PDHonline Course C396 (8 PDH)

Shallow Foundation Design for Highway Bridges

Instructor: Vincent D. Reynolds, MBA, PE

2020

PDH Online | PDH Center

5272 Meadow Estates Drive
Fairfax, VA 22030-6658
Phone: 703-988-0088
www.PDHonline.com

An Approved Continuing Education Provider

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-SA-02-054		

4. Title and Subtitle	5. Report Date
GEOTECHNICAL ENGINEERING CIRCULAR NO. 6 Shallow Foundations	September 2002
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Robert E. Kimmerling	
9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
PanGEO, Inc. 3414 N.E. 55th Street Seattle, Washington 98105	11. Contract or Grant No. DTFH61-00-C-00031
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Office of Bridge Technology Federal Highway Administration HIBT, Room 3203 400 Seventh Street, S.W. Washington, D.C. 20590	Technical Manual 14. Sponsoring Agency Code

15. Supplementary Notes

Contracting Officer's Technical Representative: Chien-Tan Chang (HIBT)
FHWA Technical Consultant: Jerry DiMaggio (HIBT)

16. Abstract

This document is FHWA's primary reference of recommended design and procurement procedures for shallow foundations. The Circular presents state-of-the-practice guidance on the design of shallow foundation support of highway bridges. The information is intended to be practical in nature, and to especially encourage the cost-effective use of shallow foundations bearing on structural fills. To the greatest extent possible, the document coalesces the research, development and application of shallow foundation support for transportation structures over the last several decades.

Detailed design examples are provided for shallow foundations in several bridge support applications according to both Service Load Design (Appendix B) and Load and Resistance Factor Design (Appendix C) methodologies. Guidance is also provided for shallow foundation applications for minor structures and buildings associated with transportation projects.

17. Key Words	18. Distribution Statement
Shallow foundation, spread footing, abutment, mat foundation, bearing capacity, settlement, eccentricity, overturning, sliding, global stability, LRFD	No Restrictions. This document is available to the public from the National Technical Information Service, Springfield, Virginia 22161

19. Security Classification (of this report)	20. Security Classification (of this page)	21. No. of Pages	22. Price
		310	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

NOTICE

GEOTECHNICAL ENGINEERING CIRCULAR NO. 6

SHALLOW FOUNDATIONS

PREFACE

This document is the sixth in a series of Geotechnical Engineering Circulars (GEC) developed by the Federal Highway Administration (FHWA). This Circular focuses on the design, procurement and construction of shallow foundations for highway structures. The intended users are practicing geotechnical, foundation and structural engineers involved with the design and construction of transportation facilities.

Other circulars in this series include the following:

- GEC No. 1 – Dynamic Compaction (FHWA-SA-95-037)
- GEC No. 2 – Earth Retaining Systems (FHWA-SA-96-038)
- GEC No. 3 – Design Guidance: Geotechnical Earthquake Engineering for Highways, Volume I – Design Principles (FHWA-SA-97-076) Volume II – Design Examples (FHWA-SA-97-077)
- GEC No. 4 – Ground Anchors and Anchored Systems (FHWA-SA-99-015)
- GEC No. 5 – Evaluation of Soil and Rock Properties (FHWA-IF-02-034)
- GEC No. 7 – Soil Nailing (under development)

This Circular is intended to be a stand-alone document geared toward providing the practicing engineer with a thorough understanding of the analysis and design procedures for shallow foundations on soil and rock, with particular emphasis on bridges supported on spread footings. Accordingly, the manual is organized as follows:

- Chapters 1 and 2 present background information regarding the applications of shallow foundations for transportation structures.
- Chapters 3 and 4 present methods used to perform foundation type selection, including the minimum level of subsurface investigation and laboratory testing needed to support design of shallow foundations.
- Chapter 5 presents the soil mechanics theory and methods that form the basis of shallow foundation design.
- Chapter 6 describes the shallow foundation design process for bridge foundation support on spread footings. **Chapters 5 and 6, together with the detailed bridge foundation design examples presented in Appendix B, provide the practical information necessary to complete shallow foundation design for a highway bridge.**

- Chapters 7 and 8 discuss the application and special spread footing design considerations for minor transportation structures and buildings.

- Chapters 9 and 10 provide guidelines for procurement and construction monitoring of shallow foundations.

- Appendix A provides recommended materials specifications for embankments constructed to support shallow foundations. The appendix also includes example material specifications used by state highway agencies that design and construct spread footings in compacted structural embankments.

- Appendix B includes five worked design examples of shallow foundations for highway bridges based on Service Load Design methodology.

- Appendix C includes practical guidance on the use of Load and Resistance Factor Design (LRFD) methodology for shallow foundation design and reworks two of the design examples from Appendix B using LRFD.

This Circular was developed for use as a desktop reference that presents FHWA recommended practice on the design and construction of shallow foundations for transportation structures. To the maximum extent possible, this document incorporates the latest research in the subject matter area of shallow foundations and their transportation applications. Attention was given throughout the document to ensure the compatibility of its content with that of reference materials prepared for the other FHWA publications and training modules. **Special efforts were made to ensure that the included material is practical in nature and represents the latest developments in the field.**

ACKNOWLEDGMENTS

Special thanks are given to the state highway agency personnel who contributed to the collection of data regarding the state of the practice in using shallow foundations for support of highway bridges. The Nevada, Michigan and Washington State Departments of Transportation are especially acknowledged for providing the case history data included in this Circular.

Notable and much appreciated contributions to the design examples included in this Circular were provided by Messrs. George Machan, Will Shallenberger and Kenji Yamasaki of Cornforth Consultants, Inc., Portland, Oregon, and by Ms. Tiffany Adams and Mr. Siew Tan of PanGEO, Inc., Seattle, Washington.

Particular gratitude is extended to Mr. Ronald G. Chassie, formerly with FHWA, Mr. Monte J. Smith of Sargent Engineers, Inc., Olympia, Washington, and Mr. James L. Withiam of D'Appolonia, Monroeville, Pennsylvania for their extremely helpful technical review, input and commentary provided on the document.

For her superior technical editing and outstanding positive attitude during the final production of this document, the author extends heartfelt thanks to Ms. Toni Reineke of Author's Advantage, Seattle, Washington.

Last but not least, the author wishes to acknowledge the members of the Technical Working Group, Mr. Mike Adams; Mr. James Brennan; Mr. Myint Lwin; Mr. Mohammed Mulla; Dr. Sastry Putcha; Mr. Benjamin S. Rivers; Mr. Jésus M. Rohena; Ms. Sarah Skeen; Mr. Chien-Tan Chang, Contracting Officers Technical Representative; and Mr. Jerry DiMaggio for their review of the Circular and provision of many constructive and helpful comments.

ENGLISH TO METRIC (SI) CONVERSION FACTORS

The primary metric (SI) units used in civil and structural engineering are:

- meter (m)
- kilogram (kg)
- second (s)
- newton (N)
- Pascal (Pa = N/m^2)

The following are conversion factors for units presented in this manual:

Quantity	From English Units	To Metric (SI) Units	Multiply by:	For Aid to Quick Mental Calculations
Mass	lb	kg	0.453592	1 lb (mass) = 0.5 kg
Force	lb kip	N kN	4.44822 4.44822	1 lb (force) = 4.5 N 1 kip (force) = 4.5 kN
Force/unit length	plf klf	N/m kN/m	14.5939 14.5939	1 plf = 14.5 N/m 1 klf = 14.5 kN/m
Pressure, stress, modulus of elasticity	psf ksf psi ksi	Pa kPa kPa Mpa	47.8803 47.8803 6.89476 6.89476	1 psf = 48 Pa 1 ksf = 48 kPa 1 psi = 6.9 kPa 1 ksi = 6.9 MPa
Length	inch foot foot	mm m mm	25.4 0.3048 304.8	1 in = 25 mm 1 ft = 0.3 m 1 ft = 300 mm
Area	square inch square foot square yard	mm^2 m^2 m^2	645.16 0.09290304 0.83612736	1 sq in = 650 mm^2 1 sq ft = 0.09 m^2 1 sq yd = 0.84 m^2
Volume	cubic inch cubic foot cubic yard	mm^3 m^3 m^3	16386.064 0.0283168 0.764555	1 cu in = 16,400 mm^3 1 cu ft = 0.03 m^3 1 cu yd = 0.76 m^3

A few points to remember:

1. In a **"soft"** conversion, an English measurement is mathematically converted to its **exact** metric equivalent.
2. In a **"hard"** conversion, a new **rounded**, metric number is created that is convenient to work with and easy to remember.
3. Only the meter and millimeter are used for length (avoid centimeter)
4. The Pascal (Pa) is the unit for pressure and stress (Pa and N/m^2)
5. Structural calculations should be shown in MPa or kPa
6. A few basic comparisons worth remembering to help visualize metric dimensions are:
    - One mm is about 1/25 inch or slightly less than the thickness of a dime
    - One m is the length of a yardstick plus about 3 inches
    - One inch is just a fraction (1/64 in) longer than 25 mm (1 in = 25.4 mm)
    - Four inches are about 1/16 inch longer than 100 mm (4 in = 101.6 mm)
    - One foot is about 3/16 inch longer than 300 mm (12 in = 304.8 mm)

CHAPTER 1

INTRODUCTION

1.1 PURPOSE AND SCOPE

This Geotechnical Engineering Circular (GEC) coalesces more than four decades of research, development and practical experience in the application of shallow foundations for support of transportation structures. The document is intended to be a definitive desk reference for the transportation professional responsible for design, procurement and construction of shallow foundations for bridges and other transportation-related structures.

This GEC draws heavily from previous published work by the Federal Highway Administration (FHWA), State (and Local) Highway Agencies (SHAs) and other authors of practical guidance related to shallow foundations. As such, this document generally does not represent "new" research but is intended to provide a single reference source for state-of-the-practice information on the design and construction of shallow foundations.

The one exception to the foregoing statement is in the area of bridge support on shallow foundations bearing on compacted structural fills. Special attention has been given to case histories and design examples on the use of shallow foundations to support abutments in compacted approach embankments.

1.2 BACKGROUND

Shallow foundations represent the simplest form of load transfer from a structure to the ground beneath. They are typically constructed with generally small excavations into the ground, do not require specialized construction equipment or tools, and are relatively inexpensive. In most cases, shallow foundations are the most cost-effective choice for support of a structure. Your house is most likely supported on shallow spread footings, and you probably supported that deck you constructed last year on pre-cast concrete pier blocks because they were inexpensive and easy to place.

Bridges, however, are frequently supported on deep foundations such as driven piling. This may be as much a result of the continued use of past practice than for any other reason and has its roots not in highway construction, but railroads. The need to maintain constant and reliable grades over vastly differing ground conditions and topography made the choice to support virtually all railroad bridges on piles rather obvious. At the time of rapid rail expansion in North America, and all over the world, the concepts of soil mechanics and geotechnical engineering had not even been conceived. Railroad engineers needed a reliable way to support bridges and trestles, and the available technology directed them to driven piles.

As the pace of highway construction increased and eventually passed that of rail construction, the knowledge base for construction of bridges passed from the rail engineers to the highway engineers. It is likely that most of the early highway engineers were, in fact, ex-rail engineers, so it is not surprising that piling would be chosen to support highway bridges.

In many cases, and for very good reasons, pile support of transportation structures is wise, if not essential. Waterway crossings demand protection from the potentially disastrous effects of scour and other water-related hazards. Poor ground conditions and transient load conditions, such as vessel impact or seismicity, also may dictate the use of deep foundations such as piles. In more recent times, congested urban and suburban environments restrict the available construction space. The use of deep foundations, such as shafts, that can be constructed in a smaller footprint may be preferable to the costs associated with shoring, subterranean utility relocation, and right-of-way acquisition that would be necessary to construct a shallow foundation.

However, many transportation bridges are associated with upland development and interchanges. These locations are frequently removed from waterway hazards and are in areas with competent ground conditions. The engineers responsible for these structures, both geotechnical and structural, should be constantly on the lookout for the potential prudent and cost-effective use of shallow foundations.

The fortuitous combination of good, competent ground conditions and an available source of good-quality, granular fill material should always be seen as an opportunity to save bridge construction costs. By supporting the bridge abutments within the compacted approach fills, cost savings will be realized from the following:

- A shortened construction schedule
- Deletion of piles placed or driven through a good-quality, compacted fill to competent foundation materials
- Reduction in concrete and steel materials costs in the case where a spread footing was detailed, but bearing below the approach fill

Various studies on the use of shallow foundations yielded the following information, which underscores the potential cost savings that can result from the judicious use of shallow foundations:

- There are currently approximately 600,000 highway bridges in the United States. The average replacement cost is about $500,000 per bridge. About 50 percent of the replacement cost is associated with the substructure. Shallow foundations can generally be constructed for 50 to 65 percent of the cost for deep foundations (Briaud & Gibbens, 1995).

- The Washington State Department of Transportation (WSDOT) constructed more than 500 highway bridges between 1965 and 1980 with one or more abutments or piers supported on spread footings (DiMillio, 1982). WSDOT continues using shallow foundations supported in structural fills on a regular basis.

- In 1986, FHWA conducted research to evaluate the settlement performance of 21 bridge foundations supported on shallow foundations on cohesionless soil. This research demonstrated that 70 percent of the settlement of the shallow foundations occurred prior to placement of the bridge deck. The average post-deck settlement of the structures monitored was less than 6 mm (¼ inch), (Gifford et al., 1987).

The following important conclusions were drawn from this research:

1) There is sufficient financial incentive to promote the use of shallow foundations, where feasible (Briaud & Gibbens, 1995);

2) The use of shallow foundation support of bridges has a proven track record on literally hundreds of bridge projects (DiMillio, 1982); and

3) Sufficient performance data exist to alleviate concerns over settlement performance of shallow foundations in most bridge support applications where good ground conditions exist (Gifford et al., 1987).

1.3 RELEVANT PUBLICATIONS

Although the Shallow Foundations GEC is intended to be a stand-alone reference document, additional detail and background on the methods and procedures collectively included can be found in the following publications:

- AASHTO Standard Specifications for Highway Bridges, 16th Edition, with 1997, 1998, 1999 and 2000 Interim Revisions (AASHTO, 1996)

- AASHTO LRFD Bridge Design Specifications, 2nd Edition, with 1999, 2000 and 2001 Interim Revisions (AASHTO, 1998)

- NHI 13212 Soils and Foundations Workshop Reference Manual (Cheney & Chassie, 2000)

- NHI 132037 Shallow Foundations Workshop Reference Manual (Munfakh et al., 2000)

- NHI 132031 Subsurface Investigation Workshop Reference Manual (Arman et al., 1997)

- NHI 132035 Module 5 – Rock Slopes: Design, Excavation and Stabilization (Wyllie & Mah, 1998)

- Foundation Engineering Handbook (Fang, 1991)

- Soil Mechanics (Lambe & Whitman, 1969)

- Rock Slope Engineering (Hoek & Bray, 1981)

1.4 SHALLOW FOUNDATION SYSTEMS APPLICATIONS

Shallow foundations principally distribute structural loads over large areas of near-surface soil or rock to lower the intensity of the applied loads to levels tolerable for the foundation soils. Shallow foundations are used in many applications in highway projects when the subsurface conditions are appropriate. Such applications include bridge abutments on soil slopes or embankments, bridge intermediate piers, retaining walls, culverts, sign posts, noise barriers, and rest stop or maintenance building foundations. Footings or mats may support column loads under buildings. Bridge piers are often supported on shallow foundations using various structural configurations.

The next chapter describes common applications for transportation structure support on shallow foundations.

CHAPTER 2

TYPES OF SHALLOW FOUNDATIONS

2.1 Introduction

The definition of a "shallow foundation" varies from author to author but generally is thought of as a foundation that bears at a depth less than about two times the foundation width. The definition is less important than understanding the theoretical assumptions behind the various design procedures. Stated another way, it is important to recognize the theoretical limitations of a design procedure that may vary as a function of depth, such as a bearing capacity equation. Common types of shallow foundations are shown in Figures 2-1 through 2-5.

2.2 Isolated Spread Footings

Isolated spread footings (Figure 2-1) are designed to distribute the concentrated loads delivered by a single column to prevent shear failure of the bearing material beneath the footing and to minimize settlement by reducing the applied bearing stress. The forces, strength and plan dimensions of the column may govern the minimum size of an isolated spread footing. For bridge columns, isolated spread footings are typically greater than 3 m by 3 m (10 feet by 10 feet). These dimensions will increase when eccentric loads are distributed to the footing. The size of the footing is a function of the loads distributed by the supported column and the strength and compressibility characteristics of the bearing materials beneath the footing. Structural design of the footing includes consideration for moment resistance at the face of the column and in the short direction of the footing, as well as shear and punching around the column.

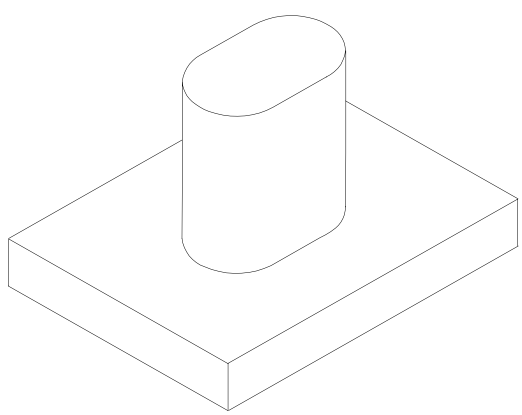


Figure 2-1: Isolated Spread Footing

2.3 CONTINUOUS STRIP SPREAD FOOTINGS

The most commonly used type of foundation for buildings is the continuous strip spread footing (Figure 2-2). Continuous or strip footings generally have a minimum length to width ratio of at least 5 (i.e., length > 5 x width). They support a single row of columns or a bearing wall to reduce the pressure on the bearing materials. These footings may tie columns together in one direction. Sizing and structural design considerations are similar to isolated spread footings with the exception that plane strain conditions are assumed to exist in the direction parallel to the long axis of the footing. The structural design of these footings is generally governed by beam shear and bending moments.

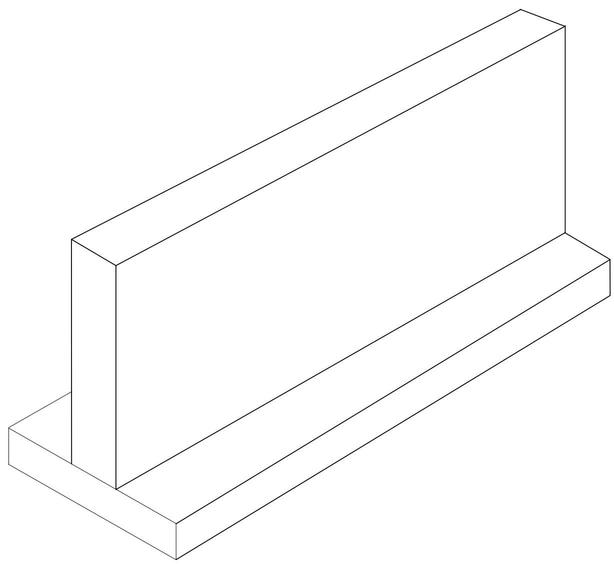


Figure 2-2: Continuous Strip Spread Footing

2.4 SPREAD FOOTINGS WITH CANTILEVER STEMWALLS

When a spread footing with a bearing wall is used to resist lateral loads applied by backfill, the foundation and the wall must perform both vertical load distribution and retaining wall functions. The foundation is designed to resist the resulting eccentric load, and the lateral earth pressures are resisted by cantilever action of the stemwall and overturning resistance provided by the footing.

2.4.1 Bridge Abutments

Bridge abutments are required to perform numerous functions, including the following:

- Retain the earthen backfill behind the abutment
- Support the superstructure and distribute the loads to the bearing materials below the spread footing
- Provide a transition to the approach fill

- Depending on the structure type, accommodate shrinkage and temperature movements within the superstructure

Spread footings with cantilever stemwalls are well suited to performing these multiple functions. The general arrangement of a bridge abutment with spread footing and cantilever stemwall is shown in Figure 2-3.

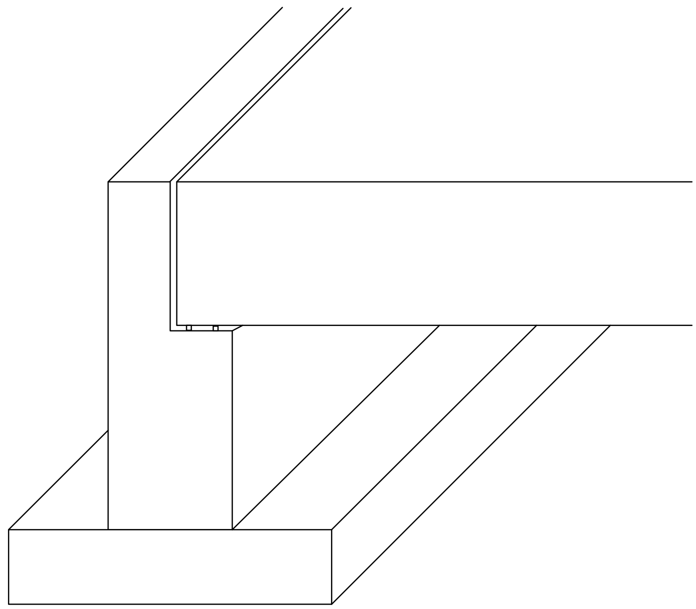


Figure 2-3: Spread Footing with Cantilever Stemwall at Bridge Abutment

2.4.2 Retaining Structures

The bases of semi-gravity concrete cantilever retaining walls (inverted “T” walls) are essentially shallow spread footings. The wall derives its ability to resist loads from a combination of the dead weight of the backfill on the heel of the wall footing and the structural cantilever of the stem (Figure 2-4).

2.4.3 Building Foundations

When a building stemwall is buried, partially buried or acts as a basement wall, the stemwall resists the lateral earth pressures of the backfill. Unlike bridge abutments and semi-gravity cantilever walls, the tops or the ends of the stemwalls are frequently restrained by other structural members (e.g., beams, floors, transverse interior walls, etc.) to provide additional structural resistance.

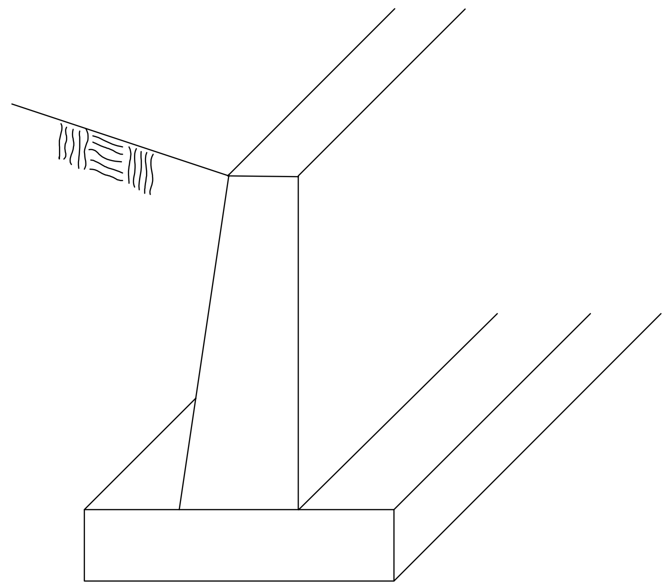


Figure 2-4: Semi-Gravity Cantilever Retaining Wall

2.5 COMBINATION FOOTINGS

Combination, or combined, footings are similar to isolated spread footings except that they support two or more columns and are rectangular or trapezoidal in shape (Figure 2-5). They are primarily used when the column spacing is non-uniform (Bowles, 1996) or when isolated spread footings become so closely spaced that a combination footing is simpler to form and construct. In the case of bridge abutments, an example of a combination footing is the so-called "spill-through" type abutment (Figure 2-6). This configuration was used during some of the initial construction of the Interstate freeways on new alignments where spread footings could be founded on competent native soils. Spill-through abutments are also used at stream crossings to make sure foundations are below the scour level of the stream.

Due to the frame action that develops with combination footings, they can be used to resist large overturning or rotational moments in the longitudinal direction of the column row.

There are a number of approaches for designing and constructing combined footings. The choice depends on the available space, load distribution among the columns supported by the footing, variations of soil properties supporting the footing and economics.

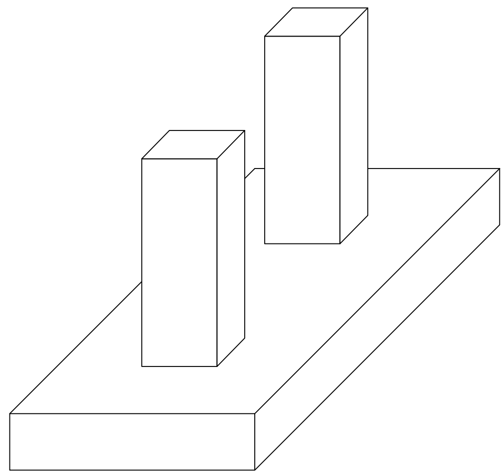


Figure 2-5: Combined Footing

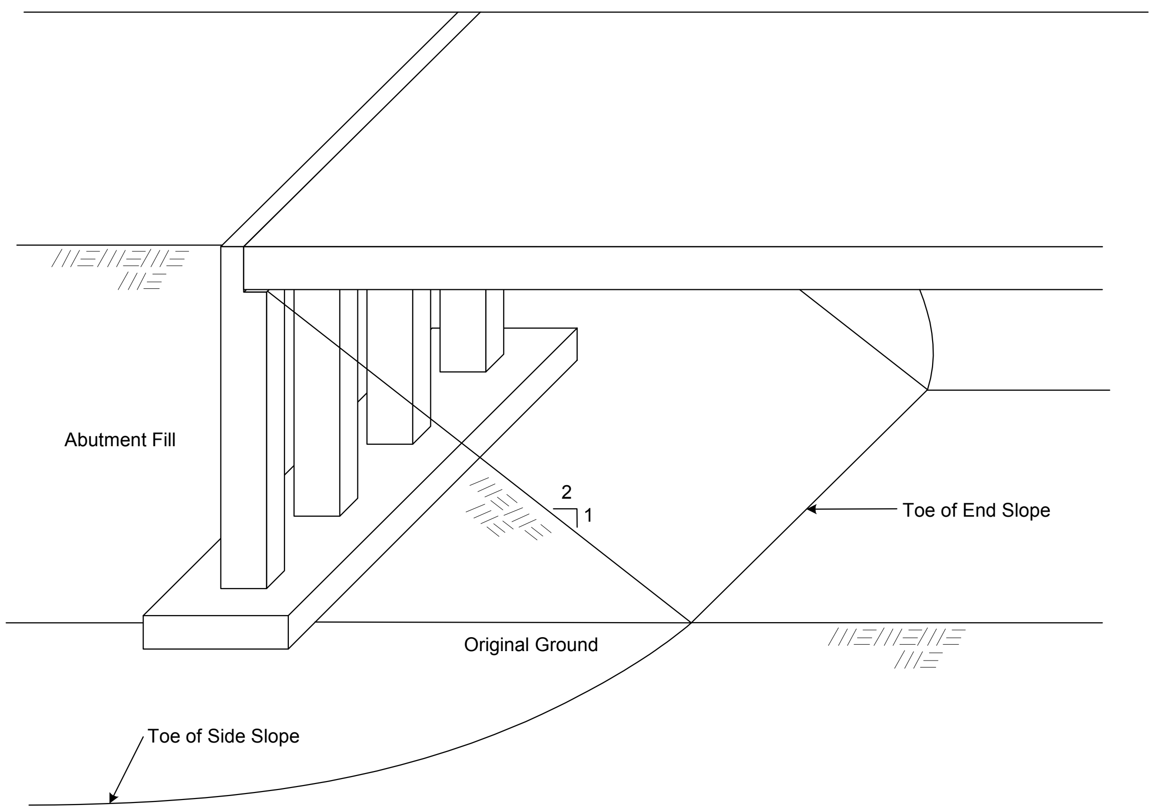


Figure 2-6: Spill-Through Abutment on Combination Strip Footing

2.6 MAT FOUNDATIONS

A mat foundation consists of a single heavily reinforced concrete slab, which underlies the entire structure or a major portion of the structure. Mat foundations are often economical when spread footings would cover more than about 50 percent of the footprint of the plan area of a structure (Peck et al. 1974). A mat (Figure 2-7) typically supports a number of columns and/or walls in either direction or a uniformly distributed load (i.e., tank). The principal advantage of a mat foundation is its ability to bridge over soft spots and reduce differential movement.

Structures founded on relatively weak soils or lightweight structures may be economically supported on mat foundations. Column and wall loads are transferred to the foundation materials through the mat foundation. Mat foundations distribute the loads over a large area, thus reducing the intensity of contact pressures. Mat foundations are designed with sufficient reinforcement and thickness to be rigid enough to distribute column and wall loads, minimizing differential settlements and tolerating larger uniform settlements. Often the mat also serves as the base floor level of building structures.

Mat foundations have limited applicability for bridge support, except where large bridge piers, such as bascules or other movable bridge supports, have the opportunity to bear at relatively shallow depth without deep foundation support. This type of application may arguably be a deep foundation, but the design of such a pier may include consideration of the base of the bascule pier as a mat. Discussion of large mat foundation design is included in Section 8.6.

A more common application of mat foundations to transportation structures might include lightly loaded rest or maintenance facilities such as small masonry block structures or sand storage bins or sheds, or box culverts constructed as a continuous structure.

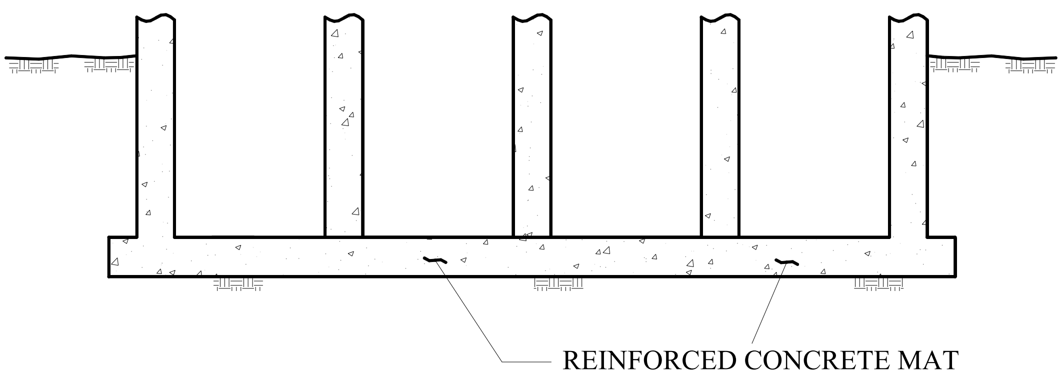


Figure 2-7: Typical Mat Foundation

2.7 Shallow Foundations in Performance Applications

High quality case histories regarding shallow foundations in transportation applications are difficult to obtain, largely because settlement data following construction are often not recorded. This makes it difficult to evaluate the performance of the foundation with respect to the settlement predictions made during design. Several relevant case histories are documented in "Performance of Highway Bridge Abutments on Spread Footings" (DiMillio, 1982). These case histories are not reproduced here. However, three new case histories are included to highlight design and construction practices in use by various state highway agencies that take advantage of the cost savings available by constructing spread footings in compacted engineered fills and in competent natural ground.

2.7.1 Case History No. 1: I-5 Kalama Interchange, Washington

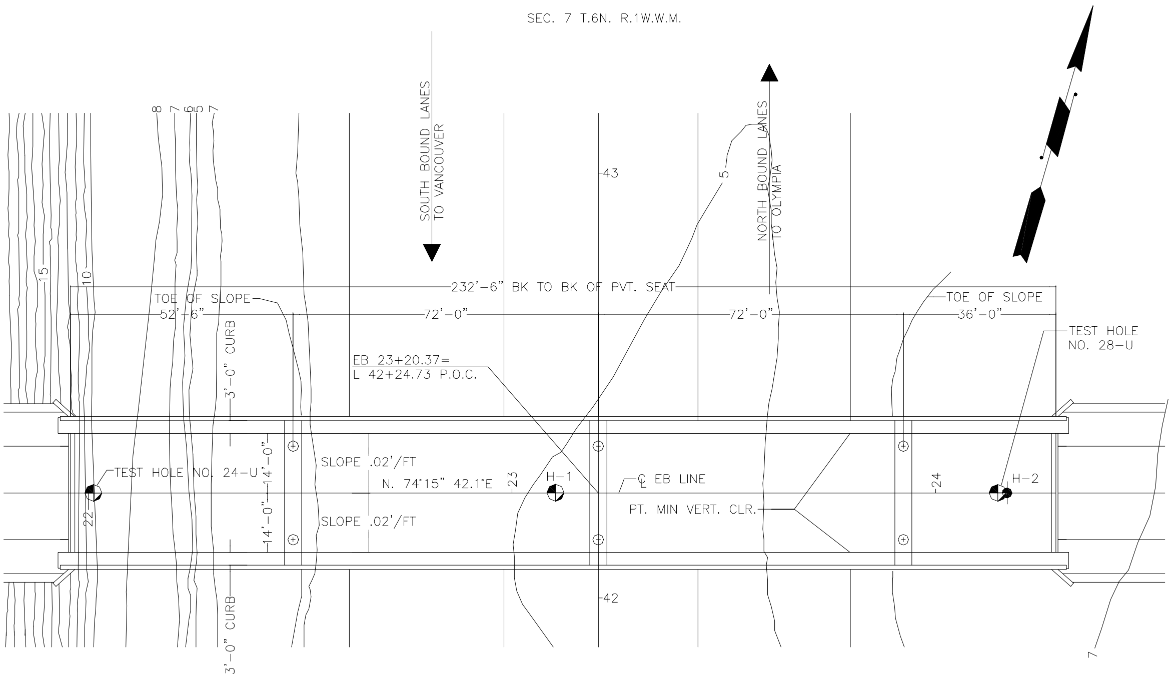
Project Background

During Interstate construction in the 1960s and 1970s, the Washington State Highway Commission, later to become the Washington State Department of Transportation (WSDOT), constructed numerous bridges with foundation support by shallow spread footings, many of which were built on structural fills (DiMillio, 1982). This case history demonstrates the significant cost savings achievable through the process of thorough site investigation, laboratory testing, application of sound geotechnical engineering and follow-through during construction with the observational method (Peck, 1969). The observational method in this application included measuring settlement as a function of time during a settlement delay period following construction of the approach embankment. These data were used to evaluate the feasibility of using shallow foundations at the abutments and to predict long-term settlement potential for the abutment footings.

The project included design and construction of a 71-m (233-ft) long by 8.5-m (28-ft) wide, four-span, prestressed concrete structure that provides east-west traffic access across Interstate 5 in Kalama, a town in southwestern Washington State. The foundation elements are numbered as Piers 1 through 5. Note that WSDOT calls out abutments as piers, so Piers 1 and 5 are the abutments. The bridge layout and the location of subsurface exploration borings are shown in Figure 2-8. The bridge was constructed in 1968 and 1969.

Subsurface Conditions

The bridge alignment is underlain by approximately 30 m (100 ft) of alluvial sediments, consisting of interbedded loose to medium dense sand, soft to medium stiff silt and occasional peat layers. Bedrock was encountered directly below the alluvium. Summary logs of three exploratory borings drilled along the bridge alignment are shown in Figure 2-9. Standard Penetration Test (SPT) N-values and moisture content test results are also indicated on the profile.



Note: Dimensions in feet (1 ft = 0.305 m)

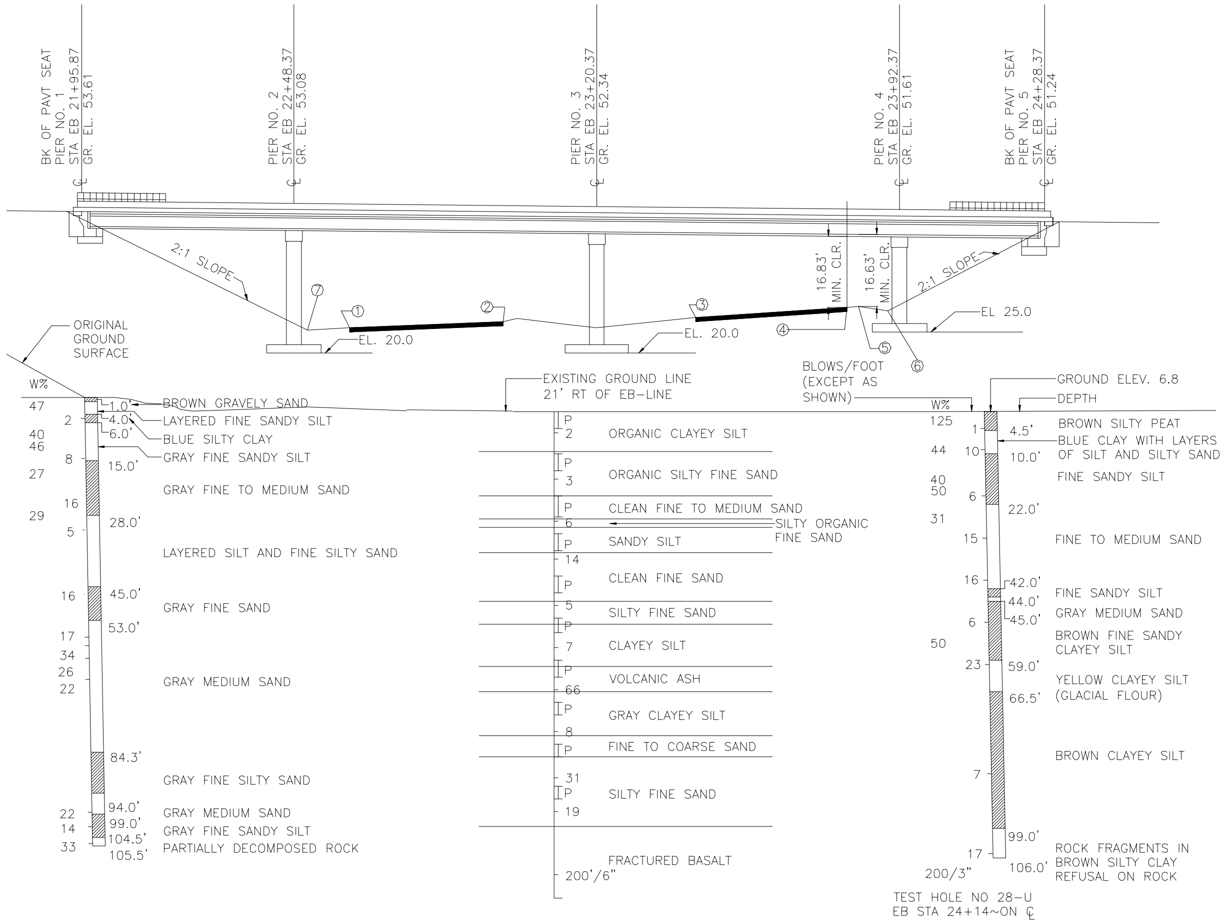
Figure 2-8: Bridge Layout and Exploration Plan, I-5 Kalama Interchange

Results of three one-dimensional consolidation tests were available and are summarized in Table 2-1.

Foundation Design Approach

In light of the excessive depth of compressible soil and the high costs associated with a deep foundation system, the Washington State Highway Commission decided to support the bridge on conventional spread footings. To reduce the potential for post-construction settlement of the site soils and to improve the level of performance for the footings, ground improvement by means of preloading was performed prior to construction of the footings. During the design and planning phase of the project, a preload period of one year was allowed in the construction schedule.

At least 3 m (10 ft) of compacted granular fill was maintained below the footings at the interior piers. Up to 14 m (45 ft) of compacted granular fill was placed at the abutments (Figure 2-9).



Note: Dimensions in feet (1 ft = 0.305 m)

Figure 2-9: Subsurface Profile, I-5 Kalama Interchange

TABLE 2-1: RESULTS OF ONE-DIMENSIONAL CONSOLIDATION TESTS, I-5 KALAMA INTERCHANGE

Sample	Soil Type	Test Results
Test Hole 24U @ 2.9 m (Pier 1/West Abutment)	Gray sandy silt	Moisture = 46%; Moist unit weight = 16.5 kN/m^3; Compression Index C_c = 0.19
Test Hole 28U @ 5.2 m (Pier 5/East Abutment)	Gray clay-silt	Moisture = 50%; Moist unit weight = 17.1 kN/m^3; Compression Index C_c = 0.22
Test Hole 28U @ 16.8 m (Pier 5/East Abutment)	Gray clay-silt	Moisture = 49%; Moist unit weight = 16.8 kN/m^3; Compression Index C_c = 0.24

The specification for Gravel Borrow included in Appendix A is currently used by WSDOT for granular fills beneath footings. This specification has not changed significantly since the time this bridge was built. A maximum allowable bearing pressure of 3 tsf was used for sizing the footings. This is the presumptive value used by WSDOT for fills constructed using Gravel Borrow. The actual design bearing pressures were 280 kPa (2.9 tsf) for Piers 1 through 4, and 244 kPa (2.55 tsf) for Pier 5.

Construction Sequencing and Performance

The preload fill was placed in two stages such that the foundation soils would gain strength during the Stage 1 preload operation and to reduce the potential for embankment instability. In mid-October 1967, Stage 1 preload fill was completed, and as much as 7.6 m (25 ft) of fill was placed. Stage 2 preload fill was placed about 6 months after completion of Stage 1. The total preload fill thickness was about 13 m (43 ft) and 14.6 m (48 ft) thick at abutment Piers 1 and 5, respectively. At the interior piers, the preload fill was about 7.6 m (25 ft) thick. Available data indicated that, in late June 1968, 0.34 m (1.1 ft) and 0.9 m (3.0 ft) of settlement had occurred under the preload fill at Piers 1 and 5, respectively. Settlement data for the interior piers preload fill are not available.

Subsequent to the preloading, footings and columns at Piers 3 through 5 were completed in August 1968. Construction at Piers 1 and 2 was delayed until November 1968 because 3 months earlier, when Piers 3 through 5 were constructed, settlement data indicated that the rate of settlement of the approach fills was still excessive.

The bridge deck was completed in December 1968. Settlement monitoring of the bridge deck was performed on a monthly basis for the following 4 months. At the end of the monitoring period in April 1969, 37 to 49 mm (0.12 to 0.16 ft) of settlement was measured at the abutments and 21 to 34 mm (0.07 to 0.11 ft) at the interior piers. Detailed settlement data are shown in Table 2-2 and are plotted in Figure 2-10. The settlement data show that vertical deflections at each pier are relatively uniform and differential settlement is small. Based on this data, the maximum angular distortion (δ'/l) between adjacent piers is less than 0.002.

The project was completed in June 1969, and the bridge has performed well since its completion more than 30 years ago. The bridge as it appears today is shown in Figure 2-11.

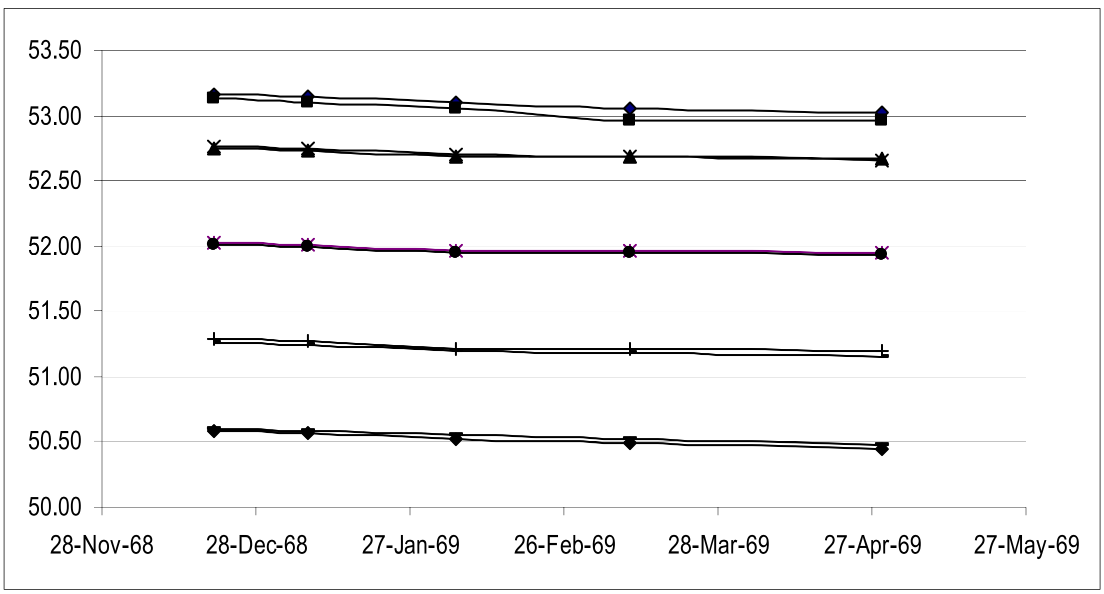
Costs

Using an inflation-adjusted cost of $20 per square foot for a prestressed girder bridge in a non-water crossing situation (source: WSDOT price records, 1975 data), the estimated contract amount for the EB-line under crossing was about $130,000. According to a project memorandum, the use of the spread footings instead of timber piles represented savings of approximately $25,000. The estimated savings were based on the assumption that a total of 124 timber piles 15 to 18 m (50 to 60 ft) in length would be adequate for supporting the bridge. The cost savings of deleting the piling therefore represented about 20 percent of the total cost of the structure.

