$, $C_{w1} = C_{w2} = 0.5$. For intermediate positions of groundwater table, values of C_{w1} and C_{w2} can be determined by interpolation.

C 4.11.4.2.1 Semi-empirical Procedures

The empirical correlation developed by Peck, et al. (1974), as given in Table C4.11.4.2.1-1, may be used to estimate allowable bearing pressure of footings on competent rock. The value of RQD in Table C4.11.4.2.1-1, should be taken as the average RQD of rock within a depth B below the base of the footing. In no instance should the allowable bearing pressure of the foundation exceed the allowable bearing stress of the concrete. Since the design values recommended by Peck et al. are based on settlement limitation, load factors should be taken as unity.

The ultimate bearing capacity of jointed or broken rock may be estimated using the semi-empirical procedure developed by Carter and Kulhawy (1988). The procedure is

Table C4.11.4.2.1-1: Allowable Bearing Pressures of Jointed Rock (After Peck, Hanson and Thornburn, 1974)

RQD	Allowable Bearing Pressure* (tsf)
100	300
90	200
75	120
50	65
25	30
0	10

*Note: If the recommended value of allowable bearing pressure exceeds the unconfined compressive strength of the rock or allowable stress of concrete, the allowable bearing pressure should be taken as the unconfined compressive strength, or the allowable stress of concrete, whichever is less.

based on the unconfined compressive strength of the intact rock core sample. Depending on rock mass quality (measured in terms of RMR or NGI system), ultimate bearing capacity of a rock mass varies from a small fraction to six times the unconfined compressive strength of intact rock core samples.

C 4.11.4.2.2 Analytic Method

Depending upon the relative spacing of joints and rock layering, bearing capacity failures for foundations on rock may take several forms. Except for the case of a rock mass with closed joints, the failure modes are different from those in soil. Procedures for estimating bearing capacity for each of the failure modes can be found in Kulhawy & Goodman (1987), Goodman (1989), and Sowers (1979).

C 4.11.4.3 Failure by Sliding

Sliding failure occurs if the horizontal component of the factored load exceeds the factored shear resistance of the soils, or the factored shear resistance at the interface between the soil and the foundation, whichever is less.

For footings on cohesionless soils, magnitude of sliding resistance depends on the roughness of the interface between the foundation and the soil. If the base of the footing is rough, as in the case where footings are cast in situ, the sliding is resisted by the full strength of the soil. For precast concrete footings, which may be smoother, the shear strength of the interface may be taken as eight-tenths of the soil strength.

For footings that rest on clay, the sliding resistance should be taken as the cohesion of the clay, or one-half the normal stress on the interface between the footing and soil, whichever is less.

The resistance to sliding may be increased by widening the base of footings, or by the use of key if footings are founded on stiff clay or rock.

C 4.11.4.4 Loss of Overall Stability (Revised Article C 4.4.9)

Equilibrium methods or analyses which employ the Modified Bishop, simplified Janbu, Spencer or other generally accepted methods of slope stability analysis may be used.

Investigation of global stability is particularly important for foundations which are located close to:

- a natural slope or on an inclined site
- an embankment or an excavation
- a river or a canal
- a lake, a reservoir or the sea shore
- mine workings
- a retaining wall.

The mode of failure will be dictated by the subsurface conditions in the vicinity of the footing. When relatively homogeneous soil conditions exist and such conditions extend below the footing, the critical failure surface will likely be circular. When subsurface conditions include a particularly weak zone or layer, or a shallow sloping rock surface, the critical failure surface will likely be planar. In many cases, both modes of failure must be analyzed to determine the more critical failure mode.

Even if overall stability is satisfactory, special exploration, testing and analyses may be required for bridge abutments or retaining walls constructed over soft subsoils where consolidation and/or lateral squeeze of the soft soil could result in unacceptable long-term settlements or horizontal movement of abutments.

C 4.12.3.1.2 Groundwater Table And Buoyancy

Unless the pile is bearing on rock, the tip resistance is primarily dependent on the effective surcharge which is directly influenced by the groundwater level. For drained loading conditions, the vertical effective stress, σ_v', is related to the position of the groundwater level and thus affects shaft capacity.

C 4.12.3.1.3 Effect Of Settling Ground And Downdrag Forces

When a soil deposit, in which, or through which, piles have been installed is subject to consolidation and settles in relation to the piles, downdrag (negative skin friction) forces are induced on the piles. The induced downdrag loads tend to reduce the usable pile capacity.

Downdrag loads are a pile capacity problem only in the case of a true end-bearing pile on very dense or hard soil or rock where the pile capacity is generally controlled by the structural strength of the pile and where settlements of the pile are negligible. In all other cases of piles bearing in compressible soils, where the pile capacity is controlled by tip resistance and shaft adhesion or friction, the problem of downdrag may be regarded as a settlement problem.

Field observation on instrumented piles have shown that the magnitude of downdrag is a function of the effective stress acting on the pile and may be computed in a way similar to the calculation of positive shaft resistance. Downdrag loads can be estimated using the α- or λ- methods. However, an allowance should be made for the possible increase in undrained shear strength as consolidation occurs, since the increase in shear strength will result in higher downdrag loads. An alternative approach would be to used the β-method where the long-term conditions after consolidation should be considered.

Downdrag loads should not be combined with transient loads (e.g., wind and traffic loads) and, therefore, only permanent loads need be included with the downdrag loads, provided that the transient loads are smaller than the downdrag loads.

Downdrag can be reduced by applying a thin coat of bitumen on the pile surface. In the case where downdrag is a capacity problem, the load factors for the downdrag load shall be the reciprocal of the performance factor used for the method of estimating shaft resistance (Table 4.10.6-2). When downdrag is a settlement problem, the load factor for the downdrag load shall be unity.

C 4.12.3.2.3b Cohesionless Soil

The settlement of pile groups in cohesionless soils can be estimated through the use of empirical correlations proposed by Meyerhof (1976). These can be expressed in the following form:

Using SPT, $$\rho = \frac{2qI\sqrt{X}}{N_{corr}} \qquad \text{(C4.12.3.2.3b-1)}$$

Using CPT, $$\rho = \frac{qXI}{2q_c} \qquad \text{(C4.12.3.2.3b-2)}$$

where q = net foundation pressure (tsf) applied at $2D_b/3$ (Figure 4.12.3.2.1-1)

X = width or smallest dimension of pile group

ρ = settlement of pile group

I = influence factor of the effective group embedment

$$I = 1 - D'/8X \geq 0.5 \qquad \text{(C4.12.3.2.3b-3)}$$

$$D' = \text{effective depth} = 2D_b/3 \qquad \text{(C4.12.3.2.3b-4)}$$

D_b = depth of embedment of piles in layer which provides support (Figure 4.12.3.2.1-1)

N_{corr} = representative average corrected (for overburden) SPT blow count over a depth X below the equivalent footing.

$$N_{corr} = \left[0.77 \log_{10}\left(\frac{20}{\sigma_v'}\right)\right]N \qquad \text{(C4.12.3.2.3b-5)}$$

N = measured SPT-N value within the seat of settlement

σ_v' = effective vertical stress (tsf)

q_c = average static cone resistance over a depth X below the equivalent footing (tsf).

C 4.12.3.2.4 Lateral Displacement

The lateral displacement of pile groups can be estimated using the procedures described in the Engineering Manual for Driven Piles (Ooi et al., 1991). The procedure was developed through a parametric study of a large number of pile groups using the theories proposed by Evans and Duncan (1982) and by Focht and Koch (1973).

C 4.12.3.3.1 Axial Loading of Piles

The design criterion for the bearing capacity of piles may be expressed as:

$$\phi_q Q_{ult} \geq \text{Axial Load Effect} \quad \text{(C4.12.3.3.1-1)}$$

where:

ϕ_q = performance factor for the ultimate bearing capacity of pile

Q_{ult} = ultimate bearing capacity of a single pile

$$Q_{ult} = Q_p + Q_s \quad \text{(C4.12.3.3.1-2)}$$

$$Q_p = \text{load carried by pile tip} = q_p A_p \quad \text{(C4.12.3.3.1-3)}$$

$$Q_s = \text{load carried by pile shaft} = q_s A_s \quad \text{(C4.12.3.3.1-4)}$$

q_p = unit tip resistance of pile

q_s = unit skin resistance of pile

A_s = surface area of pile shaft

A_p = area of pile tip

The value of ϕ_q is dependent upon the method used in estimating the pile bearing capacity and may be different for tip and shaft resistances. In this case the factored capacity may be written as:

$$\text{Factored Capacity} = \phi_{qp} Q_p + \phi_{qs} Q_s \quad \text{(C4.12.3.3.1-5)}$$

where:

ϕ_{qp} = performance factor for tip resistance

ϕ_{qs} = performance factor for shaft resistance.

The ultimate bearing capacity of a pile is derived from the tip resistance and/or shaft resistance (skin friction). Both the tip and shaft resistances develop in response to foundation displacement. The maximum values of each are unlikely to occur at the same displacement. The shaft resistance is typically fully mobilized at displacements of about 0.1 to 0.4 in. The tip capacity, however, is mobilized after the pile settles about 8% of its diameter (Kulhawy et al., 1983).

C 4.12.3.3.2 Analytic Estimates Of Pile Capacity

Three analytic methods of estimating skin friction and one analytic method of estimating tip resistance of piles are described below:

Shaft Resistance

1) α-Method

The α-method relates the adhesion between the pile and a clay to the undrained shear strength of the clay. The ultimate unit skin friction, q_s, can be expressed by:

$$q_s = \alpha S_u \quad \text{(C4.12.3.3.2-1)}$$

where:

S_u = mean undrained shear strength

α = adhesion factor applied to S_u.

Tomlinson (1987) found that the adhesion factor, α, varies with the value of the undrained shear strength, S_u, as shown in Figure C4.12.3.3.2-1.

2) β-Method

The β-method is an effective stress method for predicting skin friction of piles. The ultimate unit skin friction, q_s, is related to the effective stresses in the ground as follows:

$$\begin{aligned} q_s &= \sigma_h' \tan \delta \\ &= K \tan \delta \, \sigma_v' \\ &= \beta \, \sigma_v' \end{aligned} \quad \text{(C4.12.3.3.2-2)}$$

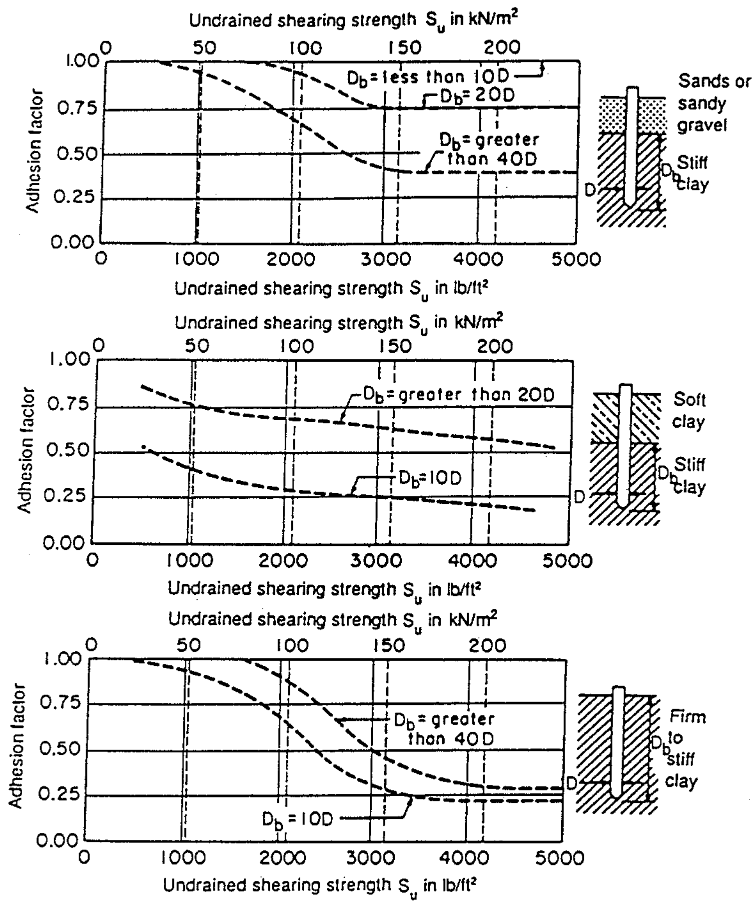


Figure C4.12.3.3.2-1:

Design curves for adhesion factors for piles driven into clay soils (After Tomlinson, 1987)

where:

σ_h' = horizontal effective stress

σ_v' = vertical effective stress

δ = angle of shearing resistance between the soil and the pile

K = coefficient of lateral earth pressure

β = Ktan δ.