TABLE 2-2: SETTLEMENT DATA – AFTER COMPLETION OF BRIDGE DECK, I-5 KALAMA INTERCHANGE

		12/20/68	1/7/69	2/5/69	3/11/69	4/29/69
Pier 1	SE Corner of S Wing Wall	53.16	53.14	53.10	53.05	53.02
	SE Corner of N Wing Wall	53.13	53.10	53.05	52.97	52.97
Pier 2	S Side of Deck	52.75	52.74	52.69	52.68	52.67
	N Side of Deck	52.77	52.75	52.70	52.68	52.66
Pier 3	S Side of Deck	52.02	52.01	51.96	51.96	51.95
	N Side of Deck	52.01	52.00	51.95	51.95	51.93
Pier 4	S Side of Deck	51.29	51.28	51.22	51.22	51.20
	N Side of Deck	51.26	51.25	51.19	51.18	51.15
Pier 5	SE Corner of S Wing Wall	50.60	50.59	50.55	50.52	50.48
	SE Corner of N Wing Wall	50.59	50.57	50.52	50.49	50.45

Note: Survey data in feet (1 ft = 0.305 m)



Note: Data in feet (1 ft = 0.305 m)

Figure 2-10: Settlement Following Completion of Bridge Deck, I-5 Kalama Interchange

Acknowledgment: Al Kilian, formerly of WSDOT, collected and compiled the information for this case history, and Dave Jenkins of WSDOT Geotechnical Branch made this information available. Mike Bauer of the WSDOT Bridge and Structures Office provided the historical cost data.

Figure 2-11: I-5 Kalama Interchange

2.7.2 Case History No. 2: I-580 Mt. Rose Highway Interchange, Nevada

Project Background

This case history is of a bridge at the Mt. Rose Highway Interchange on Interstate Highway 580, in Washoe County, Nevada. The bridge is the northbound I-580 overpass over Mt. Rose Highway and was constructed in 1994. The bridge is 107.6 m (353 ft) long and 25.8 m (84.5 ft) wide, and is a two-span concrete structure supported by two end abutments and an intermediate pier (Figure 2-12). The abutments and pier are supported by spread footings.

Subsurface Conditions

According to the geotechnical investigation (SHB AGRA, 1993), the site lies within a combination of Donner Lake Glacial Outwash and Mt. Rose Alluvial Fan Deposits. A total of 10 borings were drilled, from which the following information was obtained.

<u>North Abutment Area:</u> The predominant soil type in this area is a dense, fine- to medium-grained silty sand to sand with varying amounts of fine to coarse gravels and cobbles. Blow counts (SPT N-values) generally ranged from 25 to 75, with 50 as the average.

Figure 2-12: Mt. Rose Interchange: I-580 Overcrossing at Mt. Rose Highway, Nevada

Center Pier Area: From approximately 0.6 to 7.6 m (2 to 25 ft) below the existing surface grade, a medium dense to very dense, fine- to medium-grained silty sand to sand was encountered. This horizon contained varying amounts of fine to coarse gravels and occasional cobbles. This layer was underlain by 1.5 to 3.0 m (5 to 10 ft) thick, low to medium plasticity, hard clayey sand to clayey gravel with varying amounts of fine to coarse gravels, which graded into a very dense sandy gravel with depth.

South Abutment Area: The predominant soil type in this area is a dense to very dense, fine- to medium-grained silty sand to sand with varying amounts of fine to coarse gravels.

Ground water was not encountered during the field exploration and is believed to lie at depths greater than 30 m (100 ft) below the site.

Foundation Design Approach

As indicated above, the foundation soils consist primarily of dense, silty sand to sand. The estimated settlements of spread footings were 32 mm for the abutments and 44 to 57 mm for the center pier. These settlements were associated with a design bearing pressure of 190 kPa (2 tons per square foot) and were considered to be within tolerable limits. Thus, spread footings were selected for the abutments and pier. Due to the sloping terrain at the abutments, the abutment footings were founded on both cuts and structural fills. The maximum height of the structural fill is about 2.5 m (8 ft). The center pier is founded on natural ground. The elevation of the bridge is shown in Figure 2-13.

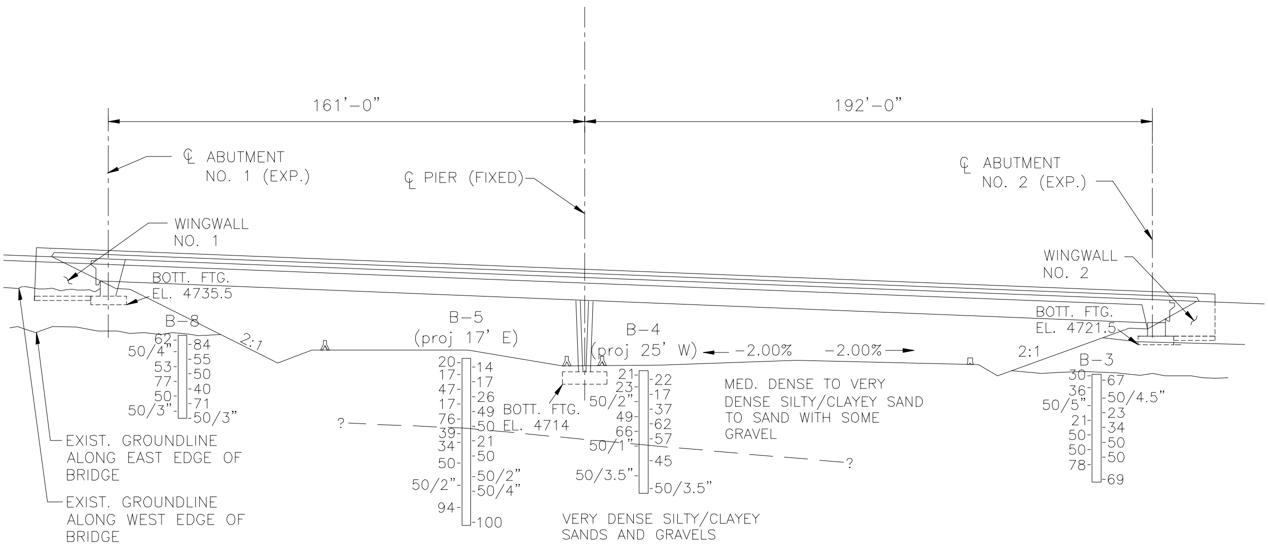


Figure 2-13: Elevation of Mt. Rose Interchange Bridge, I-580, Nevada

Construction Sequencing and Performance

The structural fills on which the footings were founded were compacted aggregate base (Nevada DOT designation is Type 1 Class B Aggregate Base), which consisted of crushed rock, 100 percent passing the 37.5 mm (1½ in) sieve and 1 to 12 percent passing the 75 µm (No. 200) sieve (other materials requirements are included in specifications in Appendix A). Construction specifications called for this material to be placed in lifts not exceeding 200 mm (8 in) in thickness, and compacted to 95 percent of the maximum density as determined by Test Method No. Nev. T101. *(Note: T101 is based on the use of the Harvard Miniature Compaction Device.)* Figure 2-14 shows the footing of the south abutment.

Settlement of the bridge structure was not monitored. According to the Nevada DOT, the structure has performed without problems since construction.

Figure 2-14: Footing at South Abutment, I-580, Nevada

Costs

This bridge was part of a project that involved about 7.2 km (4.5 mi) of new Interstate highway with numerous structures. The total construction cost of the entire project was about $52.9 million. The contract items were measured and paid on a unit-cost basis for the entire project, so direct comparison of individual bridge costs is not possible. However, the unit contract price to drive piles elsewhere on the project was $1,000 per pile, and the price for furnishing pipe piles was $65 per 0.3 m (1 ft). Assuming approximately 20 piles about 7.5 m (25 ft) in length had been driven at each abutment (and if the spread footings were not founded on structural fill), the cost savings based on deletion of the piles alone is estimated at $105,000. Assuming an average estimated cost for the entire structure of about $1,000 per square meter, the total estimated structure cost was about $2.8 million. The costs savings of using spread footings at the abutments bearing in compacted structural fills are therefore estimated to be at least 4 percent of the overall cost of the structure.

Acknowledgment: Parviz Noori, Todd Stefonowicz, and Jeff Palmer of Nevada DOT provided the information for this case history.

2.7.3 Case History No. 3: Cadillac Bypass Structure S01, Michigan

Project Background

The Cadillac Bypass project involves extending the limited-access US-131 around Cadillac to Manton, Michigan. The project includes 24 structures, 10 of which are or will be supported on spread footings. The entire project is scheduled for completion in 2004.

Structure S01, one of the structures supported on spread footings, is the northbound US-131 relocation (overpass) over S–43 Road and is 77.8 m (255.4 ft) long and 14.7 m (48.3 ft) wide. It is a three-span concrete structure with two abutments and two piers. Figure 2-15 shows an elevation of the bridge.

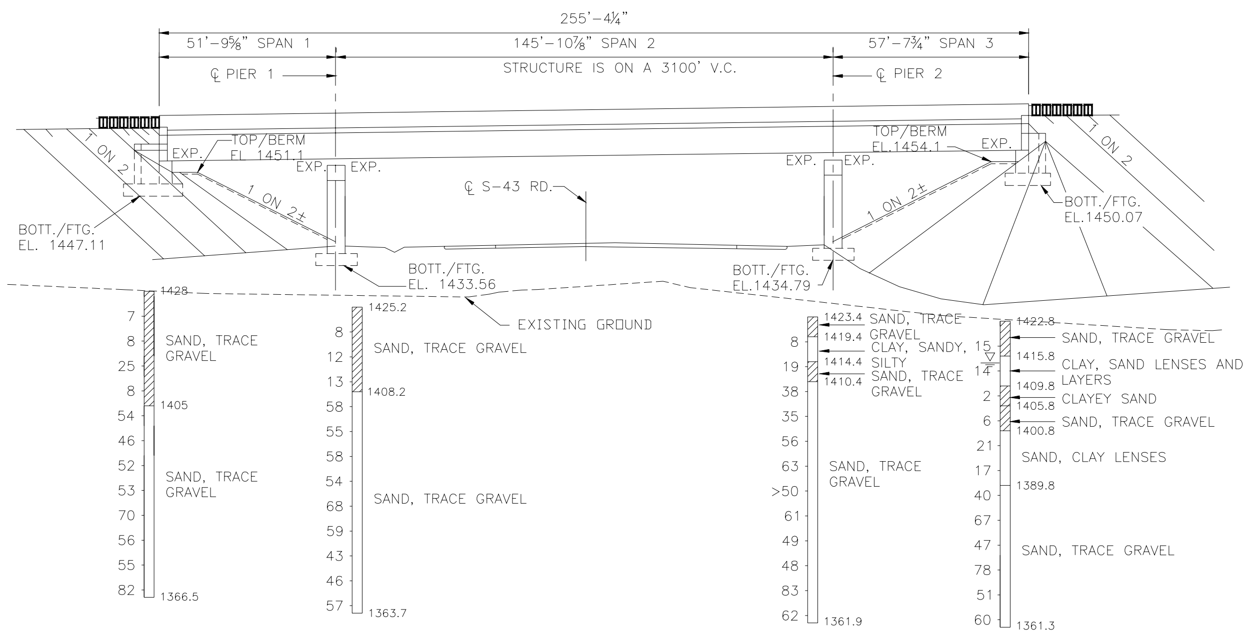


Figure 2-15: Elevation of Cadillac Bypass Structure S01, Michigan

Subsurface Conditions

Four borings were drilled at the site, one at each at abutment and pier location. Subsurface conditions at the site consist of a loose sand layer, up to about 6 m (20 ft) thick, underlain by medium dense to very dense sand with trace fine gravel. Three borings were dry at the time of completion, and the fourth had a water level 2.4 m (8 ft) below the ground surface.

Foundation Design Approach

The abutment and pier footings were placed on structural fill (Michigan DOT designation is Structure Embankment). Structure Embankment was placed on the existing ground after topsoil removal. Abutment footings were placed on the Structure Embankment. The thickness of the Structure Embankment under the abutment footings was about 7.6 m (25 ft) at the centerline.

Loose soils were removed from under the pier locations and backfilled with Structure Embankment. Pier footings were placed on Structure Embankment with a total thickness of over 4.6 m (15 ft). Figure 2-16 shows the structural fill under and above the footings of the bridge.

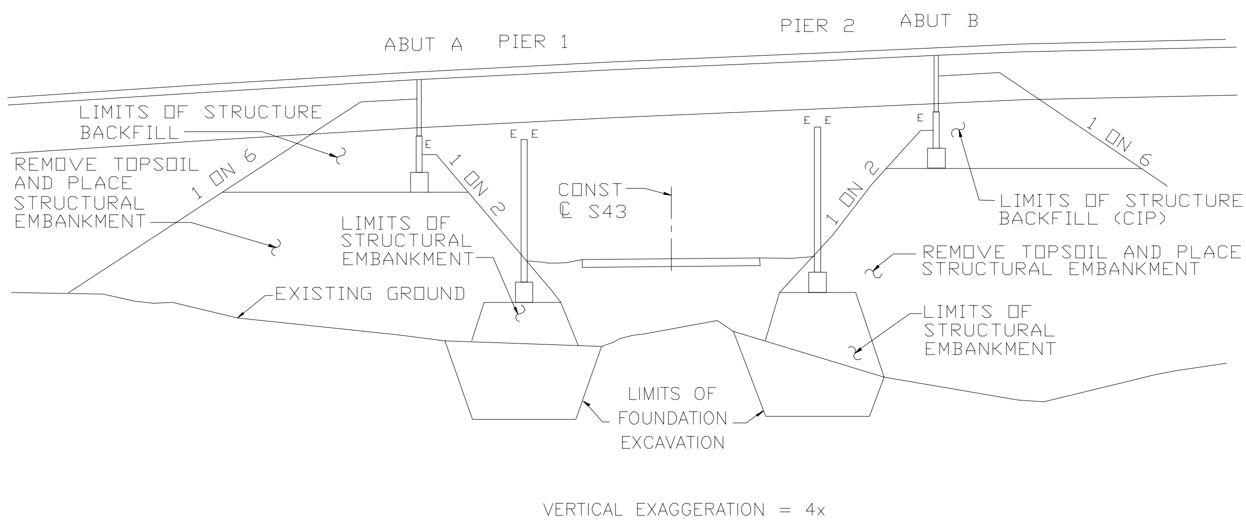


Figure 2-16: Profile Along Centerline Showing the Limits of Structural Fill,
Cadillac Bypass Structure S01, Michigan

Construction Sequencing and Performance

Structure Embankment consisted of granular material with 100 percent passing the 150 mm (6 in) sieve, 95 to 100 percent passing the 75 mm (3 in) sieve, and 0 to 15 percent passing the 75 μm (No. 200) sieve. It was placed in lifts with a thickness of 230 mm (9 in) to 380 mm (15 in) and compacted to 100 percent of the Maximum Unit Weight. The Maximum Unit Weight is determined by a method that is equivalent to the Modified Proctor Method. The specifications for the embankment material are included in Appendix A. Settlement of the structure has not been monitored.

Costs

The estimated cost for the entire project is $117 million. The cost for Structure S01 alone was about $2.3 million. Comparative costs of pile foundations were not available, but based on a rule-of-thumb cost for an in-place pile of about $2,500 and an assumption that about 18 piles per pier would be required if spread footings were not used, the cost savings is estimated at about $180,000, or about 8 percent of the total structure cost.

Acknowledgment: Richard Endres, Greg Perry and Al Rhodes of the Michigan DOT provided the information for this case history.

2.8 Pre-cast Foundation Elements, Mudsills and Temporary Footings

Many bridges require temporary support, or falsework, in order to complete the permanent structure. Falsework is, in turn, supported on temporary foundations. Because they are temporary, these foundations do not need the same design considerations as permanent structures. For instance, frost protection is usually not required, so the temporary footings can bear at shallow depth or directly on the ground surface.

However, the temporary footing still must be designed to limit the stress applied to the supporting soil such that a shear failure does not develop (see Section 5.2), and to limit the settlement to a tolerable amount (Section 5.3). The analytical procedures for checking the performance of a temporary footing are the same as for permanent structures (Chapters 5 and 6). Many transportation agencies include a requirement to perform a field plate load test (ASTM Test Method 1194 and AASHTO test method 235 are identical) to confirm the load-carrying capacity and settlement of falsework foundations. Plate load test data should be used with caution when a stiff, near surface crust overlies softer, saturated soils, since the depth of influence (see Section 3.1.5) will be less for the plate than the actual falsework foundations. In this case, site-specific settlement analyses should be required.

CHAPTER 3

SHALLOW FOUNDATION TYPE SELECTION

3.1 General Considerations

In most cases, spread footings represent the most economical foundation type if they do not have to be installed deeply into the ground. At some limiting depth, a "shallow" foundation begins to behave like and have the associated construction needs of a "deep" foundation. This limiting depth is somewhat arbitrary but generally can be taken as about two times the least-plan dimension of the foundation.

The decision to use a shallow foundation for support of a structure includes checking that an adequate margin of safety is provided against failure of the ground below the bearing depth (bearing capacity failure), and checking that deformations (settlement) under expected loading conditions will be acceptable. Design checks will also be performed to make sure the footing will not slide and that it is stable (e.g., will not overturn).

If the foundation can meet these fundamental design requirements, it also must be constructible. Constructability considerations are discussed in Section 3.3.

3.1.1 Footings on Cohesionless Soils

Granular, or cohesionless, soils are generally more suited to support of shallow foundations than cohesive soils, particularly when a foundation is supported on a structural fill. Cohesionless soils tend to be less prone to settlement under applied loads. Settlement of cohesionless soils generally occurs rapidly, as loads are applied.

Special consideration should be given to situations in which the following conditions may occur or be present:

- Water table close to or above the foundation bearing elevation. Saturated ground conditions will result in reduced effective stresses in the soils supporting the footing and in an associated reduction in the bearing capacity of the soil. See Section 4.4 for discussion of effective stress theory and computation and Chapter 5 for specific foundation design criteria and considerations.

- Steep slopes near the bearing elevation of a footing. An adequate factor of safety with respect to global stability must be maintained over the life of the structure.

- Presence of collapsible soils. Collapsible soils are generally stable when dry, but upon wetting or saturation, rapid settlement (collapse) can occur that could exceed the performance (settlement) criteria for the structure. Collapsible soils are regional in their occurrence. The potential for the presence of collapsible soils should be evaluated based on site-specific subsurface data and on local knowledge and experience.

- Presence of seismic hazards. Seismic hazards, including liquefaction potential under seismic conditions, should be evaluated. If liquefaction is possible, the dynamic stability of the footing should be checked and the potential for dynamic settlement assessed.

3.1.2 Footings on Cohesive Soils

Normally consolidated cohesive soils (clays) will experience consolidation settlement when subjected to an increase in stress such as that applied by a shallow foundation. The consolidation process and the procedure for calculating consolidation settlement are described in Section 5.3. Normally consolidated cohesive soils may also exhibit relatively low shear strength when loaded rapidly. This is an undrained loading condition. Therefore, both the bearing capacity of such soils and the potential for short- and long-term settlement must be evaluated as part of the preliminary design process when considering support of a shallow foundation on cohesive soils.

Because cohesive soils can experience large increases and decreases in volume as a result of changes in water content, bridge foundations should not be supported on embankments constructed of such materials. Expansive clays in natural conditions can also experience large volume changes. If clay soils are encountered at a site, the expansive potential should be evaluated as part of the site investigation and laboratory testing program before confirming the suitability of spread footings supported on the natural clay soils. Additional information on the identification of expansive soils can be found in the Subsurface Investigation Workshop Reference Manual (Arman et al., 1997).

Lightly overconsolidated cohesive soils (e.g., materials with over-consolidation ratios (OCR) of about 1 to 2) may also experience consolidation settlement in primary (virgin) compression under the range of stresses applied by a spread footing. Serviceability must be checked during the foundation type selection process to make sure total and differential settlements are within the acceptable performance range for the structure.

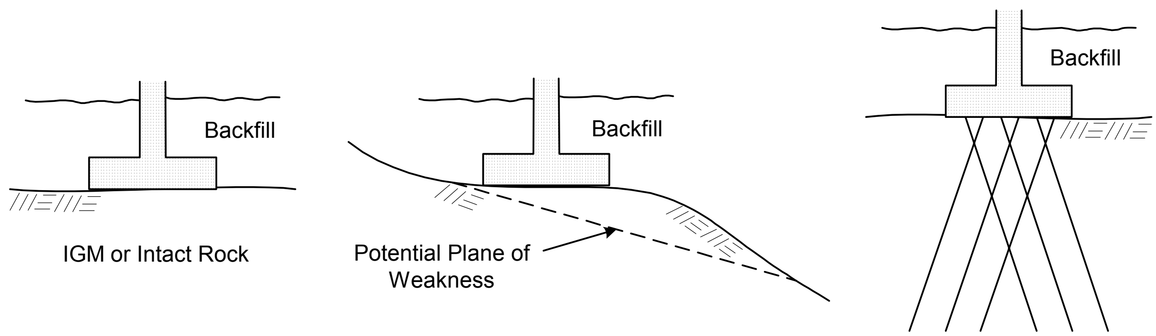
Heavily overconsolidated cohesive soils with OCRs greater than about 3 or 4 represent the most suitable cohesive soil conditions for consideration of support of shallow foundations. Heavily overconsolidated clays possess relatively high strength and low compressibility characteristics.

3.1.3 Footings on Intermediate Geomaterials (IGMs) and Rock

Intermediate geomaterial (IGM) is a term for ground conditions that are dense and stiff enough to no longer be characterized as a “soil” but are not considered intact “rock” (O’Neill et al., 1996). Even so, the term is convenient for discussions of ground conditions that differ in a geologic sense, but that behave similarly from an engineering perspective. IGMs include materials such as glacial till, completely weathered rock, poorly to moderately indurated sediments and chemically cemented materials. These materials may occur as the transition from overburden soil to intact rock. The term IGM is therefore rather broad in its definition and may carry regional connotations that are not consistent from location to location. For example, what may be considered "soft" sedimentary rock in one part of the world may be considered elsewhere to be so weak that it should be treated, for engineering purposes, as a soil.

For the purpose of shallow foundation discussions here, IGMs are considered to be sufficiently strong and stiff such that design considerations other than the strength of the material (e.g., overturning potential) will typically govern the design of a shallow foundation supported on such a material. Note that by their nature, IGMs may demand consideration of rock-like characteristics, such as mass structure and discontinuities.

Where an IGM or rock material is at or near the ground surface, the most economical foundation system to support highway structures will usually be a shallow foundation bearing in or on the IGM or rock. Three basic types of shallow foundations on IGMs or rock are shown in Figure 3-1.



(a) (b) (c)

Figure 3-1: Typical Shallow Foundations on IGM or Rock for Highway and Bridge Structures

Figure 3-1(a) illustrates a shallow footing founded directly on a relatively horizontal or mildly sloping, stable IGM or rock surface.

Figure 3-1(b) shows a shallow foundation near the crest of an IGM or rock slope. The geotechnical design of foundations near the crest of a slope should include detailed consideration of global stability, including planar, wedge, sliding and toppling failure mechanisms of the supporting slope. The potential for slope instability in IGMs and rock may be a function of the intact mass strength properties, orientation and condition of unfavorable structure or discontinuities (e.g., bedding, joints, foliations, etc.) or both. The stability analysis of IGM and rock slopes is not presented in detail here. The reader is referred to NHI Module 5, "Rock Slopes" (Wyllie & Mah, 1998) and *Rock Slope Engineering* (Hoek & Bray, 1981) for detailed discussions of rock slope stability analysis and design.

When supporting structures with large horizontal or uplift loads, shallow foundations may need to be tied into the IGM or rock to provide sufficient resistance to uplift and lateral forces and overturning moments, as portrayed in Figure 3-1(c). When considering design of a foundation in

IGM or rock conditions that will be required to resist large uplift or lateral loads, consideration should be given to the use, economics and constructability of a deep foundation socketed into the foundation material, as discussed in *Drilled Shafts: Construction Procedures and Design Methods* (O'Neill & Reese, 1999). Other options for resistance of uplift or lateral loads include micropiles and ground anchors. Refer to *Micropile Design and Construction Guidelines Implementation Manual* (Armour et al., 2000) and *GEC No. 4: Ground Anchors and Anchored Systems* (Sabatini et al., 1999) for guidance regarding these options.

Rock and IGMs can perform as competent and reliable foundation materials. However, the condition of the rock or IGM must be evaluated for degree of weathering, rock strength, durability, and the orientation and condition of discontinuities (e.g., joints, bedding or faults). An IGM or rock mass can also be shattered by blasting and/or undermined by cavities that can compromise the capability of the material to provide reliable support for a shallow foundation. Heavily overconsolidated IGMs, such as glacially overridden lakebed sediments and some residual weathered rock materials, may possess high intact strength but, if subjected to adverse stress changes or minimal shearing, will experience a drastic reduction in shear strength. Therefore, the stress history of an IGM should be considered during the foundation type selection process.

The selection of a shallow foundation supported on IGM or rock should therefore include early assessment of global stability (including structurally controlled failure mechanisms) and the potential for loss of stability due to sinkhole or subterranean cavern collapse. As with shallow foundations bearing on soils, the design of a footing on IGM or rock should also include checking bearing capacity, sliding, overturning and settlement.

3.1.4 Scour Potential

Scour is a hydraulic erosion process that lowers the grade of a water channel or river bed. This erosion is caused by flowing water. Excessive removal of the material around a shallow foundation or undermining of a footing can cause excessive deformation or structure collapse. Foundations for bridges and structures located near rivers, channels and in floodplains should be located below the limits of scour. If the scour depth is too deep for a shallow foundation to be constructed using normal methods, consideration should be given to selection of a deep foundation system for support of the structure. Additional details regarding scour potential evaluation and protection are presented in Section 6.2.2.

3.1.5 Depth of Influence and Frost Protection

Because spread footings have plan dimensions of limited areal extent, the stress induced in the ground mass below the footing will diminish with depth. In general, the ground conditions at a depth greater than about four times the lesser plan dimension of a continuous spread footing will be affected by less than about ten percent of the stresses induced under the footing (2 times the lesser plan dimension for square or nearly square footings). Closely spaced footings can have overlapping zones of influence that can combine to produce a zone of influence that extends deeper than these rules of thumb. The procedures in Section 5.3 can be used to calculate the stress at a given depth and location below a spread footing. These stress distributions are used in Chapter 5 for evaluation of bearing capacity and settlement.

Spread footings should be designed to bear below the frost depth for local conditions, or on materials that are not frost susceptible. Section 6.2.3 contains additional discussion on frost protection.

As can be seen from the stress bulbs in Figures 5-9 and 5-10, the depth of stress influence is a function of the width of the footing. The dimensions of the footing therefore play a role in the assessment of settlement and bearing capacity. Note that presumptive bearing capacities for footings on soils that can be found in the literature are typically not a function of width. This is one reason why presumptive values for footings on soils are not presented in this manual, and also why they should be used with caution, or for preliminary design purposes only.

3.2 Geotechnical Criteria in Preliminary Design

The geotechnical engineer should provide input to the preliminary design process. At this time there is usually insufficient information about pier locations and bridge length to warrant detailed subsurface information with borings. If no existing subsurface information is available in the area, preliminary borings may be appropriate to gain a general understanding of the subsurface conditions at the site. An assessment of ground conditions based on geologic and site reconnaissance, supplemented with subsurface information (either from existing information or preliminary explorations), is critical to structure layout and configuration. Some key considerations may include the following.

- Potential soft or weak ground conditions can be identified that will limit the steepness of embankment slopes. For bridges, this may dictate pier locations and overall structure length.

- Geologic hazards, such as earthquakes, volcanic eruption, flood, tsunami, avalanche, landslides, collapsible ground conditions, expansive soils, liquefaction potential, etc., should be identified. The presence of any of these conditions at a structure site can have significant impact not only on foundation type selection, but also on design, cost and/or construction schedules.

- Feasible foundations types should be evaluated. There may be sufficient existing information to conclude that shallow foundations are feasible at a site.

Early consultation with the geotechnical engineer can help avoid costly redesign, either late in the project development process or during construction.

3.3 Shoring and Construction Excavation Requirements

The design of a spread footing should be both efficient and constructible. Sometimes temporary shoring has to be installed before a spread footing can be constructed. The designer should consider whether shoring will be necessary and how it can be constructed. Some of the questions a designer should ask prior to selection of a shallow foundation include:

- Will normal excavation equipment be capable of constructing the footing, or will rock or difficult ground conditions in the excavation require special equipment? The geotechnical engineer should consider the type of equipment that will be necessary for the excavation, based on the results of the geotechnical investigation, and provide sufficient commentary in the geotechnical report for the structural designer and estimator to adequately specify the construction requirements.

- Will the excavation be stable under the temporary conditions, or will shoring be required to stabilize excavation sidewalls? The geotechnical engineer should consider the maximum slope angles at which the excavation slopes will stand without external support, and evaluate the potential for instability due to zones of weak ground and the location of the ground water table.

- Will obstructions be encountered? Buried structures, utilities, old pile foundations or boulders can result in contractor claims for additional compensation if provisions are not made for relocation or removal of obstructions in the footing excavation. The geotechnical investigation may or may not disclose the presence of obstructions, but the geotechnical engineer may provide commentary in the geotechnical report when geologic conditions or knowledge of prior site activities indicates that obstructions may be encountered. Existing utility location, including physical confirmation by potholing, is an activity that should be completed early in design to adequately assess construction costs.

- Will water be encountered in the foundation excavation? Water can pose a problem in the construction of shallow foundations. If there is a high water table, then consideration should be given to the stability of the bottom of the foundation soils just before the footing is placed. Softening or heaving of the bottom should be prevented. In some cases, dewatering and seals may be required. The geotechnical investigation should identify the presence and elevation of the water table below a site. The ground water conditions should be assessed at all times of the year, particularly when there is potential for seasonal variation of the water level.

CHAPTER 4

GEOTECHNICAL INVESTIGATION

4.1 PRELIMINARY DESIGN

Once a bridge layout or preliminary plan of the proposed structure is available, the geotechnical engineer can proceed with preliminary design activities. This process and its application are described in the *Soils and Foundations Workshop Reference Manual* (Cheney & Chassie, 2000) and are reiterated below. Preliminary design activities should include a terrain reconnaissance, which is predominantly an office study that includes collecting and reviewing existing information relevant to the project area. Sources of information include the following:

- Topographic maps available from the U.S. Geologic Survey (USGS) or the U.S. Coast and Geodetic Survey
- Geologic maps or bulletins available from the USGS or state geology agency
- Soil survey maps available from the U.S. Department of Agriculture National Conservation and Resource Survey (formerly Soil Conservation Survey)
- Well logs available from local or state water resource or protection agencies
- Air photos available from the USGS or state or private geographic services
- Construction records from nearby structures available from local building or highway jurisdictions

The last item is becoming increasingly valuable, as highway projects are frequently located in congested urban and suburban environments where numerous nearby existing structures, especially bridges, may be located. The geotechnical and subsurface information from the design and construction of these structures should be reviewed with judgment as to the reliability of the data, considering the methods available at the time of subsurface exploration. In particular, drilling methods may have been used that disturbed the soils, thus rendering Standard Penetration Test (SPT) test data unreliable. Refer to Section 4.3.1 for additional discussion on the performance and potential errors in the SPT test.

The terrain reconnaissance should focus on gaining an understanding of the geologic and geomorphic features (landforms) likely to be present at the project site. This information should be used to plan the subsurface exploration program for the site. Once this information is reviewed and understood, a field reconnaissance should be scheduled with the bridge engineer and other members of the design team to discuss foundation type alternatives and to plan a subsurface exploration accordingly. Other disciplines, such as hydraulics, environmental and general civil engineers can also benefit from, and provide valuable input during, the site reconnaissance on issues that may affect foundation type selection and location (e.g, stream stability, environmentally sensitive areas, utility location, etc.).

4.2 FIELD RECONNAISSANCE

The *area concept* of site exploration allows the geotechnical engineer to extend the results from a limited number of explorations in a particular landform to the entire geologic deposit. The area concept is a powerful tool in reducing subsurface exploration costs. The application of the area concept is described within the context of an entire transportation project in the *Soils and Foundations Workshop* and *Reference Manual* (Cheney & Chassie, 2000)

Application of the area concept requires the use of proper subsurface exploration equipment and techniques. An adequate site investigation can be accomplished only under the direction of a geotechnical engineer who knows the general limitations of the exploration equipment, as well as the general demands of the project. A site reconnaissance, preferably with the bridge engineer and other members of the design team, is recommended to assess foundation conditions, particularly for the purpose of foundation type selection.

The field reconnaissance for structures should include the following:

- Visually assess any nearby structures to determine their performance with the particular foundation type used. If settlement is suspected, and the original structure plans are available, arrange to have the structure surveyed using the original benchmark, if possible, to determine the magnitude of settlement experienced. If the structure is a bridge, review the bridge inspection reports and files to assess the performance history of the structure.

- Visually assess the surface geology and geomorphology, including looking for evidence of slope instability, rock or boulder outcrops, erosion or debris deposits, pre-existing fills, performance of existing cut and fill slopes and ground water seepage or springs.

- For water crossings, inspect any existing structure footings and the stream banks up- and downstream for evidence of scour. Take careful note of the streambed material. Often, large boulders exposed in the stream but not encountered in the borings are an indication of obstructions that could impact foundation construction, including shoring or cofferdams that may be required to construct a shallow foundation.

- Note the location, type and depth of any existing structures or abandoned foundations that may infringe on the foundation for the new structure.

- Locate subsurface and overhead utilities prior to subsurface exploration and show the utility locations on the bridge plans.

- Relate site conditions to proposed site exploration operations. Assess access for site exploration equipment, including the potential for problems with utilities (overhead and underground), site access (temporary clearing or grading, and site restoration), private property (right-of-entry) or obstructions (natural or man-made).

A field reconnaissance report form should be used to record observations and notes and to serve as a checklist of important items to note during the reconnaissance. Figure 1 of the *Soils and Foundations Workshop Reference Manual* (Cheney & Chassie, 2000) is an example of a field reconnaissance form that may be used to record data pertinent to the site.

Upon completion of the site reconnaissance, the geotechnical engineer should prepare a subsurface exploration plan or terrain reconnaissance report outlining the following:

- General suitability of the site for the proposed construction, specifically, addressing the potentially stable end-slope configuration (affects bridge length) and likely foundation types.

- Proposed exploration locations, equipment, required sampling intervals and standard test methods.

- Potential problems that may impact construction and cost.

- Beneficial shifts in alignment or pier location.

- Expected subsurface conditions so that the field exploration crew can communicate early if the encountered conditions are different.

- An estimate of subsurface exploration quantities, costs and time required for completion.

This information will be used by a variety of people to help the geotechnical engineer obtain the minimum level of subsurface investigation needed for design. The plan or report should be distributed to anyone who needs this information to proceed with their job responsibilities and may include the following persons or offices:

<u>Bridge Engineer</u> – The findings of the terrain reconnaissance may affect the bridge layout, including pier location and span length, which may, in turn, affect structure type and hence the performance expectations of shallow foundations.

<u>Project Office</u> – The design office may be requested to assist in obtaining right-of-entry permission and any necessary permits. A well-prepared exploration plan that documents the desired access requirements and the equipment and methods proposed can aid in obtaining work permits or approvals for access to sensitive areas or over water. The project office will also be advised of any potential impacts to construction cost or schedule that may result from ground conditions at the site. This helps avert the proverbial "search for the guilty" at later stages of project development and construction.

<u>Field Exploration Crew</u> – The exploration plan forms the basis for future communication between the drill crew and the geotechnical engineer. If conditions are encountered during subsurface exploration that are different from those expected and described in the plan, the drill crew should ask the geotechnical engineer if any changes should be made to the plan.

4.3 SUBSURFACE EXPLORATIONS AND IN SITU TESTING

The procedures employed in any subsurface exploration program are dependent on a variety of factors that vary from site to site. However, the project design objectives and the expected site soil conditions have a major influence on the subsurface explorations. Further, for shallow foundation consideration, the extent, location and sampling of subsurface materials will be different than if a deep foundation is anticipated. The minimum level of subsurface exploration and sampling is discussed in detail in the *Soils and Foundations Workshop Reference Manual* (Cheney & Chassie, 2000), in *Geotechnical and Foundation Engineering, Module 1 – Subsurface Investigations* (Arman et al., 1997), and in the *Manual on Subsurface Investigations* (AASHTO, 1988)

4.3.1 Standard Penetration Test

Probably the most widely used field test in the United States is the Standard Penetration Test (SPT). Both AASHTO T-206 and ASTM D-1586 have standardized this test. Despite the numerous drawbacks to the test that have been noted by various authors (see Cheney & Chassie, 2000), an extensive number of correlations have been developed based on this enormous quantity of data. Many of the design procedures in this Circular are based on the SPT.

SPT testing is therefore recommended for all transportation projects due to the simplicity and economy of the test and the usefulness of the data obtained. The SPT may be supplemented with other in situ tests as discussed in Section 4.3.3. In the SPT test, a relative measure of soil density and a soil sample are obtained. The test consists of driving the split-spoon sampler 0.46 m (18 in) with the 63-kg (140-lb) hammer falling through a drop of 0.76 m (30 in). The first 150-mm (6-in) increment is referred to as the seating load. The sum of the next two 150 mm (6-in) increments is known as the Standard Penetration Value (N). The soil sample obtained is disturbed but can be used for visual description and laboratory classification. The detailed procedures for conducting the test, as well as for limiting the potential errors frequently associated with the test, are well-documented in the *Soils and Foundations Workshop Reference Manual* (Cheney & Chassie, 2000) and in *Geotechnical and Foundation Engineering, Module 1 – Subsurface Investigations* (Arman et al., 1997).

The N-values of the SPT are an indication of the relative density of cohesionless soils and the consistency of cohesive soil. General N-value ranges are correlated with relative density and consistency in Table 4-1. It is emphasized that for gravels and clays the correlations to relative density and consistency are rather unreliable and should serve only as general estimates.

The SPT N-values are also frequently used to estimate strength and compressibility characteristics of granular soils, especially sands. The correlations shown in Figures 4-1 and 4-2 can be used to estimate the friction angle of granular soils, which then can be used to calculate bearing capacity, as described in Chapter 5. Note that Figure 4-1 is a function of both the N-value and the effective overburden stress, σ'_{vo} (see Section 4.4.2). Therefore, the N-value used to enter the chart should not be corrected for overburden stress. However, the chart assumes a rope and cathead operated hammer, with an assumed efficiency of about 60 percent for a hammer operated with two wraps of rope on the cathead.

TABLE 4-1: SOIL PROPERTIES CORRELATED WITH STANDARD PENETRATION TEST VALUES*

Sands (reliable)		Clays (relatively unreliable)	
Number of Blows per 0.3 m (1 ft), N	Relative Density	Number of Blows per 0.3 m (1 ft), N	Consistency
0 – 4	Very loose	Below 2	Very soft
4 – 10	Loose	2 – 4	Soft
10 – 30	Medium	4 – 8	Medium
30 – 50	Dense	8 – 15	Stiff
Over 50	Very dense	15 – 30	Very stiff
		Over 30	Hard

*Measured with 9.5 mm (3/8 in) I.D., 51 mm (2 in) O.D. sampler driven 0.3 m (1 ft) by a 63-kg (140 lb) hammer falling 0.76 m (30 in)

If a hammer system other than a rope and cathead is used, the N-value should be corrected to an efficiency of 60 percent. The correction is as follows:

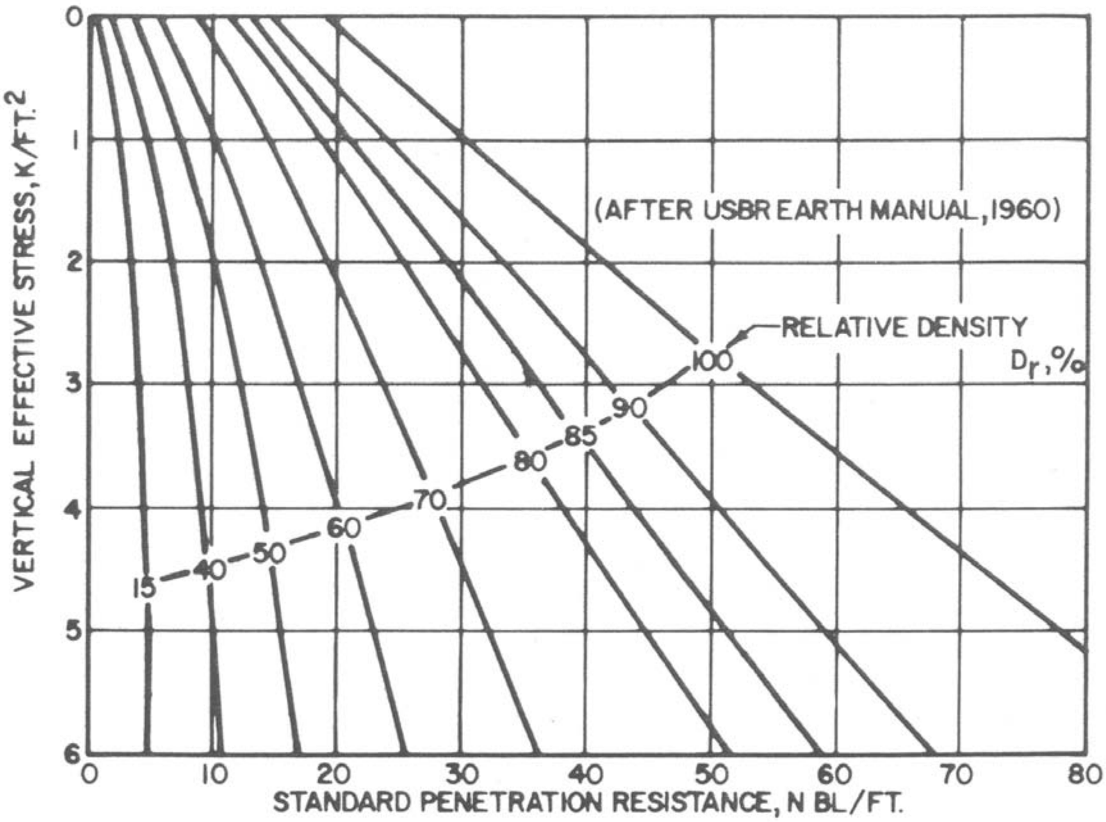
$$N_{60} = \frac{ER}{60\%}(N_{FIELD}) \tag{4-1}$$

where: N_{60} = SPT N-value corrected for hammer efficiency
ER = "Energy Ratio," the efficiency or percent of theoretical free fall energy delivered by the hammer system actually used
N_{FIELD} = Blowcount recorded in field

The use of automatic drop sampling hammers is increasing. Physical measurements of the efficiency of these hammer systems indicate ER values in the range of 70 to 90 percent, with 80 percent commonly assumed for correction of the N-values for automatic hammers.

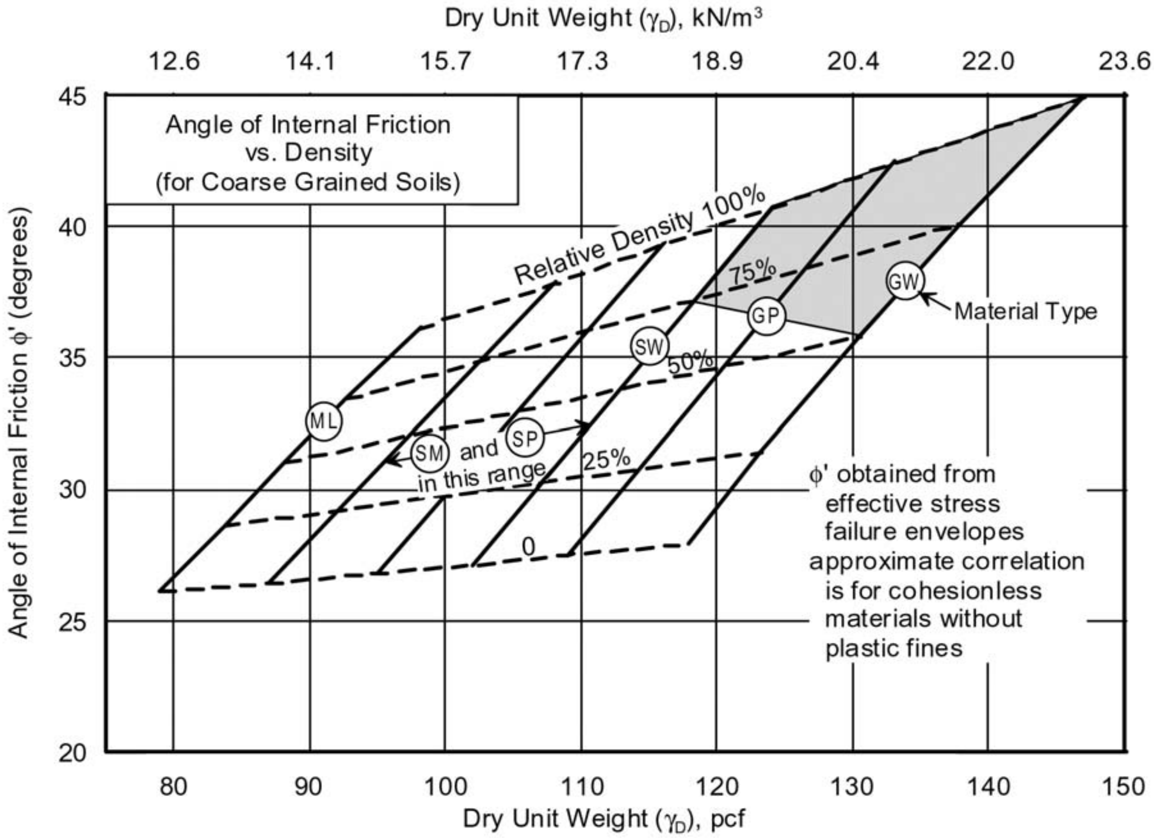
For example, if an automatic hammer with an Energy Ratio (ER) of 80 percent is used to obtain a field SPT blowcount (N_{FIELD}) of 30 blows per 0.3 m (1 ft), the blowcount corrected for hammer efficiency is:

$$N_{60} = \frac{ER}{60\%}(N_{FIELD}) = \frac{80\%}{60\%}(30) = 40$$



1 k/ft^2 = 1 ksf = 47.88 kPa
1 ft. = 0.3 m

Figure 4-1: Correlation Between Relative Density and SPT Resistance (NAVFAC, 1986a, after Gibbs & Holtz)



Note: Use caution in the shaded portion of the chart due to the potential for unreliable SPT N-values in gravels

Figure 4-2: Correlation Between Relative Density, Material Classification and Angle of Internal Friction for Coarse-Grained Soils (NAVFAC, 1986a)

4.3.2 Subsurface Exploration Program

With regard to the scope of the subsurface program for a structure, one must carefully consider the small cost of a boring in relation to the foundation cost. A 65-mm (2½-in) diameter drill hole will cost less than one 0.3-m (12-in) diameter pile. Yet the knowledge gained from that boring will permit the use of proper design techniques that may allow the cost-effective use of shallow foundations and elimination of all piles for that structure.

The number, depth, spacing and character of tests to be made in any individual exploration program are so dependent upon site conditions and the type of structure and its requirements that no rigid rules may be established. Conceptually, the exploration program must provide sufficient information to allow the geotechnical engineer to assess the strength and compressibility of the soils within the zone of influence of the foundation. To evaluate these design considerations, the geotechnical engineer will need site-specific data at each pier, abutment and retaining wall, as to the material type, the density or consistency of the material, and the location and depth of ground water.

The scale of the exploration and testing program usually depends on the importance and criticality of the structure, the magnitude of the loading, the availability of the budget and funding and the limitations of project schedule. The exploration and testing programs for foundations supporting less critical structures will be less extensive than those for critical structures such as bridges, elevated highways or official buildings, etc.

The following program will produce the minimum foundation data for a typical structure site. Soft ground conditions may require undisturbed sample explorations or in situ testing.

1. As a minimum, advance one drill hole at each pier or abutment that measures over 9 m (30 ft) in length. The drill hole pattern should be staggered at the opposite ends of adjacent footings. Piers or abutments over 9 m (100 ft) in length require one drill hole at the extremities of each element. The drill holes should be advanced using techniques that will allow in situ tests to be conducted in accordance with the test standards and that produce a hole that is stable. Appropriate drill techniques may include mud rotary with or without casing and continuous flight augers. Cone penetration tests (CPT) may be substituted for drilled test holes but should not be relied on solely as a method of obtaining subsurface data at a given bridge site. The CPT is not appropriate for all ground conditions, such as very dense soils or soil profiles with cobbles and boulders. Also, the CPT recovers no samples, so soil descriptions are based on empirical correlations, and no soil classification is possible. However, the readout of data is essentially continuous, so definition of soil layers is better than drilling and split-spoon sampling alone.

2. Estimate the boring depth from existing data obtained during the terrain reconnaissance phases in conjunction with minimum requirements that are established to help ensure that the borings are not terminated too soon or in a soft soil layer. The borings should also extend past the estimated depth of influence (Section 3.1.5), unless rock is encountered first. Minimum requirements may include criteria such as these:

   - The borings for structure foundations shall be terminated when a minimum SPT resistance of 20 blows per 0.3 m (1 foot) on the sample spoon has been achieved for 6 continuous meters (20 feet) of drilling, or

   - The boring shall extend 3 m (10 feet) into rock having an average recovery of 50 percent or greater.

   Such minimum criteria should be developed based on the design requirements of the local area and the expected subsurface conditions. More stringent criteria, such as higher blows for longer depths of penetration, should be specified where high lateral load conditions may be present (e.g., impact or seismic loads).

3. Obtain standard split-spoon samples at 1.5-m (5-ft) intervals or at changes in material. Split-spoon samples at 0.75-m (2½-ft) intervals are recommended for a minimum 4.5-m (15-ft) zone beneath the elevation where spread footings may be placed on natural soil. These spoon samples are “disturbed” samples generally not suited for laboratory

determination of strength or consolidation parameters, but they can be used for general index tests (e.g., moisture content, Atterberg limits and grain-size analysis). Undisturbed (e.g. thin-walled or Shelby tube) samples should be obtained at 1.5-m (5-ft) intervals in at least one boring in cohesive soils. For cohesive deposits greater than 9 m (30 ft) in depth, the tube sample interval can be increased to 3 m (10 ft). In soft clay deposits, in situ vane shear strength tests are recommended at 1.5- to 3-m (5- to 10-ft) intervals for measurement of intact shearing resistance.

4. Record the Standard Penetration Test data on each drill hole in accordance with ASTM D-1586. This is the most economical method presently available for procuring useful data, regardless of the frequently cited drawbacks of the test.

5. Instruct the drilling crew to perform a rough visual analysis of the soil samples and record all pertinent data on a standard drill log form. The disturbed split-spoon samples must be carefully sealed in plastic bags, placed in jars and sent to the soil mechanics laboratory for analysis. Undisturbed tube samples must be sealed and stored upright in a shockproof, insulated container normally constructed from plywood and filled with cushioning material.

6. Observe the water level in each boring, then record the depth below the top of the hole and the date of the reading on the drill log for the following:

    a. Water seepage or artesian pressure encountered during drilling. Artesian pressure may be measured by extending drill casing above the ground until flow stops. Report the pressure as the number of meters (feet) of head above ground.

    b. Water level at the end of each day and at completion of boring.

    c. Water level 24 hours (minimum) after hole completion. Long-term readings may require installation of a perforated plastic tube before abandoning the hole.

    A false indication of water level may be obtained when water is used during drilling and adequate time is not permitted after hole completion for the water level to stabilize. In low-permeability soils, such as clays, more than 1 week may be required to obtain accurate readings. In this case, the boring should be completed as a piezometer so that water levels can be measured over time.

The reasons for obtaining this minimum data are clear; the engineer must have adequate data to determine the soil type and relative density or consistency, as well as the position of the static water level. Methods such as driving open-end rod without obtaining soil samples or taking water-level readings after the last soil sample was removed must be discouraged. <u>Good communication between the driller and the foundation engineer is essential during all phases of the subsurface investigation program</u>.

4.3.3 In Situ Testing

The SPT test is considered the minimum level of in situ testing needed to complete a foundation design. Other in situ tests are available that can be employed when specific site data are desired, or to support the implementation of a particular type of engineering analysis. In the design of shallow foundations, this may include attempts to measure compressibility characteristics of the soils below the footing directly.

After the SPT, the cone penetration test (CPT) is probably the next most common in situ test performed to support foundation design activities. The CPT has the advantage of collecting nearly continuous data as the probe is pushed into the ground. The definition of the soil layering is therefore more discrete and accurate than that obtained from split-spoon sampling at 1.5-m (5-ft) intervals. Correlations to SPT N-values are available for interpretation of the CPT data. Drawbacks of the CPT include the limitation of material types that can be penetrated by the cone and the fact that no samples are recovered for material description or classification. Refer to ASTM 3441 and "The Cone Penetrometer Test" (Riaund & Miran, 1992) for details in performance of the CPT and for reduction and interpretation of the data.

Other in situ tests such as the pressuremeter test (PMT) and the flat dilatometer test can be used to measure the stress-strain properties of soils in situ, and have been used to predict the performance of shallow foundations. Refer to "The Pressuremeter Test for Highway Applications" (Briaud, 1989), "The Flat Dilatometer Test" (Riaund & Miran, 1992) and *Geotechnical and Foundation Engineering, Module 1 – Subsurface Investigations* (Arman et al., 1997) for guidance on the performance and interpretation of these tests and their results.

Recent research by Briaud and colleagues (Briaud & Gibbens, 1994, Briaud & Gibbens, 1995, and Briaud et al., 2000) has resulted in development of a new method of predicting shallow foundation settlement as a function of applied load based on in situ data obtained directly from the PMT. This procedure has potential for improving the geotechnical engineer's ability to predict settlement of a spread footing, not only under ordinary working loads, but also for higher loads that may be transient, such as earthquake or ice loads. The procedure and considerations regarding its use are presented in Section 5.3.4.3.

4.3.4 Rock and IGM Mass Characterization

A natural rock or intermediate geomaterial (IGM) mass is rarely a continuous, isotropic, homogeneous material. It can be highly variable and may be very difficult to characterize in a generalized manner. A rock mass is generally composed of intact or weathered blocks of rock separated by discontinuities (e.g., joints, bedding planes, sheared zones and faults). An IGM can exhibit variable zones of weathering or texture. The first step in a successful design of a footing on rock or IGM is to properly establish the material and discontinuity and weathering characteristics. The following are important characteristics for a shallow foundation design:

- Rock lithology (such as granite, gneiss, schist, limestone, sandstone)
- IGM genesis (e.g., weathered in place, glacially overridden, cemented)
- Strength properties (e.g., unconfined compressive strength, cohesion intercept, frictional angle)

- Deformation properties (e.g., Young's modulus, Poisson's ratio)
- Frequency of discontinuities (e.g., RQD)
- Spacing (S) of discontinuities (especially with respect to foundation width)
- Orientation of discontinuities (e.g. strike and dip, or dip and dip direction)
- Aperture, filling, and condition (open or closed) of discontinuities
- Degrees of weathering
- Ground water condition

To establish the above rock or IGM mass characteristics, the following exploration and testing methods as described in detail in various chapters of *Geotechnical and Foundation Engineering, Module 1 – Subsurface Investigations* (Arman et al., 1997) should be employed with proper planning:

- Field reconnaissance
- Geological mapping
- Drilling/coring
- Test excavation/test trench, if feasible
- In situ testing (see Table 4-2)
- Geophysical methods (see Table 4-2)
- Field load testing (see Table 4-2)
- Laboratory testing (see Table 4-2)

In addition, Module 5, "Rock Slopes" (Wyllie & Mah, 1998) includes detailed descriptions of geological mapping. In this Circular, Table 4-2 summarizes the characteristics of interest and available in situ and laboratory tests (after *Corps of Engineer Manual* EM 1110-1-2908). Note that not all of these tests will be applicable or feasible for all rock and IGM types. Some are applicable for stronger and stiffer materials (e.g., borehole jacking, unconfined compression), while others can yield useful design information for weaker and softer IGMs (e.g., pressuremeter, dilatometer).

It is common for a geotechnical engineer to provide design recommendations for a footing on rock or IGM based on limited data including rock type, core description, percentage of core recovery, Rock Quality Designation (RQD) values, and unconfined compressive strength. This information should be combined with the local geological knowledge, experience and judgment of the geotechnical engineer to develop the foundation recommendations.

4.4 Laboratory Testing

To develop a laboratory testing program that will provide data useful in design, it is important to test the sample under conditions that either exist currently in the ground or that will occur as a result of shallow foundation construction and loading. This means that the stress conditions, moisture states and load paths imposed on the representative soil sample in the laboratory should match the conditions that will be experienced by the material in the field. A detailed discussion of laboratory testing procedures is not presented here. However, the basic concepts of laboratory testing of soils to determine strength and deformation characteristics are reviewed, beginning with a discussion of the phase relationships of the constituents of soil, continuing with an

TABLE 4-2: SUMMARY OF COMMONLY USED IN SITU AND LABORATORY TESTING FOR FOOTINGS ON ROCK OR IGMS (AFTER ARMAN ET AL., 1997)

Purposes	Types of In Situ/Field Tests	Types of Laboratory Tests
Shear Strength	Pressuremeter Borehole shear test Torsional shear test	Uniaxial compression Triaxial compression Direct shear Point load
Tensile Strength		Direct tensile strength Split strength (Brazilian tensile strength tests)
Bearing Capacity	Plate bearing load test	(Same as shear strength)
Deformability	Pressuremeter Dilatometer Borehole jacking Seismic refraction survey • Cross-hole • Down-hole Plate bearing load test	Deformation moduli from: • Uniaxial compression • Triaxial compression Swell Creep
Discontinuity Study	Geologic mapping Geophysical methods • Seismic refraction • Ground penetrating radar • Electrical resistivity	Visual observation
Durability		Slake durability Freezing and thawing
Void/Sinkhole Detection	Ground penetrating radar Electrical resistivity imaging Microgravity	
Rock Anchor (Uplift Resistance)	Anchor pull test	

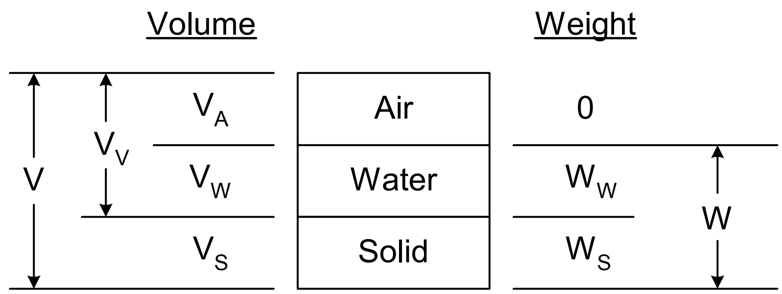
Note: See Module 1, "Subsurface Investigation" (Arman et al., 1997), for detailed descriptions of each testing procedure.

explanation of the principle of effective stress (which is used throughout this Circular), and finishing with a discussion of common laboratory tests used to determine the strength and deformation characteristics of soil.

4.4.1 Phase and Volume Relationships

Soil particles are irregularly shaped solids that are in contact with adjacent soil particles. The weight and volume of a soil sample depends on the specific gravity (ratio of the density of the soil particles to the density of water) of the soil particles (solids), the size of the space between soil particles (void or pore space), and the amount of the void space filled with water (pore water).

Soil is therefore usually a three-phase material (solid, water and gas), as shown in Figure 4-3. The proportion of the various phases, in terms of both weight and volume, can be represented schematically with the aid of the block diagram shown in Figure 4-3.



Note: Subscript *s* stands for solids, *w* for water, *a* for air and *v* for voids (the space between soil solids or alternatively, the air and water volumes combined). No subscripts are used relative to weight W and volume V of the entire soil mass.

Figure 4-3: Phase Relation Block Diagram

Relative to the block diagram, there are a number of weight-volume or phase relationships (i.e., ratios) that are useful in geotechnical engineering and are essential values or parameters used in laboratory testing.

These relationships are summarized in Table 4-3.

Of particular note is the void ratio, e, which is a general indicator of the relative strength and compressibility of a soil sample. Low void ratios generally indicate strong, incompressible soils, and high void ratios may indicate weak, compressible soils.

When a load is applied to a soil sample, the deformation that occurs will depend on the forces between the soil particles that are in contact with each other (intergranular forces) and the amount of water in the void space (pore water). Dry soils with no pore water in the void space will deform by a combination of sliding between the soil particles and deformation or crushing of the particles themselves. Intergranular sliding accounts for most of the deformation that occurs as the particles move to increase the contact area between the particles to support the increase in load.

TABLE 4-3: WEIGHT-VOLUME RELATIONSHIPS, DIMENSIONLESS RATIOS AND FUNCTIONAL RELATIONSHIPS

Unit weight

General definition

$$\gamma = \frac{W}{V} \quad (4\text{-}2)$$

where: γ = unit weight (F/L^3)
W = weight, units of force, F
V = Volume, units of length cubed, L^3

Unit weight of saturated soil is also:

$$\gamma_{sat} = \frac{W}{V} \quad (4\text{-}3)$$

Unit weight of soil solids

$$\gamma_S = \frac{W_S}{V_S} \quad (4\text{-}4)$$

Unit weight of water

$$\gamma_w = \frac{W_W}{V_W} \quad (4\text{-}5)$$

= 9.8 kN/m^3 (62.4 pcf)

Dry unit weight

$$\gamma_d = \frac{W_S}{V} \quad (4\text{-}6)$$

the ratio of the weight of soil solids (only), W_S, to the total volume V

Buoyant (or submerged) unit weight

$$\gamma' = \frac{W'}{V} \quad (4\text{-}7)$$

$$\gamma' = \gamma_{sat} - \gamma_w \quad (4\text{-}8)$$

where W' is the buoyant weight of the soil mass upon complete submergence in water, i.e., $W' = W - \gamma_w V$

Dimensionless ratios

Specific gravity of soil solids

$$G_S = \frac{\gamma_S}{\gamma_W} \quad (4\text{-}9)$$

Void ratio (a ratio of volumes, void vs. solid)

$$e = \frac{V_V}{V_S} \quad (4\text{-}10)$$

Porosity (a ratio of volumes, void vs. total)

$$n = \frac{V_V}{V} \quad (4\text{-}11)$$

Water content (a ratio of weights, water vs. solid)

$$w = \frac{W_W}{W_S} \quad (4\text{-}12)$$

Degree of saturation

$$S = \frac{V_W}{V_V} \quad (4\text{-}13)$$

Functional relationships

$$Se = wG_S \quad (4\text{-}14)$$

$$n = \frac{e}{1+e}; \quad e = \frac{n}{1-n} \quad (4\text{-}15)$$

$$\gamma = \frac{(G_S + Se)\gamma_W}{1+e} \quad (4\text{-}16)$$

$$\gamma_d = \frac{G_S \gamma_W}{1+e} \quad (4\text{-}17)$$

$$\gamma_d = \frac{\gamma}{1+w} \quad (4\text{-}18)$$

$$W_S = \frac{W}{1+w} \quad (4\text{-}19)$$

$$\gamma' = \frac{(G_S - 1)\gamma_W}{1+e} \quad (4\text{-}20)$$

Because water is essentially incompressible, the water contained in the pore space of soils that are saturated (i.e., pore spaces completely filled with water) is also a function of the ability for water to be expelled from the pore space. The permeability of a soil is a measure of the speed with which water molecules can pass through the pore space. The lower the permeability of a soil sample, the longer it takes for water to be expelled from the pore space. If a load is applied quickly to a soil sample with low permeability, the pore water will initially carry the load until the water drains from the pore space and the soil particles begin to slide and accept the load. While the load is still carried by the pore water, the water will experience increased, or excess, pore-water pressure. As the excess pore-water pressure dissipates, the soil sample will deform into a smaller volume and a denser configuration. This process is termed "consolidation."

The consolidation of soil may be explained by the spring analogy shown in Figure 4-4. The ability of the intergranular contact areas to carry load is represented by the spring. The valve represents the permeability of the soil. The water inside the piston chamber and surrounding the spring is analogous to the pore water filling the pore space of the soil sample.

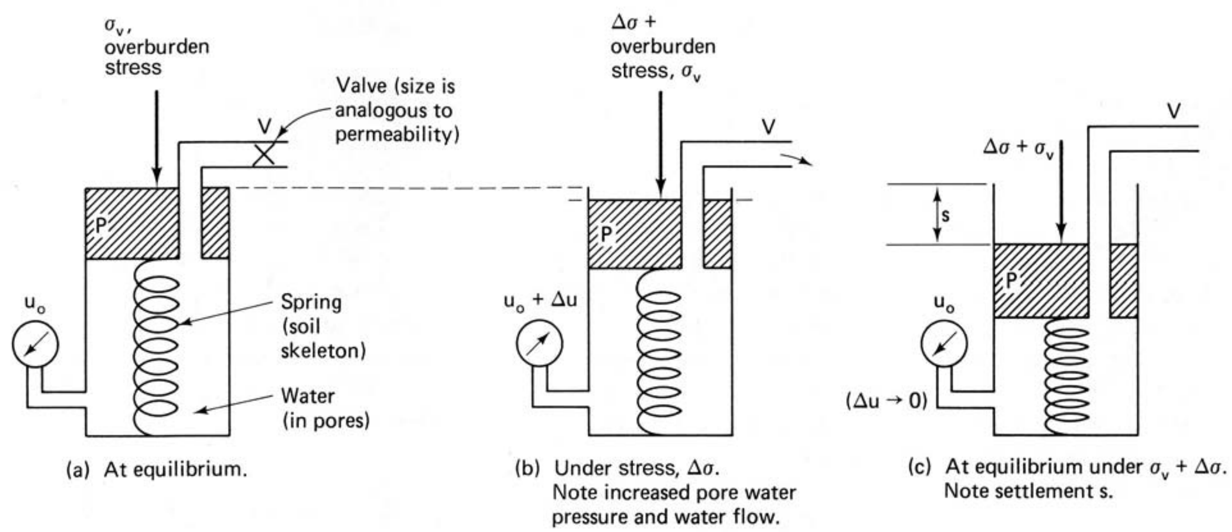


Figure 4-4: Spring Analogy as Applied to Consolidation (Holtz and Kovacs, 1981)

A load is applied to the piston with the valve closed (Figure 4-4a). With the valve closed, the incompressible water cannot be expelled, and the spring is unable to compress to accept an increase in load. Essentially the entire applied load is resisted by an increase in fluid pressure within the chamber (i.e., excess pore-water pressure).

When the valve is opened, water will begin to escape through the valve (Figure 4-4b). As water escapes, the spring shortens and begins to carry an increasing proportion of the applied load. There is a corresponding decrease in the fluid pressure within the fluid chamber. Eventually, a condition is reached (Figure 4-4c) in which the entire applied load is carried by the spring, and the pressure in the water has returned to its initial state, termed the "hydrostatic condition." Once this stage is reached, there is no further flow of water, the spring no longer continues to compress and there is no more reduction in the volume of the chamber.

4.4.2 Principle of Effective Stress

The consolidation process demonstrates the important principle of effective stress. Under an applied load, the total stress in a saturated soil sample is composed of two parts. One part acts in the water and the soil in all directions with equal intensity. This part is termed the "pore-water pressure" (or neutral stress). The other part acts solely between the areas of soil particle contact and is termed the "intergranular stress." Because the pore water has no shear strength and is essentially incompressible, only the intergranular stress is effective in resisting shearing or limiting compression of the soil. The intergranular stress is therefore called the "effective stress." The effective stress is the portion of the total stress over and above the pore-water pressure and can therefore be represented mathematically by Equation 4-21.

$$\sigma' = \sigma - u \tag{4-21}$$

Where σ is the total stress, u is the pore water pressure, and σ' is the effective stress.

Changes in the effective stress control the volumetric strain that develops in a soil. For instance, consider the load applied to a soil specimen by a laboratory oedometer as shown in Figure 4-5. The saturated specimen of clay can be compressed when sufficient pressure is applied to the soil skeleton to develop vertical deformations. However, essentially zero volume change occurs if the soil is loaded quickly – zero change in the volume of the voids ($V_V = V_W$). The water pressure increases by approximately the same amount as the total stress increases, while the effective stress does not initially change. If the load is maintained for a sufficient period of time, water will be squeezed out of the soil through the porous stones, and the increase in pore-water pressure will slowly dissipate until it is zero. After the excess pore water pressure has dissipated, the effective stress in the soil sample will be equal to the new total stress (note that in the oedometer the hydrostatic water pressure is negligible).

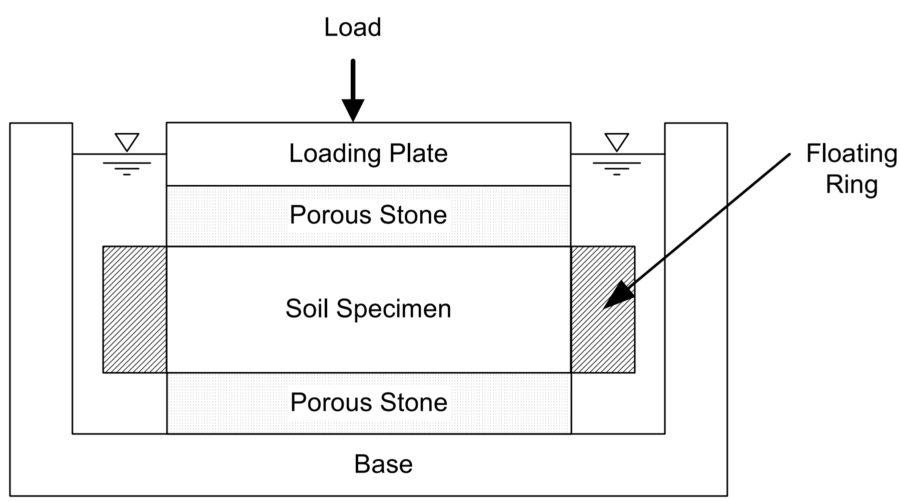


Figure 4-5: Conceptual Arrangement of Laboratory Oedometer Test

4.4.3 Overburden Stress

As mentioned in the introduction to this section, the laboratory testing required to solve soil-related problems involves simulating the conditions that exist naturally in the ground. Soils existing at a distance below the ground surface are influenced by the weight of the soil above that depth. The weight of the overlying soil exerts a pressure on the soil at that depth. This pressure is called the "overburden stress" and is a function of the unit weight of the overlying soil and the depth to the point under consideration. When a soil sample is removed from depth, the state of stress is relieved as all weight confining the sample is removed. When conducting a test on the sample, it is important to reestablish the state of stress in the soil sample that existed prior to removal of the sample from the ground. Changing the state of stress on the sample and observing the resulting strain can then provide information useful in design. The changes in stress that are applied to the sample should be representative of the changes in stress that are expected to occur as a result of constructing the shallow foundation upon the soil. As discussed, the effective stress resulting from intergranular contact is the controlling factor in evaluating the shearing resistance and compression characteristics of the soil sample.

Before requesting laboratory testing of soil samples, the engineer must estimate the total and effective stress resulting from overburden at the depth of sampling. The total or effective stress overburden stress is specified to the lab as the beginning stress state for a particular test.

The total overburden stress is computed by summing the product of the total unit weight of the soil times the layer thickness. The soil can be broken into stratigraphic layers if the unit weight varies with depth. Unit weight may be determined from undisturbed samples or estimated from SPT N-values, soil descriptions and moisture-content data. A plot of total stress and effective stress with depth is extremely useful and should be plotted for each foundation location under consideration. The plot can be constructed as follows:

1. Divide the subsurface profile into layers based on the soil stratigraphy. Layers should be selected based on material description or classification, the location of the ground water table, changes in soil unit weight and increments of about one-half the estimated footing width for a distance of about twice the footing width below the estimated bearing depth.

2. Determine the total vertical stress at the mid-point and bottom of each layer or sub-layer by multiplying the total unit weight by the thickness of each respective soil layer above the depth of calculation.

3. Determine the pore water pressures below the water table by multiplying the depth below the water table to the point of calculation by the unit weight of water (γ_w = 9.81 kN/m^3 = 62.4 pcf).

4. Calculate the effective vertical stress by subtracting the pore water pressure from the total stress at each point below the water table.

5. Plot the calculated total and effective vertical stresses versus depth or elevation.

The initial stress for laboratory tests can be read easily from this plot, which also will be used throughout the design of a shallow foundation.

Laboratory testing to determine the shearing resistance and compression characteristics of soils is relatively expensive and time consuming. The engineer should consider the cost and time required to perform such tests relative to the benefit achieved in terms of reduced foundation cost that results from an optimized design. In many cases, footings on cohesionless soils can be designed safely and effectively using shear strength and compressibility correlations based on the SPT N-values, as described in Chapter 5.

Cohesive soils, however, can exhibit both low-strength and high-compressibility characteristics that are not reliably reflected in the SPT N-values. Consolidation testing is usually essential to predict the settlement of a shallow footing founded on cohesive soils. Shear-strength testing of cohesive soils should also be considered if a bearing failure mechanism could develop under the footing.

CHAPTER 5

GEOTECHNICAL DESIGN

The design of shallow foundations involves calculating an allowable bearing pressure that will (a) maintain an adequate factor of safety relative to shear failure of the bearing soil, and (b) limit the settlement of the foundation to meet serviceability requirements. The *allowable bearing capacity* of a shallow foundation is defined as the lesser of:

- The pressure that will result in a shear failure divided by a suitable factor of safety (FS), or

- The pressure that results in a specified limiting amount of settlement.

This chapter presents and discusses the theory behind calculation of allowable bearing capacities, considering both of these important design requirements.

This chapter also presents the geotechnical procedures for computing the resistance to sliding for a shallow foundation, and the geotechnical considerations needed to ensure the foundation is stable from a global, or slope stability, perspective.

5.1 ALLOWABLE BEARING CAPACITY

The allowable bearing capacity of a spread footing historically has combined the design considerations of minimizing the potential for shear failure of the soil and limiting vertical deflection (settlement). Both of these design considerations are a function of the least footing dimension, typically called the "footing width," and designated as the variable, B or B_f. In general, for a footing bearing on essentially an isotropic, homogenous material, with no embedment (i.e., founded at the surface), the factor of safety against a shear failure developing beneath the footing will increase as the footing width, B, increases. However, as a footing's dimension increases, the depth of influence also increases. Stated another way, as the footing dimension, B, increases, the stress increase "felt" by the soil extends more deeply below the bearing elevation.

The effect of footing width on bearing capacity and settlement is shown conceptually in Figure 5-1. Note that the allowable bearing capacity of a footing is controlled by shear-failure considerations for narrow footing widths. But as the footing width increases, the allowable bearing capacity is limited by the settlement potential of the soils supporting the footing.

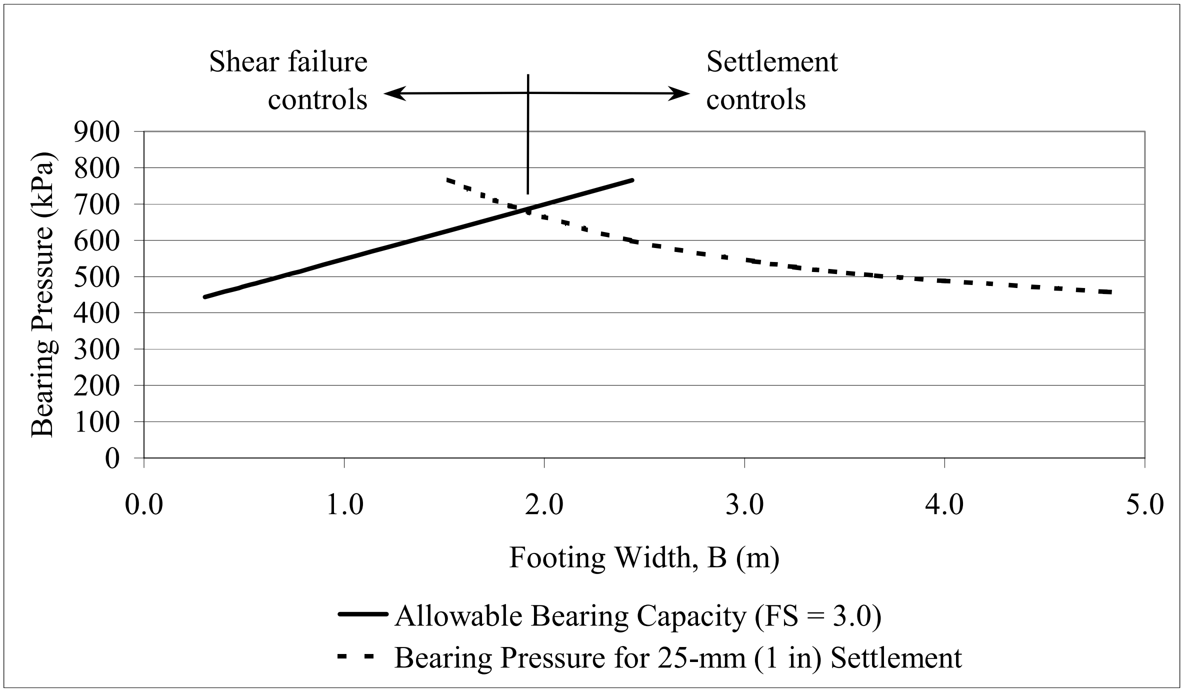


Figure 5-1: Allowable Bearing Capacity Controlled by Shear Failure Considerations versus Settlement Considerations

5.2 Bearing Capacity Theory

This section discusses bearing capacity theory and its application toward computing allowable bearing capacities for shallow foundations. Methods of calculating settlement displacements are presented in Section 5.3.

When a load is transferred through a footing to the foundation soil/rock, the subsurface materials experience settlement due to elastic (immediate) strains and long-term consolidation (elastic and/or plastic deformation) of the ground. The footing will penetrate into the foundation soil/rock when the intensity of applied loads is such that the load-carrying capacity of the foundation material is exceeded. A foundation failure will occur when the footing penetrates excessively into the ground or experiences excessive rotation (Figure 5-2). Excessive foundation movements such as penetration and rotation of the foundation may cause structural damage or collapse**. A failure caused by the vertical and lateral displacement of foundation soils due to lack of sufficient strength is called a "bearing capacity failure." The load that develops this type of subsurface collapse is called the "ultimate bearing capacity" of the soil.**

Figure 5-2: Bearing Capacity Failure of Silo Foundation
(Vesic, 1975, from Tschebotarioff, 1951)

5.2.1 Failure Mechanisms

5.2.1.1 General Shear

The kinematic conditions (strain states) that develop when a uniform, rigid-plastic, weightless soil (possessing cohesion c' and friction ϕ') reaches the ultimate bearing capacity were determined theoretically by Prandtl (1920) and Reissner (1924). When a footing is loaded to the ultimate bearing capacity, the condition of plastic flow of foundation soils develops.

Terzaghi and Peck (1948) further defined the zones of plastic equilibrium after failure of soil beneath continuous footing (Figure 5-3). As shown in Figure 5-3, a triangular wedge beneath the footing, designated as Zone I, remains in an elastic state and moves down into the soil with the footing. Radial shear develops in Zone II such that radial lines extending from the footing change length based on a logarithmic spiral until the failure plane reaches Zone III. A passive state develops in Zone III at an angle of $45^o - (\phi'/2)$ from the horizontal. This configuration of the ultimate bearing capacity failure, with well-defined shear planes developing and extending to the surface, with footing rotation, and bulging of the soil on both sides of the footing, is called "general shear." General shear-type ultimate bearing capacity failures (Figure 5-4a) are believed to be the prevailing mode of failure for soils that are relatively incompressible and reasonably strong, or saturated, normally consolidated clays that are loaded rapidly so that undrained conditions and therefore undrained soil strength governs (Coduto, 1994).

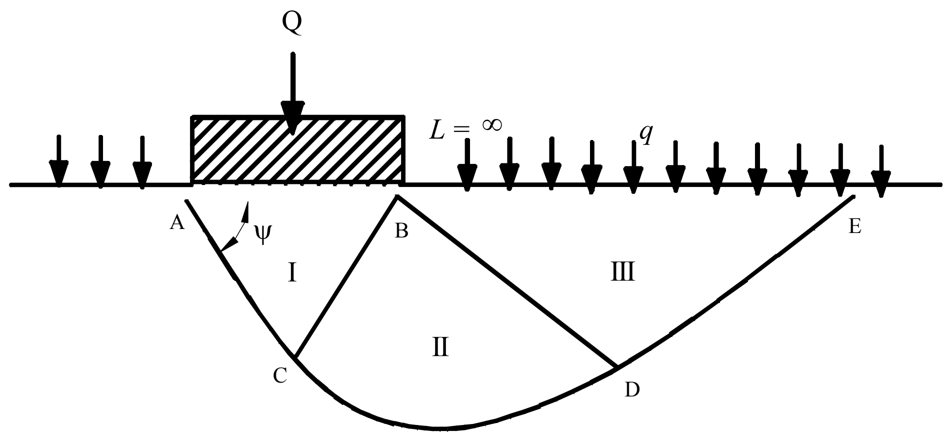


Figure 5-3: Boundaries of Zone of Plastic Equilibrium after Failure of Soil Beneath Continuous Footing

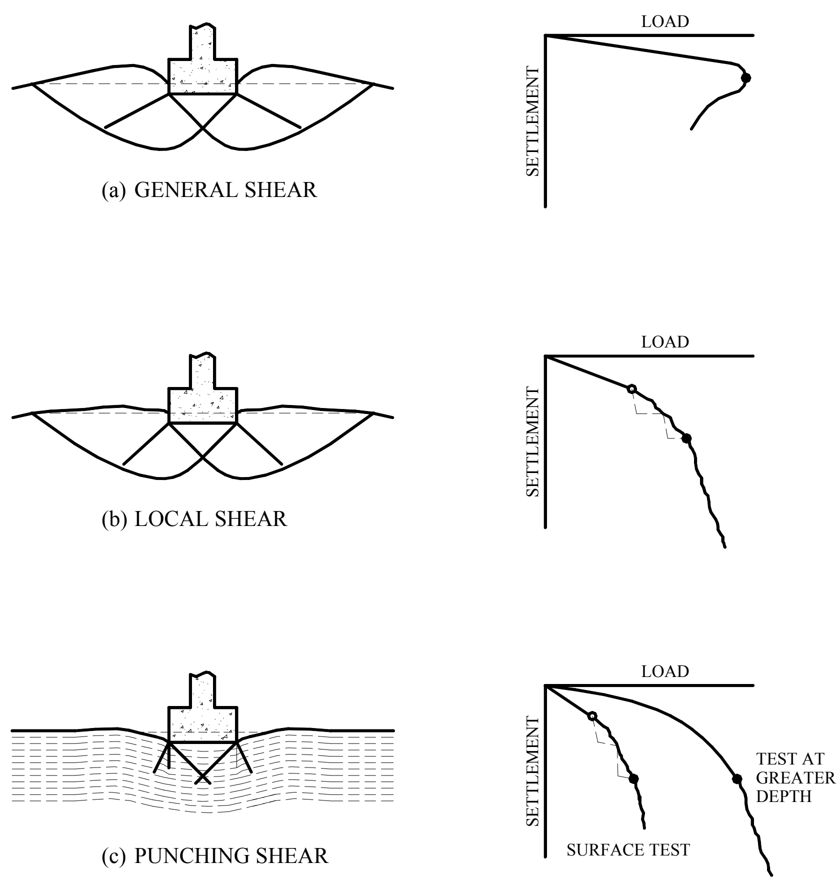


Figure 5-4: Modes of Bearing Capacity Failure (After Vesic, 1975)
(a) General Shear (b) Local Shear (c) Punching Shear

5.2.1.2 Local Shear

In some cases, the bearing capacity shear planes are not well developed, and the failure planes do not extend all the way to the ground surface. This mode of ultimate bearing capacity failure (Figure 5-4b) is called "local shear." The deformation patterns in local shear involve vertical compression (Vesic, 1975) beneath the footing, bulging of the soil at the ground surface and essentially no rotation or tilting of the footing. Local shear failures may occur in soils that are relatively loose or soft when compared to soils susceptible to general shear failure.

5.2.1.3 Punching Shear

Another type of failure observed under ultimate bearing-capacity conditions involves vertical compression of the soils beneath the footing without bulging of the soil. As shown in Figure 5-4c, the bearing load continuously increases when the footing is loaded under strain-controlled conditions ('test at greater depth'). This kinematic mode of ultimate bearing capacity failure is called "punching shear." Punching shear is considered a potential failure mode for shallow foundations when loose or compressible soils are loaded slowly under drained conditions. For instance, footings placed at great depth on dense sand or on dense sand underlain by soft, compressible soil can fail under punching-shear modes. A footing on soft clay can also fail under punching shear if it is loaded slowly.

Note that from a perspective of bridge foundation design, soils so obviously weak as to experience local or punching shear failure modes should be avoided for supporting shallow foundations. Additional guidance on dealing with soils that fall in the intermediate, or local shear range of behavior is provided in Section 5.2.3.6.

5.2.2 Bearing Capacity Equation Formulations

The gross ultimate bearing pressure is equivalent to the stress applied to the soil by the footing at ultimate (failure) conditions at the bearing elevation. In theoretical formulations, Prandtl (1920) and Reissner (1924) developed an expression for the gross ultimate bearing capacity that considered both frictional and cohesion components of soil shear strength and a uniform surcharge pressure at the base of the footing due to the depth of embedment of the footing, D_f. The gross ultimate bearing pressure ($q_{ult\ gross}$) for this case is:

$$q_{ult\ gross} = c(N_c) + q(N_q) \tag{5-1}$$

where: c = cohesion of the soil

q = γD_f = surcharge at the base of the footing

γ = Unit weight of the overburden material causing the surcharge pressure

D_f = Depth of embedment (Figure 5-5)

N_q = bearing capacity factor for the surcharge term:

$$= e^{\pi \tan\phi} \tan^2\left(45° + \frac{\phi}{2}\right) \tag{5-2}$$

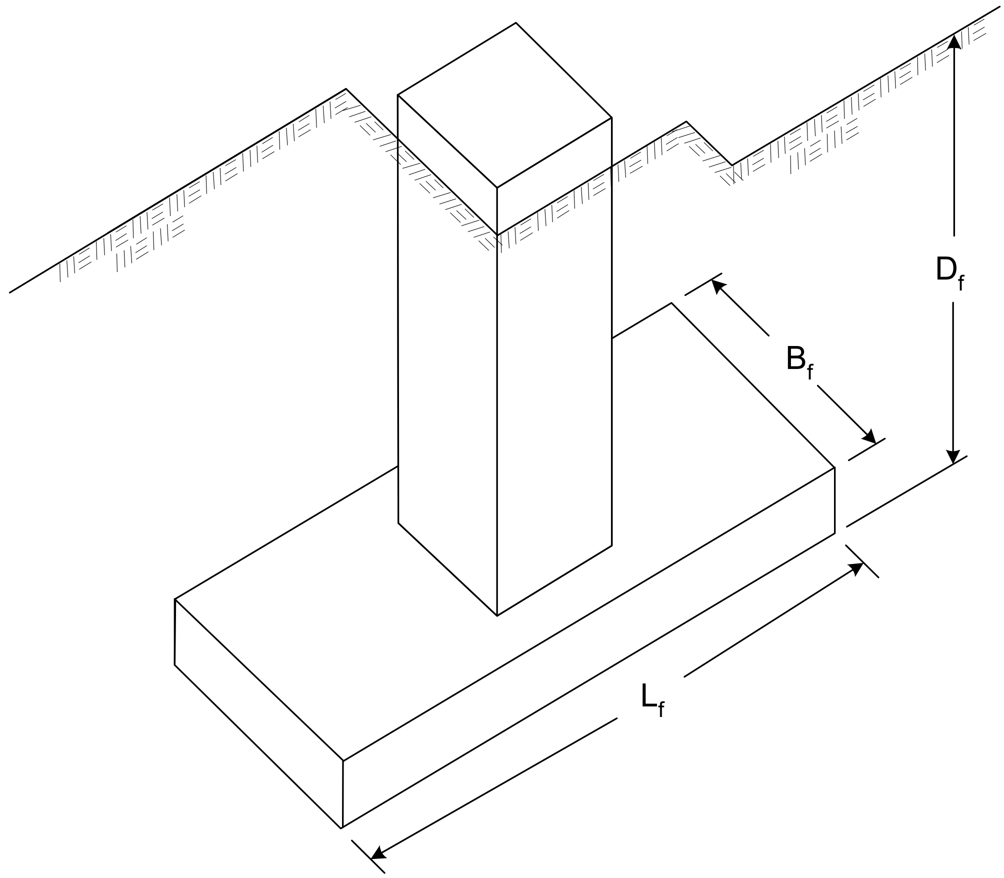


Figure 5-5: Design Terminology for Spread Footing Foundation

N_c	= bearing capacity factor for the cohesion term:		
	$= (N_q - 1)\cot\phi$	for $\phi > 0°$	(5-3)
	$= 2 + \pi = 5.14$	for $\phi = 0°$	(5-4)

The *net* ultimate bearing pressure is the difference between the gross ultimate bearing pressure and the pressure that existed due to the surcharge at the bearing depth before the footing was constructed, q (= γD_f). The net ultimate bearing pressure can thus be computed by subtracting the surcharge (q) from Equation 5-1:

$$q_{ult\ net} = q_{ult\ gross} - q \qquad (5\text{-}5)$$

$$q_{ult\ net} = c\,N_c + q\,(N_q - 1) \qquad (5\text{-}6)$$

By inspection of Equation 5-6, the actual difference between $q_{ult\ net}$ and $q_{ult\ gross}$ is generally small for cohesionless soils (i.e., $\phi > 0^o$).

Further, the structural designer will typically include the self-weight of the concrete footing and the backfill over the footing (approximately equal to $\gamma\ D_f$) in the loads that contribute to the applied bearing stress. Therefore, if the geotechnical engineer computes and reports a net ultimate bearing pressure, the effect of the surcharge directly over the footing area is counted twice. This is conservative, but generally not recommended, provided that a suitable factor of safety is maintained against bearing capacity failure. If the geotechnical engineer chooses to report an allowable bearing capacity computed from a net ultimate bearing pressure, this fact should be clearly stated in the foundation report.

Therefore, it is recommended that $q_{ult\ gross}$ be used in determination of the allowable bearing capacity of a soil (e.g., $q_{ult} \approx q_{ult\ gross}$).