Esrig and Kirby (1979) developed the relationship between β and OCR that is shown in Figure C4.12.3.3.2-2. OCR, or overconsolidation ratio is defined as the ratio of the preconsolidation pressure to the vertical effective stress.

The β-method has been found to work best for piles in normally consolidated and lightly overconsolidated clays. The method tends to overpredict skin friction of piles in heavily overconsolidated soils. Esrig and Kirby suggested that for heavily overconsolidated clays, the value of β shall not exceed 2.

3) λ-Method

Vijayvergiya and Focht (1972) recognized that the passive lateral earth pressure can be expressed as $\sigma_h' = \sigma_v' + 2S_u$, and the ultimate unit skin friction of a pile is related to this passive pressure. They proposed the following relationship:

$$q_s = \lambda(\sigma_v' + 2S_u) \qquad \text{(C4.12.3.3.2-3)}$$

where λ is an empirical coefficient shown in Figure C4.12.3.3.2-3. The value of λ decreases with pile length and was found empirically by examining the results of load tests on steel pipe piles.

Tip Resistance

The ultimate unit tip capacity of piles in saturated clay may be expressed as:

$$q_p = N_c S_u \qquad \text{(C4.12.3.3.2-4)}$$

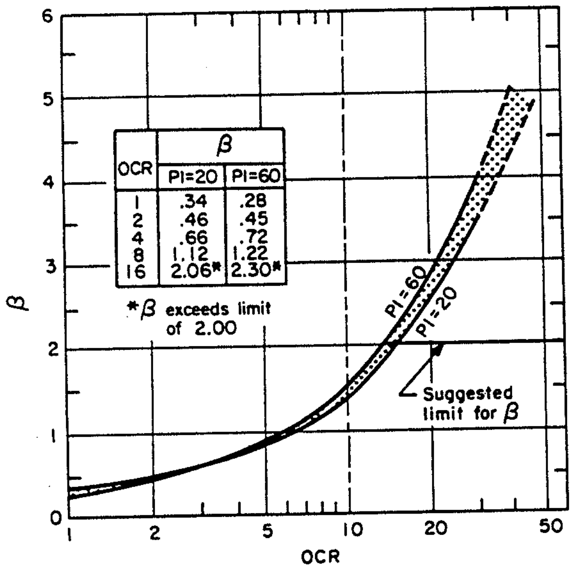


Figure C4.12.3.3.2-2: β versus OCR for full displacement piles (After Esrig and Kirby, 1979)

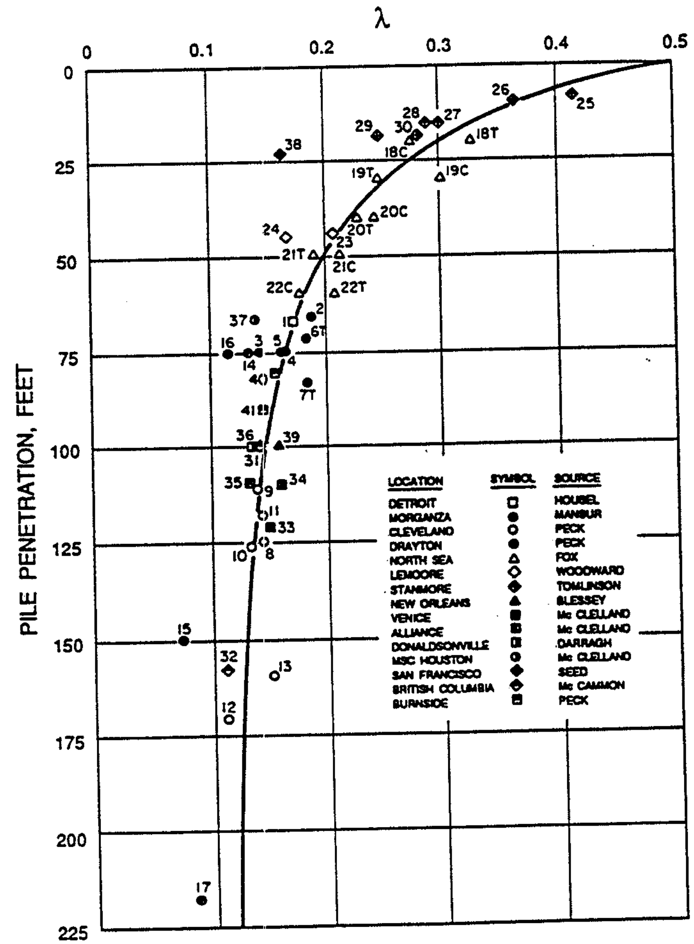


Figure C4.12.3.3.2-3: λ Coefficient for driven pipe piles (After Vijayvergiya and Focht, 1972)

where:

N_c = bearing capacity factor = 9 (Skempton, 1951)

S_u = undrained shear strength of the clay near the pile base.

C 4.12.3.3.3 Pile Capacity Estimates Based On In Situ Tests

In situ tests are widely used in cohesionless soils because obtaining good quality samples of cohesionless soils is very difficult. In situ tests parameters may be used to estimate the tip resistance and skin friction of piles. Two frequently used in situ test methods for predicting pile capacity are the standard penetration test (SPT) method (Meyerhof, 1976) and the cone penetration test (CPT) method (Nottingham and Schmertmann, 1975).

1) SPT Method

Meyerhof (1976) correlated the tip capacity and shaft resistance of piles with the SPT blow-count. This method applies only to sands and non-plastic silts.

(a) Pile Tip Capacity

The ultimate unit tip resistance for piles, q_p (in tons per square foot) driven to a depth D_b into a cohesionless soil stratum can be estimated by:

$$q_p = \frac{0.4 N_{corr}\ D_b}{D} \le q_l \qquad \text{(C4.12.3.3.3-1)}$$

where:

N_{corr} = average SPT-N value near the pile tip corrected for overburden

$$N_{corr} = [0.77\ \log_{10}(\frac{20}{\sigma_v'})]N \qquad \text{(C4.12.3.3.3-2)}$$

N = measured SPT-N value

σ_v' = vertical effective stress measured at the pile tip (in tsf)

D = pile width or diameter

q_l = limiting tip resistance (tons per square foot)

$q_l = 4N_{corr}$ for sands (C4.12.3.3.3-3)

$q_l = 3N_{corr}$ for non-plastic silts. (C4.12.3.3.3-4)

(b) Skin Friction

The skin friction of piles in cohesionless soils may be estimated using the following equation (Meyerhof, 1976):

For driven displacement piles:

$$q_s = \frac{\bar{N}}{50} \qquad \text{(C4.12.3.3.3-5)}$$

For non-displacement piles (e.g., steel-H piles):

$$q_s = \frac{\bar{N}}{100} \qquad \text{(C4.12.3.3.3-6)}$$

where:

q_s = unit skin friction for driven piles (tsf)

$\bar{N}$ = average (uncorrected) SPT-blow count along the pile shaft.

Displacement piles, which have solid sections or hollow sections with a closed end, displace a relatively large volume of soil during penetration. Non-displacement piles usually have relatively small cross-sectional areas, e.g., steel-H piles and open-ended pipe piles that do not plug. Plugging occurs when the soil between the flanges in a steel-H pile or the soil in the cylinder of an open-ended steel pipe pile adheres fully to the pile and moves down with the pile as it is driven.

2) CPT Method

The cone penetration test yields two useful parameters that can be applied to pile capacity prediction: (i) the cone penetration resistance, q_c, which is related to the tip capacity of piles and (ii) sleeve friction, f_s, which can be used to estimate the skin friction capacity. Nottingham and Schmertmann (1975) developed the following procedure for estimating pile capacity:

(a) Pile Tip Capacity

Nottingham and Schmertmann (1975) found that Begemann's procedure gives a good estimation of end bearing capacity in piles for all soil types. Begemann's procedure for estimating the tip resistance, q_p is outlined in Figure C4.12.3.3.3-1. An example of the method is shown in Figure C4.12.3.3.3-2. The minimum average cone resistance between 0.7 and 4 pile diameters below the elevation of the pile tip is obtained by a trial and error process, with the use of the minimum-path rule. The minimum-path rule is also used to find the value of cone resistance for the soil for a distance of eight pile diameters above the tip. The two results are then averaged to give the pile tip resistance.

(b) Skin Friction

Nottingham and Schmertmann's (1975) procedure can be used for computing the ultimate skin friction of piles:

$$Q_s = K_{s,c}\left[\sum_{L_f=0}^{8D} (L_f/8D) f_s a_s + \sum_{L_f=8D}^{Z} f_s a_s\right] \tag{C4.12.3.3.3-7}$$

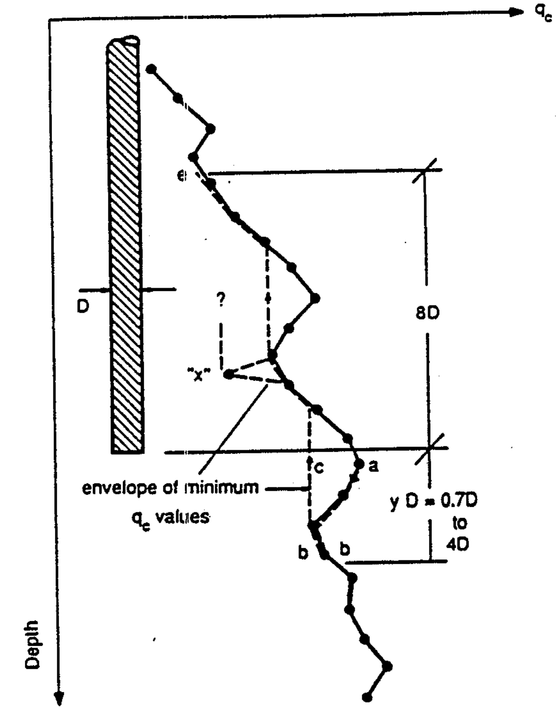
where:

Q_s = ultimate skin friction capacity of the pile

$K_{s,c}$ = correction factors: K_c for clays and K_s for sands (see Figure C4.12.3.3.3-3)

L_f = depth to point considered

D = pile width or diameter

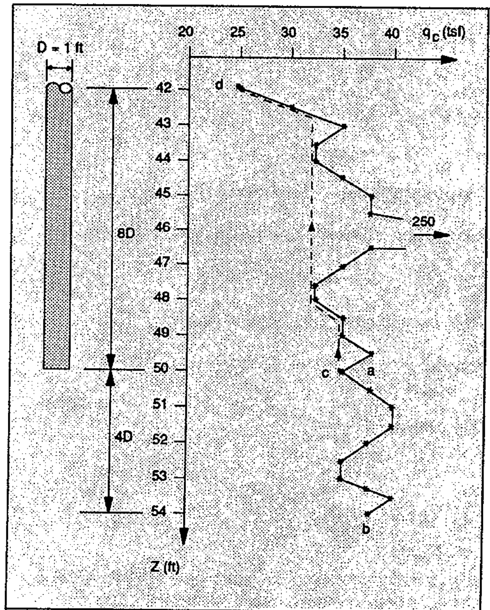


$$q_p = \frac{q_{c1} + q_{c2}}{2}$$

q_{c1} = Average of all values of q_c along path a-b-c over a distance of yD below the pile tip. Sum q_c values measured at each elevation in the downward path a-b. Sum q_c values at every elevation where a cone resistance reading is made, along the upward path b-c, but at each elevation take the minimum of (i) the q_c value at that elevation or (ii) the lowest q_c value between that elevation and the elevation of point b. This method of determining q_c is called the "minimum path" rule. Compute q_{c1} for y-values from 0.7 to 4.0 and use the minimum q_{c1} value obtained.

q_{c2} = Average q_c over a distance of 8D above the pile tip (path c-e). Use the minimum path rule as for path b-c in the q_{c1} computations. Ignore any very extreme peaks or depressions (such as "x" in the diagram above) if the soil is a sand, but include these in minimum path if the soil is a clay.