There are no analytical solutions available to extend the bearing capacity equations to include the effects of unit weight of the foundation material. Numerous investigators have proposed relationships for the inclusion of the unit weight. They involve adding an independent term (N_γ) to the ultimate gross bearing capacity equation. Analyses were developed by assuming the shapes of the failure surfaces and performing trial analyses until a solution was obtained. The revised ultimate gross bearing capacity equation for a centrically loaded strip footing on a foundation material with unit weight now becomes:

$$q_{ult} = c\,(N_c) + q\,(N_q) + 0.5\,(\gamma)(B_f)(N_\gamma) \qquad (5\text{-}7)$$

where:
- c = cohesion of the soil
- N_c = bearing capacity factor for the cohesion term
- q = surcharge at the base of the footing
- N_q = bearing capacity factor for the surcharge term
- B_f = footing width
- γ = unit weight of soil beneath the footing
- N_γ = bearing capacity factor for soil unit weight

Note that a range of values has been proposed by various authors for N_γ. The differences reported by various researchers range from about one-half to double the values presented below, depending on the friction angle selected for design (Chen & McCarron, 1991). Caquot and Kerisel's (1948) equation for N_γ is currently used by FHWA and AASHTO:

$$N_\gamma = 2\,(N_q + 1)\tan(\phi) \qquad (5\text{-}8)$$

The values of N_c, N_q, and N_γ as functions of the friction angle are included in Table 5-1 and in Figure 5-6.

Note from Equation 5-7 that the ultimate bearing capacity of a soil is a function of the cohesive and frictional strength of the soil (c and ϕ), the effective weight of the soil above and below the footing (γ or γ'), and the depth and width of the footing (D_f and B_f). Consider now the effect each of these variables has on the computed ultimate bearing capacity. For cohesive soils (i.e., $\phi = 0$, undrained loading), the last term is zero and the first term is a constant. Therefore the ultimate bearing capacity will only be a function of the cohesion and the depth of embedment of the footing.

Cohesionless soils, on the other hand, will experience large changes in ultimate bearing capacity when properties and/or dimensions are changed. The embedment effect is particularly important. Removal of soil from over an embedded footing, either by excavation or scour, can substantially reduce bearing capacity and cause footing subsidence. Similarly, a rise in the ground water level to the ground surface will reduce the effective (buoyant) unit weight of the soil, thus reducing those terms that are a function of unit weight by essentially one-half.

TABLE 5-1: BEARING CAPACITY FACTORS (AASHTO, 1996)

ϕ	N_c	N_q	N_γ	ϕ	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

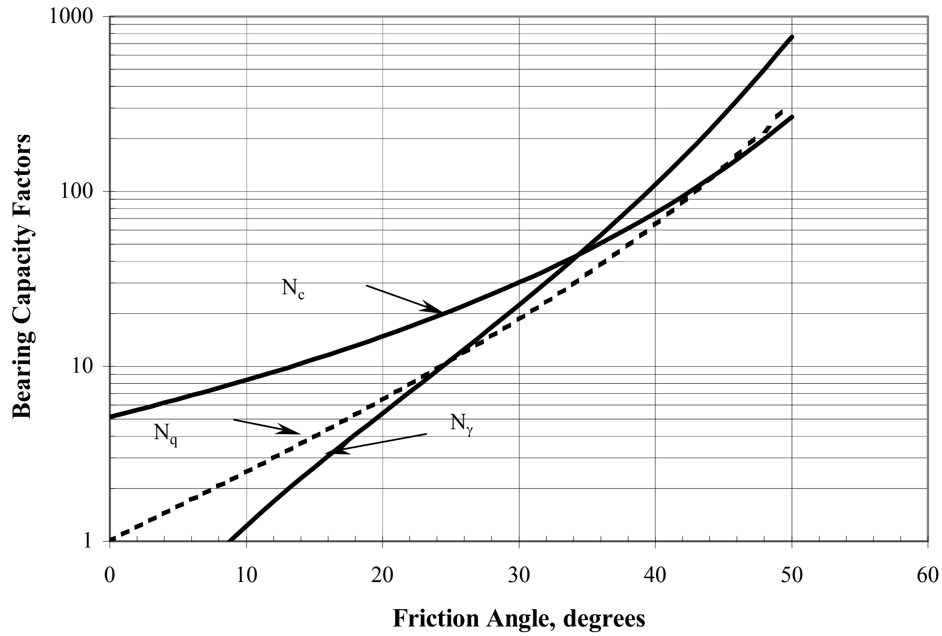


Figure 5-6: Bearing Capacity Factors versus Friction Angle (AASHTO, 1996)

5.2.3 Bearing Capacity Correction Factors

A number of factors influence the ultimate bearing capacity of shallow foundations that were not included in the derivations discussed earlier. Note that Equation 5-7 assumes a rigid strip (continuous) footing that is loaded through its centroid bearing on a level surface of homogeneous soil. Various correction factors have been proposed by numerous investigators to account for footing shape, location of the ground water table, embedment depth, sloping ground surface, an inclined base, the mode of shear (local or punching shear), inclined loading and soil compressibility. The general philosophy of correcting the ultimate bearing capacity involves using semi-empirical factors that are multiplied by each of the three terms in the bearing capacity equation to account for the particular effect. Each correction factor includes a subscript denoting the term to which the factor should be applied: "c" for the cohesion term, "γ" for the unit weight (footing width) term and "q" for the surcharge (embedment) term. Each of these factors and suggestions for their application are discussed separately below. In some, but not all, cases these factors should be used in combination. The following sections provide guidance on the use of the factors, and whether or not certain factors should be used in combination.

5.2.3.1 Footing Shape

AASHTO guidelines distinguish a strip footing as one whose length (L_f) exceeds five times the footing width (B_f). Corrections of the ultimate bearing capacity should be made for rectangular footings with L_f/B_f ratios less than 5. The shape (eccentricity) correction factors are summarized in Table 5-2. Note that for eccentrically loaded footings, AASHTO (1996) recommends using the effective footing dimensions, B'_f and L'_f, to compute the shape correction factors (refer to Section 6.4.1 for discussion of eccentricity and effective footing dimensions). However, in routine foundation design, this is not practical, since the effective dimensions will change for various load cases, and the difference in the computed shape correction factors will generally be small. Therefore the geotechnical engineer should make reasonable assumptions about the footing shape and dimensions and compute the correction factors using Table 5-2.

TABLE 5-2: SHAPE CORRECTION FACTORS (AASHTO 1996)

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors, s_c, s_γ, s_q	$\phi = 0$	$1+\left(\frac{B_f}{5L_f}\right)$	1.0	1.0
	$\phi > 0$	$1+\left(\frac{B_f}{L_f}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B_f}{L_f}\right)$	$1+\left(\frac{B_f}{L_f}\tan\phi\right)$

Note: Shape (eccentricity) factors, s, should not be applied simultaneously with inclined loading factors, i. See Section 5.2.3.7.

5.2.3.2 Location of the Ground Water Table

If the ground water table is located within the potential zone of shear below or above a footing, effective stresses will control the magnitude of overburden stress and thus the shearing resistance along that portion of the shearing surface that lies below the water table.

A simplified method for accounting for the reduction in shearing resistance is to apply factors to the two terms in the bearing capacity equation that include a unit weight term. Recall that the cohesion term is neither a function of soil unit weight nor effective stress. The factors may be computed using the following equations, or by interpolating values between those provided in Table 5-3.

$$C_{W_\gamma} = 0.5 + 0.5\left(\frac{D_W}{1.5B_f + D_f}\right) \leq 1.0 \qquad (5\text{-}9)$$

$$C_{W_q} = 0.5 + 0.5\left(\frac{D_W}{D_f}\right) \leq 1.0 \qquad (5\text{-}10)$$

TABLE 5-3: CORRECTION FACTOR FOR LOCATION OF GROUND WATER TABLE (AASHTO, 1998)

Depth of Ground Water Table, D_W	$C_{W\gamma}$	C_{Wq}
0	0.5	0.5
D_f	0.5	1.0
$> 1.5B_f + D_f$	1.0	1.0

Note: For intermediate positions of the ground water table, interpolate between the values shown above or use Equations 5-9 and 5-10.

5.2.3.3 Embedment Depth

Because the surcharge term, q, was developed considering the effect on bearing capacity as a vertical pressure applied at the footing bearing elevation, the effect of the shearing resistance as the failure surface passes through the footing embedment cover was neglected in the theory. If the backfill or cover over the footing is known to be of high-quality, compacted granular material that can be reliably assumed to remain in place over the life of the footing, this additional shearing resistance can be included by addition of the embedment depth factor, d_q, shown in Table 5-4. Otherwise, the depth correction factor can be conservatively omitted.

TABLE 5-4: DEPTH CORRECTION FACTOR (BRINCH HANSEN, 1970)

Friction Angle, ϕ (degrees)	D_f/B_f	d_q
32	1 2 4 8	1.20 1.30 1.35 1.40
37	1 2 4 8	1.20 1.25 1.30 1.35
42	1 2 4 8	1.15 1.20 1.25 1.30

Note: The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0.

5.2.3.4 Sloping Ground Surface

Placement of footings on or adjacent to slopes requires that the designer perform calculations to ensure that both the bearing capacity and the overall slope stability are acceptable. The bearing capacity equation should include corrections recommended by AASHTO as adapted from NAVFAC (1986b) to design the footings. Calculation of overall, or global, stability is discussed in Section 5.5.

The ultimate bearing capacity equation (Equation 5-7) is therefore modified to include terms N_{cq} and $N_{\gamma q}$ that replace the N_c and N_γ terms (Equation 5-11).

$$q_{ult} = c\,(N_{cq}) + 0.5\,(\gamma)(B_f)(N_{\gamma q}) \qquad (5\text{-}11)$$

Charts are provided (Figure 5-7) to determine these terms for cohesive ($\phi = 0^o$) and cohesionless ($c = 0$) soils. Note in Figure 5-7d that beyond a distance, 'b,' of two to six times the foundation width, the bearing capacity is independent of the slope angle (identical to the horizontal ground surface). Also note that the depth of embedment term goes to zero because the surcharge effect on the slope side of the footing should be ignored.

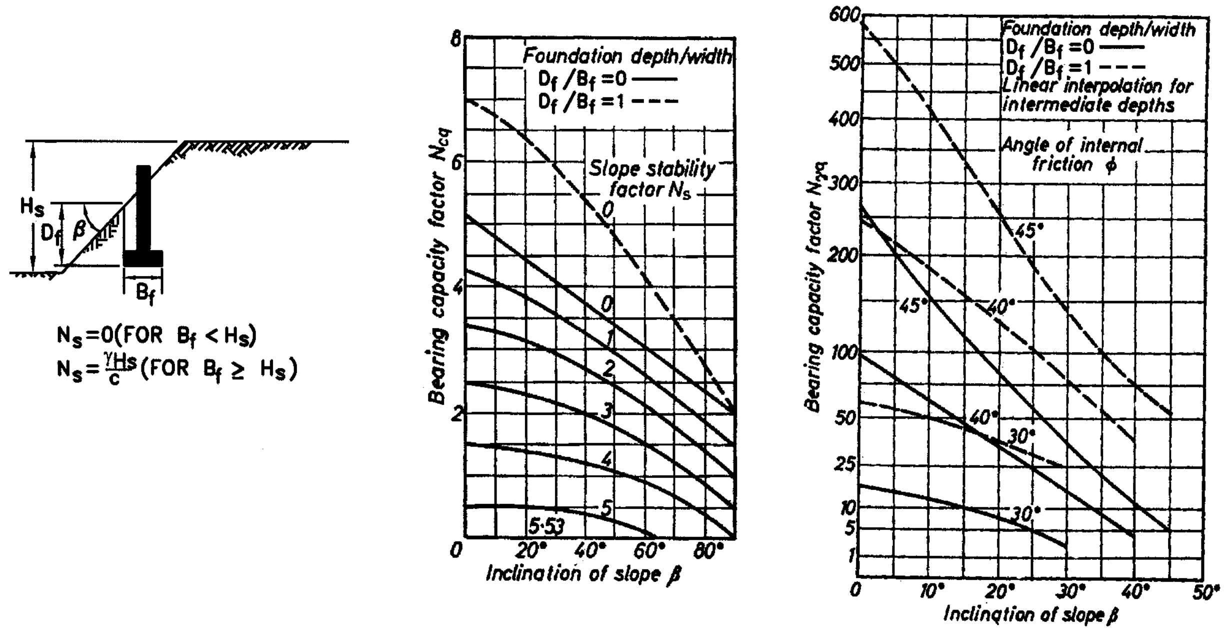
Other forms of the equation are available for cohesive soils ($\phi = 0^o$). However, because footings located on both cohesive soils and near slopes are likely to have design limitations due to either settlement or slope stability, or both, the presentation of these equations is omitted here. The reader is referred to the *NHI Shallow Foundations Workshop Reference Manual* (Munfahk et al., 2000) or the NAVFAC DM-7 (1986a, 1986b) manuals for discussion of these equations and their applications and limitations.

The form of the equation that includes the width term on cohesionless soils is useful in designing footings constructed within bridge approach fills. The calculation procedure is as follows:

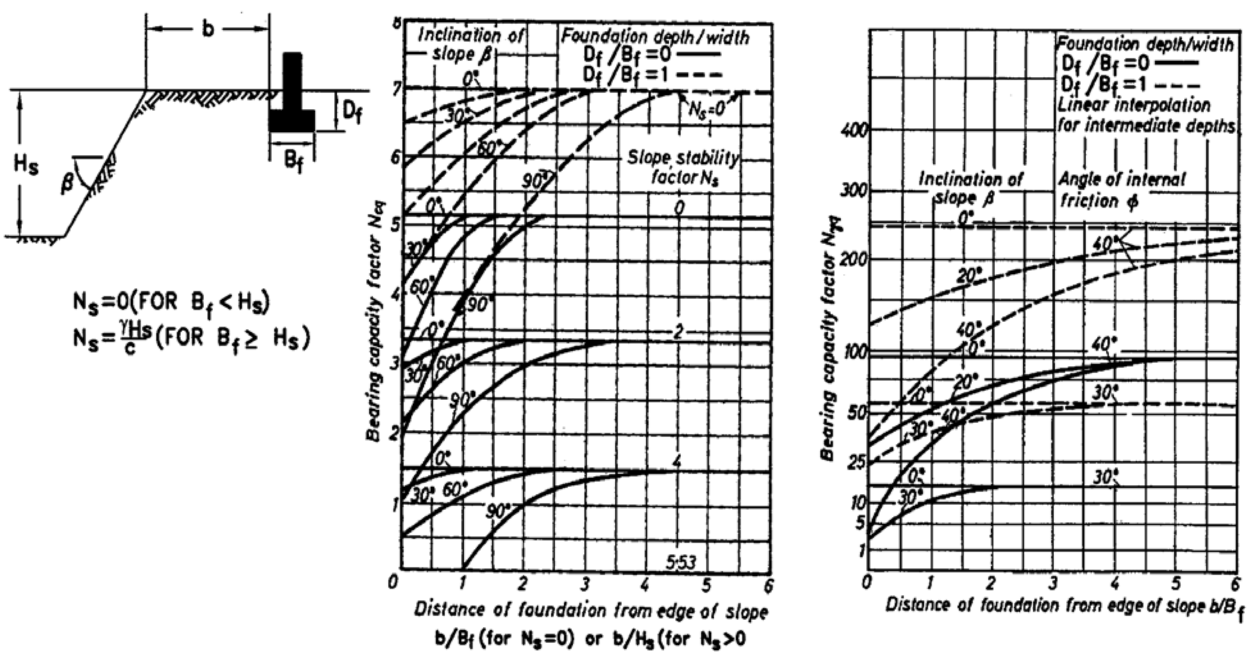
- Obtain $N_{\gamma q}$ from Figure 5-7(c) or 5-7(f) and compute the ultimate bearing capacity using Equation 5-11, or the general form of the bearing capacity expression, Equation 5-14, with the N_γ term replaced by $N_{\gamma q}$.

5.2.3.5 Inclined Base

In general, inclined footings for bridges should be avoided, or limited to shallow inclinations, α, of less than about 8 to 10 degrees from horizontal. Steeper inclinations may require keys, dowels or anchors to provide sufficient resistance to sliding. If the footing is inclined to the horizontal, the bearing capacity equation should be modified using the factors as determined from Table 5-5.



(a) Geometry (b) Cohesive Soil (φ=0) (c) Cohesionless Soil (c=0)



(d) Geometry (e) Cohesive Soil (φ=0) (f) Cohesionless Soil (c=0)

Figure 5-7: Modified Bearing Capacity Factors for Footing on Sloping Ground, (after Meyerhof, 1957, from AASHTO, 1996)

TABLE 5-5: INCLINED BASE CORRECTION FACTOR (BRINCH HANSEN, 1970)

Factor	Friction Angle	Cohesion Term (c)	Unit Weight Term (γ)	Surcharge Term (q)
Base Inclination Factors, b_c, b_γ, b_q		b_c	b_γ	b_q
	$\phi = 0$	$1-\left(\frac{\alpha}{147.3}\right)$	1.0	1.0
	$\phi > 0$	$b_q-\left(\frac{1-b_q}{N_c \tan\phi}\right)$	$(1\text{-}0.017\alpha \tan\phi)^2$	$(1\text{-}0.017\alpha \tan\phi)^2$

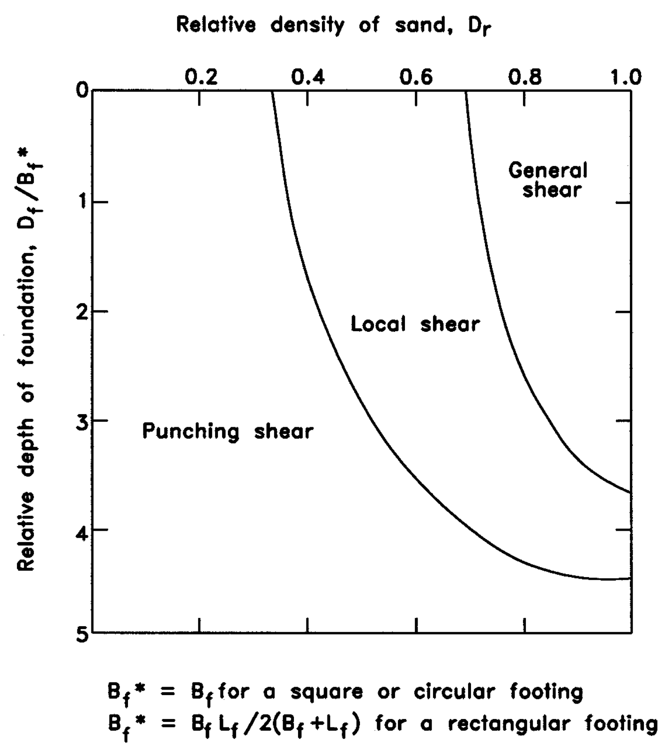
Definitions: ϕ = friction angle, degrees; α = footing inclination from horizontal, upward +, degrees

5.2.3.6 Local or Punching Shear

Several references, including AASHTO, recommend reducing the soil strength parameters if local or punching shear failure modes can develop. Figure 5-8 shows conditions when these modes can develop. The recommended reductions are shown in Equations 5-12 and 5-13.

$$c^* = 0.67c \quad (5\text{-}12)$$

$$\phi^* = \tan^{-1}(0.67\tan\ \phi) \quad (5\text{-}13)$$



Note: This chart was developed for normally consolidated native soil deposits.

Figure 5-8: Modes of Failure of Model Footings in Sand (after Vesic, 1975)

Soil types that can develop local or punching shear failure modes include loose sands, very sensitive clays [$S_t > 8$, sensitivity is defined as the ratio of the unconfined compressive strengths of an undisturbed sample to the unconfined compressive strength of a remolded sample (Terzaghi & Peck, 1967)], metastable (collapsible, see Section 3.1.1) sands and silts and brittle clays (OCR > 4 to 8; see Section 5.3.3 for discussion of OCR). These are all "problem" soil conditions that should be identified with a thorough geotechnical investigation, as described in Chapter 4. In general, these problem soils will have other characteristics that make them unsuitable for support of shallow foundations for bridges, including large settlement potential for loose sands, sensitive clays and metastable soils. Brittle clays exhibit relatively high strength at small strains, but significant reduction in strength at larger strains (strain-softening). This behavior should be identified and quantified through the field and laboratory testing program and compared to the anticipated stress changes resulting from the shallow foundation and ground slope configuration under consideration.

Although local or punching shear failure modes can develop in loose sands or when very narrow footings are used, this local condition seldom applies to bridge foundations because spread footings are not used on obviously weak soils and relatively large footing sizes are needed for structural stability (Cheney & Chassie, 2000).

The geotechnical engineer may encounter the following two situations where application of the one-third reduction according to Equation 5-13 can result in an unnecessarily over-conservative design.

The first is when considering a footing bearing on cohesionless soils that fall in the local shear portion of Figure 5-8. Note that a one-third reduction in the tangent of a friction angle of 38 degrees (a common value for good-quality, compacted, granular fill) results in a 73 percent reduction in bearing capacity factor N_q, and an 81 percent reduction in N_γ! Also note that Figure 5-8 does not consider the effect of large footing widths, such as used for support of bridges. **Therefore, provided that settlement potential is checked independently (Section 5.3.4) and found to be acceptable, spread footings on normally consolidated cohesionless soils falling within the local shear portion Figure 5-8 should <u>not</u> be designed using the one-third reduction according to Equation 5-13.**

The second situation is when considering a spread footing bearing on a compacted structural fill. Note that Figure 5-8 is for normally consolidated, natural soil deposits. The relative density of compacted structural fills as compared to compactive effort (percent relative compaction) indicates that for fills compacted to a minimum of 95 percent of maximum density (as determined by AASHTO T-180) the relative density should be at or above 75 percent (Lee & Singh, 1971). This is consistent with the excellent performance history of spread footings in structural fills (DiMillio, 1982). **The use of the one-third reduction therefore should <u>not</u> be used in the design of footings on compacted structural fills constructed with good quality, granular material**. Appendix A includes materials specifications used by State highway agencies that routinely construct bridge abutments and pier footings supported within compacted structural fills.

5.2.3.7 Inclined Loading

The inclined load case is the resultant formed by both axial and shear load components applied to the footing by the column or wall stem. If the components of this resultant (i.e., axial and shear forces) are checked against the available resistance in the respective direction (i.e., bearing capacity and sliding, respectively), the inclusion of the inclined loading factor in the bearing capacity equation can generally be omitted. The bearing capacity should, however, be evaluated using effective footing dimensions, as discussed in Section 6.4.1 and in the footnote to Table 5-2, since large moments can frequently be transmitted to bridge foundations by the columns or pier walls. **The simultaneous application of shape and load inclination factors can result in an overly conservative design.** Henceforth, the inclination factor will be omitted in calculation of bearing capacity. A check for sliding stability should be made for all footings with inclined loads.

Unusual column geometry or loading configurations should be evaluated on a case-by-case basis relative to the foregoing recommendation to omit the load inclination factors. An example might be a strut column that is not aligned normal to the footing bearing surface. In this case, it may be prudent to consider an inclined footing base for improved bearing efficiency. Keep in mind that bearing surfaces that are not level may be difficult to both construct and inspect.

5.2.3.8 Soil Compressibility

Vesic (1975) included correction factors for soil compressibility for footings on sands and gravels, c_q and c_γ. These factors were developed to effectively limit the allowable bearing capacity accounting for compression settlement of granular materials. The factors are a function of soil density and applied load, and the factors do not account for soil layering or the reduction in stress with depth below a footing (see Section 5.3). The procedures recommended in this chapter directly compute the estimated compression settlement of footings on sands and granular materials. **Therefore, the soil compressibility factors should <u>not</u> be used in calculation of ultimate bearing capacity of footings on granular soils when a separate settlement calculation is performed**.

5.2.4 Additional Considerations Regarding Bearing Capacity Correction Factors

The inherent or implied factor of safety of a settlement-limited allowable bearing capacity relative to the computed ultimate bearing capacity is usually large enough to render the magnitude of the application of the individual correction factors small. Consider the following observations:

- AASHTO (1996) guidelines recommend calculating the shape, s, factor using the effective footing dimensions, B'_f and L'_f. However, the original references (e.g., Vesic, 1975) do not specifically recommend using the effective dimensions to calculate the shape factor. Since the geotechnical engineer typically does not have knowledge of the loads causing eccentricity, it is recommended that the full footing dimensions be used to calculate the shape factor (Table 5-2) for use in computation of ultimate bearing capacity.

- Bowles (1982) also recommends that the shape and load inclination factors (s and i) should not be combined.

- In certain loading configurations, the simultaneous application of the shape factor correction, the load inclination factor correction, local shear potential, the soil compressibility factors and a suitable factor of safety can result in a computed allowable bearing capacity that is only *5 percent* or less of the uncorrected ultimate bearing capacity. This emphasizes the over-conservatism resulting from the simultaneous application of all the factors that have been proposed by various researchers.

The form of the bearing capacity equation in Equation 5-14 is therefore recommended for routine design applications.

Further, the bearing capacity correction factors were developed assuming that each correction for each of the terms involving N_c, N_γ and N_q can be found independently. The bearing capacity theory is an idealization of the response of a foundation that attempts to account for the soil properties and boundary conditions. Bearing capacity analysis of foundations is frequently limited by the geotechnical engineer's ability to accurately determine material properties as opposed to inadequacies in the theory used to develop the bearing capacity equations. Consider Table 5-1 and note that a one degree change in friction angle can result in a 10 to 15 percent change in the factors N_c, N_γ and N_q. Also note that the value of N_γ more than doubles when the friction angle increases from 35^o to 40^o. Clearly, the uncertainties in the material properties will control the uncertainty of a bearing capacity computation to a large extent. **The importance of application of the correction factors is therefore secondary to adequate assessment through field exploration and laboratory testing of the inherent strength characteristics of the foundation soils**.

It should also be noted that very few spread footings of the size used for bridge support have been load tested to failure. The evaluation of bearing capacity is based primarily on theory and laboratory testing of small-scale footings, with modification of the theoretical based on observation.

5.2.5 Bearing Capacity Calculation

The general form of the resulting ultimate bearing capacity equation, including correction terms, is thus:

$$q_{ult} = cN_c s_c b_c + qN_q C_{W_q} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{W_\gamma} s_\gamma b_\gamma \qquad (5\text{-}14)$$

where: s_c, s_γ and s_q are shape correction factors

b_c, b_γ and b_q are correction factors for the inclination of the base

d_q is a correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation (recall that the embedment is modeled as a surcharge pressure only, applied at the bearing elevation)

$C_{W\gamma}$ and C_{Wq} are correction factors considering the location of the ground water table

N_c, N_γ and N_q are obtained from Table 5-1 or Figure 5-6

N_c and N_γ are replaced with N_{cq} and $N_{\gamma q}$ for the sloping ground or footing near slope case

The friction angle of the soil is needed to obtain N_c, N_q and N_γ from either Table 5-1 or Figure 5-6. Figures 4-1 and 4-2 can be used to correlate friction angles from SPT data in cohesionless soil, or measured directly by laboratory tests or in situ testing.

5.2.6 Bearing Capacity Factors of Safety

The minimum factor of safety applied to the calculated ultimate bearing capacity will be a function of:

- The confidence in the design soil strength parameters (c and ϕ),
- The importance of the structure, and
- The consequence of failure.

Typical minimum factors of safety for shallow foundations are in the range of 2.5 to 3.5. For most bridge foundation applications, a minimum factor of safety against bearing capacity failure of 3.0 is recommended (Cheney & Chassie, 2000).

This factor of safety was selected through a combination of applied theory, experience and consensus for comparison to unfactored load combinations [allowable stress, or Service Load Design (SLD)]. As such, the uncertainty in both the magnitude of the loads and the available soil bearing strength were combined into this single factor of safety. The general factor of safety equation in allowable stress, or SLD, is as follows:

$$\sum Q_i \leq \frac{R_n}{FS} \qquad (5\text{-}15)$$

where: Q_i = applied stress (by the spread footing)
R_n = yield stress of the member (or ultimate bearing capacity of the soil)
FS = the applied factor of safety

Load Factor Design (LFD) was first recommended by AASHTO in the late 1970s. At that time, uncertainty in the loads was captured in the load factors. However, it was not until the implementation of the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO, 1994) that an attempt was made to incorporate the uncertainty of the foundation soil resistances in performance, or resistance (φ), factors.

A performance or resistance factor is not strictly the reciprocal of the factor of safety. It is more properly a partial factor of safety that incorporates the potential for variation in the computed capacity or resistance. **It must be emphasized that performance (resistance) factors for geotechnical design of shallow foundations were developed using a combination of reliability-based calibrations, calibration with SLD (fitting) and engineering judgment.** The performance (LFD – AASHTO, 1996) and resistance (LRFD – AASHTO, 1998) factors were therefore selected to match existing practice and to mimic the factors of safety used in allowable stress design. The general LFD equation is:

$$\gamma \sum \beta_i Q_i \leq \varphi R_n \qquad (5\text{-}16)$$

where: γ = load factor for load combination,

β_i = load factor for particular load, Q_i

Q_i = particular service load or stress

φ = resistance factor

R_n = nominal strength of member (e.g., yield strength for steel)

In the years between adoption of LFD for calculating factored loads and the development of resistance factors for soils, agencies that elected to implement LFD found it necessary to make interim provisions to increase the allowable bearing capacities for spread footings provided by the geotechnical engineers. If this was not done, they found that their LFD designs were noticeably more conservative than they had been using SLD (allowable stress) design.

One approach used to reduce this added conservatism was to multiply the allowable bearing capacities by some factor, say 1.5 to 2. Although this procedure may result in foundation designs that are generally similar to those developed using allowable stress methods, the approach should generally be avoided, because the settlement computations used to develop the allowable bearing capacity were based on unfactored loads, not factored (increased) loads.

It was not until the adoption of the Load and Resistance Factor Design method (AASHTO, 1994) that these inconsistencies were at least partially resolved. Appendix C contains two of the example problems presented in Appendix B that are solved using the LRFD method. Notice that the designs are not radically different. This is to be expected, since the LRFD resistance factors in AASHTO were selected such that designs would be similar to past practice.

Clearly, the structural designer and the geotechnical engineer must be consistent in communication of their design criteria and methodologies. A mixed approach of SLD for foundation capacities and LFD or LRFD for load calculation can result in very conservative designs. **The importance of good communication between the structural and geotechnical engineers has never been more important. Miscommunication about the SLD factors of safety used to develop allowable bearing capacities can result in overconservative designs when LFD or LRFD load factors are used to increase the design loads that are then used to size a foundation.** This is especially true for AASHTO SLD Load Groups VII through XI (AASHTO, 1996, see Section 6.3 for a discussion of load groups and their definitions), which are the rare, or extreme event, loading conditions resulting from earthquake, ice loading and ice plus wind, respectively. If the allowable bearing capacity provided by the geotechnical engineer is used to size a shallow foundation when one of these load conditions controls, the designer will frequently conclude that a shallow foundation must be enormous and therefore impractical. Expensive deep foundations such as piles or shafts are often unnecessarily selected when this miscommunication occurs.

5.2.7 Presumptive Bearing Capacities

5.2.7.1 Soil

The use of presumptive bearing capacities for footings bearing in <u>soils</u> is <u>not</u> recommended for final design of shallow foundations for transportation structures, especially bridges. Guesses about the geology and nature of a site and application of a presumptive value from generalizations in the literature are not a substitute for an adequate site-specific subsurface exploration and laboratory testing program. An exception is preliminary evaluation of shallow foundation feasibility and estimation of footing dimensions for preliminary constructability or cost evaluation.

Local sources of suggested presumptive values, available in engineering geology or geotechnical literature, are generally more realistic than capacities generalized to soil classification or found in design codes such as AASHTO (1996, 1998) or the NAVFAC manuals (NAVFAC, 1986a and 1986b).

5.2.7.2 Rock

For conditions of intact sound rock that is stronger and less compressible than concrete, footings on rock with small applied loads are generally stable and thus do not require extensive study of the strength and compressibility characteristics of rock behavior. However, site investigation is still required to confirm the consistency and extent of rock formations beneath a shallow foundation.

Allowable bearing capacities for footings on relatively uniform and sound rock surfaces are documented in applicable building codes and engineering manuals. Many different definitions for sound rock are available. In simple terms, however, “sound rock” can generally be defined as rock mass that does not disintegrate after exposure to air or water, and with discontinuities that are unweathered, closed or tight (less than about 3 mm wide) and spaced no closer than 1 m apart. Table 5-6 presents allowable bearing pressures recommended in selected local building codes (Goodman, 1989). These values were developed based on experience in sound rock formations, with an intention of satisfying both bearing capacity and settlement criteria in order to provide a satisfactory factor of safety. In most cases, it is prudent to follow the codes since these code-recommended values tend to be more conservative based on local experience. However, for supporting heavy structures, use of presumptive values may lead to overly conservative and costly foundations. In such cases, most codes do allow for variance if the request is supported by an engineering report.

In areas where building codes are not available or applicable, other recommended presumptive bearing values, such as those listed in Table 5-7, may be used to determine the allowable bearing pressure for sound rock. For footings designed using these published values, the elastic settlements are generally less than 13 mm. Where rock is reasonably sound, but fractured, the presumptive values listed in Tables 5-6 and 5-7 should be reduced to prevent potential excessive settlement. Most building codes also provide reduced recommended bearing pressures to account for the degree of fracturing. For example, Table 5-8 presents the allowable rock bearing pressures in accordance with the New York City Building Code (2001).

TABLE 5-6: ALLOWABLE BEARING PRESSURES FOR FRESH ROCK OF VARIOUS TYPES[1,2,3,4] (GOODMAN, 1989)

Rock Type	Age	Location	Allowable Bearing Pressure (MPa)
Massively bedded limestone[5]		U.K.[6]	3.8
Dolomite	L. Paleoz.	Chicago	4.8
Dolomite	L. Paleoz.	Detroit	1.0 – 9.6
Limestone	U. Paleoz.	Kansas City	0.5 – 5.8
Limestone	U. Paleoz.	St. Louis	2.4 – 4.8
Mica schist	Pre-Camb.	Washington	0.5 – 1.9
Mica schist	Pre-Camb.	Philadelphia	2.9 – 3.8
Manhattan schist	Pre-Camb.	New York	5.8
Fordham gneisse	Pre-Camb.	New York	5.8
Schist and slate	-	U.K.[6]	0.5 – 1.2
Argillite	Pre-Camb.	Cambridge, MA	0.5 – 1.2
Newark shale	Triassic	Philadelphia	0.5 – 1.2
Hard, cemented shale	-	U.K.[6]	1.9
Eagleford shale	Cretaceous	Dallas	0.6 – 1.9
Clay shale	-	U.K.[6]	1.0
Pierre shale	Cretaceous	Denver	1.0 – 2.9
Fox Hills sandstone	Tertiary	Denver	1.0 – 2.9
Solid chalk	Cretaceous	U.K.[6]	0.6
Austin chalk	Cretaceous	Dallas	1.4 – 4.8
Friable sandstone and claystone	Tertiary	Oakland	0.4 – 1.0
Friable sandstone (Pico formation)	Quaternary	Los Angeles	0.5 – 1.0

Notes:
1. According to typical building codes; reduce values accordingly to account for weathering or unrepresentative fracturing
2. Values from Thornburn (1966) and Woodward, Gardner and Greer (1972).
3. When a range is given, it relates to usual range in rock conditions.
4. Sound rock that rings when struck and does not disintegrate. Cracks are unweathered and open less than 10 mm.
5. Thickness of beds greater than 1 m, joint spacing greater than 2 mm; unconfined compressive strength greater than 7.7 MPa (for a 100-mm cube).
6. Institution of Civil Engineers Code of Practice 4.

TABLE 5-7: PRESUMPTIVE VALUES OF ALLOWABLE BEARING PRESSURES FOR SPREAD FOUNDATIONS ON ROCK (MODIFIED AFTER NAVFAC, 1986b)

Type of Bearing Material	Consistency In Place	Allowable Bearing Pressure (MPa)	
		Range	Recommended Value for Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Hard, sound rock	5.8 to 9.6	7.7
Foliated metamorphic rock: Slate, schist (sound condition allows minor cracks)	Medium-hard, sound rock	2.9 to 3.8	3.4
Sedimentary rock; hard cemented shales, siltstone, sandstone, limestone without cavities	Medium-hard, sound rock	1.4 to 2.4	1.9
Weathered or broken bedrock of any kind except highly argillaceous rock (shale). RQD less than 25	Soft rock	0.8 to 1.2	1
Compaction shale or other highly argillaceous rock in sound condition	Soft rock	0.8 to 1.2	1

Notes:

1. For preliminary analysis or in the absence of strength tests, design and proportion shallow foundations to distribute their loads using presumptive values of allowable bearing pressure given in this table. Modify the nominal value of allowable bearing pressure for special conditions described in notes 2 through 8.
2. The maximum bearing pressure beneath the footing produced by eccentric loads that include dead plus normal live load plus permanent lateral loads shall not exceed the above nominal bearing pressure.
3. Bearing pressures up to one-third in excess of the nominal bearing values are permitted for transient live load from wind or earthquake. If overload from wind or earthquake exceeds one-third of nominal bearing pressures, increase allowable bearing pressures by one-third of nominal value.
4. Extend footings on soft rock to a minimum depth of 40 mm below adjacent ground surface or surface of adjacent floor, whichever elevation is the lowest.
5. For footings on soft rock, increase allowable bearing pressures by 5 percent of the nominal values for each 0.3 m of depth below the minimum depth specified in note 4.
6. Apply the nominal bearing pressures of the three categories of hard or medium hard rock shown above where the base of the foundation lies on rock surface. Where the foundation extends below the rock surface, increase the allowable bearing pressure by 10 percent of the nominal values for each additional 0.3 m of depth extending below the surface.
7. For footings smaller than 1 m in the least lateral dimension, the allowable bearing pressure shall be the nominal bearing pressure multiplied by the least lateral dimension (in meters).
8. If the above-recommended nominal bearing pressure exceeds the unconfined compressive strength of intact specimen, the allowable pressure equals the unconfined compressive strength.

TABLE 5-8: ALLOWABLE BEARING PRESSURES FOR SPREAD FOUNDATIONS ON ROCK IN NEW YORK CITY (NEW YORK CITY BUILDING CODE, 2001)

Description	Classifications	$q_{allowable}$ (MPa)
Hard sound rock	Crystalline rocks such as Fordham Gneiss, Ravenswood Gneiss, Palisades Diabase and Manhattan Schist. Characteristics: the rock rings when struck with pick or bar; does not disintegrate after exposure to air or water; breaks with sharp fresh fracture; cracks are unweathered and less than 3 mm (1/8 in) wide, generally no closer than 0.9 m (3 ft) apart; core recovery with a double tube, diamond core barrel is generally 85 percent or greater for each 1.5-m (5-ft) run	5.8
Medium sound rock	Includes the above crystalline rocks, plus Inwood marble and serpentine. Characteristics include above hard rock characteristics, except that cracks may be 6 mm (¼ inch) wide and slightly weathered, generally spaced no closer than 0.6 m (2 ft) apart; core recovery with a double tube, diamond core barrel is generally 50 percent or greater for each 1.5-m (5-ft) run	3.8
Intermediate sound rock	Includes the all of the above rock types plus cemented shale and sandstone of the Newark formation. Characteristics: the rock gives dull sound when struck with pick or bar; does not disintegrate after exposure to air or water; broken pieces may show weathered surfaces; may contain fracture and weathered zones up to 25-mm (1-in) wide spaced as close as 0.3 m (1 ft); core recovery with a double tube, diamond core barrel is generally 35 percent or greater for each 1.5-m (5-ft) run	1.9
Soft rock	Includes all of the above rocks in partially weathered condition, plus cemented shale and sandstones. Characteristics: rock may soften on exposure to air or water; may contain thoroughly weathered zones up to 75 mm (3 in) wide but filled with stiff soil; core recovery with a double tube, diamond core barrel is less than 35 percent for each 1.5-m (5-ft) run, but standard penetration resistance in soil sampling is more than 50 blows per 0.3 m (1 ft)	0.8

Peck et al. (1974) presented an empirical correlation of presumptive allowable bearing pressure with Rock Quality Designation (RQD), as shown in Table 5-9. AASHTO recommends using presumptive values proposed by Peck et al. for selecting the allowable bearing pressures; but 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 lower of the two values. Although the suggested bearing values of Peck et al. (1974) are substantially greater than most of the other published values and ignore the effects of rock types and conditions of discontinuities, they provide a useful guide for an upper-bound estimation as well as an empirical relationship between allowable bearing values and intensity of fracturing and jointing (Table 5-9). Note that with a slight increase of the degree of fracturing of the rock mass, when the RQD value drops from, say, 100 percent to 90 percent, the recommended bearing capacity value is reduced drastically from 29 to 19 MPa.

Rock Quality Designation (RQD) is defined as the total sum of pieces of rock core greater than 100 mm (4 in) in length. [Example: Rock core run sample is 1.5 m (60 inches). Summation of pieces of rock core greater than 100 mm (4 in) is 0.5 m (18 in). Therefore, the RQD = 0.5/1.5 = 0.33 or 33 percent.]

In no instance should the allowable bearing capacity exceed the allowable stress of the concrete. Furthermore, Peck et al. (1974) also suggested that the average RQD for the bearing rock within a depth of footing width (B_f) below the base of the footing should be used if the RQD values within the depth are relatively uniform. If rock within a depth of $0.5B_f$ is of poorer quality, the RQD of the poorer quality rock should be used to determine the allowable bearing capacity.

TABLE 5-9: SUGGESTED VALUES OF ALLOWABLE BEARING CAPACITY (PECK ET AL., 1974)

RQD (%)	Rock Mass Quality	Allowable Pressure (MPa)
100	Excellent	29
90	Good	19
75	Fair	12
50	Poor	6
25	Very Poor	3
0	Soil-like	1

Note: Rock Quality Designation (RQD) is defined as the total sum of pieces of rock core greater than 100 mm (4 in) in length. See example calculation in text.

5.2.8 Allowable Bearing Capacities on Soil

An acceptable and safe design can be achieved by first calculating the ultimate bearing capacity of a spread footing using the procedure in Section 5.2.5 and applying an adequate factor of safety (Section 5.2.6), then performing a settlement analysis to ensure that the foundation will function within performance limits under the applied stress (Sections 5.3.3, 5.3.4 and 5.3.5). If the settlement at the allowable bearing stress is excessive, the allowable bearing capacity must be reduced until the calculated settlement is within tolerable limits. Note that the soils within a zone about 1.5 times the footing width below the footing are the most significant when considering shear failure and settlement. It is therefore beneficial to obtain good data about the subsurface soils in this zone. SPT tests and soil sampling on 0.75-m (2.5-ft) spacing should be considered the minimum level of subsurface data obtained through this zone. Sampling intervals can be reduced to the usual 1.5 m (5-foot) spacing below this level. Borings still must be extended at least through the zone of influence for settlement considerations (two to four times the estimated least footing width), as described in Section 4.3.2.

Remember that allowable bearing capacity also includes consideration for limiting the potential for the footing to settle. The design of shallow foundations for bridges is frequently governed by settlement rather than shear failure considerations, due to the large footing widths. However, an adequate margin of safety should be maintained for all expected loading conditions relative to the potential for shear failure to develop below the footing.

5.2.9 Allowable Bearing Capacities on IGMs and Rock

The geotechnical engineer should use site investigation, in situ testing and lab testing to confirm material properties of IGMs and rock. Judgment must then be applied to the theoretically computed bearing capacities combined with local experience (presumptive bearing values) when selecting an allowable bearing capacity for design. To preserve stability, the geotechnical engineer must evaluate the potential for global stability or rock mass discontinuities to limit the allowable bearing capacity. Recall that the bearing capacity of some sedimentary rock can dramatically decrease when exposed to weathering and moisture.

5.2.10 Allowable Bearing Capacities on Compacted Structural Fills

Allowable bearing capacities for spread footings constructed on compacted structural fills will be a function of the quality of the fill (e.g., gradation, durability, drainage), the degree of compaction achieved in the field, and the strength and compressibility of the embankment foundation materials. For fill materials such as sands, the design allowable bearing capacity can be determined rationally through laboratory testing such as the direct shear test, conducted at the proper level of compaction and applied stress. The friction angle yielded from such a testing program can be used to directly calculate an allowable bearing capacity.

It is typically desirable and recommended that the compacted structural fills supporting spread footings be a select and specified material that includes sand- and gravel-sized particles. Direct shear testing of such materials requires large test equipment and is therefore not practical on a project-by-project basis. The design of spread footings on compacted sand and gravel materials is therefore a combination of experience and infrequent lab testing on specified gradations of select fill materials. Materials specifications are then developed based on the specified gradations that ensure good quality control during construction. This procedure helps ensure that the conclusions from the lab tests are valid for the construction practices used to place the fills.

Table 5-10 lists the presumptive allowable bearing capacities used by some state highway agencies for design of spread footings supported on compacted structural fills. The material and construction specifications associated with these bearing capacities are included in Appendix A.

TABLE 5-10: PRESUMPTIVE BEARING CAPACITY VALUES FOR FOOTINGS CONSTRUCTED ON COMPACTED STRUCTURAL FILLS

Agency	Allowable Bearing Capacity	Associated Anticipated Settlement	Fill Material Specified[1,2]
Washington State DOT	290 kPa (3 tsf)	< 40 mm (1½ in)	Gravel Borrow
Nevada DOT	190 kPa (2 tsf)	< 32 mm (1¼ in)	Type 1A Aggregate Base
Michigan DOT	170 kPa (1¾ tsf)	Not available	Granular Material Class III

[1] Specifications included in Appendix A.
[2] Specifications must state levels of control quality for fill material (e.g., gradation, durability), lift thickness and compactive effort.