Figure C4.12.3.3.3-1: Pile End-Bearing Computation Procedure After Begemann (After Nottingham and Schmertmann, 1975)



$$q_{c2} = \frac{[4(35) + 11(32.5) + 30 + 25]}{17}$$

$$= \underline{32.5 \text{ tsf}}$$

y	yD (ft)	Z + yD (ft)	q_{c1} (tsf)
0.7	0.7	50.7	(35+37.5+37.5+35)/4 = 36.3
1.0	1.0	51.0	(35+37.5+40+40+37.5+35)/6 = 37.5
1.5	1.5	51.5	(35+37.5+40+40+40+40+37.5+35)/8 = 38.1
2.0	2.0	52.0	(35+37.5+40+40+37.5+37.5+37.5+37.5+35.7+35)/10 = 37.5
2.5	2.5	52.5	(35+37.5+40+40+37.5+35+6(35))/12 = 36.3
3.0	3.0	53.0	(35+37.5+40+40+37.5+35+35+7(35))/14 = 36.1
3.5	3.5	53.5	(35+37.5+40+40+37.5+35+35+40+40+7(35))/16 = 36.6
4.0	4.0	54.0	(35+37.5+40+40+37.5+37.5+35+40+37.5+2(37.5)+7(35))/18 = 36.7

$$q_p = (36.1 + 32.5)/2 = 34.3 \text{ tsf}$$

Figure C4.12.3.3.3-2: Example of the End Bearing Computation Procedure Using the Minimum Path Rule

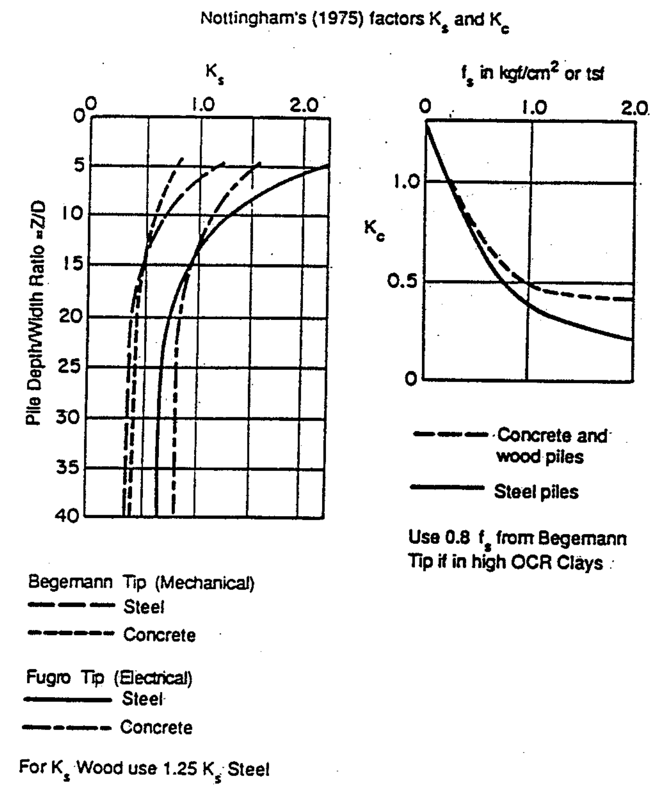


Figure C4.12.3.3.3-3: Shaft friction correction factors (After Nottingham and Schmertmann, 1975)

f_s = unit local sleeve friction resistance from CPT at the point considered

a_s = pile perimeter

.Z = total embedded pile length.

C 4.12.3.3.4 Piles Bearing On Rock

The ultimate unit end-bearing capacity (q_p) of piles driven to rock may be estimated from the uniaxial compression strength of the rock as follows (Canadian Geotechnical Society, 1985):

$$q_p = 3 q_u K_{sp} d \qquad \text{(C4.12.3.3.4-1)}$$

where:

q_u = average uniaxial compression strength of the rock core

K_{sp} = dimensionless bearing capacity coefficient (Figure C4.12.3.3.4-1)

$$K_{sp} = \frac{3 + s_d/D}{10(1 + 300 t_d/s_d)^{0.5}} \qquad \text{(C4.12.3.3.4-2)}$$

d = dimensionless depth factor = $1 + 0.4 H_s/D_s \leq 3.4$

s_d = spacing of discontinuities

t_d = width of discontinuities

D = pile width or diameter

H_s = depth of embedment of pile socketed into rock

= 0 for piles resting on top of bedrock

D_s = diameter of socket.

This method is not applicable to soft stratified rocks, such as shale or limestone. When this method is applicable, the rocks are usually so sound that the structural capacity will govern the design (Fellenius et al., 1989). This method is applicable only if (1) s_d > 1 ft; (2) t_d < 0.25 in. for unfilled discontinuities or t_d < 1 in. for discontinuities filled with soil or rock; and (3) D > 1 ft.

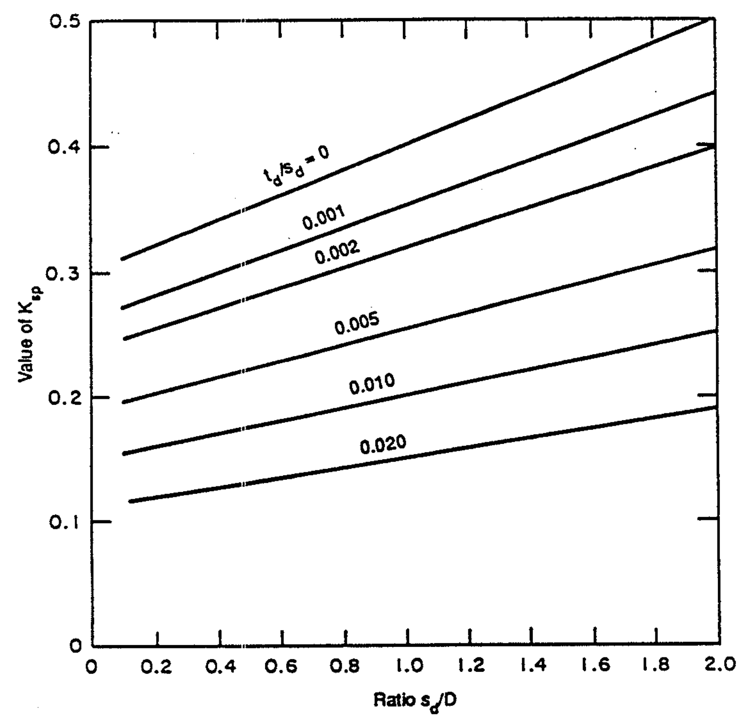


Figure C4.12.3.3.4-1: Bearing capacity coefficient, K_{sp} (After Canadian Foundation Engineering Manual, 1985)

C 4.12.3.3.5 Pile Load Test

Load testing can be performed as either a routine load test or a high-level load test. A routine test is usually carried out for the purpose of proof testing and involves limited quality control of the test data, a limited number of test piles, and only limited analysis of the test results. A high-level test is usually carried out before finalizing the design and involves more reliable test data, a testing program and method that are adapted for the particular site conditions and problems, and a detailed analysis of the results.

Test Arrangement and Load Test Methods

Every load test must be arranged in conformity with ASTM D1143-81. If the minimum distance values recommended in the ASTM standard are reduced, the reliability and usefulness of the test results could be impaired. The test load is usually applied by means of a hydraulic jack, which is also acting as a load cell. However, where a higher accuracy and confidence in the test results is needed, such as in high-level testing, a separate load cell should be used.

Measurements of the movement of the pile end by means of a telltale to the pile end and, therefore, the compression of the pile, should be considered for high-level testing and, wherever possible, also for routine tests on long piles.

Where the objective is to determine the factored axial bearing capacity of the pile for a limit state design the quick tests methods (ASTM D1143-81, reapproved 1987) are technically preferable over the slow methods. For uplift of piles, see ASTM D3689-83, and for lateral load testing of piles, see ASTM D3966-81.

Interpretation of Load Test Results

Tests, other than proof tests, should be carried to failure. The designated failure load shall be based on the shape of the load-movement diagram.

A lower bound value for vertical pile capacity may be estimated using Davisson's method (NAVFAC, 1982).

C 4.12.3.3.7a Single Pile Uplift Capacity

The design requirement for uplift is as follows:

$$\phi_u Q_s \geq P_{x,y} \qquad \text{(C4.12.3.3.7a-1)}$$

where:

Q_s = ultimate uplift capacity due to shaft resistance

$P_{x,y}$ = factored tensile load effect in the pile

ϕ_u = performance factor for uplift capacity.

The performance factors for axial tension are lower than those for compression. One reason for this is that piles in tension unload the soil; this reduces the overburden effective stress and hence the uplift skin friction resistance of the pile.

C 4.12.3.3.7b Pile Group Uplift Capacity

For pile groups in cohesionless soil, the weight of the block that will be uplifted shall be estimated using a spread of load of 1 in 4 (Figure C4.12.3.3.7b-1) from the base of the pile group.

In cohesive soils, the uplift resistance of the block in undrained shear is given by (Figure C4.12.3.3.7b-2):

$$Q_{ug} = (2XZ + 2YZ)\overline{S}_u + W_g \qquad \text{(C4.12.3.3.7b-1)}$$

where:

Q_{ug} = ultimate uplift resistance of the group

X = width of the group

Y = length of the group

Z = depth of the block of soil below pile cap

S_u = average undrained shear strength along pile shaft

W_g = weight of the block of soil, piles and pile cap.

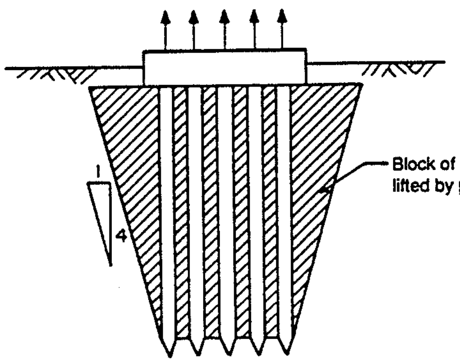


Figure C4.12.3.7.2-1: Uplift of group of closely-spaced piles in cohesionless soils

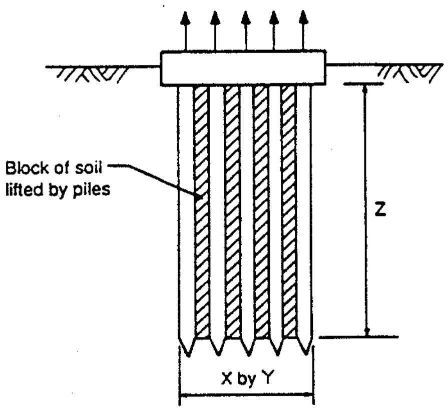


Figure C4.12.3.7.2-2: Uplift of group of piles in cohesive soils (After Tomlinson, 1987)

C 4.12.3.3.8 Lateral Load

The resistance of piles to carry lateral loads is usually governed by lateral movement criteria at the service limit state, Article 4.12.3.2, or structural failure of the piles at the strength limit state.

When groups of piles are subjected to lateral loads, there is interaction among the piles, through the ground between them. As a result, groups of piles deflect more than single piles subjected to the same lateral load per pile, and the bending moments in the piles in the group are also larger. These factors should be accounted for in design.

C 4.12.3.3.9 Batter Pile

When lateral loads acting on a foundation are large, batter piles provide an effective way of transmitting loads to the soil. The degree of batter will depend on the type of pile and the magnitude of the lateral loads. Installation by driving is feasible for batters as large as 1 horizontal:2 vertical (Tomlinson, 1987).

There are situations where the use of batter piles may be undesirable. These include conditions involving large settlements in compressible clays. Settlement induces bending moments in the shafts of batter piles (Tomlinson, 1987).

According to Tomlinson, the greatest efficiency is achieved by using piles battered in opposite directions. He describes a simple graphical procedure for estimating the compressive and tensile forces in pile groups containing not more than 3 rows of piles. The procedure is based on the assumption that (a) the battered piles are pinned at their point of intersection, (b) vertical piles in the group do not carry lateral loads and (c) batter piles carry only axial loads. Tomlinson's procedure does not consider pile-soil-pile interaction, pile stiffness, soil stiffness, and pile head fixity, all of which can significantly affect the distribution of forces in piles in a pile group. Nevertheless, Tomlinson's graphical procedure is useful for obtaining a preliminary pile layout, and is reasonably accurate if the lateral load is less than 20 percent of the vertical load (Department of the Army, in press).

If the pile group has more than three rows, Tomlinson's simple procedure is not applicable, and as mentioned previously, it may be inaccurate if the lateral loads are large. More complex methods based on linear elastic and non-linear elastic soil response are available for analyzing two and three

dimensional pile groups. These methods are often quite involved and require the use of a computer.

Hrennikoff's (1950) linear elastic procedure may be used to solve for the pile forces and displacements in pile groups that can be modelled in two dimensions. Saul (1968) expanded Hrennikoff's solution to three dimensions. O'Neill, Ghazzaly and Ha (1977) and O'Neill and Tsai (1984) have developed a method of analysis for three dimensional pile groups that considers non-linear soil response and pile-soil-pile interaction.

C 4.12.3.3.10a Cohesive Soil

The efficiency of pile groups in cohesive soil is diminished from the individual pile case due to overlapping zones of shear deformation in the soil surrounding the piles.

In cohesive soils, the resistance for a pile group depends on whether the cap is in firm contact with the ground beneath. If the cap is in firm contact, the soil between the pile and the pile group behave as a unit.

At small pile spacings, a block type failure mechanism may prevail while individual pile failure may occur at larger pile spacings. It is necessary to check for both failure mechanisms and design for the case that yields the minimum capacity.

C 4.12.3.3.10b Cohesionless Soil

For piles driven into sand, the group capacity is never less than the sum of the individual pile capacities, because of the increase in density caused by driving. Thus, the efficiency factor is always taken as unity for pile groups in sand.

C 4.12.3.3.10c Pile Group in Strong Soil Overlying a Weak or Compressible Soil

Meyerhof (1976) suggested that if the distance between the pile tip and the weak deposit (H) is less than 10 pile diameters, the ultimate unit tip resistance will be:

$$q_p = q_o \frac{(q_l - q_o)H}{10D} \leq q_l \qquad \text{(C4.12.3.3.10c-1)}$$

where:

q_l = limiting unit tip resistance in the upper stratum

q_o = limiting resistance in the lower weak stratum.

The limiting unit tip resistance of piles in sands and non-plastic silts may be estimated using Equations C4.12.3.3.3-3 and C4.12.3.3.3-4, respectively. The unit tip resistance of piles in cohesive soils may be estimated using Equation C4.12.3.3.2-4.

C 4.12.4.1 Buckling Of Piles

Piles which extend for a portion of their lengths through water or air (e.g., pile bent piers) shall be assumed to be fixed at some depth below the ground. Stability shall be determined in accordance with provisions for compression members in Sections 8, 9, and 10 using an equivalent length of the pile equal to the laterally unsupported length plus an embedded depth to fixity.

The depth to fixity can be calculated as follows (Davisson and Robinson, 1965)

For clays Depth to fixity = 1.4R (C4.12.4.1-1)

For sands Depth to fixity = 1.8T (C4.12.4.1-2)

where:

$$R = \left|\frac{E_p I_p}{E_s}\right|^{0.25} \qquad \text{(C4.12.4.1-3)}$$

E_p = modulus of elasticity of pile

I_p = moment of inertia of pile

E_s = soil modulus = 67 S_u for clays (C4.12.4.1-4)

S_u = undrained shear strength of clays

$$T = \left|\frac{E_p I_p}{n_h}\right|^{0.2} \qquad \text{(C4.12.4.1-5)}$$

n_h = rate of increase of soil modulus with depth (Table C4.12.4.1-1).

Table C4.12.4.1-1 Coefficient of horizontal subgrade reaction n_h (lb/in^3)

Density	Dry or Moist Sand	Submerged Sand
Loose	30	15
Medium	80	40
Dense	200	100

The effect of pile spacing on the soil modulus has been studied by Prakash (1990). He found that, at pile spacings greater than 8 times the pile width, neighboring piles have no effect on the soil modulus or buckling capacity. However, at a pile spacing of 3 times the pile width, the effective soil modulus is reduced to 25% of the value applicable to a single pile. For intermediate spacings, modulus values can be estimated by interpolation.

C 4.13.3 GEOTECHNICAL DESIGN

Drilled shafts may be constructed using the dry, casing or wet method of construction, or a combination of methods. In every case, hole excavation, concrete placement and all other aspects of shaft construction shall be performed in conformance with the provisions of the construction specifications in Division II (NCHRP Project 12-34).

When the quality of a completed shaft is suspect, the presence and effect of defects should be determined by destructive and nondestructive testing.

The performance of drilled shaft foundations can be greatly affected by the method of construction, particularly side resistance. The designer should consider the effects of ground and groundwater conditions on shaft construction operations and delineate, when necessary, the general method of construction to be followed to ensure the expected performance. Because shafts derive their capacity from side and tip resistance which are a function of the condition of the materials in direct contact with the shaft, it is important that the construction procedures be consistent with the material conditions assumed in the design. Softening, loosening or other changes in soil and rock conditions caused by the construction method could result in a reduction in shaft capacity and an increase in shaft displacement. Therefore, evaluation of the effects of shaft construction procedure on load capacity should be considered an inherent aspect of the design.

Drilled shafts constructed in dry, noncaving soils can usually be excavated without lateral support of the hole. Other ground conditions where caving, squeezing or sloughing soils are present require installation of a steel casing or use of a slurry for support of the hole. Such conditions and techniques may result in loosening of soil around the shaft, or altering the frictional resistance between the concrete shaft and surrounding soil.

C 4.13.3.1.1 Downdrag Loads

Relative downward soil movements of about 0.1 to 0.5 in. are sufficient to mobilize full downdrag load loading on shafts.

Downdrag loads are a capacity problem only in the case of a true end-bearing shaft on very dense or hard soil or rock where the capacity is controlled by the structural strength of the shaft and where the settlement is negligible. In all other cases of shafts bearing in compressible soils, where the capacity is controlled by tip resistance and shaft adhesion or friction, the problem of downdrag loads may be regarded as a settlement problem.

The magnitude of downdrag loads may be computed in a way similar to that of calculating positive shaft resistance. When designing for downdrag, it should be considered that the downdrag loads do not combine with live loads (e.g., wind and traffic loads) and therefore, only permanent loads need to be included with downdrag loads. Downdrag loads are especially important when the factored live loads are smaller than the factored downdrag loads.

In the case where downdrag is a capacity problem (for end bearing piers), the load factors for the downdrag load shall be the reciprocal of the performance factor used for the method of estimating the ultimate shaft resistance (see Table 4.10.6-2). When downdrag is a settlement problem, the load factor for the downdrag load shall be unity. Downdrag loads are especially important in settlement calculations when the live loads are smaller than the downdrag loads.

C 4.13.3.1.2 Uplift

(See Piles)

Evaluation of potential uplift loads on drilled shafts extending through expansive soils requires evaluation of the swell potential of the soil and the extent of the soil strata which may affect the shaft. One reasonably reliable method for identifying swell potential is presented in Table C4.13.3.1.2-1 which classifies swell potential as a function of the Atterberg limits, soil suction and percent swell from oedometer tests (Reese and O'Neill, 1988). The thickness of the potentially expansive stratum must be identified by examination of soil samples from borings for the presence of jointing, slickensiding or a blocky structure, and changes in color, and laboratory testing for determination of soil moisture content profiles.

TABLE C4.13.3.1.2-1 Method for Identifying Potentially Expansive Soils (Reese and O'Neill, 1988)

Liquid Limit LL (%)	Plastic Limit PL (%)	Soil Suction Pressure (ksf)	Potential Swell (%)	Swell Classification
> 60	>35	> 8	> 1.5	High
50 - 60	25 - 35	3 - 8	0.5 - 1.5	Marginal
< 50	< 25	< 3	< 0.5	Low

C 4.13.3.2.1 General

Settlement of drilled shafts installed in sand and rock are usually small, and they occur fairly rapidly. However, shafts in clay may settle over a longer period of time as the clay consolidates under the imposed load.

See also Article C 4.12.3.2.1.

C 4.13.3.2.3a Settlement of Single Drilled Shafts

Reese and O'Neill (1988) have summarized load-settlement data for drilled shafts in dimensionless form as shown in Figures C4.13.3.2.3a-1 through C4.13.3.2.3a-4. Figures C4.13.3.2.3a-1 and C4.13.3.2.3a-2 show the load-settlement curves in side resistance and in end bearing for shafts in clay. Figures C4.13.3.2.3a-3 and C4.13.3.2.3a-4 are similar curves for shafts in sand. These curves provide a useful guide for estimating short-term settlements of drilled shafts.

The values of the load-settlement curves in side resistance were obtained at different depths, taking into account elastic shortening of the shaft. While elastic shortening may be small in relatively short shafts, it may be quite substantial in longer shafts. The amount of elastic shortening in drilled shafts varies with depth. Reese and O'Neill (1988) have described an approximate procedure for estimating the elastic shortening of long drilled shafts.