5.2.11 Overstress Allowances for Bearing Stresses

Under Service Load Design (AASHTO, 1996), certain load groups (e.g., seismic) are permitted to exceed the allowable bearing stress of a member (in this case the foundation bearing material) by a specified percentage that ranges from 25 to 50 percent. These overstress allowances are permitted for short-duration, infrequently occurring loads and may also be applied to calculated allowable bearing capacities.

5.3 SETTLEMENT ESTIMATION

The two primary considerations that affect the selection and design of shallow foundations are bearing capacity (discussed in the previous section) and the settlement potential of ground within the zone of influence below the foundation. This section discusses the parameters and methods of analysis used to estimate the magnitude and time-rate of settlement for shallow foundations.

Total settlement of a shallow foundation will result from one, or more likely, a combination of the following:

a) Immediate Compression: This settlement may result from elastic compression of the material supporting the foundation, or from reduction in the pore space in nonsaturated soils due to expulsion of air from the void space. In cohesionless soils, nearly all the settlement that results from an increase in stress is associated with immediate, or elastic, compression.

b) Primary Consolidation: Consolidation settlement occurs when saturated, fine-grained soils experience an increase in stress. The water in the pore spaces initially carries the load. Then, as the water is expelled from the pore space, the soil experiences a reduction in volume and a decrease in the pore space between soil particles. The process can be slow to rapid and is strongly a function of the permeability of the soil in the direction of drainage. The process slows and eventually stops once all the excess pore water pressure induced by the stress increase is dissipated. Primary consolidation is principally a concern for fine-grained, or cohesive soils, since coarser grained soils are typically permeable enough that any volume change that occurs as a result of expulsion of water from void space occurs as rapidly as the loads are applied.

c) Secondary Compression: Some soils, after first experiencing primary consolidation settlement, continue to strain after excess pore-water pressures are dissipated. This process is termed secondary compression, or "creep." Organic soils and some inorganic silts are soil types that can exhibit both primary consolidation and secondary compression, or creep, settlement.

Initial compression accounts for the major portion of consolidation in granular soils. Primary compression accounts for the major portion of consolidation in cohesive soils. Primary and secondary compression both contribute significantly to consolidation settlement in organic soils.

Differential settlement occurs when one load-bearing member of a structure experiences total settlement of a different magnitude than an adjacent load-bearing member. Because differential settlement will introduce load and stress in the structure above the foundations, limiting differential settlement may frequently be of more interest in the design of a structure than total settlement. If total settlements are limited, of course, differential settlements will be limited to an even greater extent.

Transportation structures, especially bridges, are not exceptionally tolerant of differential settlement. Deformation limitations will therefore frequently form the upper bound of allowable soil bearing capacities used to design shallow foundations.

5.3.1 Serviceability Criteria

The tolerable movements a structure can withstand depend on the type of structure, the expected service life and the consequences of excessive movements. Tolerable movements are frequently described in terms of angular distortion between members. "Angular distortion" is defined as the ratio of the differential settlement (δ') between footings to the span length (l), or δ'/l. The maximum angular distortion should be limited to some acceptable value to minimize structural strength and serviceability problems. Excessive differential displacements can induce significant structural cracking.

Empirical studies of the performance of bridges subjected to various maximum angular distortions between adjacent foundations concluded that the serviceability criteria would be met if the angular distortion for simple span bridges is less than 0.005 and, for continuous span bridges, less than 0.004 (Moulton et al., 1982, 1985). Using these criteria, a continuous span bridge having a span length of 30 m (100 ft) can tolerate a differential settlement of about 130 mm (5 in) between supports. In practice, bridges are typically designed for tolerable settlements much smaller than this. It is not uncommon for continuous span structures to be designed for differential settlements between adjacent piers of about 25 mm (1 in) or less, and for simply supported spans of about 38 to 50 mm (1½ to 2 in).

DiMillio (1982) reports in the FHWA research report "Performance of Highway Abutments on Spread Footings" that moderate differential movements of 25 to 75 mm (1 to 3 in) caused very little distress in any of the structures studied. Of the 148 bridges with spread footings studied, the foundation movements observed were small, and the movements did not cause significant distress.

For large footings on non-uniform soil profiles, or footings subjected to large eccentric loads, footing rotation may be a concern. In these cases the magnitude of the rotation can be computed by considering the potential differential settlement due to the difference in the applied loads or the non-uniformity of the soil profile.

It should be noted that tolerable differential movement between adjacent columns within a single pier or bent may be significantly smaller than between adjacent piers on the same structure due to the rigidity and relatively short length of the superstructure cross beam. The geotechnical and structural engineers should confer when there is potential for adjacent individual spread footings supporting columns within the same pier to experience differential settlements.

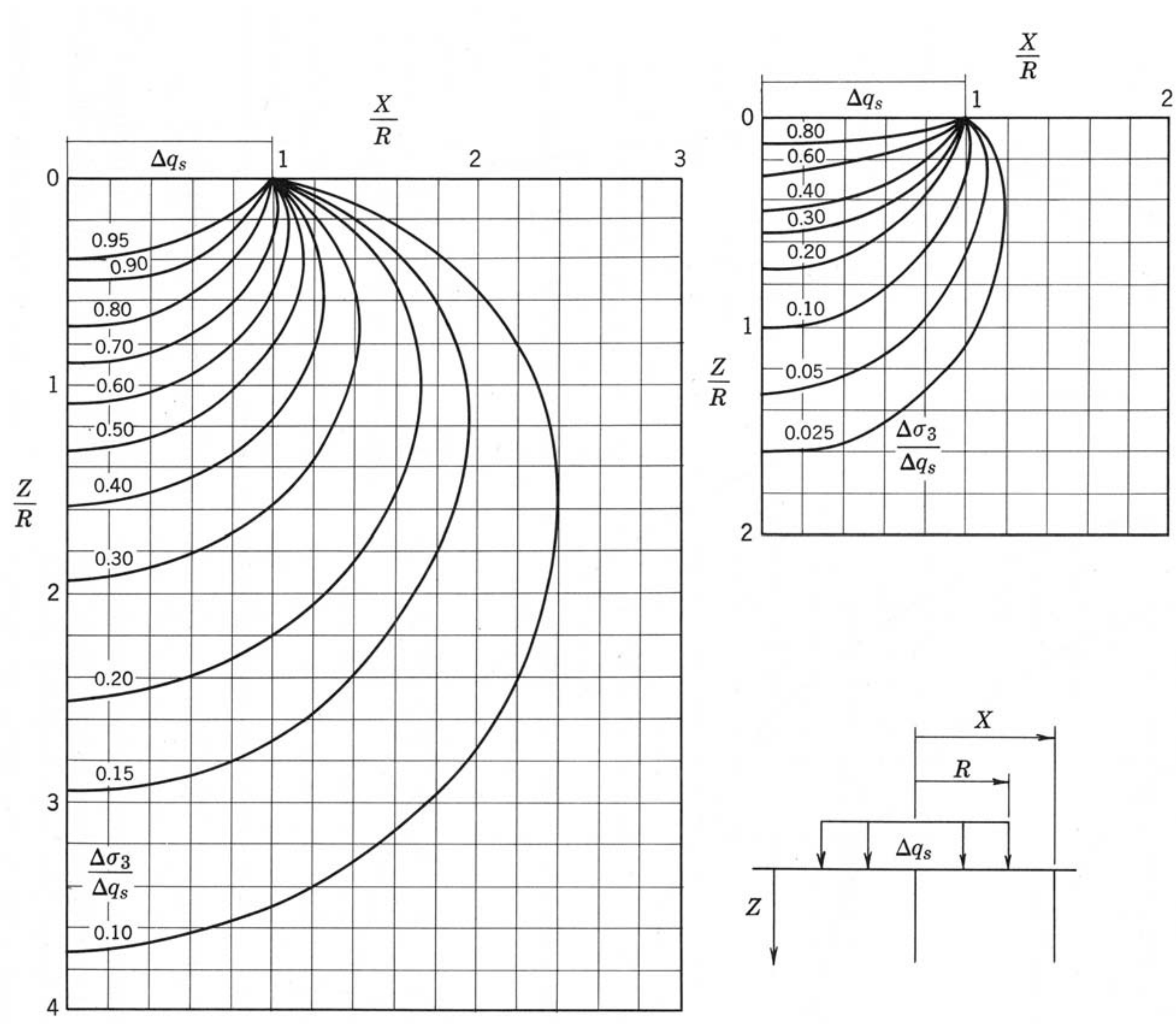
In some situations, the total movements may be the criteria that are critical in ensuring that serviceability issues are met. An example might be a simply supported, single-span structure with predicted settlements of several tenths of a meter at both abutments. While the structure itself is not at structural or performance risk due to the total, but relatively uniform movements, the resulting sag in the roadway profile may be unacceptable for vehicle ride, safety or aesthetic issues.

5.3.2 Vertical Stress Distribution

This section discusses methods to determine changes in total vertical stress ($\Delta\sigma_v$) due to foundation loading from a spread footing. The following methods for calculating total vertical stress change assume uniformly loaded foundations. Other references (Poulos & Davis, 1974) may be consulted for other load distributions.

Methods based on the theory of elasticity are the easiest and most commonly used methods to determine the changes in total stress due to applied foundation pressures. Most closed-form solutions based on the theory of elasticity involve the assumptions that the soil is homogeneous, isotropic and elastic. Lambe and Whitman (1969) report that the stresses predicted under these assumptions agree remarkably well with the relatively few comparisons that have been made to actual measured stresses, but the authors go on to point out that the computed stresses may be in error by as much as 25 percent or more. The charts in Figures 5-9 to 5-11 can be used to estimate the stress increases beneath circular, strip and rectangular loaded areas. The loadings are assumed to be uniform, an assumption that has been found to yield reasonable results for settlement prediction purposes.

Note that the principle of superposition may be used with Figure 5-11 to evaluate stresses resulting from irregular or multiple loaded areas. The FHWA EMBANK program (Urzua, 1993) uses these procedures to compute stress increases beneath embankments, which can be used for settlement analysis of approach embankments.



Note: In the above Figures, $\Delta\sigma_v$ and $\Delta\sigma_h$ are analogous to, may be substituted for, $\Delta\sigma_1$ and $\Delta\sigma_3$, respectively.

Figure 5-9: Stresses Under Uniform Load on Circular Area (Lambe & Whitman, 1969)

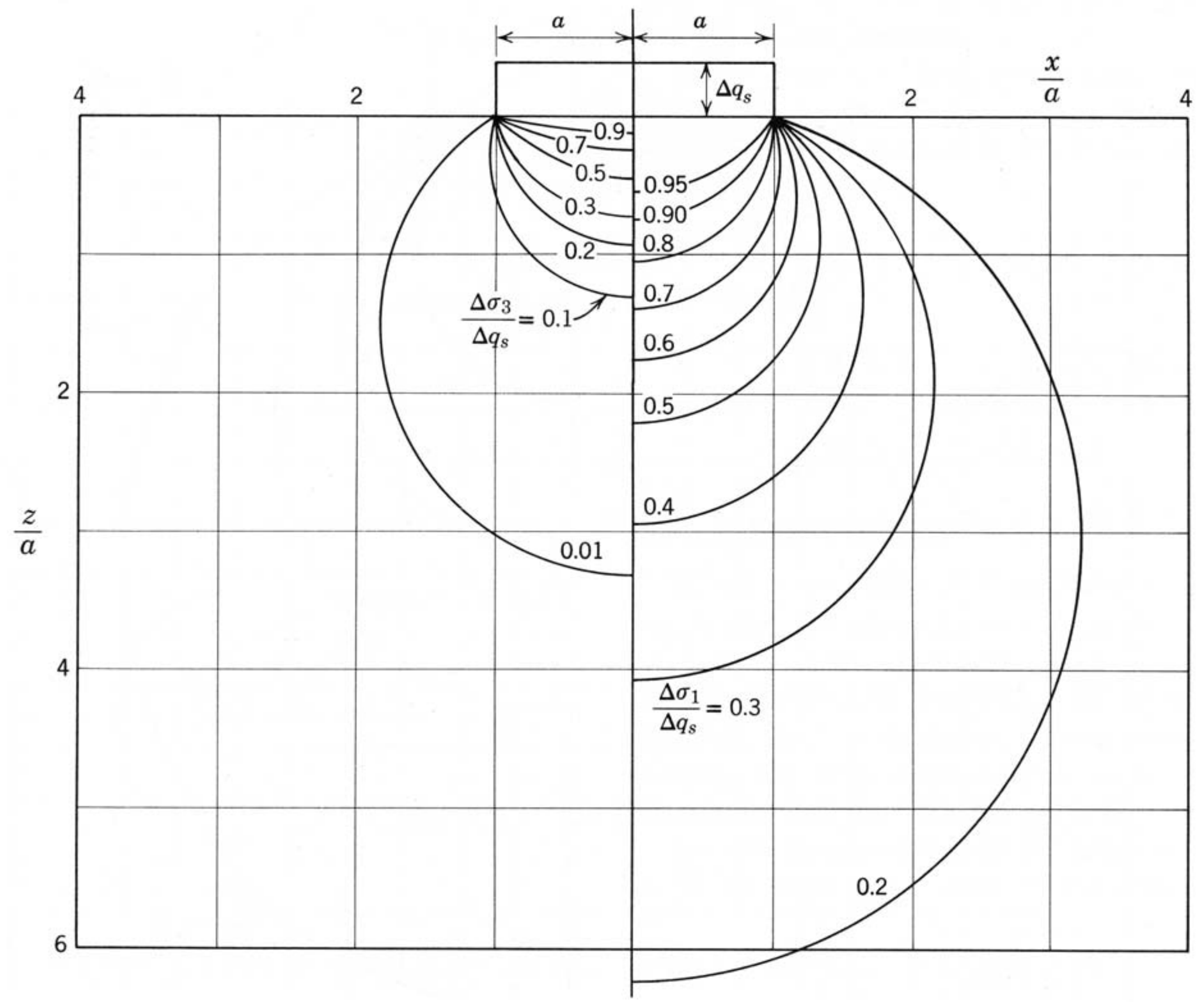


Figure 5-10: Stresses Under Uniform Strip Load (Lambe & Whitman, 1969)

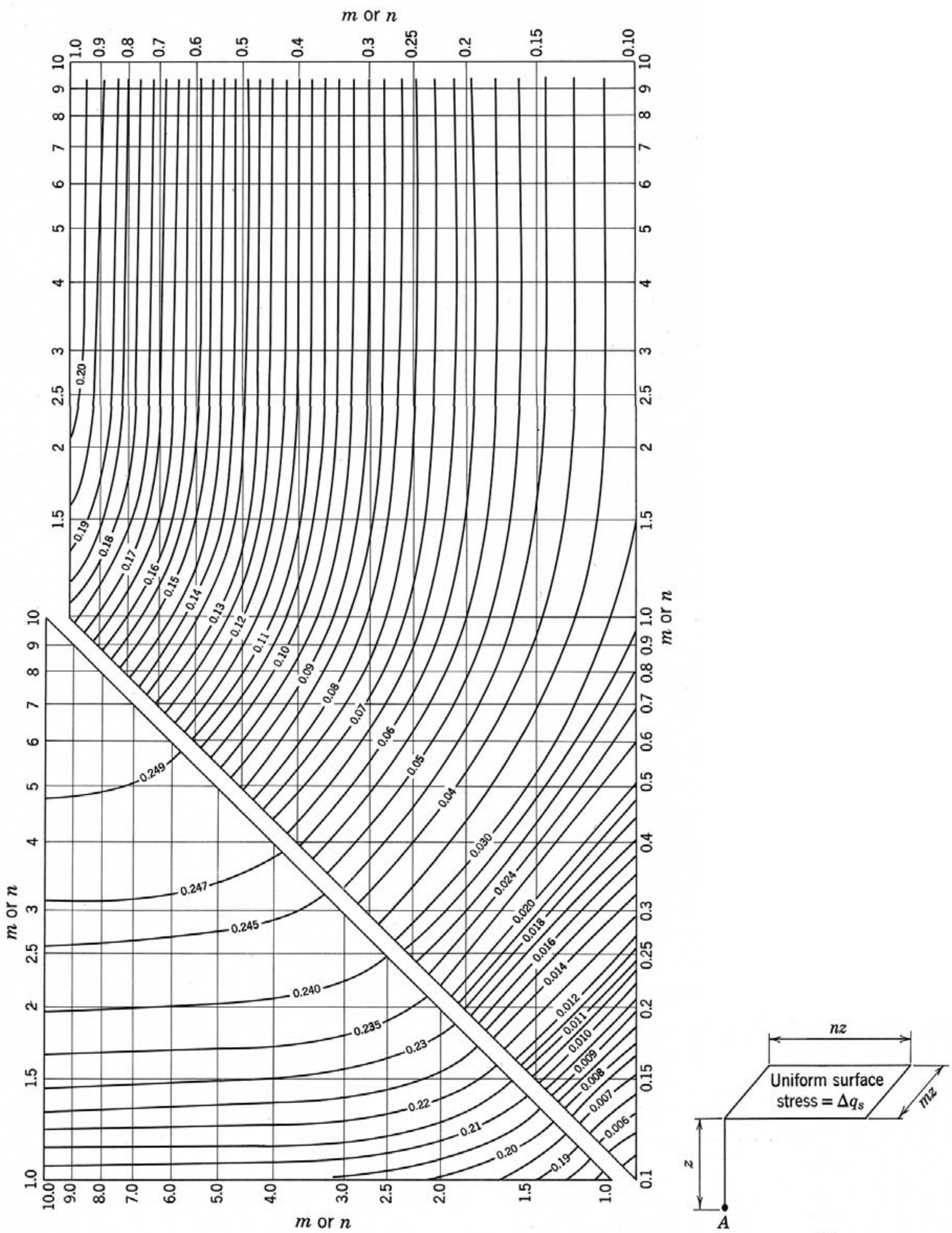


Figure 5-11: (a) Chart for Use in Determining Vertical Stresses Below Corners of Loaded Rectangular Surface Areas on Elastic, Isotropic Material
(b) At Point A, $\Delta\sigma_v = \Delta q_s \times f\,(m,n)$ (From Newmark, 1942)

Alternatively, stress distributions can also be obtained by assuming a 2-to-1 distribution of stress below the footing, as shown in Figure 5-12. This is much simpler to compute and is a reasonably accurate approximation of the decrease in stress with depth due to an applied surficial load of limited areal extent.

A comparison of the simplified 2-to-1 distribution with the actual distributed stress according to elastic theory is shown in Figure 5-13. This distribution can be computed as a function of applied stress according to:

$$\frac{\Delta\sigma_v}{q} = \frac{B_f L_f}{(B_f + Z)(L_f + Z)} \tag{5-17}$$

where: $\Delta\sigma_V$ = Change in vertical stress at depth Z below the footing bearing elevation

q = Stress applied by the footing at the bearing elevation

Z = Depth below footing bearing elevation to point of interest, usually the center or midpoint of a soil layer or sublayer where a settlement computation is to be made

B_f = Width of footing

L_f = Length of footing

5.3.3 Footings on Cohesive Soils

Primary and secondary consolidation of clays or clayey deposits may result in substantial settlements when the structure is founded on saturated cohesive soils. Settlement resulting from primary consolidation may take weeks to years to be completed. Furthermore, because soil properties may vary beneath the foundation, the duration of the primary consolidation and the amount of settlement may also vary with the location of the footing, resulting in differential settlement between footing locations. If such settlements are not within tolerable limits, the structure may not perform as expected or intended.

When a foundation is wide compared to the compressible layer thickness beneath it, a large portion of the soil will settle vertically (one-dimensionally) with very little lateral displacement because of the constraining forces exerted by the neighboring soil elements. Based on measurements of lateral displacements beneath embankments and tanks, it is usually reasonable to assume that the settlements beneath a foundation will be one-dimensional even for narrow foundations, provided an adequate factor of safety is applied to bearing capacity (Terzaghi & Peck, 1967). Desiccated crust or overburden above the base of the footing will tend to minimize the amount of lateral displacements. Experience suggests that reasonably good estimates of settlement can be predicted assuming one-dimensional compression as long as the factor of safety is at least three (3) against bearing capacity failure. One-dimensional consolidation theory is presented in this section and is used to calculate settlement estimates for shallow foundations on cohesive soils.

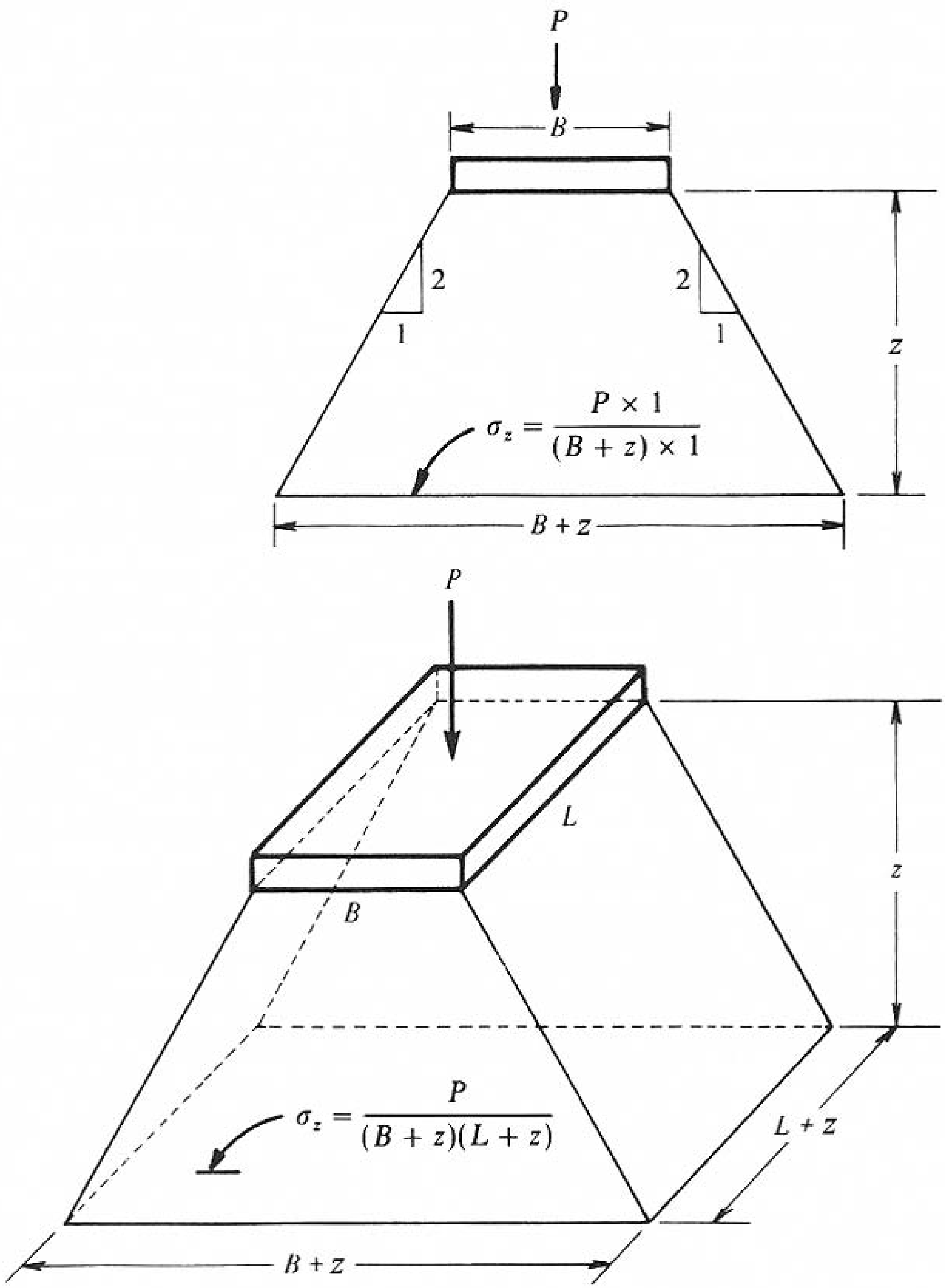


Figure 5-12: Distribution of Vertical Stress by the 2:1 Method
(Chen and McCarron, 1991)

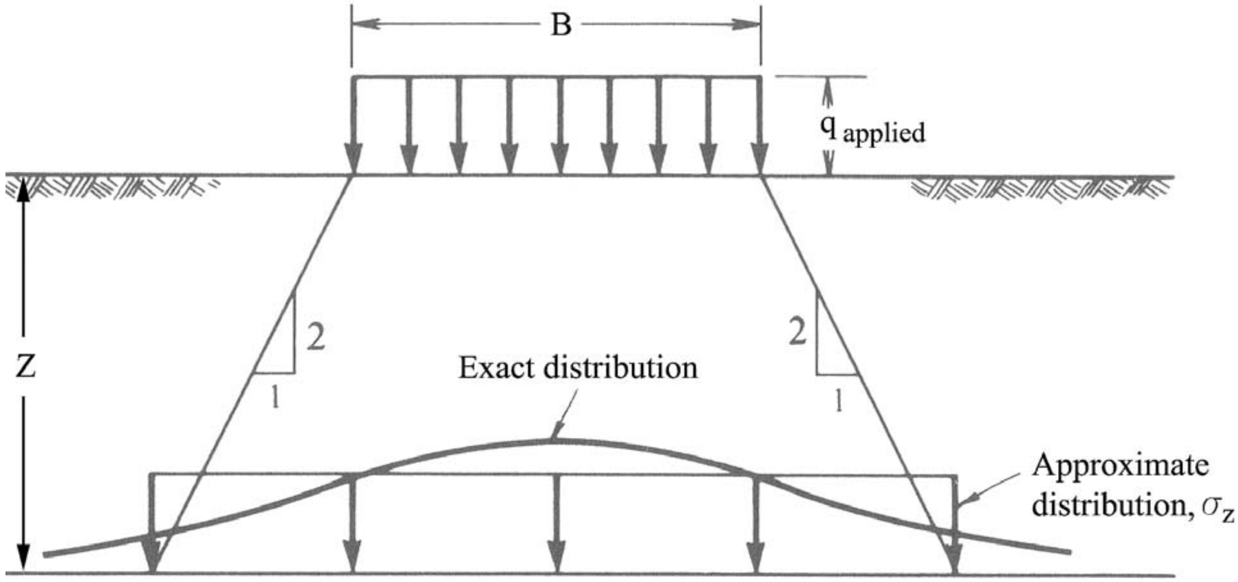


Figure 5-13: Comparison of Vertical Stress Increase Below a Uniformly Loaded Square Area as Computed by Elastic Theory and Approximated by the 2:1 Method (Perloff, 1975)

One-dimensional consolidation testing (ASTM D-2435) on undisturbed soil samples is recommended for determining the compressibility of clays. The procedure for obtaining undisturbed samples and specifying the appropriate load increments for conducting the test is described in detail in the *Soils and Foundations Workshop Reference Manual* (Cheney & Chassie, 2000).

Some natural deposits of cohesive soils have undergone heavy compression in their geologic history due to the weight of glaciers, the weight of overlying soil that was subsequently eroded away or dessication (drying) of the soils. These soils are called "preconsolidated" or "overconsolidated" and have been subjected to larger stresses in the past than at present. Preconsolidated soils can exhibit compressibility that is up to an order of magnitude less than normally consolidated soils. Preconsolidated soils can therefore be reloaded to a stress level lower than the preconsolidation pressure without experiencing large settlements. **Identification of preconsolidated soils through consolidation testing can allow the cost-effective use of shallow foundations on such soils.**

Settlement due to consolidation can be estimated from the slope of the one-dimensional consolidation test void ratio (e = volume of void/volume of solids) versus the logarithm of the vertical effective stress (σ'_v) curve. Figure 5-14 shows a typical consolidation curve from a one-dimensional consolidation test and the recommended procedure to obtain the preconsolidation pressure (σ'_p) developed by Casagrande (1936). The following definitions apply to the variables in the drawing.

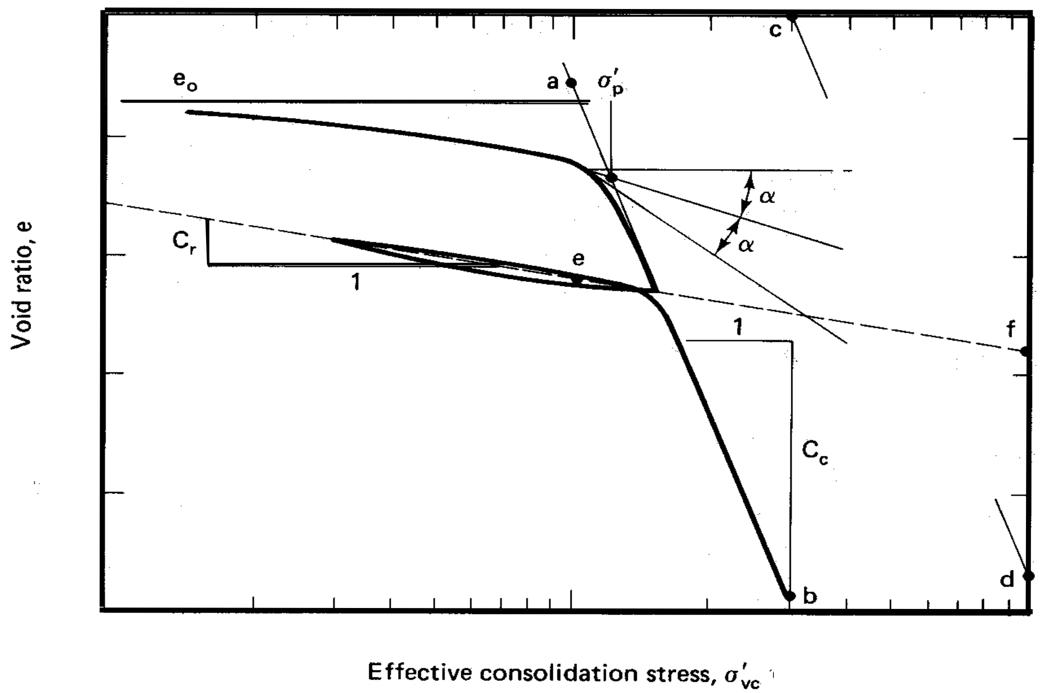


Figure 5-14: Void Ratio versus Log Effective Vertical Stress Curve from a Consolidation Test on Clay and the Casagrande Procedure for Determining the Preconsolidation Pressure (after Holtz & Kovacs, 1981)

Preconsolidation pressure, σ'_p – The maximum pressure to which a soil has been loaded in the past. It is useful to carefully establish a maximum and minimum value of σ'_p and plot the range as a function of depth on the effective stress versus depth chart for the foundation location under consideration.

Compression Index, C_C – The slope of virgin compression in the e-log(σ'_{vc}) curve. Virgin compression is the soil response to stresses higher than the preconsolidation pressure.

Recompression Index, C_R – The slope of the e-log(σ'_{vc}) curve at pressures less than the preconsolidation pressure. In general, the recompression index is approximately one-tenth the magnitude of the compression index in virgin compression.

Initial Void Ratio, e_o – The void ratio at the initial overburden stress, σ'_{Vo}.

The overconsolidation ratio (OCR) is used to distinguish overconsolidated soils from normally consolidated soils and is defined as the ratio of the preconsolidation pressure (σ'_p) at depth z to the existing vertical normal effective pressure (σ'_{vo}) at the same depth, z.

$$OCR = \sigma'_p/\sigma'_{vo} \qquad (5\text{-}18)$$

Soils with an OCR=1 are normally consolidated, while those with an OCR>1 are preconsolidated. The magnitude of expected settlement depends on the magnitude of loading of the subsurface soils relative to the magnitude of σ'_p. The magnitude of σ'_p is determined from the corrected one-dimensional consolidation test results.

Sampling disturbance can mask whether a soil is normally consolidated or overconsolidated, resulting in quantitatively poor estimates of settlements. Therefore, the highest quality samples should be obtained to measure the soil shear strength, compressibility and degree of preconsolidation in laboratory tests. The principal error in computing settlement for homogeneous clays is in the estimating of the degree of overconsolidation (Peck et al., 1974). The designer should use all available evidence to estimate the degree of preconsolidation. The undrained shear strength (c_u) and plasticity index (PI) can be used as a check (Peck et al., 1974) to estimate if a soil deposit is overconsolidated. The ratio of the undrained shear strength (c_u) and the vertical effective stress for normally consolidated clays has been correlated to plasticity index (Skempton, 1948; Bjerrum & Simons, 1960) and can be computed according to Equation 5-19.

$$\frac{c_u}{\sigma'_v} = 0.10 + 0.004\ (\mathrm{PI}) \qquad (5\text{-}19)$$

If the c_u/σ'_v values in the field are significantly higher than Equation 5-19, the soil is probably overconsolidated. Furthermore, if the deposit is known to be normally consolidated (i.e., OCR=1), the undrained shear strength determined from in situ or laboratory tests can be checked with this relationship.

The settlement induced by a shallow foundation can be kept to a minimum if the combined effect of the initial effective stress and the stress increase transmitted by the footing is lower than the preconsolidation pressure, σ'_p. When site and economic conditions permit, the foundations may be designed to distribute structure loads such that they remain below preconsolidation stresses.

Numerous correlations have been made between the compression index, C_c, and common index tests for normally consolidated soils and several are included in Table 5-11. **Note that the values of C_c can vary by as much as a factor of 5 (using the average trend line) in these empirical correlations for cohesive soils. They should not be used for final design of footings on cohesive soils.**

The primary consolidation settlement, S_c, of a foundation resting on layers of normally consolidated soils (σ'_p=σ'_{vo}) can be computed from:

$$S_c = \sum_{i}^{n} \frac{C_c}{1+e_o} H_o \log_{10} \frac{\sigma'_{vf}}{\sigma'_{vo}} \qquad (5\text{-}20)$$

TABLE 5-11: CORRELATIONS FOR COMPRESSION INDEX, C_C

Correlation	Soil	Source
$C_c = 0.009\ (LL-10)^{(1)}$	Clay of medium to low sensitivity (S_t<4) [(2)]	Terzaghi & Peck (1967)
$C_c = 0.0115\ w_n^{(3)}$	Organic soils, peat	ASCE (1994)
$C_c = 0.04$ to $0.006^{(4)}$	Uniform silts	Hough (1959)
$C_c = 0.015$ to $0.02^{(4)}$	Uniform sand, loose	Hough (1959)
$C_c = 0.004$ to $0.008^{(4)}$	Uniform sand, dense	Hough (1959)

[1] LL=liquid limit
[2] S_t=sensitivity=Undisturbed undrained shear strength/Remolded undrained shear strength
[3] w_n= natural water content
[4] $C_c = 1/C'$ where C′ is the bearing capacity index (Figure 5-19). *Note:* These are for cohesionless soils, but are included here for comparison purposes.

where: S_c = primary consolidation due to the change in void ratio of the soil
C_c = compression index of the normally consolidated portion of the e–log σ'_v curve
e_o = initial void ratio
H_o = thickness of layer *n*
σ'_{vo} = initial effective vertical stress at the center of layer *n*
σ'_{vf} = final effective vertical stress at the center of layer *n*

The final effective vertical stress is computed by adding the stress change due to the foundation load to the initial vertical effective stress. The change in the vertical effective stress is computed using the methods discussed in Section 5.3.2. The total settlement will be the sum of the compression in each of the *n* layers of soil.

Normally, the slope, C_c, of the virgin portion of the e-log σ'_v curve is determined from the corrected one-dimensional consolidation curve measured on specimens taken from each relevant soil in the stratigraphic column.

Sometimes the consolidation data are presented in terms of vertical strain (ε_v) instead of void ratio (Figure 5-15). In this case, the slope of the virgin portion of the ε_v versus log σ_v curve is denoted as $C_{c\varepsilon}$, and the settlement is computed using Equation 5-21 for normally consolidated soils.

$$S_c = \sum_{1}^{n} H_o C_{c\varepsilon}\ \log_{10}(\sigma'_{vf}/\sigma'_{vo}) \qquad (5\text{-}21)$$

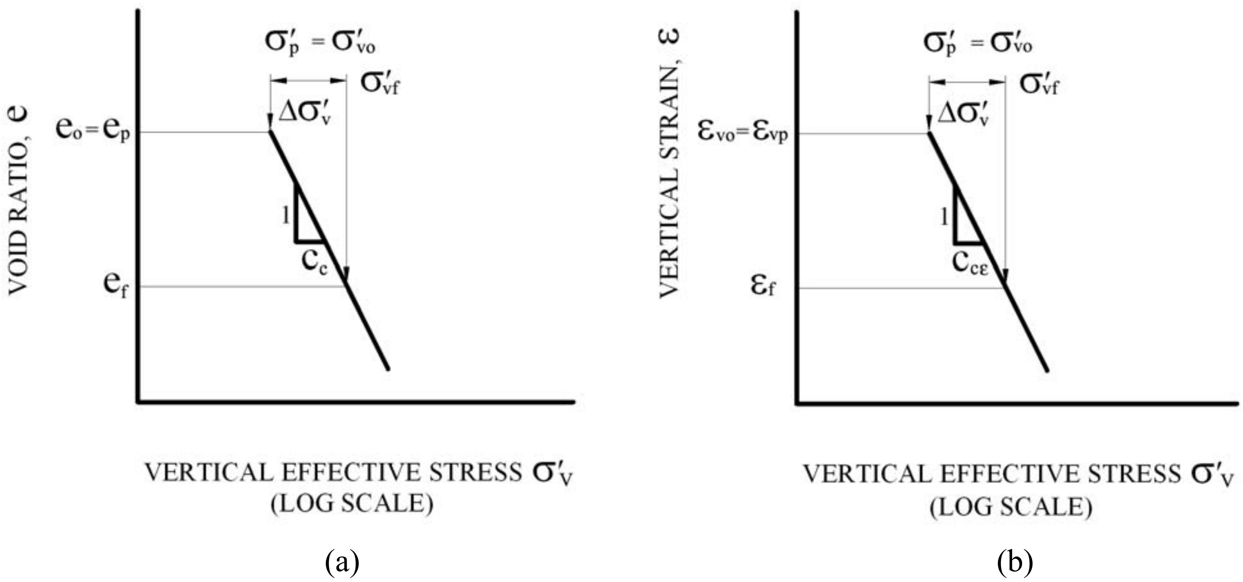


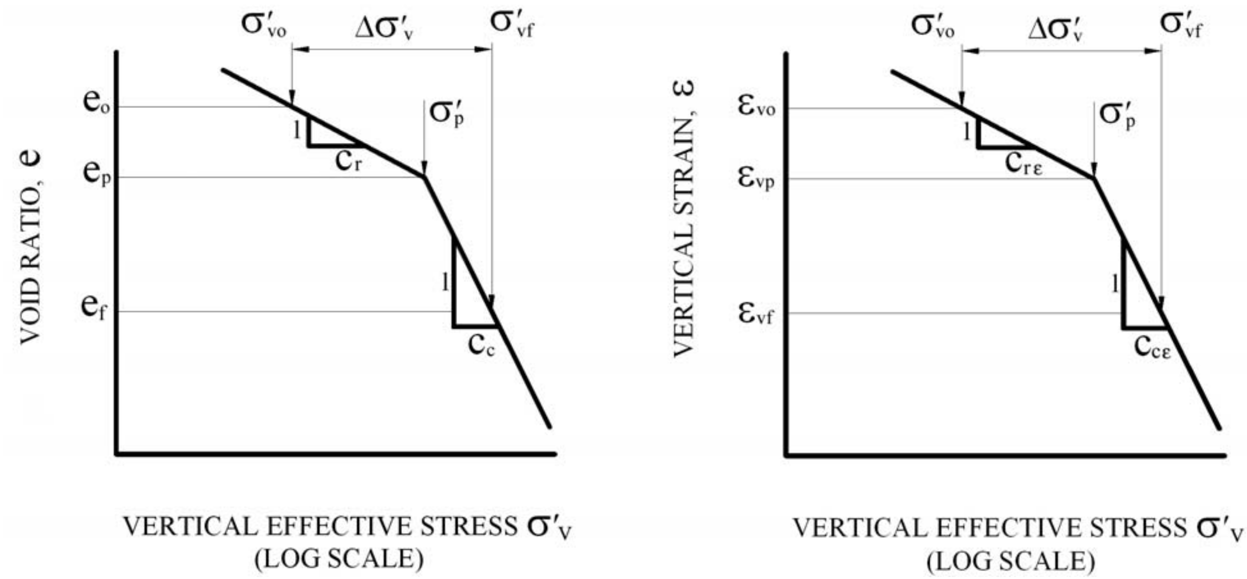
Figure 5-15: Typical Consolidation Curve for Normally Consolidated Soil
(a) Void Ratio and (b) Vertical Strain versus Vertical Effective Stress

If the water content of a clay layer below the water table is closer to the plastic limit than the liquid limit, or if the ratio of c_u/σ'_v exceeds the value from Equation 5-19, then the soil is likely overconsolidated. As a result, the field state of stress (before footing loads are applied) will reside on the recompression portion of the e versus log σ'_v curve. When a footing applies a load to the overconsolidated layer, the state of stress will first follow the recompression portion of the curve, until the preconsolidation pressure is reached. For stress states larger than the preconsolidation pressure, the virgin portion of the curve is followed. The settlements for the case of *n* layers of over-consolidated soils (OCR>1) are computed from Equation 5-22 (Figure 5-16a) or Equation 5-23 (Figure 5-16b).

$$S_c = \sum_1^n \frac{H_o}{1+e_o}\left(C_r \log_{10}\frac{\sigma'_p}{\sigma'_{vo}} + C_c \log_{10}\frac{\sigma'_{vf}}{\sigma'_p}\right) \tag{5-22}$$

$$S_c = \sum_1^n H_o \left(C_{r\varepsilon} \log_{10}\frac{\sigma'_p}{\sigma'_{vo}} + C_{c\varepsilon} \log_{10}\frac{\sigma'_{vf}}{\sigma'_p}\right) \tag{5-23}$$

The total settlement is computed by subdividing each compressible layer within the zone of influence, Z_I, into a sufficient number of sublayers, *n*, such that the initial and final stress calculated at the center of each layer is representative of the average stress for the layer, and the material properties are reasonably constant within the layer. The layers are typically 1.5 to 3 m thick in highway bridge applications. In cases where the various stratigraphic layers represent combinations of both normally and preconsolidated soils, the settlement is computed using the appropriate combinations of Equations 5-20 through 5-23.



(a) (b)

Figure 5-16: Typical Consolidation Curve for Overconsolidated Soil
(a) Void Ratio and (b) Vertical Strain versus Vertical Effective Stress

5.3.3.1 Time Rate of Consolidation

Consolidation occurs when a saturated compressible layer of soil is loaded and water is squeezed out of the layer. The time required for the (primary) consolidation process to end will depend on both the compressibility and the permeability of the soil.

The rate of consolidation is usually of lesser concern for foundations because superstructure damage will occur once the differential settlements become excessive. Shallow foundations are designed to withstand the settlement that will ultimately occur during the life of the structure, regardless of the time it takes for the settlement to occur. It is generally much more important to predict the time rate of the settlement of an approach embankment on soft clay, particularly if the embankment is surcharged in order to allow use of a spread footing. The procedure for computing the time-rate of settlement is beyond the scope of this document. However, design of approach embankments and surcharges for dealing with settlement of approach embankments is included in the *Soils and Foundations Workshop Reference Manual* (Cheney & Chassie, 2000). These procedures can be used to manage the potential for approach embankment settlement to allow the use of a spread footing supported in the approach fill. Example 3 in Appendix B shows the elements of this procedure.

5.3.3.2 Secondary Compression

Soils susceptible to secondary compression (creep) are generally too poor to consider supporting a shallow foundation on them. Even if the primary consolidation can be managed with surcharging, the uncertainty associated with predicting the magnitude of secondary compression settlements is generally too great to reliably found a spread footing on or over creep-susceptible soils.

5.3.4 Footings on Cohesionless Soils

The design of shallow foundations on cohesionless soils (sand, gravel and non-plastic silt), either as found in situ or as engineered fill, is often controlled by settlement potential, rather than bearing capacity limitations. The method used to estimate settlement of footings on cohesionless soils should therefore be reliable so that the predicted settlement is rarely less than the observed settlement, yet still reasonably accurate so that designs are not inefficient. Most settlement estimation methods are based on either elasticity theory or semi-empirical correlation to large databases of soil test data.

Settlement estimation methods based on elastic theory can be unreliable due to the difficulty of measuring the elastic modulus of granular soils. Modulus estimation is difficult because true undisturbed sampling of cohesionless soils is neither simple nor practical for routine foundation design. The modulus of granular soils can be estimated from in situ tests but, again, the reliability can be variable.

Semi-empirical methods are therefore the predominant techniques used to estimate settlements of shallow foundations on cohesionless soils. These methods have been correlated to large databases of simple and inexpensive tests such as the Standard Penetration Test (SPT) and the Cone Penetrometer Test (CPT).

Two major studies (Burland & Burbidge, 1984; and Gifford et al., 1987) on shallow foundations in sands included 76 independent case histories that are relevant to highway bridge shallow foundations. The Gifford et al. study compiled data from 21 case histories to compare the predictive capabilities of 11 methods of computing settlements of shallow foundations on sands.