Long-term settlements of drilled shafts in clay are not reflected in Figures C4.13.3.2.3a-1 and C4.13.3.2.3a-2. Consolidation settlements should be added to the short-term settlements. However, since drilled shafts are usually installed

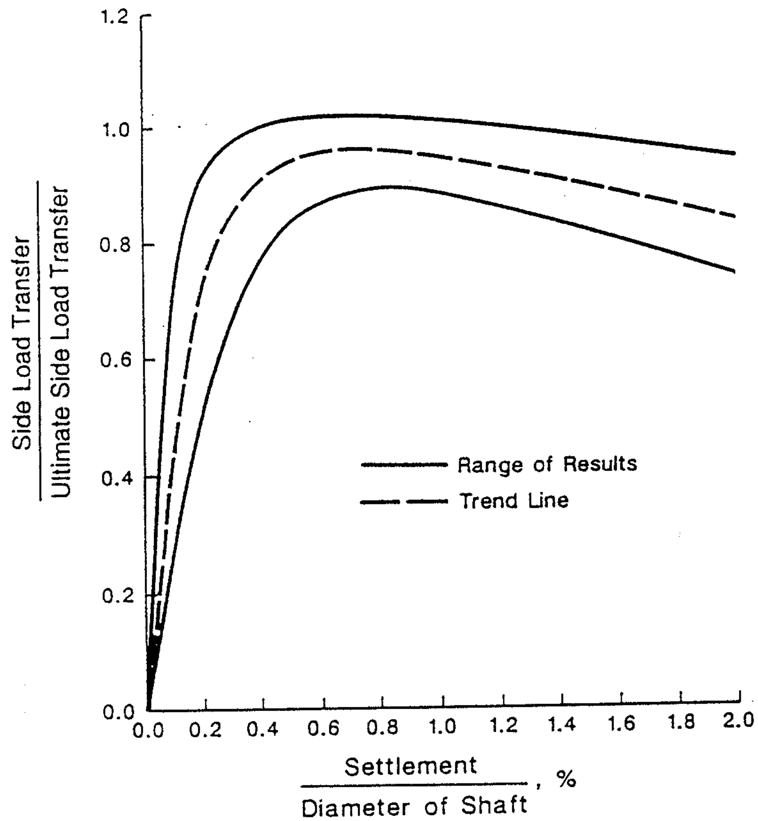


Figure C4.13.3.2.3a-1: Normalized Curves Showing Load Transfer in Side Resistance Versus Settlement for Drilled Shafts in Clay (From Reese and O'Neill, 1988)

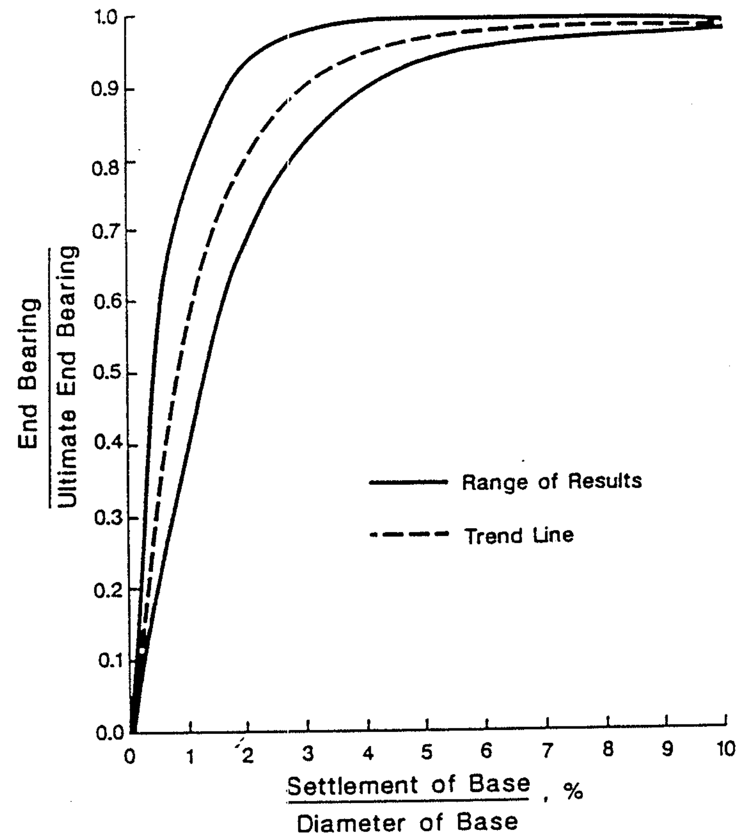


Figure C4.13.3.2.3a-2: Normalized Curves Showing Load Transfer in End Bearing Versus Settlement for Drilled Shafts in Clay (From Reese and O'Neill, 1988)

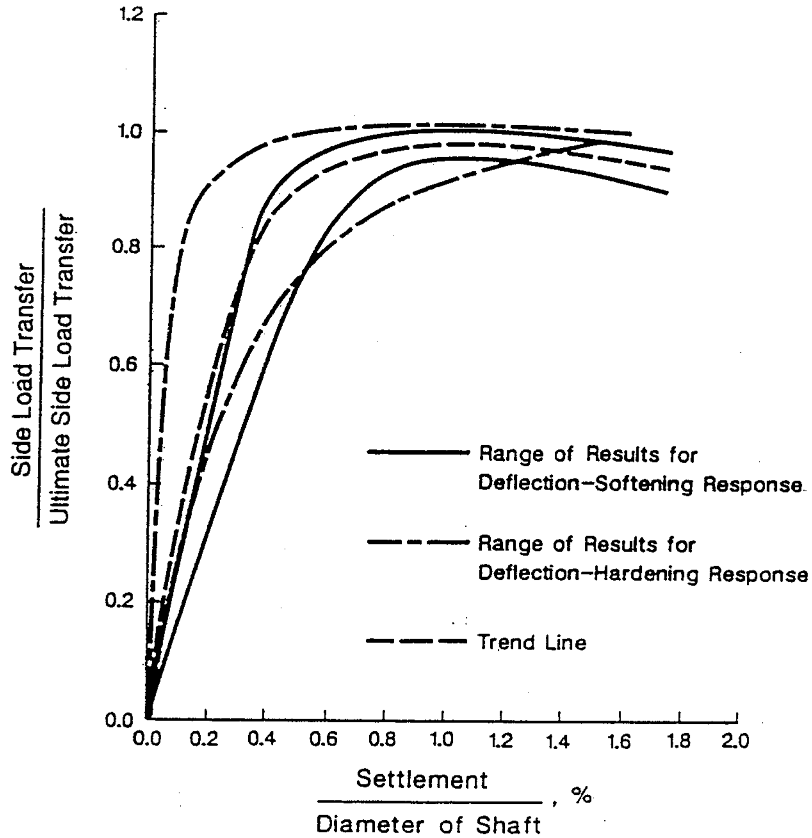


Figure C4.13.3.2.3a-3: Normalized Curves Showing Load Transfer in Side Resistance Versus Settlement for Drilled Shafts in Cohesionless Soils (From Reese and O'Neill, 1988)

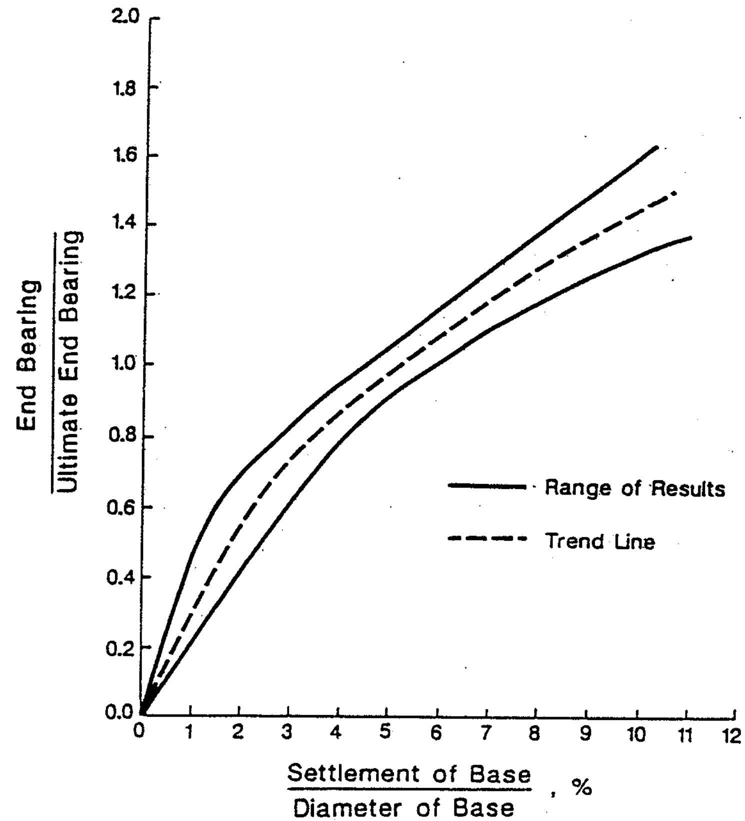


Figure C4.13.3.2.3a-4: Normalized Curves Showing Load Transfer in End Bearing Versus Settlement for Drilled Shafts in Cohesionless Soils (From Reese and O'Neill, 1988)

in heavily overconsolidated soils, consolidation settlements are usually small.

Settlements induced by loads in end bearing are different for shafts in sand and in clay. While drilled shafts in clay typically have a well-defined break in a load-displacement curve for shafts in sand, there is no well defined failure at any displacement. The load of drilled shafts in sand continues to increase as the settlement increases beyond 5% of the base diameter. Q_p is typically fully mobilized at displacements of 2% to 5% of the base diameter for shafts in clay. The ultimate unit end bearing is defined arbitrarily as the bearing pressure required to cause settlement equal to 5% of the pier diameter, even though this does not correspond to complete failure of the soil beneath the base of the pier.

The curves in Figures C4.13.3.2.3a-1 and C4.13.3.2.3a-3 also show the settlements at which the ultimate side resistance is mobilized. Q_s is typically fully mobilized at displacements of 0.2% to 0.8% of the shaft diameter for shafts in clay. For shafts in sand, this value is 0.1% to 1.0%.

C 4.13.3.2.4 Lateral Displacement

The lateral displacement of single drilled shafts and groups of drilled shafts can be estimated using the procedures described in the Engineering Manual for Drilled Shafts (Ooi et al., 1991)

C 4.13.3.3.1 Axial Loading Of Drilled Shafts

See Article C 4.12.3.3.1.

C 4.13.3.3.2 Analytic Estimates Of Drilled Shaft Capacity In Cohesive Soils

Drilled shafts in cohesive soils may be designed by total and effective stress methods of analysis, for undrained and drained loading conditions, respectively. Shafts in cohesionless soils shall be designed by effective stress methods of analysis for drained loading conditions or empirical methods based on in situ test results. The α-method, a total stress method for drilled shafts in cohesive soils, is presented below.

Shaft resistance

α-method

For shafts in cohesive soil loaded under undrained loading conditions, the ultimate unit side resistance may be estimated as follows:

$$q_s = \alpha S_u \qquad \text{(C4.13.3.3.2-1)}$$

where

S_u = mean undrained shear strength

α = adhesion factor.

Refer to Table C4.13.3.3.2-1 for guidance regarding selection of α. The values of α shown in Table C4.13.3.3.2-1 reflect the values for clay (clay is defined as a material with $S_u < 2$ tsf) and a transitional material between clay and rock (2 tsf $< S_u <$ 9 tsf) as recommended by Reese and O'Neill (1988).

Environmental, long-term loading or construction factors may dictate that the depth ignored in estimating the shaft resistance should be greater than the 5 ft indicated in Table C4.13.3.3.2-1. Refer to Figure C4.13.3.3.2-1 for identification of portions of a drilled shaft not considered in contributing to the shaft resistance.

The adhesion factor α is an empirical factor used to correlate the results of full scale load tests with a material property or characteristic of the cohesive soil. The adhesion factor is usually related to S_u and is derived from the results of full-scale pile and drilled shaft load tests. Use of the approach presumes the measured value of S_u is the correct value and that all shaft behavior resulting from construction and loading can be lumped into a single parameter. Neither presumption is strictly correct, but the approach is used due to its simplicity in dealing with a complex problem.

The upper five feet of the shaft is ignored in estimating Q_s to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading and low lateral stresses from freshly-placed concrete. The lower one diameter above the shaft tip or top of enlarged base is ignored due to the development of tensile cracks in the soil near these regions of the shaft and a corresponding reduction in lateral stress and side resistance.

TABLE C4.13.3.3.2-1 Recommended values of α for drilled shafts in clay (After Reese and O'Neill, 1988)

Location Along Drilled Shaft	Undrained Shear Strength, S_u	Value of α
From ground surface to depth along drilled shaft of 5 ft*	-	0
Bottom 1 diameter of the drilled shaft or 1 stem diameter above the top of the bell	-	0
All other points along the sides of the drilled shaft	< 2 tsf	0.55
	2 - 3 tsf	0.49
	3 - 4 tsf	0.42
	2 - 3 tsf	0.38
	3 - 4 tsf	0.35
	2 - 3 tsf	0.33
	3 - 4 tsf	0.32
	2 - 3 tsf	0.31
	> 9 tsf	Treat as Rock

* The depth of 5 ft may need to be increased if the drilled shaft is installed in expansive clay, or if there is substantial groundline deflection from lateral loading.

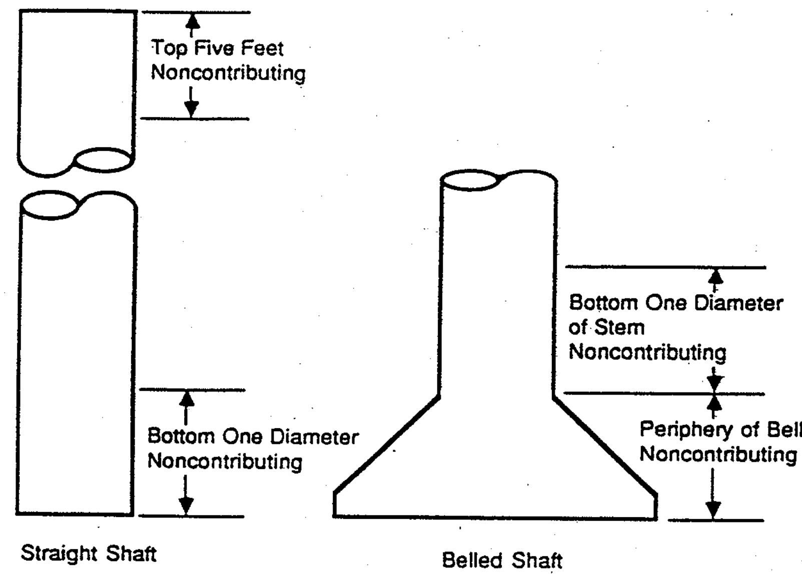


Figure C4.13.3.3.2-1: Explanation of Portions of Drilled Shafts Not Considered in Computing Side Resistance (From Reese and O'Neill, 1988)

Bells or underreams constructed in stiff fissured clay often settle sufficiently to result in the formation of a gap above the bell which will eventually be filled by slumping soil. Slumping will tend to loosen the soil immediately above the bell and decrease the side resistance along the lower portion of the shaft.

The value of α for driven piles is often considered to vary as a function of S_u. Values of α for drilled shafts are recommended in Table C4.13.3.3.2-1 based on the results of back-analyzed full-scale load tests. This recommendation is based on eliminating the upper five feet and lower one diameter of the shaft length during back-analysis of load test results. The load tests were conducted in insensitive cohesive soils. Therefore, if shafts are constructed in sensitive clays or clay-like shales or mudstones, values of α may be different than those shown in Table C4.13.3.3.2-1. Other values of α may be used if based on the results of load tests.

Tip resistance

For axially loaded shafts in cohesive soil subjected to undrained loading conditions, the ultimate unit tip resistance of drilled shafts may be estimated using the following equation (Reese and O'Neill, 1988):

$$q_p = N_c S_u \leq 80 \text{ ksf} \qquad \text{(C4.13.3.3.2-2)}$$

where:

N_c = bearing capacity factor

$$N_c = 6[1 + 0.2(Z/D_p)] \leq 9 \qquad \text{(C4.13.3.3.2-3)}$$

D_p = diameter of drilled shaft

Z = penetration of shaft.

The value of S_u shall be determined from the results of in situ and/or laboratory testing of undisturbed samples obtained within a depth of two diameters below the tip of the shaft. If the soil within two diameters of the tip is of soft consistency, the value of N_c shall be reduced by one third.

If $D_p > 75$ in. and shaft settlements will not be evaluated, the value of q_p shall be reduced as follows (Reese and O'Neill, 1988):

$$q_{pr} = F_r q_p \leq 80 \text{ ksf} \qquad \text{(C4.13.3.3.2-4)}$$

where:

$$F_r = \frac{2.5}{aD_p + 2.5b} \leq 1.0 \qquad \text{(C4.13.3.3.2-5)}$$

$$a = 0.0071 + 0.0021 \frac{Z}{D_p} \leq 0.015 \qquad \text{(C4.13.3.3.2-6)}$$

$$b = 0.45\sqrt{S_u(\text{ksf})} \quad \text{where } 0.5 \leq b \leq 1.5 \qquad \text{(C4.13.3.3.2-7)}$$

D_p = Tip diameter in inches.

The limiting value of 80 ksf for q_p and q_{pr} is not a theoretical limit but a limit based on the largest measured values. A higher limiting value may be used if based on the results of a load test.

The use of Equation C4.13.3.3.2-2 to estimate the tip resistance for drilled shafts having diameters greater than 75 inches is not recommended because the deformations required to fully mobilize the calculated value of Q_p will normally be greater than can be tolerated by highway structures. Therefore, for large diameter shafts founded on stiff to hard clay, the limiting value for the ultimate unit tip resistance shall be reduced as described in Equation C4.13.3.3.2-4.

C 4.13.3.3.3 Estimation Of Drilled-Shaft Capacity In Cohesionless Soils

While many field load tests have been performed on drilled shafts in clays, very few have been performed on drilled shafts in sands. The shear strength of cohesionless soils can be characterized by an angle of internal friction (ϕ') or empirically related to its SPT blow count (N). Methods of estimating shaft resistance and end bearing are presented below.

Shaft Resistance

Table C4.13.3.3.3-1 summarizes five methods of predicting shaft resistance of bored piles in sand. Quiros and Reese (1977) and Reese and O'Neill (1988) limited the unit side resistance to 2 tsf, corresponding to the maximum value ever measured, and Touma and Reese (1974) limited the unit side resistance to 2.5 tsf. These values however, are not theoretical limits. Higher values can be used if verified by load tests.

Table C4.13.3.3.3-1 Summary of Procedures for Estimating Side Resistance (q_s) of Drilled Shafts in Sand

REFERENCE	DESCRIPTION
Touma and Reese (1974)	$q_s = K\sigma_v' \tan\phi' < 2.5$ tsf where $K = 0.7$ for $D_b \le 25$ ft $K = 0.6$ for 25 ft $< D_b \le 40$ ft $K = 0.5$ for $D_b > 40$ ft
Meyerhof (1976)	$q_s\ (\text{tsf}) = \dfrac{N}{100}$
Quiros and Reese (1977)	$q_s\ (\text{tsf}) = 0.026N < 2$ tsf
Reese and Wright (1977)	$q_s\ (\text{tsf}) = \dfrac{N}{34}$ for $N \le 53$ $q_s\ (\text{tsf}) = \dfrac{N - 53}{450} + 1.6$ for $53 < N \le 100$
Reese and O'Neill (1988)	$q_s\ (\text{tsf}) = \beta\sigma_v' \le 2$ tsf for $0.25 \le \beta \le 1.2$ where $\beta = 1.5 - 0.135\sqrt{z}$

where N = uncorrected SPT blow count

σ_v' = vertical effective stress

ϕ' = friction angle of sand

K = load transfer factor

D_b = embedment of drilled shaft in sand bearing layer

β = load transfer coefficient

z = depth below ground surface in feet.

It may be noted that the side resistance of drilled shafts in sand can be estimated using (a) the friction angle [Touma and Reese (1974)] or (b) the SPT blow count [Meyerhof (1976), Quiros and Reese (1977), and Reese and Wright (1977)].

The friction angle of sands can be correlated to the Standard Penetration Test blow count or the cone resistance as given in Table C4.13.3.3.3-2.

Table C4.13.3.3.3-2 Friction Angle of Sands Related to Test Values

Consistency	ϕ'	SPT-N	q_c (kg/cm^2)
Very loose	< 30°	0 - 4	< 20
Loose	30° - 35°	4 - 10	20 - 40
Medium	35° - 40°	10 - 30	40 - 120
Dense	40° - 45°	30 - 50	120 - 200
Very Dense	> 45°	> 50	> 200

Reese and O'Neill (1988) proposed a method for uncemented sands that uses a different approach in that the shaft resistance is independent of the soil friction angle or the SPT blow count. According to them, the friction angle approaches a common value due to high shearing strains in the sand caused by stress relief during drilling.

Tip Resistance

Load tests show that large settlements are required to mobilize the maximum end bearing resistance of drilled shafts in sands. Since large settlements are not tolerable in most structures, the procedures presented in Table C4.13.3.3.3-3 for calculating the ultimate unit end bearing capacity (q_p) are based

Table C4.13.3.3.3-3 Summary of Procedures for Estimating Base Resistance, q_p (tsf) of Drilled Shafts in Sand

REFERENCE	DESCRIPTION
Touma and Reese (1974)	Loose: q_p (tsf) $= 0$ Medium Dense: q_p (tsf) $= \frac{16}{k}$ Very Dense: q_p (tsf) $= \frac{40}{k}$ $k = 1$ for $D_p < 1.67$ ft & $k = 0.6D_p$ for $D_p \geq 1.67$ ft Applicable only if $D_b > 10D$
Meyerhof (1976)	$q_p \text{ (tsf)} = \frac{2N_{corr}D_b}{15D_p} < \frac{4}{3}N_{corr}$ for sand $< N_{corr}$ for nonplastic silts
Quiros and Reese (1977)	Same as Touma and Reese (1974)
Reese and Wright (1977)	$q_p \text{ (tsf)} = \frac{2}{3}N$ for $N \leq 60$ q_p (tsf) $= 40$ for $N > 60$
Reese and O'Neill (1988)	q_p (tsf) $= 0.6N$ for $N \leq 75$ q_p (tsf) $= 45$ for $N > 75$

where N_{corr} = SPT blow count corrected for overburden pressure

$= [0.77\log_{10}(20/\sigma_v')]N$

N = uncorrected SPT blow count

D_p = base diameter of drilled shaft in ft

D_b = embedment of drilled shaft in sand bearing layer

on a downward movement equal to either 1 inch [Touma and Reese (1974) and Quiros and Reese (1977)] or 5% of the base diameter [Reese and Wright (1977) and Reese and O'Neill (1988)].