Gifford et al. conducted a monitoring program to evaluate the actual time-settlement behavior of typical spread footing foundations on cohesionless soil. Over a period of 3 years, they monitored 21 bridge foundations on shallow spread footings from initial construction through completion of the structures. The data from this instrumentation program then were used to evaluate the accuracy of several settlement calculation methods. The methods that Gifford et al. evaluated in their final study were Burland & Burbidge (1984), D'Appolonia et al. (1968), Hough (1959), Peck & Bazaraa (1969), Schmertmann (1970), and Oweis (1979). For each of the five settlement prediction methods, Gifford et al. plotted measured settlement versus calculated settlement for each bridge foundation as shown in Figure 5-17.

In the 1990s, FHWA funded a study to examine the load-settlement behavior of large footings on cohesionless soils (Briaud & Gibbens, 1994, Briaud & Gibbens, 1995, and Briaud et al., 2000). The study included measuring settlement as vertical loads were applied to five square test footings ranging in size from 1 m by 1 m to 3 m by 3 m. The subsurface conditions at the National Geotechnical Experimental Site on the Riverside Campus of Texas A&M University were extensively evaluated using a wide range of exploration and testing procedures, including SPT, CPT, PMT, dilatometer tests, triaxial tests, borehole shear tests, and other measurements. As part of a prediction symposium, thirty-one authors submitted papers with alpha predictions of the load that would induce 25 mm (1 in) of settlement. Although an effort was made to compare predicted versus measured settlements for the numerous settlement computation methods used by the authors, it was concluded that there was too much variability in the way the various

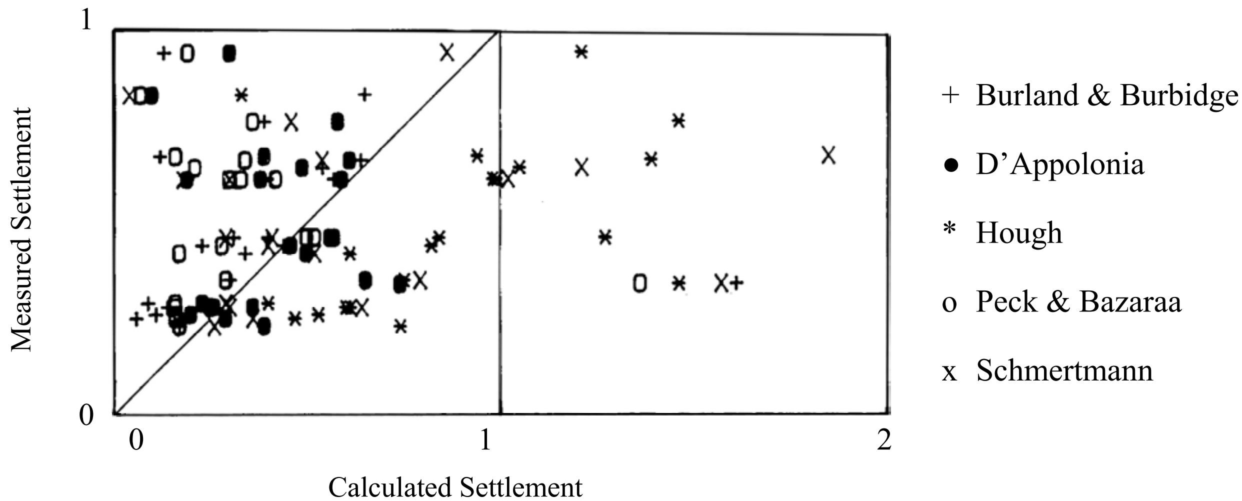


Figure 5-17: Calculated versus Measured Settlements (Gifford et al., 1987)

authors applied the methods to draw valid conclusions about the reliability and accuracy of the various methods. The most promising development resulting from this research is a method for computing settlement versus applied load using pressuremeter data. The method is presented in summary form in Section 5.3.4.3. This method is of particular interest in deformation-based analysis (as opposed to force-based analysis) and holds promise for evaluation of extreme load cases such as earthquake. The method may also be of notable value in the implementation of Load and Resistance Factor Design (LRFD), as discussed in Appendix C.

Considering the performance expectation of any settlement estimation method, the following criteria are important:

- The method should be reliable. That is, it should predict settlement with reasonable accuracy over a relatively wide range of soil conditions.
- The method should not exhibit a large standard deviation when comparing predicted settlement to actual settlement. A method that exhibits a large standard deviation in this regard has the potential for underpredicting settlement to a large extent, a condition that may result in poor structure performance.
- The method should be easy to implement and be based on site-specific data that are readily available and inexpensive to obtain.
- The method should have a substantial history of use with satisfactory results.

Unfortunately, no method for estimating settlement on granular soils does an outstanding job of meeting all these criteria. However, the studies discussed above do conclude that at least some of the methods meet most of these criteria reasonably well. Using these criteria, two methods were selected as currently preferred and recommended procedures for estimating settlement of foundations on granular soils: the Hough method and the D'Appolonia method. The load-settlement curve method recently developed by Briaud is also presented as a promising new method for computing deflections of spread footings on cohesionless soils (specifically sands) over a range of applied loads when pressuremeter test data are available.

The data from the Gifford study indicate that the Hough method, on average, overpredicts settlement by a factor of about 2, and is thus less than desirable with respect to accuracy. However, the Hough method is relatively reliable in that it is conservative and consistent. The Gifford study found that the method has a relatively low standard deviation of calculated versus predicted measurements ($s_o = 0.82$). The Hough method is based on the SPT test and is easy and inexpensive to implement. Most notably, it is already currently used by many highway agencies, so it has a vast history of satisfactory application in the design of shallow bridge foundations across the United States. It also has the desirable attributes of rationally accounting for both the increase in stress induced below a footing and the layering of soil deposits.

The Gifford study found the D'Appolonia method to be the most statistically accurate method evaluated. The standard deviation of calculated versus predicted measurements was also the lowest of the methods studied ($s_o = 0.51$). It computes settlement based on the modulus of compressibility of sand as correlated to the SPT test; therefore it is easy and inexpensive to implement. The method has the additional advantage of explicitly accounting for the effect of soil preloading, although the database to which preloaded modulus values were correlated was limited. Potential current drawbacks of the D'Appolonia method include the following:

- It has limited application history for highway bridges, especially as compared to the Hough method.
- It does not expressly account for soil layering.
- The distribution of stress below the loaded area is accounted for with an empirical correction factor that does not suggest a limiting depth of stress influence.
- The population of the database used to develop the method's modulus correlation as a function of SPT N-values is limited.
- Because it is statistically more "accurate," it is also potentially unconservative in practical application. That is, about half the normally distributed data will fall on the unconservative side of predicted versus measured settlement.

The data from the five full-scale load tests (Briaud & Gibbens, 1995) were used by the author to compare the performance of both the Hough and D'Appolonia methods of predicting the load that would result in 25 mm (1 in) of settlement. Note that this was not a true alpha prediction. Nevertheless, the field performance data from the five load tests is excellent, and the two methods were applied as strictly as possible according to the procedures described below. The comparison yielded the following conclusions:

- The Hough method was found to over-predict settlement by an average factor of about 1.8 for the five footings (the standard deviation, s_o, was 0.53). In no case, however, did the Hough method under-predict the observed settlement.

- The D'Appolonia method under-predicted the settlement by an average factor of about 0.77 for the five footings, but the accuracy was generally better than Hough, and the scatter in the predicted settlement values was also less (standard deviation, $s_o = 0.22$).

The data from the five load tests therefore generally corroborate the conclusions from the Gifford et al. and Burland & Burbidge studies.

Therefore, it is currently recommended that the Hough method be used as a primary design and analysis tool. The D'Appolonia or load-settlement methods could be employed as alternates, or when a more accurate estimate of settlement is desired, recognizing that these alternate methods have less history of use in the design of highway bridges. Naturally, if another method is locally or regionally known to provide more reliable estimates of settlement on granular soils, and the method meets the criteria noted above, the geotechnical engineer should exercise judgment and use the best method available.

5.3.4.1 Hough Method

Hough (1959) developed an empirical method for predicting settlements of shallow foundations on cohesionless soils that follows the same approach as that used for calculating consolidation settlement of clay layers. Note that the method is applicable only for normally consolidated cohesionless soils. Cheney and Chassie (2000) recommend that the SPT blowcounts be corrected for overburden pressure before correlating the N-values to the bearing capacity index, C′. An overburden correction by Bazaraa (1967) was recommended by Cheney and Chassie (2000). Since that time, many researchers have studied the effect of overburden stress on the SPT N-value, largely in support of liquefaction hazard assessment procedures. Recent consensus by the 1996 and 1998 National Center for Earthquake Engineering Research (NCEER) (Youd et al., 2001) concluded that the correction proposed by Liao & Whitman (1986) (shown in Figure 5-18) could be used for routine engineering applications. Therefore, the correction by Liao & Whitman is included here as part of the Hough procedure, in particular because it is easy to calculate and can be used without charts in simple computation spreadsheets. The soil is divided into layers, and the change in effective vertical stress at the mid-height of the layer as a result of the applied load is estimated using elastic theory.

The total settlement by the Hough method is calculated as follows:

1. Correct SPT blowcounts for overburden stress using Figure 5-18.

2. Determine bearing capacity index (C′) from Figure 5-19 using corrected SPT blowcounts, N′, determined in Step 1.

3. Subdivide subsurface soil profile into approximately 3-m (10-ft) layers based on stratigraphy to a depth of about three times the footing width.

4. Calculate the effective vertical stress, σ'_{vo}, at the midpoint of each layer and the average bearing capacity index for that layer.

5. Calculate the increase in stress at the midpoint of each layer, $\Delta\sigma_v$, using either Figure 5-9, 5-10 or 5-11, or the 2:1 method (Figure 5-12).

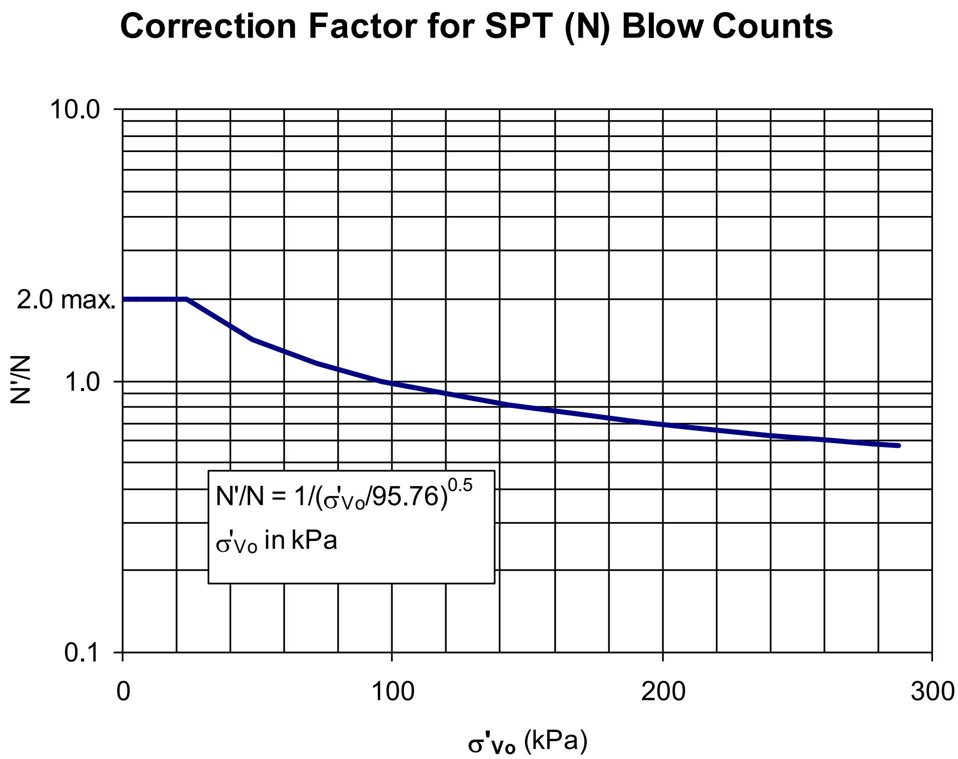


Figure 5-18: Corrected SPT (N) versus Overburden Pressure (after Liao & Whitman, 1986)

6. Calculate the settlement in each layer, ΔH, under the applied load using the following formula:

$$\Delta H \quad = H_o \frac{1}{C'} \log\left(\frac{\sigma'_{vo} + \Delta\sigma_{v_f}}{\sigma'_{vo}}\right) \tag{5-24}$$

7. Sum the incremental settlements to determine the total settlement.

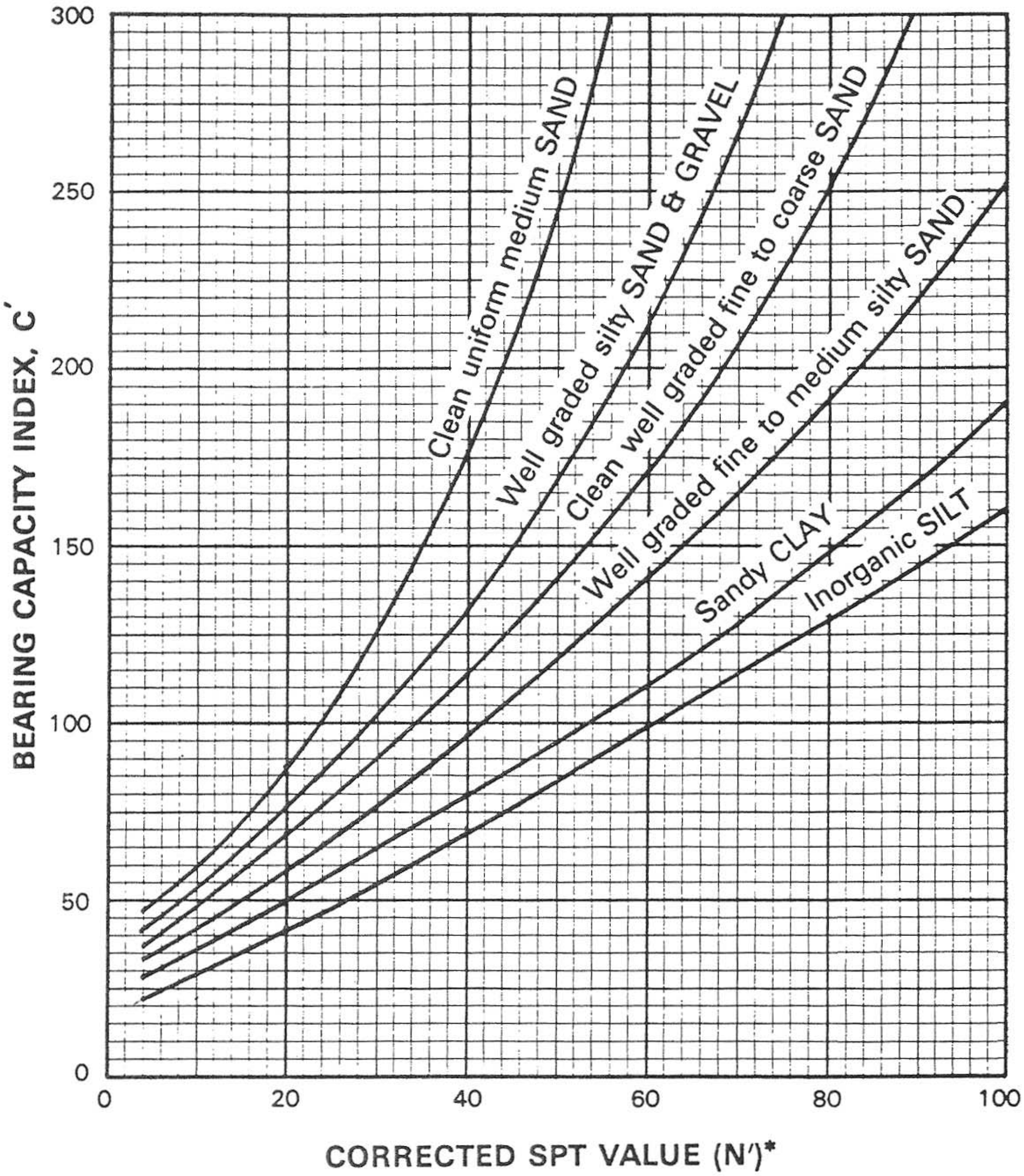
5.3.4.2 D'Appolonia Method

The D'Appolonia method calculates settlement using the following basic equation:

$$\Delta H = \left(\frac{\Delta\sigma_V B_f}{M}\right)\mu_o \mu_1 \tag{5-25}$$

where:
ΔH = settlement in sand or sand and gravel
$\Delta\sigma_v$ = applied stress beneath footing
B_f = footing width
μ_O = correction factor for embedment – see Figure 5-20 (dimensionless)
μ_1 = correction factor for thickness of sand layer – see Figure 5-20 (dimensionless)
M = modulus of compressibility of sand – see Figure 5-21

Any consistent set of units can be used for ΔH, $\Delta\sigma_v$, B_f and M.



*N′—*SPT (N) Value Corrected for Overburden Pressure.*

Reference: Hough, "Compressibility as a Basis for Soil Bearing Value" ASCE 1959

Figure 5-19: Bearing Capacity Index versus Corrected SPT (Cheney & Chassie, 2000, modified from Hough, 1959)

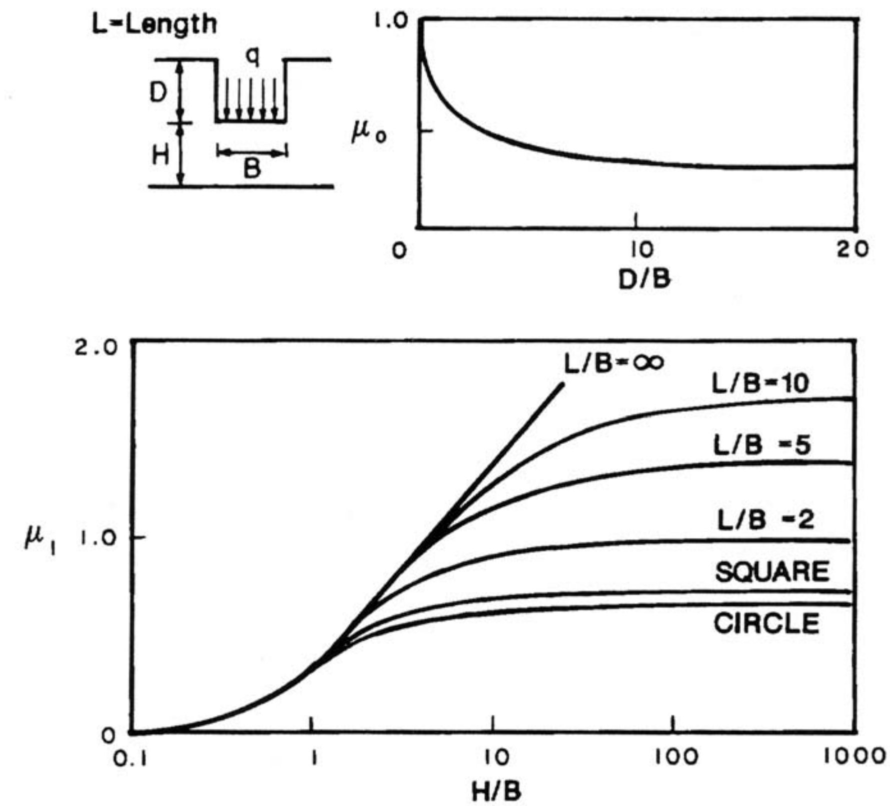
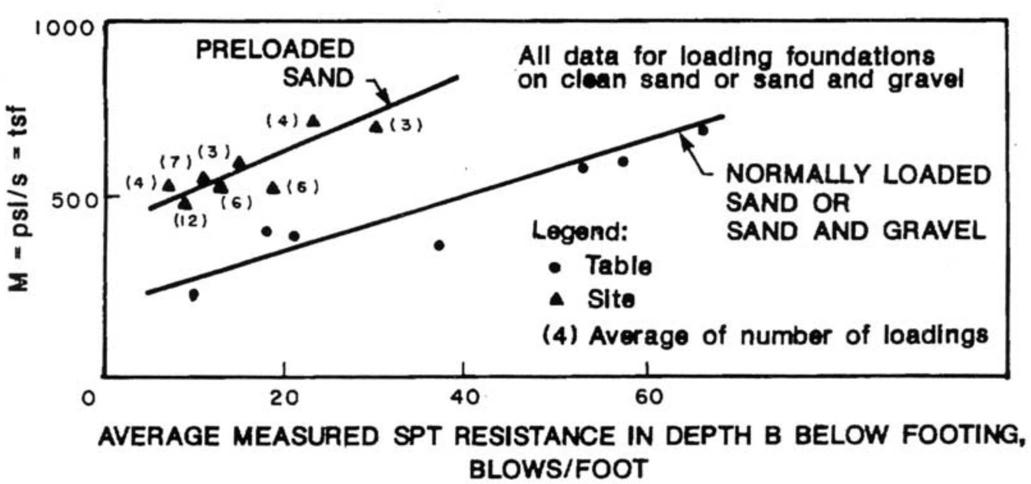


Figure 5-20: Correction Factors for Embedment and Layer Thickness
(Christian & Carrier, 1978)



Note: In the above Figure, "Table" refers to a tabulation of load versus settlement data from seven case histories, including six bridge footings, by D'Appolonia et al., (1970), and "Site" refers to the load versus settlement data obtained by D'Appolonia at al., (1968) at a large steel mill site in northern Indiana.

Figure 5-21: Modulus of Compressibility (D'Appolonia, 1968, 1970)

Further discussion of the application and use of other methods to calculate settlement can be found in the *NHI Shallow Foundations Reference Manual* (Munfakh et al., 2000). Methods covered in the Reference Manual include Schmertmann (1970), Peck & Bazaraa (1969), and Terzaghi & Peck (1967).

5.3.4.3 Load-Settlement Curve Method

In recent research, Briaud and colleagues (Briaud & Gibbens, 1994, Briaud & Gibbens, 1995, and Briaud et al., 2000) developed a procedure to estimate settlements of spread footings on sand using pressuremeter test (PMT) data. They developed the procedure because they had observed that the movement of the soil below a footing during large scale load tests showed an analogy to the movement of soil around an expanding pressuremeter probe during a pressuremeter test. The procedure is as follows:

1. Perform pre-bored pressuremeter tests within the depth of influence of the footing, typically at depths of 0.5B, 1B and 2B below the bearing elevation of the footing. Refer to *The Pressuremeter Test for Highway Applications* (Briaud, 1989) or *The Pressuremeter* (Briaud, 1992) for details regarding performance of the test and reduction of the test data.

2. Reduce the data and average the pressuremeter curves into a single pressuremeter curve.

3. Transform the average pressuremeter curve point-by-point into the load-settlement curve for the footing using Equations 5-26 through 5-35. Equation 5-26 aims at ensuring strain compatibility. The footing limit pressure is defined as the footing pressure corresponding to a settlement equal to 10 percent of the footing width ($\rho/B = 0.1$). Correspondingly, the pressuremeter limit pressure is defined as the pressure reached when the relative increase in cavity radius is equal to 0.42. Equation 5-26 ensures that when ρ/B is equal to 0.1, $\Delta R/R_0$ is equal to 0.42, and therefore the two limit pressures are corresponding.

$$\frac{\rho}{B} = \frac{1}{4.2}\left(\frac{\Delta R}{R_0}\right) \tag{5-26}$$

where: ρ = Vertical settlement of the footing
B = Footing width
$\Delta R/R_0$ = Ratio of the increase in cavity radius to the original radius from average pressuremeter test

Once Equation 5-26 is satisfied, Equation 5-27 is used to transform the pressuremeter pressure, p_p, into the footing pressure, p_f, through the gamma function, Γ.

$$p_f = f_{L/B}\, f_e\, f_\delta\, f_{\beta,d}\, \Gamma(p_p) \tag{5-27}$$

where: p_f = Transformed footing pressure
p_p = Pressuremeter pressure
Γ = Gamma function computed according to Equation 5-28
$f_{L/B}$ = Influence factor for footing shape, Equation 5-29
f_e = Influence factor for load eccentricity, Equations 5-30 & 5-31

f_δ = Influence factor for load inclination, Equations 5-32 & 5-33
$f_{\beta,d}$ = Influence factor for influence of slope, Equations 5-34 & 5-35

The gamma function, Γ, transforms the pressuremeter pressure, p_p, into the footing pressure, p_f, for a square footing loaded vertically at its center on a flat ground surface. This function was developed using data from 22 footing load tests from 9 different sand and silt sites. The gamma function recommended for design was chosen as the mean function through the data points minus one standard deviation to be on the safe side (conservative). The gamma function is computed according to Equation 5-28.

$$\Gamma = \frac{0.9}{100\left(\frac{\rho}{B}\right)+0.5}+0.8 \tag{5-28}$$

where: ρ = Vertical settlement of the footing
B = Footing width

The influence factors, f, for more complex loading conditions were developed from numerical simulations because there are insufficient load tests available for these load cases to correlate the factors directly to performance data. The influence factors were compared with the few existing applicable load tests and with other existing recommendations for influence factors.

The influence factor for footing shape can be computed using Equation 5-29.

$$f_{L/B} = 0.8+0.2\left(\frac{B}{L}\right) \tag{5-29}$$

where: L = Footing length
B = Footing width

The influence factor for eccentric loading can be computed using Equations 5-30 and 5-31, depending on the settlement location of interest.

$$f_e = 1-0.33\left(\frac{e}{B}\right) \quad \text{at the center of the footing} \tag{5-30}$$

$$= 1-\left(\frac{e}{B}\right)^{0.5} \quad \text{at the edge of the footing} \tag{5-31}$$

where: B = Footing width
e = Eccentric distance due to applied moment in the direction of B

The influence factor for inclined loading can be computed using Equations 5-32 and 5-33, depending on the settlement location of interest.

$$f_\delta = 1-\left(\frac{\delta}{90}\right)^2 \quad \text{at the center of the footing} \tag{5-32}$$

$$f_\delta = 1 - \left(\frac{\delta}{360}\right)^{0.5} \quad \text{at the edge of the footing} \tag{5-33}$$

where: δ = Inclination of load from the vertical, in degrees.

The influence factor for a footing located adjacent to a slope can be computer using Equations 5-34 and 5-35, depending on the slope inclination and the proximity of the footing to the slope.

$$f_{\beta,d} = 0.8\left(1 + \frac{d}{B}\right)^{0.1} \quad \beta = 18.4 \text{ degrees (3:1 slope)} \tag{5-34}$$

$$= 0.7\left(1 + \frac{d}{B}\right)^{0.15} \quad \beta = 26.6 \text{ degrees (2:1 slope)} \tag{5-35}$$

where: β = Slope inclination from the horizontal
d = Horizontal distance from top of slope to closest edge of footing
B = Footing width

Based on the comparison data studied to date, the load settlement curve method shows significant promise for accurately predicting the settlement of a spread footing on cohesionless soils over a fairly large range of applied loads. The method may therefore prove uniquely useful in deformation-based design. Deformation-based design is not covered here, but holds promise for analysis of structures under extreme loading conditions, such as earthquake.

Important considerations regarding use of the load-settlement curve method:

1. The researchers that developed this method acknowledge that it is new and that use in parallel with existing design approaches on full scale bridge projects, with performance measurements, is needed before the method can be implemented as a validated, everyday design tool.

2. Pressuremeter tests are still used infrequently, as compared to the SPT, hence the skills needed to perform and interpret the test results may be locally limited.

3. The method develops load settlement relationships in normalized displacement (i.e., ρ/B). This form should be reduced to displacement versus footing width for a tolerable magnitude of settlement (as defined by the performance requirements of the structure) in order to be used in the recommended design procedures in this GEC (see Chapter 6 and the design examples in Appendices B and C).

5.3.5 Footings on Structural Fills

Calculation of settlement within the structural fill due to a spread footing supported in the fill requires an assumption about the compressibility of the constructed fill material. Because structural fills should be constructed using good-quality granular materials, the estimation of settlement lends itself to application of the methods discussed in the previous section. To do so, an assumption must be made about the representative SPT N-value of the compacted fill.

The Gifford et al., (1987) study used a corrected (for overburden pressure) SPT N-value of 32 blows per foot as a representative value for estimating settlement in structural fills. This would correspond to a relative density, D_R, of about 85 percent at an overburden stress of about 100 kPa (1 tsf) (Figure 4-1). Correlations of relative density to percent relative compaction suggest that this value of D_R is at or above 95 percent relative compaction (Lee & Singh, 1971) and therefore approximates the compaction criteria for structural fill used by most highway agencies. Although Lee & Singh (1971) do not specifically state the relative compaction criteria (i.e., standard or modified Proctor), the comparisons were for granular soils and the modified Proctor method is mentioned. It is therefore assumed that the comparisons were based on the modified Proctor test. **In the absence of other SPT data in structural fills, the settlement of a footing supported on structural fill can be estimated using an assumed corrected N-value (N′) of 32 and the Hough method for computing settlement**.

5.3.6 Footings on IGMs and Rock

As discussed in Section 3.1.3, the assumption is made in this GEC that intermediate geomaterials (IGMs) are stiff and strong enough that bearing capacity and settlement considerations will generally not govern design of a spread footing supported on such material. If a settlement estimate is necessary for shallow foundations supported on IGM or rock, a method based on elasticity theory will generally be the best approach. As with any of the methods for estimating settlement that use elasticity theory, a major limitation is the engineer's ability to accurately estimate the modulus parameter(s) required by the method.

Equation 5-36 may be used to compute settlement on rock based on Young's modulus of the intact rock. In this equation, the stress applied at the top of the rock surface can be calculated using the stress distribution methods shown in Figures 5-9 through 5-12.

$$\delta_v = \frac{C_d \Delta\sigma_V B_f (1 - \nu^2)}{\alpha_E E_{IR}} \tag{5-36}$$

where: δ_v = Vertical settlement at surface
C_d = Shape and rigidity factors (Table 5-12)
$\Delta\sigma_v$ = Change in stress at top of rock surface due to applied footing load
B_f = Footing width or diameter
ν = Poisson's ratio (Table 5-13)
E_{IR} = Young's modulus of intact rock (Table 5-14)
α_E = Modulus reduction factor based on RQD (Equation 5-37)

$$\alpha_E = 0.0231(\text{RQD}) - 1.32 \geq 0.15 \tag{5-37}$$

where: RQD = Rock Quality Designation, in percent.

TABLE 5-12: SHAPE AND RIGIDITY FACTORS, C_d, FOR CALCULATING SETTLEMENTS OF POINTS ON LOADED AREAS AT THE SURFACE OF AN ELASTIC HALF SPACE WITH INFINITE DEPTH (AFTER WINTERKORN & FANG, 1975)

Shape	Center	Corner	Middle of Short Side	Middle of Long Side	Average
Circle	1.00	0.64	0.64	0.64	0.85
Circle (rigid)	0.79	0.79	0.79	0.79	0.79
Square	1.12	0.56	0.76	0.76	0.95
Square (rigid)	0.99	0.99	0.99	0.99	0.99
Rectangle (length/width):					
1.5	1.36	0.67	0.89	0.97	1.15
2	1.52	0.76	0.98	1.12	1.30
3	1.78	0.88	1.11	1.35	1.52
5	2.10	1.05	1.27	1.68	1.83
10	2.53	1.26	1.49	2.12	2.25
100	4.00	2.00	2.20	3.60	3.70
1000	5.47	2.75	2.94	5.03	5.15
10000	6.90	3.50	3.70	6.50	6.60

TABLE 5-13: SUMMARY OF POISSON'S RATIO FOR INTACT ROCK (AASHTO, MODIFIED AFTER KULHAWY, 1978)

Rock Type	No. of Values	No. of Rock Types	Poisson's Ratio, v			Standard Deviation
			Maximum	Minimum	Mean	
Granite	22	22	0.39	0.09	0.20	0.08
Gabbro	3	3	0.20	0.16	0.18	0.02
Diabase	6	6	0.38	0.20	0.29	0.06
Basalt	11	11	0.32	0.16	0.23	0.05
Quartzite	6	6	0.22	0.08	0.14	0.05
Marble	5	5	0.40	0.17	0.28	0.08
Gneiss	11	11	0.40	0.09	0.22	0.09
Schist	12	11	0.31	0.02	0.12	0.08
Sandstone	12	9	0.46	0.08	0.20	0.11
Siltstone	3	3	0.23	0.09	0.18	0.06
Shale	3	3	0.18	0.03	0.09	0.06
Limestone	19	19	0.33	0.12	0.23	0.06
Dolostone	5	5	0.35	0.14	0.29	0.08

TABLE 5-14: SUMMARY OF YOUNG'S MODULUS FOR INTACT ROCK (MODIFIED AFTER KULHAWY, 1978)

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, E_{IR} (kPa x 10^6)			Standard Deviation
			Maximum	Minimum	Mean	
Granite	26	26	99.97	6.41	52.67	24.48
Diorite	3	3	111.69	17.10	51.36	42.68
Gabbro	3	3	84.11	67.57	75.84	6.69
Diabase	7	7	104.11	68.95	88.25	12.27
Basalt	12	12	84.11	28.96	56.12	17.93
Quartzite	7	7	88.25	36.47	66.12	16.00
Marble	14	13	73.77	4.00	42.61	17.17
Gneiss	13	13	82.05	28.47	61.09	15.93
Slate	11	2	26.13	2.41	9.58	6.62
Schist	13	12	68.95	5.93	34.27	21.92
Phyllite	3	3	17.31	8.62	11.79	3.93
Sandstone	27	19	39.16	0.62	14.69	8.20
Siltstone	5	5	32.82	2.62	16.48	11.38
Shale	30	14	38.61	0.01	9.79	10.00
Limestone	30	30	89.63	4.48	39.30	25.72
Dolostone	17	16	78.60	5.72	29.10	23.72

Additional corrections can be made for complex geologic conditions. The *NHI Shallow Foundations Workshop Reference Manual* (Munfakh et al., 2000) includes discussion of layered and anisotropic rock conditions and methods for correcting the deformation calculation.

The elastic modulus of IGMs and some rocks may be measurable by in situ testing with equipment such as the pressuremeter, the flat dilatometer, plate load tests or flat jacks. The reader is referred to FHWA publications, including *The Pressuremeter Test for Highway Applications* (FHWA-IP-89-008) and *The Flat Dilatometer Test* (FHWA-SA-91-044), or the relevant ASTM test method for details regarding performance and interpretation of the in situ test data.

Alternatively, some modulus values are available from various researchers based on the rock type. Sources include Bieniawski (1984, 1989), Kulhawy (1975) and Lama and Vutukuri (1978). Tables 5-13 and 5-14 summarize suggested values for Poisson's ratio and elastic modulus as presented in AASHTO (1996) and modified after Kulhawy (1978).

5.4 Sliding and Lateral Stability

A shallow foundation will resist lateral loads by a combination of the shear resistance along the base of the footing and the lateral earth pressure that develops in front of the footing in the direction of loading. The shear resistance along the base will develop its full capacity at very small deflections. The lateral earth pressure is termed “passive resistance” and requires much larger deflections to develop the full passive capacity. By the time the passive resistance is fully developed, the shearing resistance is fully mobilized and may even by reduced to some residual value less than the peak shearing resistance. For this reason, the design of most shallow foundations conservatively ignores passive resistance.

Sliding resistance between the base of a shallow foundation and a granular soil will be governed by the coefficient of friction (tan δ) that develops between the foundation soil and the footing. The value of the coefficient of friction is a function of the soil type and the roughness of the footing. For concrete cast against cohesionless or granular material, the coefficient of friction, tan δ, will be equal to the tangent of the friction angle (tan ϕ) for the soil supporting the footing. If the bottom of the footing consists of something other than concrete cast against the ground, then the coefficient of friction (tan δ) is not solely a property of the soil but is also a function of the footing material type and roughness. In the absence of specific data, Table 5-15 may be used to estimate the coefficient of friction between soil and various footing material types.

TABLE 5-15: ULTIMATE FRICTION FACTORS FOR DISSIMILAR MATERIALS (NAVFAC, 1986B)

Interface Materials	Coefficient of Friction, tan δ	Friction Angle, δ (degrees)
Mass concrete on the following 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 (masonry on foundation materials has same friction factor)	0.30 to 0.35	17 to 19

If the base of the footing rests on clay, the sliding resistance will be limited by the adhesion that develops between the bearing surface of the footing and the cohesive bearing material, c_a. In practice, c_a is typically taken as equal to the undrained shear strength (cohesion, c_u) of the clay.

The general equation for sliding resistance for footings with planar bearing surfaces on frictional (drained) materials is:

$$F_R = (W + P_v) \tan \delta \tag{5-38}$$

where: F_R = ultimate sliding resistance
W = weight of the footing and soil backfill over the footing
P_v = loads applied to the top of the footing
δ = friction angle between the base of the footing and frictional bearing material

The general equation for sliding resistance for footings with planar bearing surfaces on cohesive (undrained) materials is:

$$F_R = c_a B_f \tag{5-39}$$

where: F_R = ultimate sliding resistance
c_a = adhesion between the base of the footing and cohesive bearing material
B_f = footing width

Because drained and undrained conditions will not occur simultaneously, Equations 5-38 and 5-39 should not be combined for use at the same time.

A minimum factor of safety (FS) of 1.5 against sliding should be used for design of bridge foundations:

$$FS_{sliding} = \frac{F_R}{F_{sliding}} \geq 1.5 \tag{5-40}$$

where: $FS_{sliding}$ = factor of safety with respect to sliding
F_R = ultimate sliding resistance
$F_{sliding}$ = sum of horizontal loads acting to cause footing to slide

Equations 5-38 and 5-39 assume the bottom surface of the footing is planar. If sliding resistance as determined by Equation 5-39 for a footing on clay soils is insufficient, the footing width should be increased to provide additional sliding resistance. If the footing width becomes impractical in terms of economic or constructability factors, consideration may be given to including a key constructed under the bearing surface of the footing. A typical configuration of a key is shown in Figure 5-22. Keys are typically difficult to construct while maintaining the intact shearing resistance of the soil adjacent to the key.

In reality, keys constructed in granular soils are not very efficient in developing passive pressures in front of the key. Due to the deflections required to develop passive pressures described above, the frictional shearing resistance will already be fully mobilized. The effect of a key in granular soils is more properly considered as increasing the effective length of the shearing surface.

In stiff, overconsolidated clays and soft rock, the additional resistance provided by a key can be computed using Figure 5-22 and Equation 5-41.

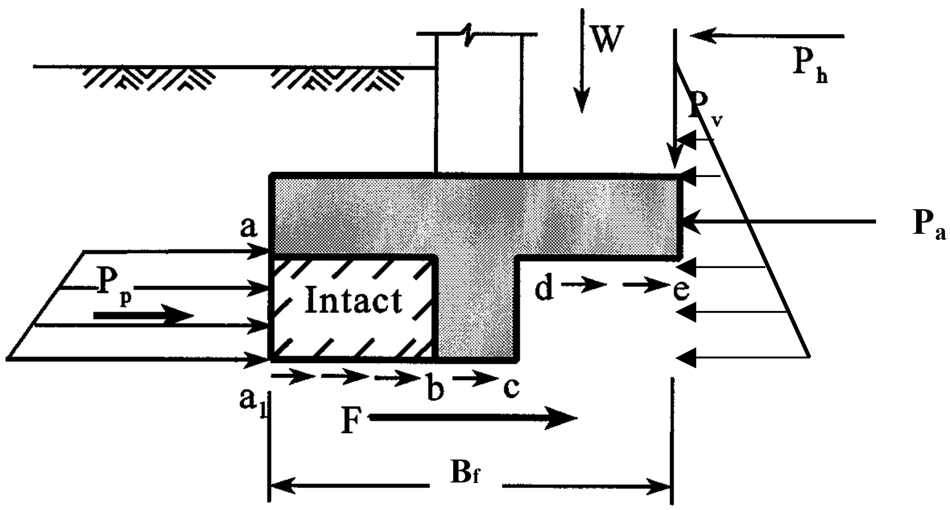


Figure 5-22: Resistance Against Sliding from Keyed Foundation in Cohesive Soils and Soft Rock

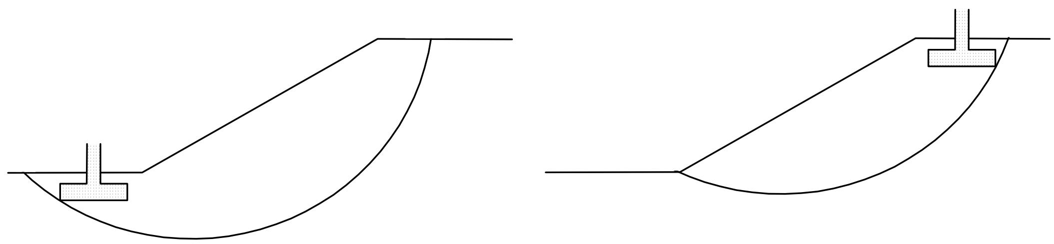
The general equation for sliding resistance of footings on cohesive soils provided with a key is:

$$F_R = c_a(B_f - \overline{a_1 b}) + c_u(\overline{a_1 b}) + P_p \tag{5-41}$$

where:
- F_R = ultimate sliding resistance
- c_a = adhesion between the base of the footing and cohesive bearing material
- B_f = footing width
- $\overline{a_1 b}$ = distance from point a_1 to point b
- c_u = undrained shear strength of cohesive soil
- P_p = passive resistance force

5.5 GLOBAL STABILITY

Global stability of shallow foundations should be evaluated when the geometry of the ground surface in any direction away from the footing suggests that the assumptions associated with the bearing capacity theory used to calculate the bearing capacity of the footing are not valid, and the stress applied by the footing loads is acting to destabilize the slope geometry. Figure 5-23 depicts slope configurations and footing locations that act to either improve or degrade the global stability of the slope.



(a) (b)

Figure 5-23: Global Slope Stability with Respect to Shallow Foundation Locations
(a) Footing Acting to Stabilize Slope, (b) Footing Acting to Destabilize Slope

In cases where the stress applied by the footing acts to destabilize the slope, the global stability of the footing should be checked using limit equilibrium methods such as the method of slices (Bishop, 1955; or Spencer, 1967).

Detailed guidance for the evaluation of slope stability is not included in this Circular. The *Soils and Foundation Workshop Reference Manual* (Cheney & Chassie, 2000) includes fundamental discussion of slope stability analysis methods considering the presence of a loaded footing within the slope. The reference work TRB Special Report 247: *Landslides, Investigation and Mitigation* (TRB, 1996) includes more detailed discussion on the analysis of global slope stability.

CHAPTER 6

SHALLOW FOUNDATION DESIGN FOR BRIDGES

This chapter presents the practical application of shallow foundation design for bridge structures. A design process is presented in a step-by-step format that will result in a satisfactory solution for most applications of shallow foundation bridge support. The design process involves multiple engineering disciplines. The discipline generally responsible for each design step is identified, whether it be geotechnical, structural, hydraulic, environmental or general civil.

Worked foundation design examples that follow this process are included in Appendix B for spread footing support of bridge piers in the following applications:

- Interior bridge pier supported on a spread footing founded in native soil
- Integral stub abutment supported on compacted structural fill
- Stub seat-type abutment footing supported in compacted structural fill
- Full-height seat abutment founded in native soil
- Interior bridge pier supported on a spread footing on native rock

A discussion of the application of Load and Resistance Factor Design (LRFD) to the design of spread footings is included in Appendix C. The first and third examples in Appendix B are re-worked in Appendix C using LRFD methodology.

It should be noted that these examples are intended to convey one approach and process that can lead to a satisfactory design of bridge support on a shallow foundation. The examples should not be construed as representations of the *only* approach that can lead to a satisfactory design. Also note that the examples only check one or, at most, a few specific load cases. In practice, every complete design should consider all applicable load cases.

6.1 DESIGN PROCESS AND FLOW CHART

The design process of a bridge supported on shallow foundations includes the steps shown in Table 6-1 and also in the design process flow chart, Figure 6-1.

6.2 GENERAL CONSIDERATIONS

The depth and size of a shallow foundation for support of a bridge need to be selected to ensure that the bearing materials are able to support the loads imparted without shear failure or excessive settlement. This includes verifying that the bearing materials are nonliquefiable during earthquakes. The base of the footing also needs to be below the frost depth for frost susceptible soils, and below the probable scour during a design high water extreme event.