Reese and O'Neill (1988) recommend that for base diameters greater than 50 in., q_p should be reduced to q_{pr} as follows:

$$q_{pr} = \frac{50}{D_p} q_p \qquad \text{(C4.13.3.3.3-1)}$$

where q_{pr} = reduced base resistance for $D_p > 50$ in.

D_p = diameter of the base of the shaft (in.)

q_p = ultimate unit end bearing resistance calculated using one of the methods in Table C4.13.3.3.3-3

Use of Equation C4.13.3.3.3-1 assumes that q_p is estimated using the Reese and O'Neill (1988) equation in Table C4.13.3.3.3-3.

Meyerhof's expression for base resistance stems from the idea that the tip resistance increases linearly with embedment up to a limiting depth of 10 shaft diameters; thereafter, the tip resistance remains constant with depth.

Five methods [Touma and Reese (1974), Meyerhof (1976), Quiros and Reese (1977), Reese and Wright (1977) and Reese and O'Neill (1988)] have been presented for estimating the side resistance and end bearing capacities of drilled shafts in sands and gravels. Comparison of these methods shows that they may result in widely divergent estimates of capacity for the same conditions. Unfortunately, the information available from field load tests at present is insufficient to determine which of the methods is most reliable and most generally applicable.

Due to the shortage of field data, it is not possible at present to determine with precision what values of performance factors should be used for drilled shafts in sands and gravels. Accordingly, the best procedure appears to be to estimate the capacity using all of the applicable methods, and to select the factored capacity using judgment, and any available experience with similar conditions. The inherent great variability of the capacities of drilled shafts in sand logically suggests that values of performance factors for shafts in sands should be smaller than for shafts in clay.

C 4.13.3.3.4 Axial Capacity In Rock

Typically, the axial compression load on a shaft socketed into rock is carried solely in side resistance until a total shaft settlement on the order of 0.4 inches occurs. At this displacement, the ultimate side resistance, Q_{SR}, is mobilized and slip occurs between the concrete and rock. As a result of this slip, any additional load is transferred to the tip, and it is assumed that side resistance reduces to zero. This assumption is conservative because a portion of the fully mobilized side resistance will remain after failure of the bond along the shaft-rock socket interface. Alternative procedures are available which can be used to proportion the socket load between side and tip resistance (e.g., Carter and Kulhawy, 1988).

Where the rock socket capacity is derived from side resistance, the settlements within the socket will be small. If the rock socket capacity is derived from tip resistance, the settlements will be larger and must be checked as an integral part of the design.

The design procedures assume the socket is constructed in reasonably sound rock that is little affected by construction (i.e., does not rapidly degrade upon excavation and/or exposure to air or water) and which is cleaned prior to concrete placement (i.e., free of soil and other debris).

The design procedure presented in this article assumes that: (a) the rock strength measured during site investigation will not deteriorate during construction when water or drilling fluids are used, (b) the drilling fluid used will not form a lubricated film on the sides of the socket, and (c) the bottom of the socket is properly cleaned out. This is especially important if the capacity of the drilled shaft is based on end bearing.

The design procedure proposed by Reese and O'Neill (1988) for bearing capacity of drilled shafts socketed in rock assumes that the load is carried entirely by the shaft if the computed settlement is less than 0.4 in. Conversely, loads that cause settlements greater than 0.4 in. are assumed to be carried entirely by the base of the drilled shaft. This method is conservative since loads are assumed to be carried entirely in side resistance or entirely in end bearing, and no allowance is made for the loads to be carried by a combination of side resistance and end bearing. The steps in the design procedure are as follows:

1. Estimate the settlement of the portion of the drilled shaft that is socketed in rock. This consists of two components: (a) the elastic shortening of the drilled shaft, ρ_e, which can be computed as follows:

$$\rho_e = \frac{(\Sigma P_i)H_s}{A_{soc}E_c} \qquad \text{(C4.13.3.3.4-1)}$$

where H_s = depth of the socket

ΣP_i = working load at the top of the socket

A_{soc} = cross-sectional area of the socket

E_c = Young's modulus of concrete in the socket, considering the stiffness of any steel reinforcement,

and (b) settlement of the base of the drilled shaft, ρ_{base}, which can be computed as follows:

$$\rho_{base} = \frac{(\Sigma P_i)I_\rho}{D_s E_r} \qquad \text{(C4.13.3.3.4-2)}$$

where I_ρ = influence coefficient (from Figure C4.13.3.3.4-1)

D_s = diameter of the base of the drilled shaft socket

E_r = modulus of the in situ rock, taking the joints and their spacing into account.

The Young's modulus of the in situ rock, E_r, can be estimated as follows:

$$E_r = K_E E_i \qquad \text{(C4.13.3.3.4-3)}$$

where E_i = intact rock modulus found either by testing or by means of Figure C4.13.3.3.4-2

K_E = modulus modification ratio, related to the rock quality designation (RQD), as shown in Figure C4.13.3.3.4-3.

2. Calculate $\rho_e + \rho_{base}$. If the sum is less than 0.4 in., compute the ultimate capacity based on side resistance alone (Step 3). If the sum is greater than 0.4 in., compute the ultimate capacity based on base resistance alone (Step 4).

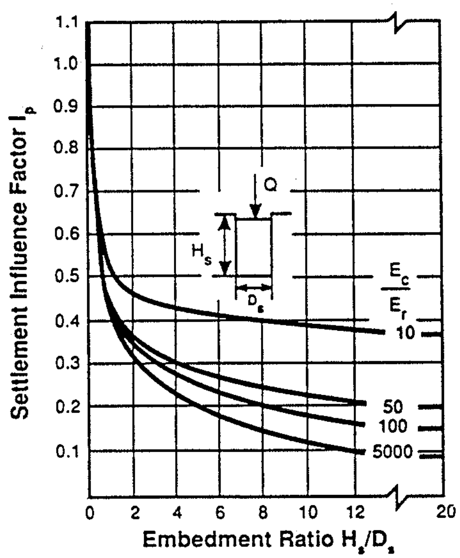


Figure C4.13.3.3.4-1:

Elastic Settlement Influence Factor as a Function of Embedment Ratio and Modulus Ratio (After Donald, Sloan and Chiu, 1980, as presented by Reese and O'Neill, 1988)

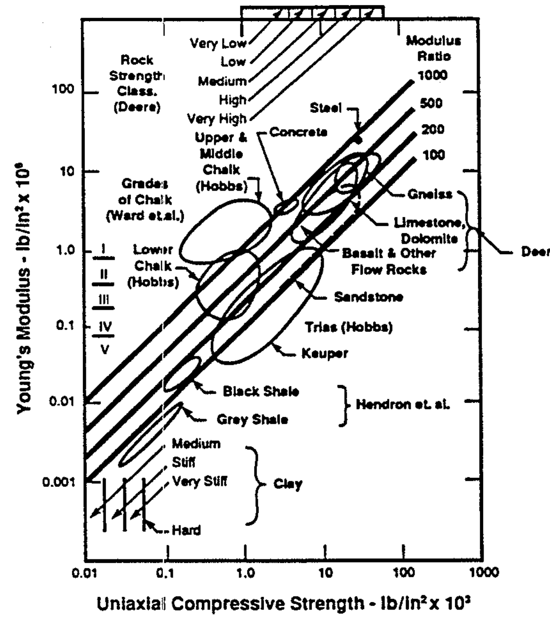


Figure C4.13.3.3.4-2: Engineering Classification of Intact Rock (After Deere, 1968, and Peck, 1976, as presented by Reese and O'Neill, 1988)

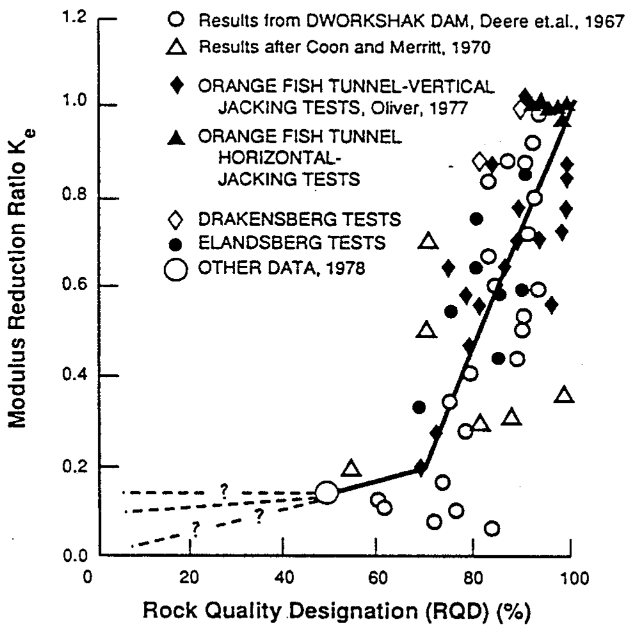


Figure C4.13.3.3.4-3:

Modulus Reduction Ratio as a Function of RQD (After Bieniawski, 1984, as presented by Reese and O'Neill, 1988)

3. Estimate the side resistance of drilled shafts socketed in rock as follows: if the uniaxial compressive strength of the rock is less than or equal to 280 psi, then the ultimate unit side resistance (q_s) is given by (Carter and Kulhawy, 1988):

$$q_s = 0.15 q_u \qquad \text{(C4.13.3.3.4-4)}$$

where q_u is the uniaxial compressive strength of the rock. If the uniaxial compressive strength of the rock or concrete in the drilled shaft, whichever is less, is greater than 280 psi, then q_s is given by (Horvath and Kenney, 1979):

$$q_s = 2.5\sqrt{q_u} \qquad \text{(C4.13.3.3.4-5)}$$

where q_s and q_u are in psi.

4. Estimate the base resistance of the drilled shaft socket from the uniaxial compression strength as follows (Canadian Geotechnical Society, 1985):

$$q_p = 3 q_u K_{sp} d \qquad \text{(C4.13.3.3.4-6)}$$

where q_u = average uniaxial compression strength of the rock core

K_{sp} = dimensionless bearing capacity coefficient

$$K_{sp} = \frac{3 + s_d/D_s}{10[1 + 300 t_d/s_d]^{0.5}} \quad \text{(Figure C4.13.3.3.4-4)}$$

(C4.13.3.3.4-7)

d = dimensionless depth factor

$d = 1 + 0.4 H_s/D_s \le 3.4$

s_d = spacing of discontinuities

t_d = width or thickness of discontinuities

D_s = diameter of drilled shaft socket

PART 6

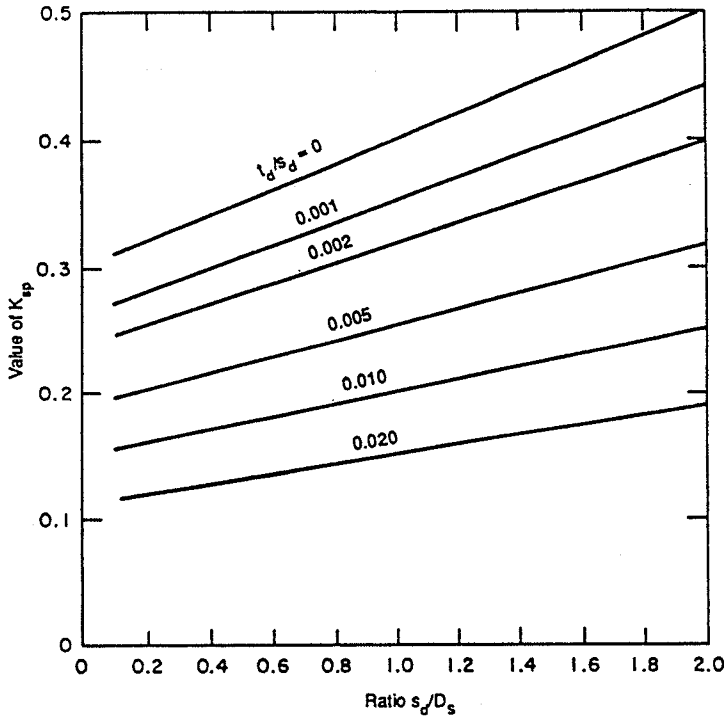


Figure C4.13.3.3.4-4: Bearing Capacity Coefficient, K_{sp} (After Canadian Geotechnical Society, 1985)

H_s = depth of embedment of drilled shaft socket = 0 for drilled shafts resting on top of bedrock.

This method is not applicable to soft stratified rocks, such as shale or limestone. When this method is applicable, the rocks are usually so sound that the structural capacity will govern the design (Fellenius et al., 1989). This method is applicable only if (a) $s_d > 1$ ft, (b) $t_d < 0.25$ in. for unfilled discontinuities or $t_d < 1$ in. for discontinuities filled with soil or rock debris, and (c) $D_s > 1$ ft.

Alternatively, estimate the base resistance of drilled shafts socketed in rock using results from pressuremeter tests as follows (Canadian Geotechnical Society, 1975):

$$q_p = K_b(p_1 - p_o) + \sigma_v \qquad \text{(C4.13.3.3.4-8)}$$

where p_1 = limit pressure determined from pressuremeter tests averaged over a distance of 2 diameters above and below the base

p_o = at rest horizontal stress measured at the base elevation

σ_v = total vertical stress at the base elevation

K_b = dimensionless coefficient which depends on the socket diameter to socket depth ratio as given in Table C4.13.3.3.4-1.

TABLE C4.13.3.3.4-1 Relationship of H_s/D_s With K_b for Drilled Shafts Socketed Into Rock (After Canadian Geotechnical Society, 1985)

H_s/D_s	0	1	2	3	5	7
K_b	0.8	2.8	3.6	4.2	4.9	5.2

C 4.13.3.3.5 Load Test

Load tests are conducted on full-scale drilled shaft foundations to provide data regarding load capacity, load-displacement response, and shaft performance under the design loads to permit assessment of the validity of the design assumptions for a particular site.

As a minimum, the written test procedures should include the following:

- Apparatus for applying loads including reaction system and loading system.
- Apparatus for measuring movements.
- Apparatus for measuring loads.
- Procedures for loading including rates of load application, load cycling and maximum load.
- Procedures for measuring movements.
- Safety requirements.
- Data presentation requirements and methods of data analysis.
- Drawings showing the procedures and materials to be used to construct the load test apparatus.

As a minimum, the results of the load test(s) shall provide the load-deformation response at the top of the shaft.

Tests can be conducted for compression, uplift, or lateral loading, or combinations of loading. Full-scale load tests in the field provide data which include the effects of soil, rock, and groundwater conditions at the site, the dimensions of the shaft, and the procedures used to construct the shaft. Because the results of full-scale load tests can differ even for apparently similar ground conditions, care must be exercised in generalizing and extrapolating the test results to other locations.

Load Test Procedures

Load testing should be conducted whenever special site conditions or combinations of load are encountered, or when structures of special design or sensitivity (e.g., large bridges) are to be supported on drilled shaft foundations.

Load testing procedures generally provide flexibility in the methods of applying incremental loads and approaches to defining a duration of loading. The typical types and suitability of the various test loading rates and durations are summarized below:

- Maintained Load
  The test procedure in most common use (ASTM, 1989) consists of incremental loading to 200% of the design load to determine load capacity and load-deformation behavior.

- Constant Time Interval
  Similar to the Maintained Load test procedure, except loads for the Constant Time Interval test are maintained on the shaft for a constant interval to permit determination of the yield load as a function of creep rate.
- Constant Rate of Movement
  The test procedure permits application of load to shafts at a constant rate of movement that permits a significant reduction in the testing times in comparison to the previous procedures.

The Osterberg Method (Osterberg, 1984) uses a special flat pressure cell installed below the tip of the shaft, as shown in Figure C4.13.3.3.5-1, to load the shaft internally in a way that makes it possible to differentiate the contribution of tip and side resistance to load capacity. The Osterberg Method is the only current test procedure specifically developed for load testing of drilled shafts.

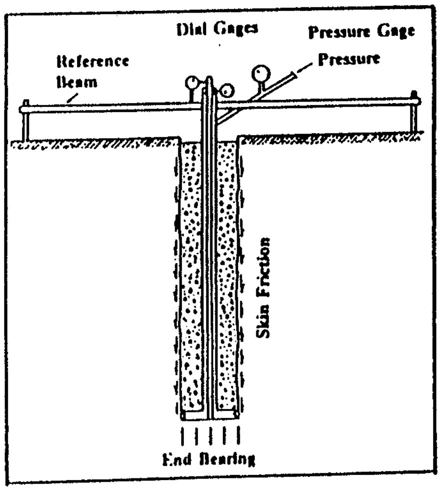
The Osterberg Method is ideal for proof loading a large number of drilled shafts because it is simple, quick, and inexpensive as compared to other methods of field testing. Load tests conducted at constant time intervals or rates of movement are also relatively quick, and characterize foundation behavior during undrained conditions in fine-grained soils.

The maintained load test procedure and other procedures which provide data in regard to time-dependent movements, are the most costly and time consuming, and may or may not provide information indicative of actual long-term performance. When it is anticipated that the bridge foundations will be subjected to live loads of an intermittent or cyclic nature, a test method incorporating this condition may be warranted.

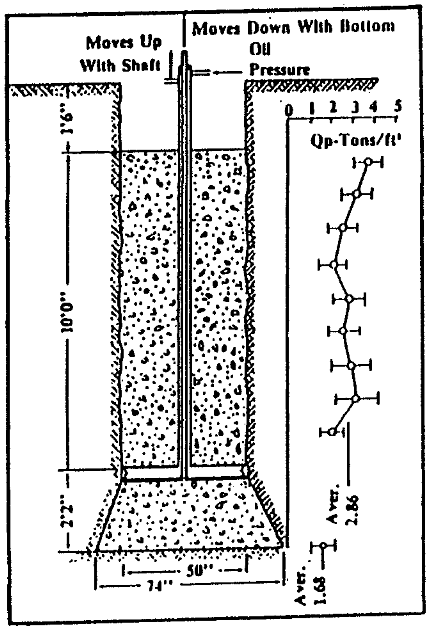
There are two basic types of lateral load tests; these are standard tests and comprehensive tests. The standard test addresses the need to determine the lateral load-deflection behavior at the ground line. The comprehensive test is conducted to determine the lateral load-deflection behavior at depths below the ground line for purposes of developing site-specific p-y relationships along the length of the shaft. Additionally, depending on the particular requirements at a site, comprehensive testing could include consideration of fixity at the top of the shaft, cyclic loading, or group effects.

C 4.13.3.3.6a Uplift Capacity of a Single Drilled Shaft

The performance factors for uplift are lower than those for axial compression because drilled shafts in tension unload the



(a) End Bearing Test



(b) Shaft Resistance Test

Figure C4.13.3.3.5-1: Osterberg Load Test Apparatus for Drilled Shafts (After Osterberg, 1984)

soil; this reduces the overburden effective stress and hence the uplift side resistance of the drilled shaft, as discussed in Article 4.12.3.3.7a.

The uplift capacity of a belled drilled shaft should be calculated assuming that the bell behaves as an anchor (Reese and O'Neill, 1988). Any skin friction above the bell should be discounted. The uplift capacity (Q_s bell) of a belled drilled shaft may be calculated as follows:

$$Q_s \text{ bell} = q_s \text{ bell } A_u \qquad (C4.13.3.3.6a\text{-}1)$$

where

q_s bell = unit uplift capacity of a belled drilled shaft

$$q_s \text{ bell} = N_u S_u \qquad (C4.13.3.3.6a\text{-}2)$$

A_u = annular area between the bell and the shaft

$$A_u = \pi\,(D_p^2 - D^2)/4 \qquad (C4.13.3.3.6a\text{-}3)$$

N_u = uplift bearing capacity factor

S_u = undrained shear strength averaged over a distance of two bell diameters ($2D_p$) above the base. If the soil above the founding stratum is expansive, S_u should be averaged over $2D_p$ above the bottom of the base, or over the depth of penetration of the drilled shaft in the founding stratum, whichever is less

D_p = diameter of the bell

D = diameter of the shaft.

The value of N_u varies from 0 at $D_b/D_p = 0.75$, to a value of 8 at $D_b/D_p = 2.5$, where D_b is the depth below the top of the founding stratum. As shown in Figure C4.13.3.3.6a-1, the top of the founding stratum should be taken at the base of the zone of seasonal moisture change. This method conservatively neglects the uplift resistance due to the soil section and the weight of the drilled shaft.

C 4.13.3.3.6b Group Uplift Capacity

See Article C 4.12.3.3.7b.

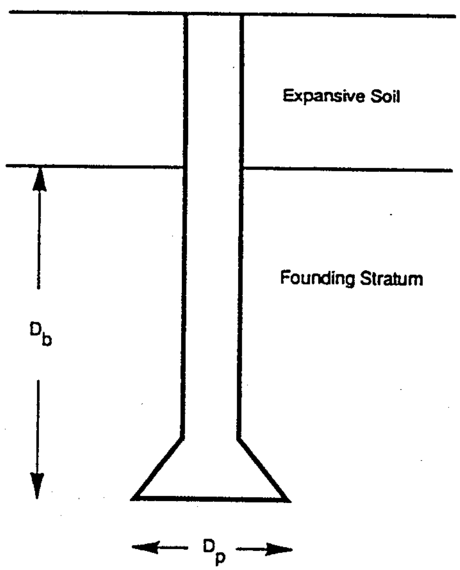


Figure C4.13.3.3.6a-1: Uplift of an underreamed drilled shaft

C 4.13.3.3.7 Lateral Load

The design of laterally loaded drilled shafts is usually governed by lateral movements criteria of Article 4.13.3.2 or structural failure of the drilled shaft, Article 4.13.4.

See Article C 4.12.3.3.8.

C 4.13.3.3.8 Group Capacity

At loads somewhat less than the ultimate, a concentric load applied to a rigid cap supported by a group of drilled shafts is distributed mainly to the corner shafts in square, rectangular, or triangular group plans, and to perimeter shafts in circular group layouts. As the applied load approaches the capacity of the group, the load transferred to each shaft tends to equalize.

Drilling of a hole for a shaft near an existing shaft reduces the effective stresses against both the side and base of the existing shaft. As a result, the capacities of individual drilled shafts within a group tend to be equal to or less than the corresponding capacities of isolated shafts.

C 4.13.3.3.8a Cohesive Soil

The efficiency of groups of drilled shafts in cohesive soil is diminished from the individual shaft case due to the overlapping zones of shear deformation in the soil surrounding the shafts. When a cap is in contact with the ground surface, the cap tends to lead to a block failure mechanism which does not result in mobilization of sufficient shear stress in the ground mass between individual shafts to necessitate a reduction in the soil strength, provided the center-to-center spacing is greater than three diameters.

C 4.13.3.3.8b Cohesionless Soil

The capacity of drilled shaft groups in sand is less than the sum of the individual shaft capacities due to overlap of shear zones in the soil between adjacent shafts and the lossing of the soil during construction. The recommended reduction factors are based in part on theoretical considerations and on limited load test results.

C 4.13.4.1

See Article C 4.12.4.1.