TABLE 6-1: STEPS IN DESIGN PROCESS FOR BRIDGE SUPPORTED ON SHALLOW FOUNDATIONS

Step	Design Activity	Responsible Discipline(s)
1.	Develop **preliminary bridge layout**. Establish the desired bridge type, size and location. Define span lengths and pier locations, considering geometrical and environmental constraints.	Structural, in coordination with general civil and environmental considerations and geotechnical for approach stability
2.	Determine the shallow foundation feasibility based on **review of existing geologic and subsurface data**. Competent bearing material must be present within a reasonable distance from the ground surface. A preliminary assessment of approach embankment stability should be conducted to evaluate potential impacts to abutment locations and span lengths (Section 4.1).	Geotechnical, in coordination with structural, general civil and environmental
3.	A **site reconnaissance** with the structural and general civil engineer should be completed at this stage to evaluate constructability of foundation types (Section 4.2).	Geotechnical, in coordination with structural, general civil and environmental
4.	Determine the depth of the footing so that it will not be susceptible to **scour potential** or **frost** (Section 6.2).	Hydraulic, with geologic input from geotechnical
5.	Determine the **loads** applied to the footing and **tolerable total and differential settlement** for each pier (Section 6.3).	Structural
6.	Determine the design soil properties from the **subsurface exploration** and **laboratory testing** program (Sections 4.3 and 4.4).	Geotechnical
7.	Calculate the **allowable bearing capacity**, considering both the potential for bearing capacity failure (Section 5.1) and settlement (Section 5.2)	Geotechnical
8.	Calculate the **allowable sliding** and **passive soil resistance** (Section 5.4) for evaluating lateral stability of the footing.	Geotechnical
9.	In cases where overall stability of the footing may govern the design (e.g., footings on or near slopes), perform a **global stability analysis** of the footing using service (unfactored) loads (Section 5.5).	Geotechnical
10.	Size the **footing dimensions** using service (unfactored) loads and the effective footing dimensions (Section 6.4.1).	Structural
11.	Check the stability of the footing for **sliding** and **overturning** using service loads (Sections 5.4 and 6.4).	Structural
12.	Complete the **structural design** of the footing using factored loads per Load Factor Design requirements and assuming a trapezoidal distribution of contact pressure beneath the footing (Section 6.5). The design should consider the moment at the face of the column, one-way shear and two-way shear.	Structural

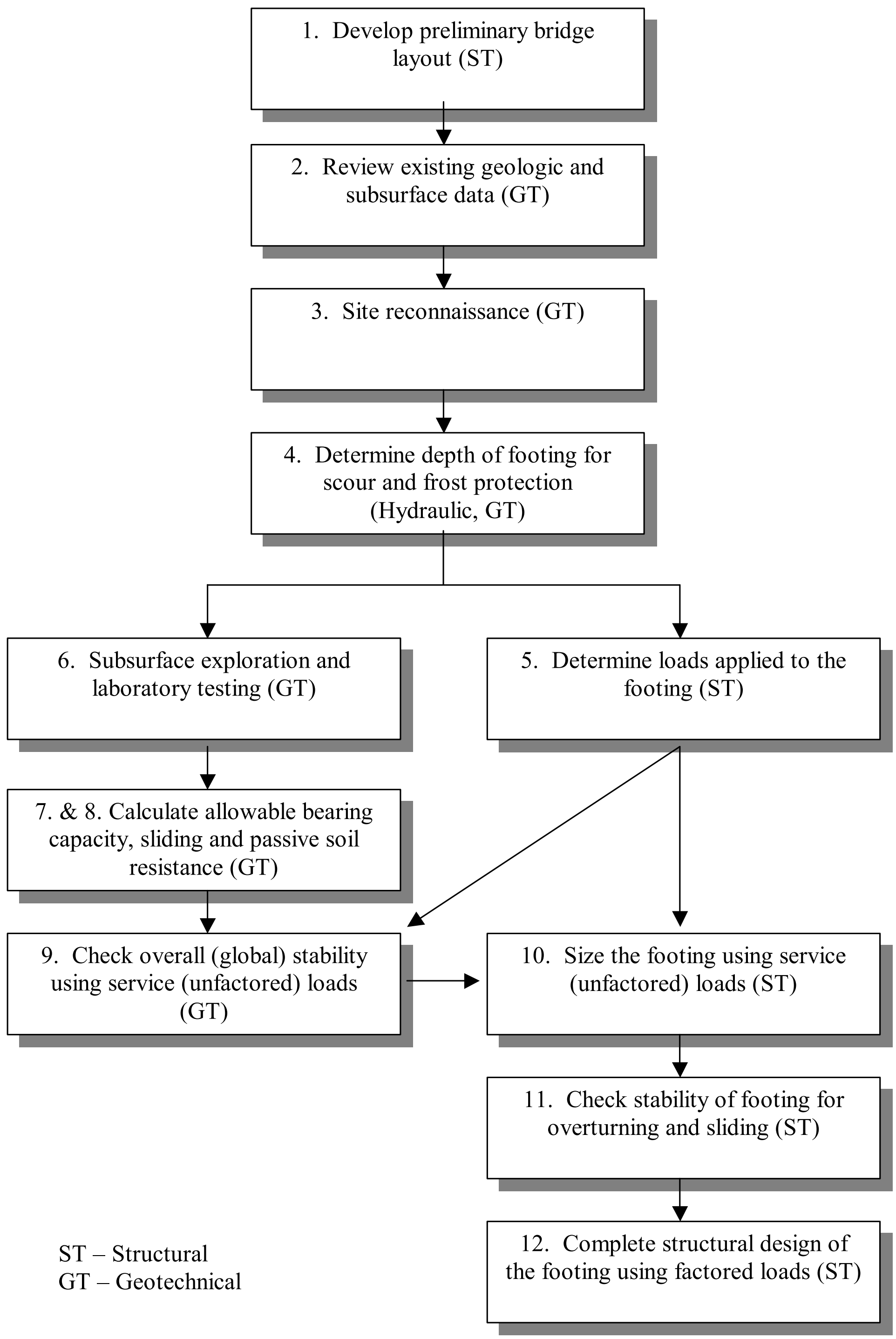


Figure 6-1: Design Process Flow Chart – Bridge Shallow Foundation

6.2.1 Scour Potential and Depth

Scour is localized erosion of the channel bed that occurs around flow obstructions such as piers and bridge abutments, at channel contractions such as bridges, and on the outside of channel bends. It can also be the result of long-term erosion of the channel bed that can occur during a structure's life.

Scour is a site-specific process that is a function of the flow velocity and duration, the geometry of the structural elements exposed to the flow of water, the geomorphology of the channel and the properties of the foundation and channel bed materials. The process of scour is complex. A multidisciplinary team of hydraulic, geotechnical and structural engineers should evaluate the risk of scour-induced failure at each structure site.

This section describes the assessment of scour susceptibility for spread footing foundations. Basic scour concepts and techniques for scour protection are presented. The discussion focuses on scour at bridges, but concepts presented also apply to culverts, retaining walls and other transportation structures built near channels and rivers. Information presented here is consistent with FHWA publications on stream stability and scour, and the reader is encouraged to read the following for a detailed discussion of scour analysis and scour countermeasure design:

- HEC 20 – *Stream Stability at Highway Structures* (Lagasse et al., 2001a)
- HEC 23 – *Bridge Scour and Stream Instability Countermeasures* (Lagasse et al., 2001b)

Scour assessment requires a determination of the cumulative effects of the three main components of scour: aggradation/degradation, contraction scour and local scour. It also requires an evaluation of potential changes in channel geometry and location that may occur during a structure's design life.

Aggradation and degradation are long-term changes in the channel bed elevation. They can be caused by human activities within the watershed or naturally occurring changes in basin hydrology and geology. "Aggradation" is an increase in channel bed elevation caused by an increase in sediment load or decrease in sediment transport in the watercourse. It is common upstream of dams and reservoirs where flow velocities are reduced and sediment deposition tends to occur. "Degradation" is a lowering of the channel bed that can be caused by reductions in sediment load or increases in a stream's sediment transport. Degradation is common in urbanizing basins due to increases in flood magnitude and flow duration. It can also occur downstream of dams and in channels subject to aggregate mining, where sediment loads are reduced.

Contraction scour is the lowering of the channel bed due to localized narrowing of a channel as caused by a bridge or culvert constriction or an encroachment, such as a bridge approach embankment, into the channel. A channel contraction will cause a reduced flow area, an increase in velocity at the constriction and increased erosive power of the flow. An increase in discharge in the main channel, as overbank flows return at the bridge opening, also increases contraction scour magnitude.

Local scour occurs at bridge piers and abutments due to complex flow patterns that occur in the vicinity of these obstructions. Pier scour is caused by strong turbulence that occurs at the base of a pier as flows are deflected down toward the channel bed and accelerated around the pier. Pier scour magnitude is related to pier width, pier shape and alignment to flow. It is also related to the depth and velocity of the flow upstream of the structure. Effective pier width is the most influential parameter and is a factor in most pier scour equations. Greater effective pier widths result in deeper scour depth potential. Abutment scour is caused by the turbulent mixing of flows in the channel with flows obstructed by the abutment.

Natural channels continuously change shape and location within a floodplain. In addition to the vertical components of scour described above, the potential for changes in channel geometry and location must be assessed. Channel migration can expose structures to flows not anticipated at the time of design. It can also increase scour potential due to changes in flow direction or increase velocities at a structure. A foundation located in a floodplain should be designed for the worst scour condition that can occur during its life. If lateral stream migration could expose a shallow foundation or abutment, it should be designed for the same scour condition as foundations in the main channel.

Scour can undermine shallow foundations or remove sufficient overburden to redistribute foundation forces causing foundation displacement and detrimental stresses to structural elements. Table 6-1 provides general guidelines for design of shallow foundations. One of the hazards of placing a structure in a river or channel is the potential for scour around the foundations. For new structures the foundation should be designed deep enough such that scour protection is not required (Richardson & Davis, 1995). Deep foundation systems may be preferable from both constructability and/or cost perspectives.

6.2.2 Scour Design Considerations for Shallow Foundations

Richardson & Davis (1995) provides the following guidance on scour design considerations for shallow foundation support of bridges at stream crossings.

a. Spread Footings on Soil

- Ensure that the top of the footing is below the sum of the long-term degradation, contraction scour and lateral migration (excluding local scour).

- Place the bottom of the footing below the total scour line.

- The top of the footing can act as a local scour arrestor.

b. Spread Footings on Rock Highly Resistant to Scour

Place the bottom of the footing directly on the cleaned rock surface for massive rock formations (such as granite) that are highly resistant to scour. Avoid small embedment (keying) since blasting to achieve keying frequently damages the rock structure beneath the footing and makes it more susceptible to scour. Keys into softer rock or IGMs may be feasible using pneumatic and hydraulic excavation tools without detriment to the quality of the underlying bearing materials. If footings on massive hard rock surfaces require lateral constraint, steel dowels should be drilled and grouted into the rock below the footing level.

c. Spread Footings on Erodible Rock or IGMs

Weathered or other potentially erodible rock formations need to be carefully assessed for scour. An engineering geologist familiar with the area geology should be consulted to determine if rock, soil or other criteria should be used to calculate the support for the spread footing foundation. The decision should be based on an analysis of intact rock cores, including rock quality designations and local geology, as well as hydraulic data and anticipated structure life. An important consideration may be the existence of a high quality rock formation below a thin, weathered zone. For deep deposits of weathered rock, the potential scour depth should be estimated and the footing base placed below that depth. Excavation into weathered rock should be made with care. If blasting is required, light, closely spaced charges should be used to minimize overbreak beneath the footing level. Loose rock pieces should be removed and the zone filled with clean concrete. In any event, the final footing should be poured in contact with the sides of the excavation for the full designed footing thickness to minimize water intrusion below footing level. Guidance on scourability of rock formations is given in FHWA memorandum "Scourability of Rock Formations," dated July 19, 1991.

AASHTO specification requires footings for stream piers and each abutment to be founded at least 1.8 m (6 ft) below the streambed. For existing structures identified as scour susceptible, scour countermeasures are often required to protect foundations from scour conditions not identified at the time of design.

There are four general types of scour protection:

- Localized armoring
- River training
- Modifications to the foundations
- Monitoring

Localized armoring techniques include the following:

- Rock riprap
- Gabions and slope mattress
- Pre-cast concrete blocks
- Grouted riprap

Discussion of other armoring systems, including concrete slope pavement, grouted fabric, sand/cement bags and soil cement, and a thorough treatise on current technology for stream instability and bridge scour countermeasures, are provided in HEC 23 (Lagasse et al., 2001b).

6.2.3 Frost Depth

Footings founded in seasonal and permanently frozen grounds must be embedded below the depth of frost penetration (frost depth) to provide adequate frost protection. This is required to prevent heaving of the footings due to volumetric expansions of the subgrade soils from freezing and/or to prevent settling due to loss of shear strength and stiffness from thawing.

In general, for frost action to occur, the following conditions must apply:

- Presence of frost-susceptible soil
- Availability of water
- Freezing conditions

Fine-grained soils with low cohesion tend to be more frost susceptible. Silty soil containing a high percentage of particles smaller than the No. 200 U.S. Standard sieve tends to have a network of pores and fissures that promote frost penetration. Common frost-susceptible soils include silts (ML, MH), silty sands (SM) and low-plasticity clays (CL-CL/ML). In regions where freezing of the ground occurs during the winter months, lenses of clear ice will form within the fine-grained soils and result in a sequence of frozen soil and ice. If there is a source of water in liquid phase, such as a water table available under the frost depth, the water will be drawn toward these ice lenses by capillary and suction actions, further expanding the lenses, and consequently producing surface heave. In addition, the degree of air temperature and duration of below-freezing temperatures also influence the depth of penetration. The U.S. Army Corps of Engineers proposed a relationship between frost penetration depth and the freezing index. The "freezing index" is a product of the number of days below freezing and the average number of degrees below freezing during the freezing season. Refer to Corps of Engineers' Technical Manual TM 5-852-6 (U.S. Army Corps of Engineers, 1988) for calculation methods to determine frost depths in soils.

Footings should be located below the maximum depth of frost penetration to minimize the potential for freeze/thaw movements. The maximum depth of frost penetration generally is established by local experience or from published maps. The use of generalized maps showing large regions is discouraged. Local building codes, agency design policy and experience should be consulted for appropriate design values.

Discussion about special considerations for foundations in permanently frozen regions (permafrost regions) is beyond the scope of this document. Readers are referred to *Canadian Foundation Engineering Manual* (1985) for special principles and practices needed for successful design and construction in cold environments.

6.2.4 Expansive and Collapsible Soils

Both expansive and collapsible soils are regional in occurrence. Neither soil type is well suited for shallow foundation support without some sort of mitigation for the potential for large volume changes in these soils due to changes in moisture content. Because deep foundations are frequently used for bridge support in both soil types, detailed guidance for ground treatment to permit use of shallow foundations for bridge support is not included here.

Expansive clays experience increases in volume due to increases in moisture content. The magnitude of the volume change can cause severe damage to structures. In general, deep foundations are preferred for bridge support in expansive soil terrain, unless the potential for the volume change can be mitigated. The most common and effective strategy for mitigating expansion potential is to excavate and replace the expansive soil with sufficient nonexpansive material to counteract the swell potential. This is generally practical only if a ready source of nonexpansive material is located nearby. For approach embankment and abutment design, the expansion potential should be evaluated considering the construction and loads of both. The *Shallow Foundations Workshop Reference Manual* (Munfakh et al., 2000) provides the procedures for laboratory testing for expansion potential and analysis of the results.

Collapsible soils, when dry, can provide adequate support for relatively high loads. However, upon wetting after the load has been applied, the soils can experience dramatic decreases in volume. It is most important to identify collapse potential of foundation soils at a bridge site. Due to the difficulties in obtaining undisturbed samples of these materials, exploration and testing alone may not identify the collapse potential. Local experience and review of geologic mapping are therefore essential to identification of collapsible soil deposits.

6.2.5 Drainage Considerations

When a shallow foundation supports a stemwall, and there is a difference in elevation of the backfill on either side of the stemwall so that unbalanced lateral earth pressures will act on the stemwall, the footing and the stemwall combine to function as a retaining structure. This is a typical configuration for bridge abutments. As with most retaining structures, the material retained by the structure should be drained such that hydrostatic pressures do not also act against the abutment.

Abutments are important structures that perform multiple functions. Drainage details, including weep holes and perforated drainage pipes, are an essential inclusion in the plans and specifications. Most highway agencies use standard details and specifications to ensure adequate drainage behind abutments.

Additional discussion of abutment and retaining wall drainage requirements and construction details are provided in Chapter 9.

6.2.6 Seismic Considerations and Dynamic Stability

Seismic hazards should be assessed as part of the foundation type selection process. Shallow foundations are susceptible to excessive movements or damage if the bearing soils are subject to liquefaction or lateral spreading due to an earthquake. GEC No. 3, *Geotechnical Earthquake Engineering for Highways* (Kavazanjian et al., 1997), presents guidance on assessing seismic hazards at a site, including evaluation of liquefaction and lateral spreading potential. In general, shallow foundations will not be an appropriate choice for support of bridge piers when there is potential for liquefaction to develop under the design seismic event unless the liquefaction potential is mitigated by ground improvement. Refer to Munfakh et al., (2000) for discussion of design and construction techniques to improve poor ground conditions and allow use of spread footings.

If liquefaction and lateral spreading are not expected at a site, shallow foundations may be a suitable and cost-effective choice for support of a bridge, provided the foundation is designed to adequately resist the seismic loading conditions appropriate for the design event. Seismic design checks should, at a minimum, include the following (others may also apply):

- Design for seismic loads that act through the column or stemwall to the footing. A minimum factor of safety of about 1.0 to 1.15 should be maintained against bearing capacity or sliding failure of the footing.

- AASHTO (1996) permits eccentricity due to seismic loading to fall within the middle two-thirds (2/3) of the footing, based on a trapezoidal distribution of earth pressure (see Section 6.4). However, equivalent rectangular soil pressure distribution is recommended for sizing footings in this GEC; therefore, when using the rectangular pressure distribution, the seismic eccentricity should be kept within the middle one-half of the footing.

- Recall that seismic loads act in opposite directions at different times during the earthquake. Be sure to check for stability (overturning and sliding) as well as bearing capacity using loads applied in the direction that results in the most critical condition. Check to see if the foundation experiences uplift. A deep foundation may be needed to resist uplift forces.

- Design for seismic lateral earth pressures that can act on the footing. This is typically done using the Mononobe–Okabe equations as described in AASHTO (1996, 1998).

- Check global stability of any slope that supports the footing, or is supported by the footing, by including a seismic coefficient in the slip circle or other method of calculating a factor of safety of a slope. In general, the calculated factor of safety should be greater than 1.0 when the applied horizontal seismic coefficient is one-half the peak ground acceleration (Hynes-Griffin & Franklin, 1984).

6.3 LOADS

Calculation of loads applied to shallow foundations is presented in AASHTO (1996, 1998). Numerous load groups provide for a variety of possible loading conditions and also consider the special loading requirements of non-routine structures, such as long span bridges. A summary of load cases and the conditions they are intended to check is included in Table 6-2.

The designer should be aware of the possibility that a non-codified loading condition may actually control the design of a foundation. For instance, the construction sequence and methods should be considered when evaluating the critical load case for a shallow foundation. If an abutment may be fully backfilled prior to application of the superstructure loads, the overturning load of the backfill may be the most critical loading condition the foundation must ever resist.

TABLE 6-2: SERVICE LOAD DESIGN LOAD GROUPS

<table>
<tr><th>Load Group</th><th>Description</th></tr>
<tr><td colspan="2">Groups I though III check the structure without consideration for long term creep, shrinkage, shortening, or thermal movements.</td></tr>
<tr><td>I</td><td>Load combination with normal vehicles using the bridge without wind.</td></tr>
<tr><td>IA</td><td>Load combination for vehicles smaller than an H 20 using the bridge without wind.</td></tr>
<tr><td>IB</td><td>Load combination with permit vehicles using the bridge without wind.</td></tr>
<tr><td>II</td><td>Load combination with maximum wind, without vehicles.</td></tr>
<tr><td>III</td><td>Load combination with normal vehicles using the bridge with wind.</td></tr>
<tr><td colspan="2">Groups IV to VI check the structure with consideration for long term movements.</td></tr>
<tr><td>IV</td><td>Load combination with normal vehicles using the bridge without wind, and with consideration of long term shrinkage, shortening, and thermal movements.</td></tr>
<tr><td>V</td><td>Load combination with maximum wind without vehicles, and with consideration of long term shrinkage, shortening, and thermal movements.</td></tr>
<tr><td>VI</td><td>Load combination with normal vehicles using the bridge with wind, and with consideration of long term shrinkage, shortening, and thermal movements.</td></tr>
<tr><td colspan="2">Group VII is for seismic loading.</td></tr>
<tr><td>VII</td><td>Load combination for seismic loading.</td></tr>
<tr><td colspan="2">Groups VIII and IX check the structure for ice loads on the piers.</td></tr>
<tr><td>VIII</td><td>Load combination with normal vehicles using the bridge without wind and ice acting on the bridge.</td></tr>
<tr><td>IX</td><td>Load combination with maximum wind and ice, and without vehicles.</td></tr>
<tr><td colspan="2">Group X is for culvert design, the other load groups do not apply to culverts.</td></tr>
<tr><td>X</td><td>Load combination for culverts with normal vehicles using the culvert.</td></tr>
</table>

6.3.1 Vertical Loads and Pressures

Vertical loads should be calculated in accordance with the design specification selected as the design criteria for the project. Calculation of infrequent loads, such as seismic, should consider the potential range of soil and ground water conditions that may exist under and around the footing over the life span of the structure. It is important to ensure that the structure will perform well over the range of expected conditions that may occur, considering the potential for the following:

- Seasonal variations in ground water level
- Variation in the depth of cover over a footing due to scour or excavation
- Increased loads due to changes in permitted loads or the addition of overlays
- Intermediate construction conditions or equipment loading that may exceed design live loads

Section 5.2.2 discusses the concepts of gross and net bearing pressures. An additional consideration that is related to the concept of net bearing pressure is the calculation of loads due to the self-weight of the footing and the backfill over the footing.

Consider a footing constructed in an excavation below existing grade and then subsequently backfilled. The increase in stress experienced by the soil below the bearing elevation will be the net increase due to a change in the unit weight of material that is replaced over the top. This is usually only due to the concrete in the footing itself, unless a specified select backfill material replaces the soil excavated to construct the footing. Because the concrete has a higher unit weight than the soil it replaces, there is slight increase in bearing stress below the footing. For most applications, including bridge foundations, this effect is usually less than one or two percent of the loads applied by the column and is therefore commonly ignored.

Alternatively, consider the effect of a mat foundation constructed well below the existing grade. In this case, the soil removed during construction is replaced largely by air, so there is potentially a net *decrease* in the stress due solely to the self-weight of the shallow foundation and the lack of backfill.

The decision to include or ignore the effect of the footing self-weight and backfill should be evaluated on a case-by-case basis considering the following effects that the increase in stress will have on the design conditions checked while sizing the footing:

- The bearing stress increases, so the factor of safety with respect to bearing capacity will decrease, and the settlement will also potentially increase. These are undesirable effects. If the effect of self-weight is more than one or two percent of the applied column or structure loads, the weight should be included when sizing the footing.

- The effect of a net stress increase on eccentricity (overturning) or sliding of a footing will be to stabilize the foundation. It is therefore conservative to ignore the increase when computing the factor of safety against overturning (eccentricity), or sliding.

Mat foundations below grade with no backfill will experience the opposite effects.

6.3.2 Lateral Loads and Pressures

The transfer of shear applied by the column may apply lateral loads on a shallow foundation. The computation of these loads should be done in accordance with the appropriate section of AASHTO or the selected design criteria. For the case of abutments that retain fill, the lateral earth pressures and associated design loads must also be computed to size and design the footing.

Lateral earth pressures are determined based on the characteristics of the fill material behind the abutment. The different states of stress that should be considered include active, at-rest, passive and seismic. Computation of seismic earth pressures is not included here. The reader is referred to GEC No. 3, *Geotechnical Earthquake Engineering for Highways* (Kavazanjian et al., 1997) for design guidance on calculating lateral earth pressures due to seismic loads. In addition, a soil pressure distribution has to be determined for traffic surcharge. For the active, passive and at-rest states, the pressure distribution can be assumed to be triangular, with the resultant applied at H/3 from the bottom of the footing. The general equation for computing lateral loads due to lateral earth pressures is:

$$P = \frac{1}{2} K \gamma' H^2 \tag{6-1}$$

where: P = Lateral load, typically computed per unit length of wall or abutment
K = Lateral earth pressure coefficient
γ' = Effective unit weight of soil acting against the wall or abutment
H = Height of retained backfill behind the wall or abutment

The lateral load will be computed based on the earth pressure condition expected to develop behind the wall or abutment, active, at-rest or passive. The loads and the lateral earth pressure coefficients are typically written with a subscript of ‘a,’ ‘o’ or ‘p,’ for active, at-rest, or passive earth pressure, respectively, to denote the earth pressure computed.

6.3.2.1 Active Pressure Distribution

The pressure distribution is determined using calculations based on Coulomb theory. The active pressure coefficient, K_a, is computed from Equation 6-2. This is the general case for Coulomb lateral earth pressure theory. It does not account for surcharge or seismic loads.

$$K_a = \frac{\cos^2(\phi'-\theta)}{\cos^2\theta\cos(\theta+\delta)\left[1+\sqrt{\dfrac{\sin(\phi'+\delta)\sin(\phi'-\beta)}{\cos(\theta+\delta)\cos(\theta-\beta)}}\right]^2} \tag{6-2}$$

where: K_a = Active earth pressure coefficient
δ = Wall/backfill interface friction angle
β = Angle of backfill slope behind the wall to the horizontal ($\beta = 0^o$ for horizontal backfill surface
θ = Angle of back wall face inclination ($\theta = 0^o$ for vertical wall face)
ϕ' = Effective angle of internal friction for the soil

Figure 6-2 shows these terms.

For the case of a vertical wall (no batter) and where the ground surfaces above and below the wall are level, the active pressure coefficient may be calculated using the simpler Rankine equation:

$$K_a = \tan^2\left(45° - \frac{\phi'}{2}\right) \tag{6-3}$$

where: K_a = Active earth pressure coefficient
ϕ' = Effective angle of internal friction for the soil

The Rankine equation assumes no friction between the wall and the soil backfill. Note that Equation 6-3 does not include consideration for surcharge or seismic loads.

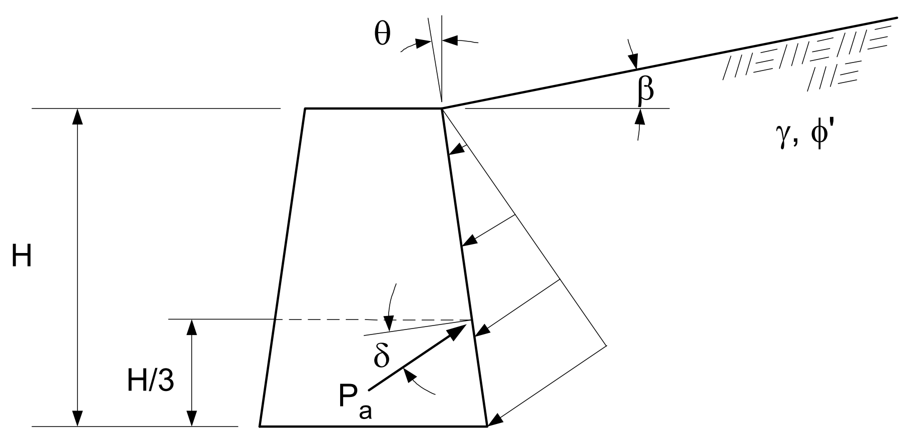


Figure 6-2: Coulomb Lateral Earth Pressure Definitions

6.3.2.2 At-Rest Pressure Distribution

The at-rest coefficient can be determined from the following equation developed by Mayne and Kulhawy (1982):

$$K_o = (1 - \sin\phi')(OCR)^{\sin(\phi')} \tag{6-4}$$

where: K_o = At-rest pressure coefficient
ϕ' = Effective angle of internal friction for the soil
OCR = Overconsolidation (preconsolidation) ratio (see Section 5.3.3)

Some agencies design abutment walls for at-rest earth pressures. However, if the wall is free to rotate such that the displacement at the top of the abutment is about 0.2 percent of the height of the wall (i.e., 0.002H), then active earth pressures can be assumed to develop.

6.3.2.3 Passive Pressure Distribution

The passive pressure distribution should be determined for special abutment cases. The passive pressure is generally ignored in front of the abutment due to the large lateral displacements required to fully mobilize passive lateral earth pressures and the potential for poor compaction or subsequent removal of the backfill. The passive earth pressure coefficient may be computed using Coulomb earth pressure theory or estimated from charts such as Figures 5.5.2C and 5.5.2D in the AASHTO "Standard Specifications" (1996).

6.3.2.4 Equivalent Fluid Methods

The AASHTO "Standard Specifications" (1996) state that structures that resist loads due to lateral earth pressures should not be designed for less than an equivalent fluid weight of 4.8 kN/m^3 (30 pounds per cubic foot). If the above methods yield pressures lower than this equivalent fluid weight, the equivalent fluid pressure should be used as a minimum. Note that these earth pressures only apply to gravity and semi-gravity cantilever walls.

6.3.2.5 Live Load Surcharge

If live loads (traffic) can occur within a horizontal distance behind the top of a wall or abutment wall equal to one-half of the wall height (measured from the heel of the footing), the lateral load for design should be increased to account for the additional lateral earth pressure that will act on the wall.

The live load surcharge is estimated as an equivalent soil surcharge behind the wall or abutment. However, if there is an approach slab, the live loads are applied to the corbel on the abutment backwall. The equivalent height of soil to use in calculating the live load surcharge is 0.6 m (2 ft). The unit weight for this imaginary surcharge is typically taken as 19.6 kN/m^3 (125 pcf). The pressure distribution from this equivalent soil height is uniform along the back of the wall, and the resultant is halfway up from the base of the wall. The lateral load due to live load surcharge is then calculated as:

$$P_{LS} = K\gamma' h_{eq} H \tag{6-5}$$

where P_{LS} = Lateral load due to live load surcharge, in units of force per unit length of wall
K = Lateral earth pressure coefficient, either at-rest, K_o, or active, K_a
γ' = Effective unit weight of soil acting against the wall or abutment
h_{eq} = Equivalent height of soil surcharge representing live load (0.6 m)
H = Height of retained backfill behind the wall or abutment

The general loading configuration for lateral earth pressures acting on walls or abutments is shown in Figure 6-3.

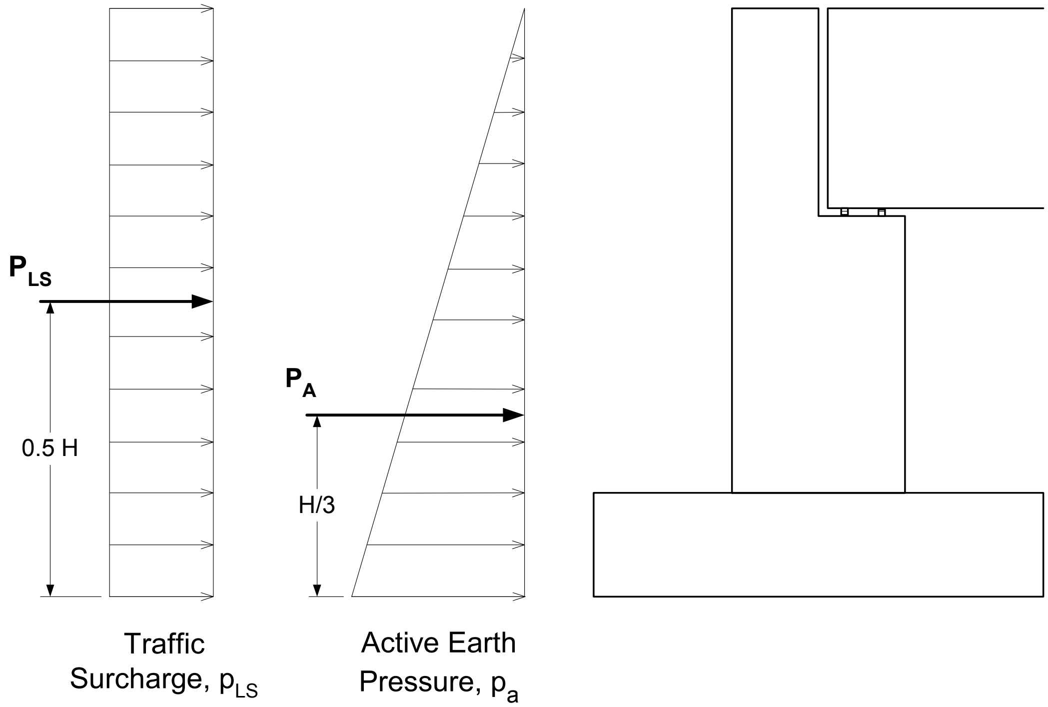


Figure 6-3: Lateral Earth Pressures on Abutments

6.4 SIZING THE FOOTING

Once all the loads acting on the footing have been calculated, the footing should be sized such that:

- The footing is stable under eccentrically applied loads (i.e., the footing will not overturn) and such that negative contact stress does not develop beneath the footing (soil cannot transfer tensile stress). This is determined by computing the effective footing dimensions (Section 6.4.1).

- The maximum applied bearing stress is less than the allowable bearing capacity, considering the potential for both shear failure and settlement. This is determined using the effective footing area computed per Section 6.4.1 and comparing it with the calculated allowable bearing capacity (Section 6.4.2).

- The footing will not slide due to lateral loads (Section 6.4.3).

6.4.1 Effective Footing Dimensions and Overturning Stability

Eccentric loading occurs when a footing is subjected to a combination of vertical loads and moments, or moments induced by shear loads transferred to the footing. Abutments and retaining wall footings are examples of footings subjected to this type of load condition. Moments can also be applied to interior column footings due to skewed superstructures, impact loads from vessels or ice, seismic loads or loading in any sort of continuous frame. The effect of the eccentric load is to distribute the load over a smaller area than the entire footing area. The eccentricity correction is usually applied by reducing the width (B_f) and length (L_f) such that:

$$B'_f = B_f - 2e_B \tag{6-6}$$

$$L'_f = L_f - 2e_L \tag{6-7}$$

In these equations, e_B and e_L are the eccentric distances (Figure 6-4) in the B_f and L_f directions, respectively, and are computed by dividing the applied moment by the applied vertical load. Recall that it is important to maintain consistent sign conventions and coordinate directions. The reduced footing dimensions B'_f and L'_f are termed the effective footing dimensions. When eccentric load occurs in both directions, the bearing pressure is assumed to act over a fictitious area (A') of the footing (AASHTO, 1998):

$$A' = B'_f L'_f \tag{6-8}$$

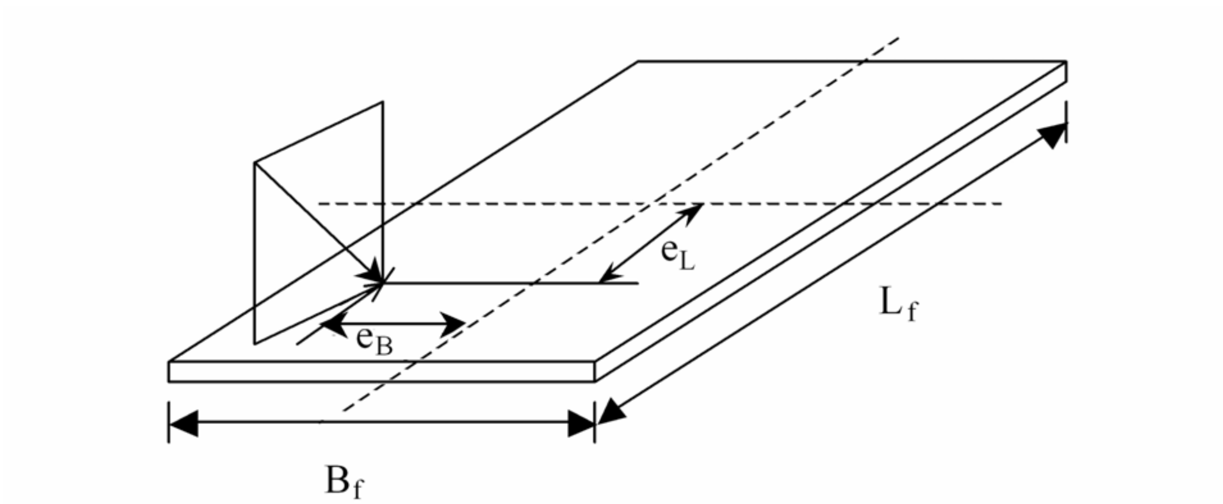
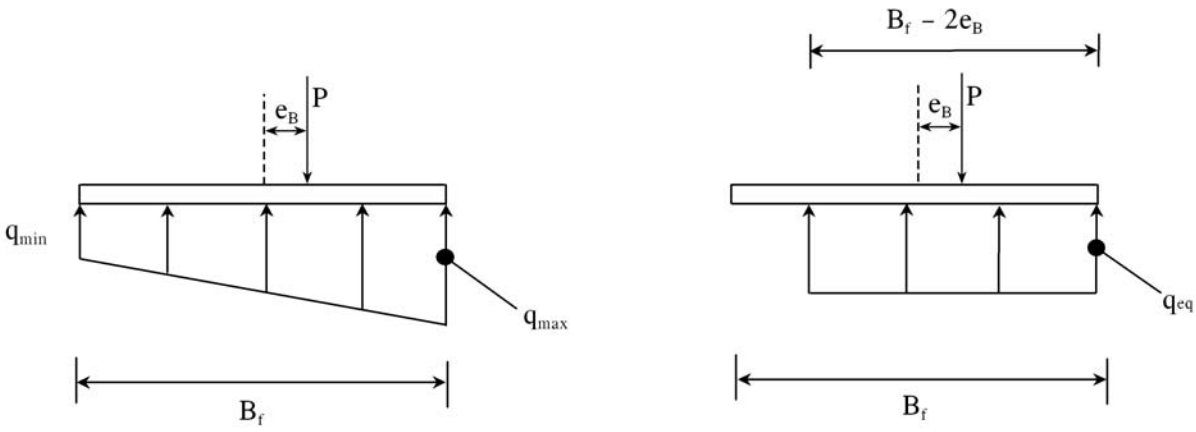


Figure 6-4: Notations for Footings Subjected to Eccentric, Inclined Loads (after Kulhawy, 1983)

Note that the concept of effective footing dimensions and area is an approximation made to account for eccentric loading and is not representative of the actual stress distribution under a footing subjected to eccentric loads. Structural design of the footing is performed using trapezoidal or triangular stress distributions that more conservatively model the stress distribution under an eccentrically loaded footing as described in Section 6.6. A comparison of the two loading distributions is shown in Figure 6-5.



(a) (b)

Figure 6-5: Eccentrically Loaded Footing with (a) Linear Pressure Distribution (Structural Design) and (b) Equivalent Uniform Pressure Distribution (Sizing the Footing)

Footings on soil should be designed such that the eccentricity in any direction (e_B or e_L) is less than one-sixth (1/6) of the (actual) footing dimension in the same direction to ensure that zero contact pressure does not exist beneath the footing (AASHTO, 1996). For footings on rock, the eccentricity should be less than one-fourth (1/4) of the (actual) footing dimension. If the eccentricity does not exceed these limits, separate calculation for stability with respect to overturning need not be performed.

6.4.2 Allowable Bearing Stress

Using the procedures in Sections 5.2 and 5.3, an allowable bearing capacity should be computed that provides a minimum factor or safety of 3.0 against bearing failure in shear and limits the settlement of the footing to a tolerable amount. It is typically convenient to present the allowable bearing capacity as a range of bearing stresses plotted as a function of (effective) footing widths. Alternatively, settlements could be computed and tabulated for a variety of applied stresses and footing widths.

An acceptable footing design may be obtained by sizing the footing such that the applied bearing stress over the effective footing area, A′, assuming a rectangular pressure distribution, is less than the allowable bearing pressure computed using the procedures in Chapter 5. Because the pressure distribution is assumed to be uniform, B_f in Equation 5-14 may be replaced by B'_f.

6.4.3 Sliding

The loads that act laterally on the footing should be compared to the allowable sliding resistance computed according to the procedures in Section 5.4. If the computed factor of safety is less than 1.5, the footing width should be increased. Addition of a key may also be considered, keeping in mind the limitations of key construction and effectiveness described in Section 5.4.

6.5 DESIGN OF SHALLOW FOUNDATIONS FOR INFREQUENT TRANSIENT LOADS

Bridge design may need to include consideration of loading conditions that rarely, if ever, occur during the life of the structure. Some of these load conditions can impose large loads on the structure and the foundations. Examples of loads that may impact bridge design include ice load, wind load, vessel impact and, notably, earthquake or seismic loads. These large loads can control the design and size of a foundation.

The performance objective of the structure, and therefore the foundations, should be selected before establishing the design criteria for the structure. It may be considered that structural collapse is not acceptable, but that some structural damage is tolerable if one of these transient loads is applied to the structure.

The concept of allowable bearing capacity that includes consideration of limiting both bearing failure in shear and settlement does not lend itself to design for infrequent transient loads. If some structural damage, but no collapse, is considered an acceptable result from the application of such a load, it is not reasonable to design the foundations for total settlement (i.e., settlement due to "normal" loads plus any additional settlement due to the transient load) that is limited to, say, 25 mm (1 in).

When considering infrequent transient loads Service Load Design provisions include an allowance for overstress above the allowable capacity of a member. For soils, however, the 125 to 150 percent overstress allowance means that there is still a factor of safety against a collapse (bearing failure in shear) of at least 2.0. Compared to other construction materials, such as concrete and steel, this is still a large margin of safety under this type of load. The design codes have evolved, and will continue to evolve, with regard to achieving consistent levels of performance and safety. One step in this process is AASHTO's adoption of the Load and Resistance Factor Design (LRFD) methodology (AASHTO, 1998), which includes provisions for considering the performance of a structure under typical, or service loads, that are generally controlled by deflection (e.g., settlement), and strength or extreme event loads that may be controlled by the ultimate strength of a material (e.g., ultimate bearing capacity). Further discussion of the LRFD method as applied to shallow foundations is included in Appendix C.

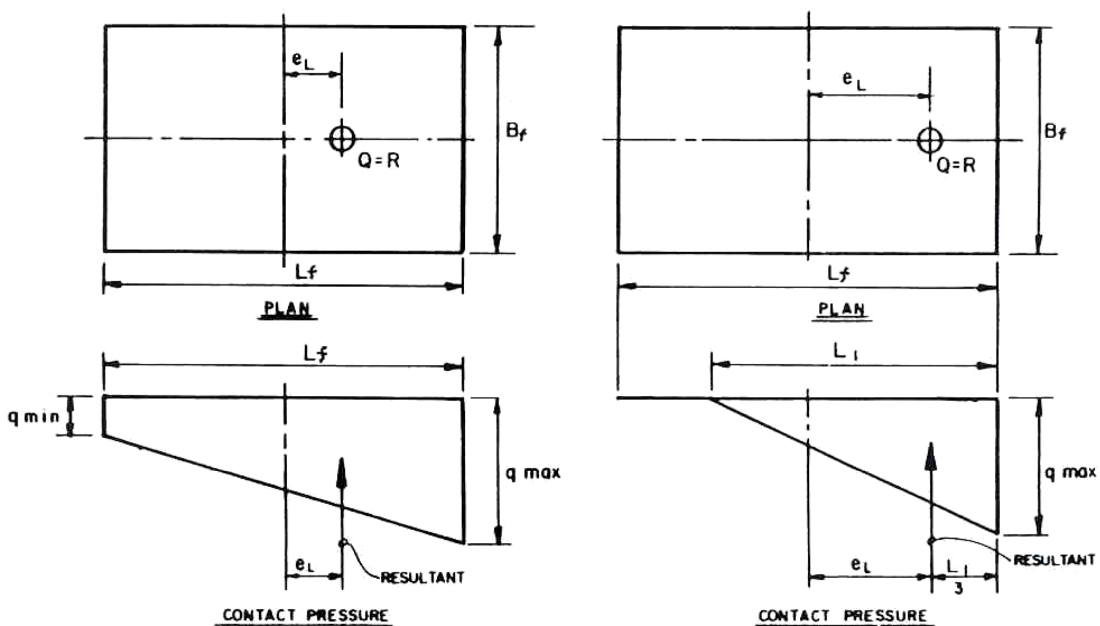
6.5.1 Soil–Structure Interaction Analyses and Shallow Foundation Stiffness

An additional consideration for shallow foundations designed to resist large, transient loads is the effect of soil–structure interaction. The loads calculated for a structure subjected to a large transient load will depend on how the structure responds to the application of the load. The structure response is, in turn, dependent on the stiffness of the foundation and the foundation soils. An accurate prediction of the loads will depend on an accurate prediction of the foundation stiffness.

Of particular interest to designers is the response of foundations to transient lateral loads, such as those applied by seismic loads or vessel impact. For shallow foundations, the foundation stiffness will therefore be a function of the shear modulus and Poisson's ratio of the soil. A detailed description of evaluating soil and foundation stiffnesses is not included here. The reader is referred to GEC No. 3: *Geotechnical Earthquake Engineering for Highways* (Kavazanjian et al., 1997) and the course materials and design examples developed for National Highway Institute's *Seismic Design of Bridges* (Mast et al., 1996) for guidance on calculating foundation loads due to transient lateral forces.

6.6 Structural Design of the Footing

The contact pressure beneath a spread footing is, in reality, a function of the stiffness of the footing and the compressibility and strength of the soil. However, the most common practical approach is to assume that the footing is rigid with a linear distribution of contact pressure. This results in either a trapezoidal or triangular shaped contact pressure distribution, as shown in Figure 6-6. The maximum and minimum pressures beneath the footing can be computed using Equations 6-9 through 6-13.



(a) (b)

Figure 6-6: Contact Pressure for Footing Loaded Eccentrically About One Axis: (a) for $e_L \leq L_f/6$, and (b) for $L_f/6 < e_L < L_f/2$ (AASHTO, 1996)

For $e_L \leq L_f/6$:

$$q_{max} = Q[1 + 6(e_L/L_f)]/B_fL_f \qquad (6\text{-}9)$$

$$q_{min} = Q[1 - 6(e_L/L_f)]/B_fL_f \qquad (6\text{-}10)$$

For $L_f/6 < e_L < L_f/2$:

$$q_{max} = 2Q/(3B_f[(L_f/2) - e_L]) \quad (6\text{-}11)$$

$$q_{min} = 0 \quad (6\text{-}12)$$

$$L_1 = 3[(L_f/2) - e_L] \quad (6\text{-}13)$$

where: L_1 = the length of the triangular-shaped contact pressure distribution

Note: When the eccentricity is in the B_f direction, the minimum and maximum contact pressures are computed with Equations 6-9 through 6-13 by replacing e_L with e_B, L_f with B_f, and B_f with L_f.

The structural design of the footing should then be performed in accordance with Section 8 of the AASHTO Standard Specifications (1996). The soil pressures used for this design are the customary trapezoidal or triangular pressures shown in Figure 6-6 and calculated using Equations 6-9 through 6-13, not the equivalent uniform rectangular pressures used to size the footing dimensions. The following failure modes should be considered during structural design of the footing:

- Consider the moment in the footing as a function of the column connection (i.e., pinned, fixed or sliding, an equivalent rectangular column is used).

- Increase moment (flexural) steel in the short direction as specified in the AASHTO Standard Specifications (1996).

- When compression is induced at the face of the column, investigate shear at "d" away from the column face in compression (where "d" is the full depth of the footing). If tension is induced at the face of the column, investigate shear at the face of the column.

- Consider punching or two-way shear around the column.

For abutments, each of the structural elements is designed similar to spread footings. In addition, the backwall and stem wall are designed for the design soil pressures that can be applied. Wingwalls, if included, are designed for the maximum soil pressures that can be applied. The connection of the wingwall to the stem wall is critical so that it will not fracture.

CHAPTER 7

MINOR STRUCTURE SUPPORT ON SHALLOW FOUNDATIONS

Transportation projects frequently include minor structures such as signs, signals and illumination. Because these structures are both relatively tall and lightweight, the controlling load for design of their foundations is nearly always overturning due to wind loading or cantilever arms. As a result, the moments applied at the foundation connection tend to be large as compared to the axial load due to the dead weight of the structure.

Spread footings must resist these moments by transferring the moment to an effective bearing area that is small due to the large eccentricity; therefore, spread footings tend to be large for support of signs, signals and illumination. Most agencies have developed standard plans for supporting minor structures on small, shaft-type foundations that are more efficient at resisting the applied lateral shears and moments than spread footings because they spread the load over a large, lateral surface area.

However, there are some ground conditions, such as very soft soils, that may lend themselves to minor structure support on a spread footing. Because the axial loads are not very large for minor structures, settlement considerations may not govern the allowable bearing capacity. An example might be a deep soft clay or organic silt deposit that provides poor near-surface lateral resistance against a shaft or pile foundation. In this situation a large spread footing could be a cost-effective alternate to a pile group.

7.1 Geotechnical Investigation

The geotechnical investigation for minor structures should include the same considerations as presented in Chapter 4, but the depth and frequency of explorations may be reduced according to identified similar geologic conditions for groups of structures.

7.2 Allowable Bearing Stresses

Allowable bearing stresses for minor structure support on shallow foundations follow the same principles and procedures presented in Chapter 5.

7.2.1 Settlement Limitations

Settlement considerations for minor structures should include determining how much the foundation might tilt when supporting a tall, slender structure. Visually perceptible tilt of signals and illumination towers, while not necessarily a problem for the function of the structure, will certainly be noticeable, and reported, by the public. As mentioned above, however, the controlling design load may be wind, which is transient and therefore not likely to be of sufficient duration to induce consolidation settlement of cohesive soils. If the structure includes a cantilever arm, however, consolidation settlement should be computed and kept within tolerable limits.

The potential for differential settlement of sign bridge supports should be checked if the applied bearing stresses are sufficient to induce consolidation settlement.

7.2.2 Bearing Capacity Limitations

As mentioned in the introduction to this section, shallow foundations for minor structures may be cost-effective when deep, soft soils are present. Punching shear mechanisms can develop in these types of soils. This is a situation in which the one-third reduction in the soil strength parameters discussed in Section 5.2.3.6 should be included when computing the ultimate bearing capacity.

7.3 OVERTURNING

As mentioned in Section 6.4.1, the factor of safety against overturning does not need to be calculated if the eccentricity is within certain prescribed limits that ensure negative bearing stress does not occur. For lightweight structures that have large overturning moments, these design criteria may be overly conservative. Recall that the assumption of the actual distribution of stress beneath a spread footing is that the footing is perfectly rigid with respect to the soil. For wide spread footings that would be needed to withstand the overturning moment applied by, say, a signal mast arm, the assumption of perfect rigidity is not valid. Both the footing and the supporting soil will deform under the applied load, and the stresses will redistribute accordingly. If the eccentricity is allowed to exceed the limits stated in Section 6.4.1 for a minor structure foundation, then a separate computation for overturning should be performed. A minimum factor of safety against overturning (i.e., the resisting moments divided by the moments acting to overturn the structure) should be greater than about 2.0 for permanent dead loads and greater than about 1.5 for infrequent transient loads such as the maximum wind force.

7.4 SLIDING

The design of shallow foundations supporting minor structures should be checked, following the procedures in Sections 5.4 and 6.4, so that sliding will not occur.

CHAPTER 8

SHALLOW FOUNDATION DESIGN FOR BUILDINGS

Foundation design for buildings is typically performed in accordance with the current version of the Uniform Building Code (UBC) or the International Building Code (IBC). These codes provide minimum criteria for design of building foundations, including considerations for frost protection; maximum presumptive bearing stresses; and dead, live, wind, seismic and other loads.

8.1 GEOTECHNICAL INVESTIGATION

The field and laboratory procedures for geotechnical investigation of a building site are the same as described in Chapter 4. The extent of sampling and testing should be commensurate with the performance expectation and importance of the structure. A maintenance facility, for instance, may include several one- and two-story structures, including an equipment bay, that are constructed as slabs-on-grade with spread footing support of lightweight column and wall loads. The subsurface investigation for this type of facility should be sufficient to describe the vertical and lateral extent and variability of foundation bearing soils beneath the structures. A 1,500 to 2,000 m^2 facility will typically require a minimum of three to five borings or test pits that extend at least 3 to 9 m (10 to 30 ft). The required quantity of subsurface information will depend on the geotechnical engineer's confidence that the subsurface conditions are adequately defined to proceed with confidence in designing a shallow foundation system.

8.2 SHALLOW FOUNDATION TYPES

The types of shallow foundations used to support buildings are similar to those described in Chapter 2, with the notable difference being generally much smaller dimensions.

8.2.1 Isolated Spread Footings

Isolated spread footings are frequently used to support individual or occasionally multiple column loads for buildings. Depending on the anticipated potential for differential movements or the potential for high floor loads, the footings may or may not be constructed integral with slabs-on-grade. It may be preferable for slabs to bear on the subgrade soils independently from footings. For lightly loaded structures on compressible soil, it can be economical to partially overexcavate the poor foundation soils and replace them with compacted granular backfill to limit the potential for settlement or bearing capacity problems.

8.2.2 Continuous Strip Footings

Continuous strip footings are frequently used to support perimeter and load-bearing walls. Where designed and constructed continuous with slabs-on-grade, the footings can be built as thickened edges to achieve the minimum depth requirements for bearing capacity and frost protection.

8.2.3 Mat Foundations

Mats are heavily reinforced concrete foundations that usually cover the entire plan area of a building. Mat foundations may be designed to minimize differential settlements. Mats are usually more economical and preferable when column loads are so large that more than 50 percent of the plan area would be covered by spread footings (Peck et al., 1974). Mats can be used to create deep basements, distribute column loads more uniformly and provide basement slabs (water barrier). Mats may be used when the foundation soil has a low bearing capacity or large differential settlements are anticipated.

Mats, by design, are rigid elements that distribute column and wall loads over a larger area, thus reducing the magnitude of the vertical stress on the foundation soil. This not only decreases the stress but also produces a more uniform settlement profile and subsequently reduces differential settlement. Figures 8-1(a) and (b) present the pressure distribution profiles versus depths for footing and mat foundations, respectively, assuming that the total load from the superstructure is the same for both a and b. As shown in Figure 8-1, the applied stress on the subgrade soil directly below the spread footings is higher than directly below the mat; however, this condition reverses with depth. Additionally, a mat foundation is preferred for a subsurface soil profile containing erratic changes in strength and/or stiffness (Figure 8-2) because the mat tends to bridge over these erratic loose zones better than isolated footings and is expected to undergo smaller differential displacements (about 50 percent less).

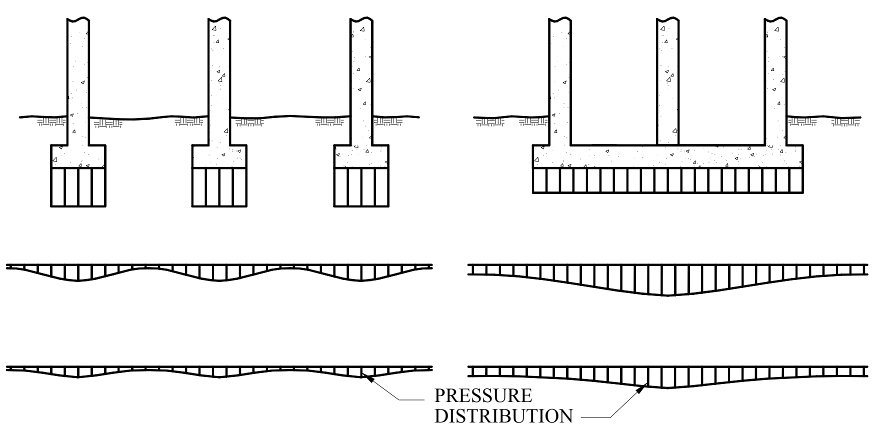


Figure 8-1: Distribution of Pressure in Soil Beneath Buildings Supported by (a) Widely Spaced Footings and (b) Concrete Mat

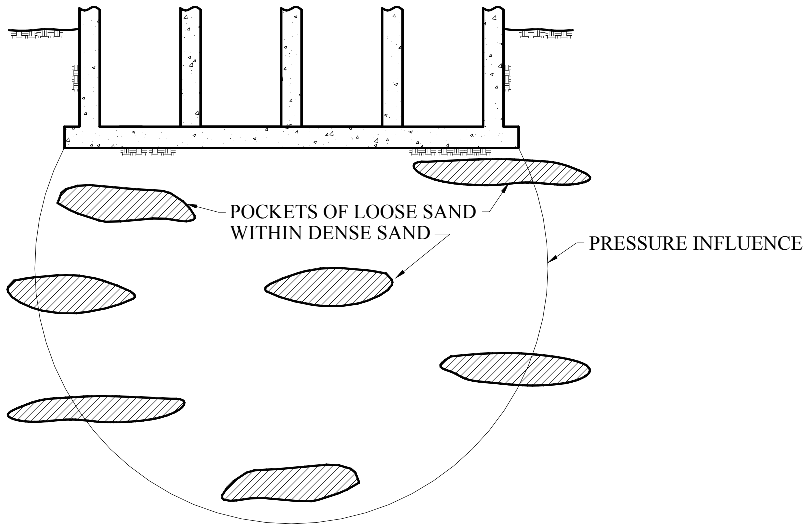


Figure 8-2: Mat Foundation on a Dense Sand Stratum with Erratic Distribution of Pockets of Loose Sand

8.3 ALLOWABLE BEARING STRESSES

8.3.1 Settlement Limitations

Tolerable total and differential movements for buildings, like bridges, are strongly a function of the materials and methods used to construct the building. Steel frame buildings are capable of withstanding surprisingly large differential movements without experiencing structural distress or damage. However, such movement may result in objectionable appearances of wall facings and coverings. Masonry structures, on the other hand, are relatively settlement intolerant and may experience structural distress (cracking) if angular distortions exceed 1/300.

In practice, bearing stresses are typically limited so that total settlements are on the order of 25 mm (1 in) and differential settlements about half the total settlement.

In designing building foundations, considerable benefits can be derived by permanently excavating a portion of the soil by including a basement. This reduces the net increase in stresses on the subgrade, which in turn reduces or eliminates consolidation even in normally consolidated soils.

8.3.2 Bearing Capacity Limitations

Due to the relatively small width of spread footings for buildings, the potential for bearing capacity failure may control the design of footings, especially foundations on clay. Knowing that most building foundations for lightweight one- and two-story frame construction have widths of about 0.5 to 1.0 m, and referring to Figure 5-1, it can be seen that these values fall to the left, in the bearing capacity limited portion of the chart.

8.4 OVERTURNING AND ECCENTRICITY

Spread footings for buildings are designed to maintain a minimum factor of safety against overturning of at least 1.5. Eccentricity may also be checked using the procedures in Chapter 6, although this is generally not part of the code requirements for design of buildings.

8.5 SLIDING AND PASSIVE RESISTANCE

The procedures for calculating sliding and passive resistance for building spread footings are described in Chapter 5. A minimum factor of safety of 1.5 should be provided for sliding stability. Due to the generally small width of spread footings used for buildings, inclusion of the passive resistance may be necessary. Because of the differences in strains required to develop sliding and passive resistance, as described in Section 5.4, a practical approach is to include only one-half of the computed passive resistance when combining both sliding and passive components. For narrow footing widths, simply deepening the footing to increase passive resistance may be more practical than attempting construction of a key.

8.6 DESIGN OF MAT FOUNDATIONS AND FLOOR SLABS

The design of mat foundations should account for bearing capacity, as well as immediate and long-term settlements. The bearing capacity and settlement of a mat foundation may be determined using the methods presented in Chapter 5. Due to the large size of mat foundations, bearing capacity will generally not govern design. Controlling deflections will usually be the most critical aspect of design. Mat settlements can be controlled to some degree with the lower applied soil pressures as compared to isolated or strip spread footings. Also, the volume of soil displaced by the foundation will result in a lower net applied stress that will limit settlement.

Mat foundations on cohesive soils (clays) may be controlled by deep-seated bearing capacity. The subsurface explorations must be carried to sufficient depth to assess both deep-seated bearing capacity and the zone of stress influence (Section 5.3) for settlement computation. Due to the large dimensions of a mat, the zone of influence will be deeper than for isolated or strip spread footings.

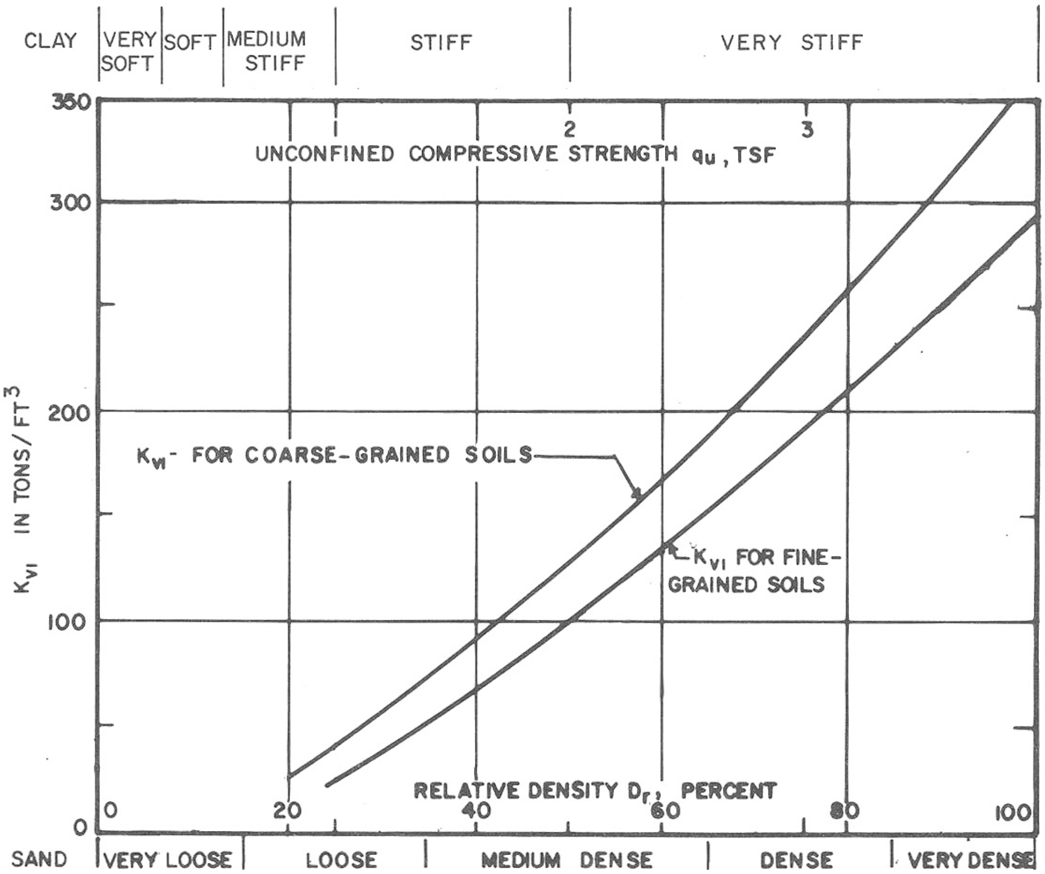
A common design approach for mat foundations uses beam on elastic foundation theory to compute the soil-structure interaction and deflections. The reinforcement of the mat can then be designed based on the deflections computed within the mat. The beam on elastic foundation approach models the support of foundation soils as a bed of discrete elastic springs (Winkler foundation). The soil reaction at any particular point beneath a mat is:

$$q_{applied} = K_v y \tag{8-1}$$

where:
$q_{applied}$ = Applied stress
K_v = Modulus of subgrade reaction (force/length^3)
y = Vertical displacement

The modulus of subgrade reaction can be obtained from Figure 8-3. Note that the notation for the modulus is K_{V1} in Figure 8-3. The modulus of subgrade reaction can also be obtained from plate load tests but should be corrected for size and shape effects according to Figure 8-4.

The validity of the beam on elastic foundation approach has been called into question due to the fundamental assumption regarding the relative rigidity of the mat as compared to the soil. Also, the modulus of subgrade reaction is not an intrinsic soil property, and is a function not only of the stiffness of the soil, but the stiffness, shape and depth of the mat. Despite these limitations, the method is still a practical and commonly used way of computing displacements to design the structural reinforcement of the mat.



DEFINITIONS

ΔH_i = IMMEDIATE SETTLEMENT OF FOOTING
q = FOOTING UNIT LOAD IN tsf
B = FOOTING WIDTH

D = DEPTH OF FOOTING BELOW GROUND SURFACE
K_{VI} = MODULUS OF VERTICAL SUBGRADE REACTION

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)

SHALLOW FOOTINGS $D \leq B$

FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{4\,q\,B^2}{K_{VI}\,(B+1)^2}$$

FOR $B \geq 40$ FT:

$$\Delta H_i = \frac{2\,q\,B^2}{K_{VI}\,(B+1)^2}$$

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$

FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{2\,q\,B^2}{K_{VI}\,(B+1)^2}$$

NOTES: 1. NONPLASTIC SILT IS ANALYZED AS COARSE-GRAINED SOIL WITH MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH.
2. VALUES OF K_{VI} SHOWN FOR COARSE-GRAINED SOILS APPLY TO DRY OR MOIST MATERIAL WITH THE GROUNDWATER LEVEL AT A DEPTH OF AT LEAST 1.5B BELOW BASE OF FOOTING. IF GROUNDWATER IS AT BASE OF FOOTING, USE $K_{VI}/2$ IN COMPUTING SETTLEMENT

Figure 8-3: Modulus of Subgrade Reaction (NAVFAC, 1986a)

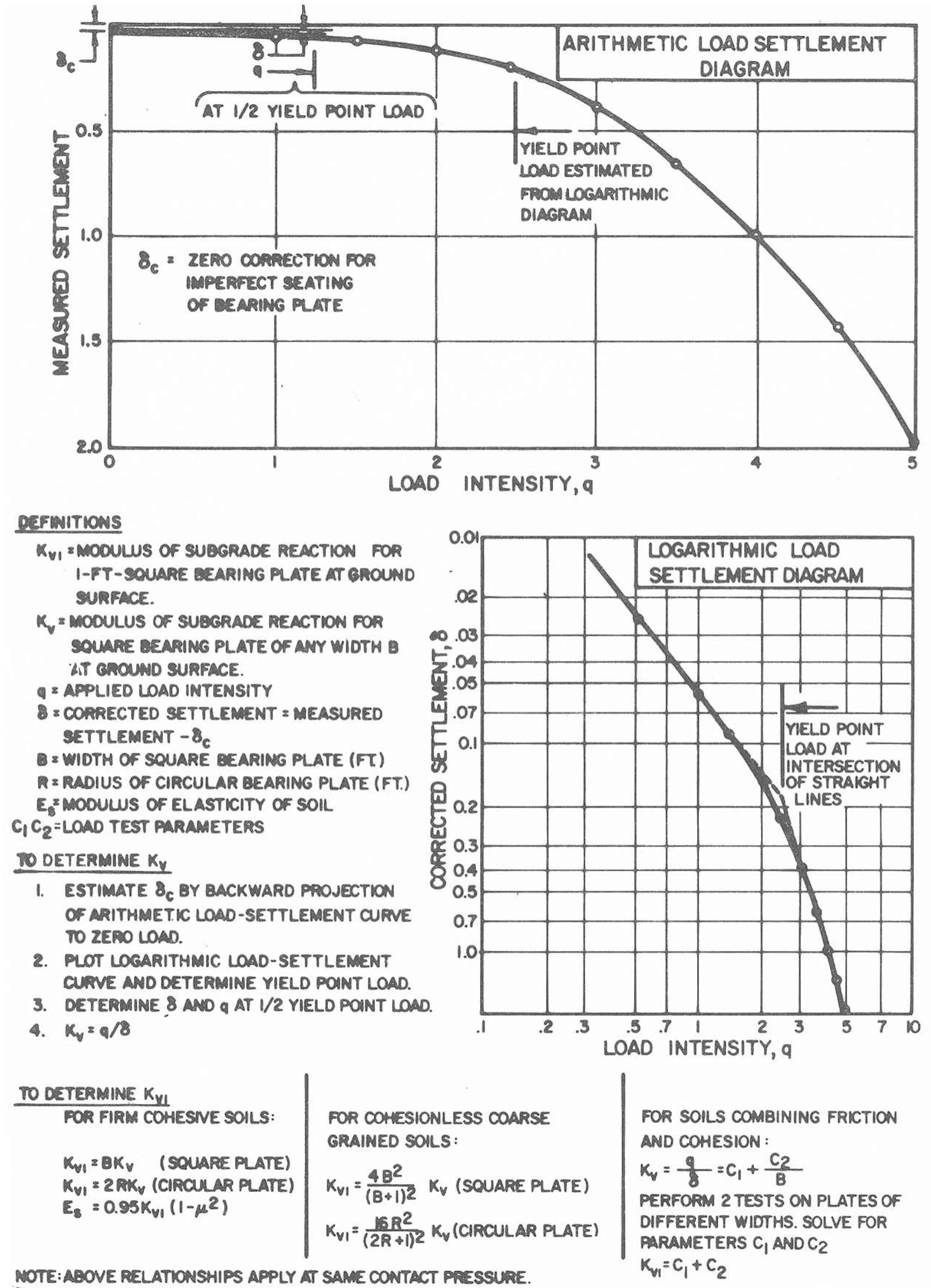


Figure 8-4: Analysis of Plate Load Test Results (NAVFAC, 1986a)

(This page intentionally blank)

CHAPTER 9

CONSTRUCTION DOCUMENTS

9.1 Design-Bid-Build Procurement

9.1.1 Plans, Specifications and Estimates

Plans are typically developed to show footing locations, dimensions and details for steel reinforcement, required footing elevations, and the embedment and backfill dimensions. A foundation data sheet is usually provided to show summary subsurface data in spatial relation to the foundations. Material descriptions, ground water levels, SPT data and other subsurface investigation information are particularly valuable to the contractor for evaluation of excavation, shoring, dewatering and other construction details that are usually left up to the contractor to design or select. The geotechnical report should be made available to contractors.

The plans should detail the drainage and backfill requirements for abutments and retaining walls. For bridges, the abutment area is critical because it performs many functions. To prevent the notorious "bump at the end of the bridge," the abutment backfill must be properly compacted and not prone to material loss due to migration of fine-grained material. Refer to the *Soils and Foundations Workshop Reference Manual* (Cheney & Chassie, 2000) for specific details regarding materials that can be specified to promote good compaction of the backfill. Underdrain filter material or prefabricated geocomposite drains can be specified to prevent the fine-grained soil migration problem. Drainage details should be provided to prevent build up of pore water pressures behind the stemwall.

Specifications typically include requirements for concrete, steel reinforcement and construction procedures. The FHWA Standard Specifications (FHWA, 1996) include Section 208, "Structure Excavation and Backfill for Selected Major Structures," Section 209, "Structure Excavation and Backfill," Section 552, "Structural Concrete" and Section 554 "Reinforcing Steel." Items covered by these specifications include material requirements, preparation and compaction of a suitable bearing level in soil or rock, temporary cut slopes and shoring, requirements for shop drawings, protection of foundation conditions during construction, concrete delivery and placement, curing requirements and acceptance requirements.

The specifications should reference the availability of the geotechnical report. Special provisions might be necessary where special foundation soil/rock treatment or ground water control is required. Soft or liquefiable foundation soils may need special treatment to develop firm foundation conditions. When constructing in tight and sensitive areas, such as next to existing structure foundations, special shoring requirements might be necessary.

Pay items may include Structure Excavation, Foundation Fill, Structural Backfill, Shoring and Bracing and Cofferdams. The actual footings may be considered incidental to structure construction. Alternatively, concrete and steel would be measured and paid for according to unit quantities and prices.

9.1.2 Structural Fills

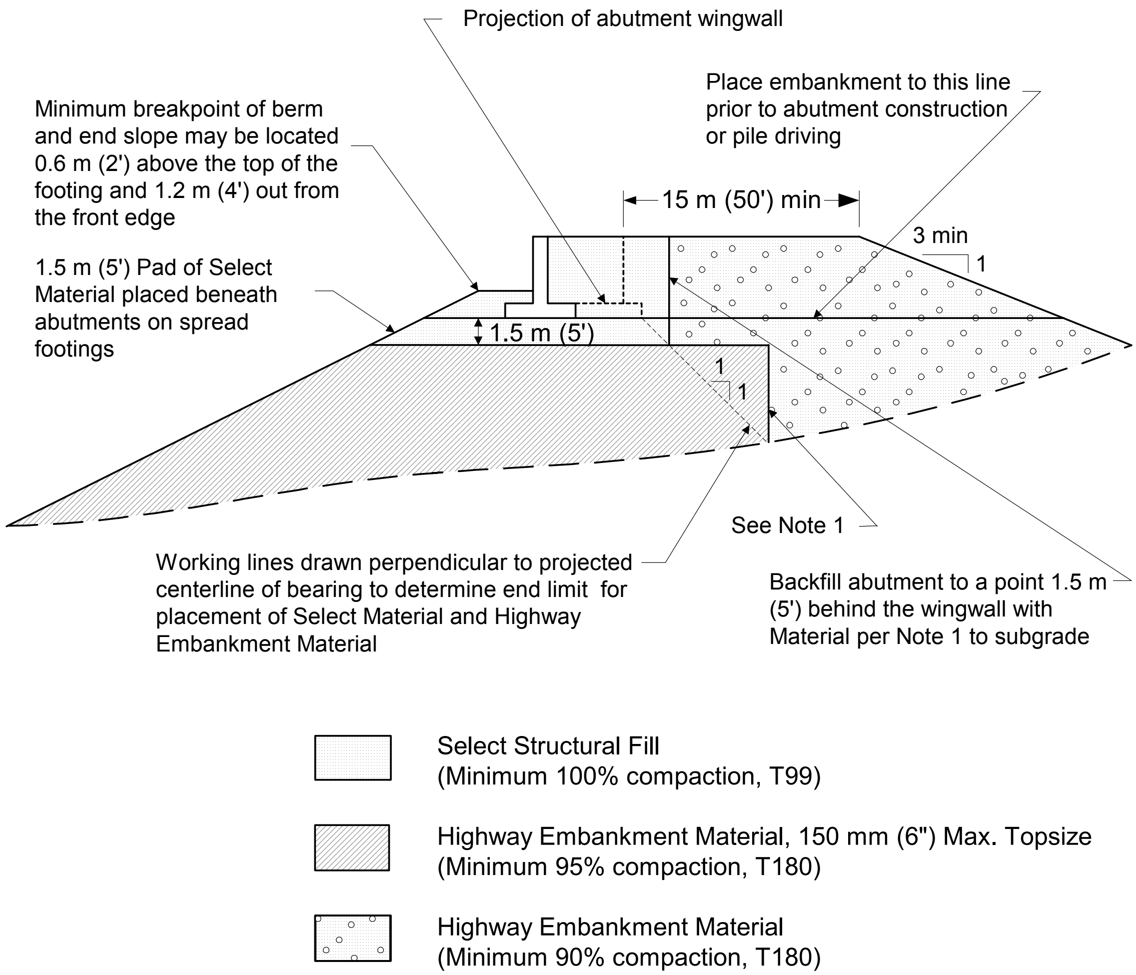
Special details and specifications are necessary when constructing a compacted structural fill to support a bridge foundation such as an abutment. Figure 9-1, adapted from Cheney and Chassie (2000) shows recommended details for construction of a structural approach embankment. Note the zones of specified materials and compaction requirements. The materials specifications are chosen to provide a material that is capable of being compacted to a firm, non-yielding condition that will be suitable for support of a bridge abutment spread footing. The specification for select material recommended in the *Soils and Foundations Workshop Reference Manual* (Cheney & Chassie, 2000) is included in Appendix A. The specifications used by the Washington, Nevada and Michigan state transportation departments for structural fills beneath spread footing abutments are also included in Appendix A.

Select structural fill and highway embankment material should not include unsuitable material. The specifications for these materials should prohibit unsuitable or deleterious material such as peat, muck, wood, organic waste, coal, charcoal or any other material that would perform poorly in an embankment.

9.2 DESIGN-BUILD PROCUREMENT

The design-build procurement approach should include owner-provided geotechnical data, to the extent sufficient for normal foundation design. A Geotechnical Data Report (GDR) is recommended. Geotechnical Interpretive Reports and Geotechnical Baseline Reports may also be prepared to document the expected surface and subsurface conditions. These documents will help to reduce uncertainty during bidding and reduce the potential for risk contingencies. The GDR should identify any special conditions that might affect structural fill and shallow foundation design/construction and should provide the test results for necessary geotechnical properties. Sometimes the alignment and structure locations are not fixed at the time of the Design-Build procurement; therefore, the contractor may need to provide supplemental explorations and testing.

Special Provision specifications should include the requirements the contracting agency wants to impose on the contractor to achieve the desired foundation performance – for example, codified design criteria, tolerable settlement criteria, estimated scour depths, seismic criteria and adherence to standard specifications. A submittal and review process should be prescribed, along with a minimum quality control and assurance plan.



Note 1: Highway embankment material and select material shall be placed simultaneously of the vertical payment line

Figure 9-1: Structural Approach Fill Details (after Cheney & Chassie, 2000)

(This page intentionally blank)

CHAPTER 10

INSPECTION AND CONSTRUCTION MONITORING

Because of the limitations of geotechnical explorations and geologic interpretations, there is always the possibility that significant deviations from anticipated subsurface conditions might be encountered. In addition, construction operations can change the behavior of in-place soils, as well as the foundation support and stability of nearby facilities. Therefore, observations by an experienced geotechnical engineer during construction are recommended to identify additional concerns in the field and to provide adjustments to the design when necessary.

The contracting agency's construction personnel normally provide inspection and monitoring. The geotechnical report should discuss the anticipated ground conditions at the footing bearing level. In addition, the geotechnical engineer should explain how to identify the expected ground conditions to the inspection staff and be sure the inspectors know how to contact the designers if any questionable conditions are observed. Geotechnical staff may need to make site visits to observe questionable subsurface conditions and to oversee specialized instrumentation.

Additional discussion and guidance regarding inspection and monitoring can be found in the Guide to Earthwork Construction (TRB, 1990).

10.1 INSPECTION

Inspection of foundation construction is only as good as the qualifications and experience of the inspector. Most problems can be avoided by having, in addition to a qualified and cooperative contractor, a competent inspector who follows systematic inspection and documentation procedures and communicates issues with the foundation designer. The inspector should be aware of conditions that could affect the intended performance of the foundation and structure. This can be accomplished by reviewing the plans and specifications, along with the geotechnical report, and consulting with the geotechnical engineer. Foundations are usually designed to:

- Perform within settlement tolerances,
- Maintain an adequate safety margin with respect to bearing capacity failure,
- Provide frictional resistance against sliding,
- Provide resistance against lateral earth pressures, and
- Provide sufficient support to prevent slope instability.

Accepting materials or procedures different from those specified could compromise the design intent and potentially lead to distress or failure. Exceeding a criterion can sometimes lead to problems such as over-compaction behind an abutment wall, which could lead to increased stresses on the foundation element.

Ground water impacts can be serious, possibly contributing to the disturbance of in-place soils. When dewatering operations may be necessary, they should be mentioned in the construction considerations section of the geotechnical report. Ground water control using sumps will have different effects on foundation soils and slopes than the use of dewatering wells. For instance, the stability of temporary sloped excavations may be compromised if water is allowed to seep directly from the sidewalls of the excavation for removal from a sump. In this case dewatering wells would locally lower the ground water level and improve the stability of the slopes. Conversely, dewatering in a clay soil profile can induce consolidation settlement by the resulting increase in vertical effective stress, a condition the geotechnical engineer may not have allowed for or considered in the settlement computations.

Improper construction methods and procedures can potentially lead to foundation and slope failures. The inspector should regularly (e.g., daily) observe foundation and slope conditions for potential weaknesses that may need to be observed and addressed by the geotechnical engineer.

Surface water and precipitation can deteriorate the condition of foundation bearing materials. If the ground conditions expected at the bearing elevation are considered moisture sensitive, the geotechnical report should identify this potential and make a recommendation that the bearing materials be protected from the effects of surface moisture. Some rocks and IGMs can also be adversely affected by moisture, seepage and relaxation due to excavation.

The bottoms of footing excavations can be probed with small diameter rods to confirm bearing resistance. More quantitative information can be obtained using cone penetration tests and nuclear densometers. Test pit excavation into and below the bearing stratum should be avoided due to the difficulty in compacting the backfill.

Routine inspection should include checking that procedural requirements are followed, materials are provided that meet performance criteria and satisfactory compaction/density test results are obtained. The inspector should review the geotechnical report for special issues that should be observed during construction.

10.2 MATERIALS

10.2.1 Line, Grade and Bearing Elevation

The agency inspector should confirm that surveyed locations and elevations of the footing elements conform to the plans and shop drawings. Where foundation elevations and location vary from those specified, the actual dimensions should be documented and then checked with the designer for concurrence.

10.2.2 Bearing Stratum

The in-place bearing stratum of soil or rock should be checked to verify the in situ condition and the degree of improvement achieved by the contractor's preparation approach. Some soil types can become remolded and weakened from disturbance. If the conditions deviate from those anticipated in the geotechnical report and/or the plans and specifications, the geotechnical engineer should be consulted to determine if additional measures are necessary.

10.2.3 Structural Fill Materials

The fill requirements should be strictly adhered to because the fill must perform within expected limits with respect to strength (bearing capacity) and, more importantly, within tolerance for differential settlement. Sometimes the area for construction of the fill is small, such as abutment and wingwall backfill, which may necessitate the use of smaller compaction equipment.

10.3 MONITORING

When monitoring the construction of structural fills that will support shallow foundations, particular attention should be paid to the following:

- The material should be tested for gradation and durability at sufficient frequency to ensure the material placed meets the specification.

- The specified level of compaction must be obtained in the fill. Testing, if applicable, should be performed in accordance with standard operating procedures and at the recommended intervals or number of tests.

- If a surcharge fill is required for pre-loading, it should be verified that the unit weight of the surcharge fill meets the value assumed in design.

The elevation of constructed foundations should be checked before and after the structural load is applied. This will serve as a baseline for long-term bridge monitoring. Subsequently, additional survey measurements could be made to confirm satisfactory performance or to identify whether potentially harmful settlements are occurring. It may be important to check the completion of fill settlements before foundation construction if soft compressible soils exist below (e.g., settlement plates, horizontal inclinometers, etc.). The lateral displacement potential can be greater than the vertical movements; therefore, if conditions warrant, monitoring may also include complete survey coordinates and possibly more accurate instrumentation.

Monitoring may also be necessary to evaluate the impact on neighboring ground and facilities. Such concerns could be monitored with simple survey tag lines (with benchmark and monitoring hubs) and telltales to measure lateral deviations and vertical subsidence/heave. Higher reliability expectations may require more sophisticated instrumentation, such as inclinometers, strain gages, extensometers and tiltmeters. Pre-existing condition surveys of neighboring structures could be valuable, particularly in congested urban areas.

The instrumentation program should be developed considering the anticipated performance, risks and potential consequences. Parameters should be identified that are critical to project success and appropriate instrumentation selected. A key to successful use of instrumentation is to measure, plot and interpret the data in a timely manner to be able to take corrective measures, if needed.

(This page intentionally blank)

REFERENCES

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

AASHTO (1994). "LRFD Bridge Design Specifications, 1st Ed.," American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1996). "Standard Specifications for Highway Bridges, 16th Ed. (with 1997, 1998, 1999 and 2000 Interim Revisions)," American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1998). "LRFD Bridge Design Specifications, 2nd Ed. (with 1999, 2000 and 2001 Interim Revisions)," American Association of State Highway and Transportation Officials, Washington, D.C.

Arman, A., Saintani, N., Castelli, R., and Munfakh, G. (1997). "Geotechnical and Foundation Engineering, Module 1 – Subsurface Investigations," *Publication No. FHWA NHI-97-021,* NHI Course No. 13231, Federal Highway Administration, National Highway Institute, Arlington, Virginia.

Armour, T., Groneck, P., Keeley, J. and Sharma, S. (2000). "Micropile Design and Construction Guidelines Implementation Manual," *Publication No. FHWA-SA-97-070*, Federal Highway Administration, Washington, D.C.

ASCE (1994). "Settlement Analysis," *Technical Engineering and Design Guides as Adapted from the U.S. Corps of Engineers*, American Society of Civil Engineers, No. 9.

Bazaraa, A.R.S. (1967). "Use of the Standard Penetration Test for Estimating Settlements of Shallow Foundations on Sand," Ph.D. Thesis presented to University of Illinois, Urbana.

Bieniawski, Z.T. (1984). *Rock Mechanics Design in Mining and Tunneling,* A. A. Balkema Publishers, Rotterdam.

Bieniawski, Z.T. (1989). *Engineering Rock Mass Classifications,* John Wiley & Sons, New York.

Bishop, A.W. (1955). "The Use of the Slip Circle in the Stability Analysis of Slopes." *Geotechnique*, Vol. 5, No. 1, pp. 7-17.

Bjerrum, L. and Simons, N.E. (1960). "Comparison of Shear Strength Characteristics of Normally Consolidated Clays," *Proceedings,* ASCE Research Conference on Shear Strength of Cohesive Soils, pp. 711-726.

Bowles, J.E. (1982). *Foundation Analysis and Design*, 3rd Ed., McGraw-Hill Book Co., New York.

Briaud, J.-L. (1989). "The Pressuremeter Test for Highway Applications," *Publication No. FHWA-IP-89-008*, Federal Highway Administration, Washington, D.C.

Briaud, J.-L. (1992). *The Pressuremeter*, A. A. Balkema Publishers, Brookfield, Vermont.

Briaud, J.-L. and Gibbens, R. (1994). "Predicted and Measured Behavior of Five Spread Footings on Sand," *ASCE Geotechnical Special Publication No. 41*.

Briaud, J.-L. and Gibbens, R. (1995). "Large Scale Load Tests and Data Base of Spread Footings on Sand," *Publication No. FHWA-RD-95-000*, Federal Highway Administration, Washington, D.C.

Briaud, J.-L., Hossain, K., Barfknecht, J. and Lee, J.H. (2000). "Load Settlement Curve Method for Spread Footings on Sand," *Publication No. FHWA-RD-2000-XXX*, Federal Highway Administration, Washington, D.C.

Brinch Hansen, J. (1970). "A Revised and Extended Formula for Bearing Capacity," *Bulletin No. 28*, Danish Geotechnical Institute, Copenhagen.

Burland, J.B. and Burbidge, M.C. (1984). "Settlement of Foundations on Sand and Gravel," *Proc. Instn Civil Engrs,* Part 1, 78, December, pp. 1325-1381.

Canadian Foundation Engineering Manual (1992). 3rd Ed., Canadian Geotechnical Society, Technical Committee on Foundations, c/o BiTech Publishers Ltd., Vancouver, B.C.

Carter, J.P., and Kulhawy, F.H. (1988). "Analysis and Design of Foundations Socketed into Rock," *Report No. EL-5918*, Empire State Electric Engineering Research Corporation and Electric Power Research Institute, New York, N.Y.

Caquot, A. and Kerisel, F. (1948). "Tables for the Calculation of Passive Pressure, Active Pressure and Bearing Capacity of Foundations," Gauthier-Villars, Paris.

Casagrande, A. (1936). "The Determination of the Preconsolidation Load and Its Practical Significance," Discussion 34, *Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering*, Cambridge, III, pp. 60-64.

Chen, Wai-Fah and McCarron, W.O. (1991). "Bearing Capacity of Shallow Foundations," *Foundation Engineering Handbook,* 2nd Ed., Van Nostrand Reinhold, New York, pp. 144-165.

Cheney, R.S. and Chassie, R.G. (2000). "Soils and Foundation Workshop Reference Manual," *Publication No. FHWA NHI-00-045*, Federal Highway Administration, Washington, D.C.

Christian, J.T. and Carrier, W.D. (1978). "Janbu, Bjerrom and Kjaernsli's Chart Reinterpreted," *Canadian Geotechnical Journal*, Vol. 15.

Coduto, D.P. (1994). *Foundation Design: Principles and Practices*, Prentice Hall, Inc., Englewood Cliffs, New Jersey.

D'Appolonia (1989). "Recommended Specifications for the Design of Foundations, Retaining Walls and Substructures," *Report No. NCHRP 12-35,* National Cooperative Highway Research Program, Transportation Research Board and National Research Council.

D'Appolonia, D.J., D'Appolonia, E. and Brissette, R.F. (1968). "Settlement of Spread Footings on Sand," *Journal of Soil Mechanics and Foundation Division,* ASCE, Vol. 94 (SM3).

D'Appolonia, D.J., D'Appolonia, E. and Brissette, R.F. (1970). "Settlement of Spread Footings on Sand," (closure), *Journal of Soil Mechanics and Foundation Division,* ASCE, Vol. 96 (SM2).

DiMillio, A.F. (1982). "Performance of Highway Bridge Abutments Supported by Spread Footings on Compacted Fill," *Report No. FHWA/RD-81/184*, FHWA, Washington, D.C.

Fang, Hsai-Yang (1991). *Foundation Engineering Handbook*, 2nd Ed., Van Nostrand Reinhold, New York.

FHWA (1996). "Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects," *FP-96.*

Gifford, D.G., Kraemer, S.R., Wheeler, J.R. and McKown, A.F. (1987). "Spread Footings for Highway Bridges," *FHWA Report No. RD-86/185*.

Goodman, R.E. (1989). *Introduction to Rock Mechanics,* 2nd Ed., John Wiley & Sons, New York.

Hoek, E. and Bray, J.W. (1981). *Rock Slope Engineering,* 3rd Ed., The Institution of Mining and Metallurgy, London.

Holtz, R.D. and Kovacs, W.D. (1981). *An Introduction to Geotechnical Engineering,* Englewood Cliffs, NJ: Prentice Hall.

Hough, B.K. (1959). "Compressibilty as the Basis for Soil Bearing Value," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 85, Part 2.

Hynes-Griffin, M.E. and Franklin, A.G. (1984). "Rationalizing the Seismic Coefficient Method," *Miscellaneous Paper GL-84-13*, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi.

Kavazanjian, E., Jr., Matasović, T. Hadj-Hamou and Sabatini, P.J. (1997). "Geotechnical Engineering Circular No. 3, Design Guidance: Geotechnical Earthquake Engineering for Highways," *Report No. FHWA-SA-97-076*, Federal Highway Administration, Washington, D.C.

Kulhawy, F. H. (1975). "Stress Deformation Properties of Rock and Rock Discontinuities," *Engineering Geology*, Vol. 9, pp. 327-350.

Kulhawy, F. H. (1978). "Geomechnical Model for Rock Foundation Settlement," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 4 (GT2), pp. 211-217.

Kulhawy, F.H. (1983). "Transmission Line Structure Foundations for Uplift and Compression Loading," *Report No. EL-2870*, Electric Power Research Institute.

Kulhawy, F. H. and Carter, J. P. (1992). *Engineering in Rock Masses,* Butterworth and Heinemann Ltd., Oxford.

Kulhawy, F. H. and Goodman, R. E. (1987). "Foundations in Rock," *Ground Engineer's Reference Book,* Bell, F.G., Ed., Butterworths, London.

Lagasse, P.F., Schall, J.D. and Richardson, E.V. (2001a). "Hydraulic Engineering Circular No. 20 (HEC-20) – Stream Stability at Highway Structures," 3rd Ed., *Report No. FHWA NHI-01-002*, Federal Highway Administration, Washington, D.C.

Lagasse, P.F., Zevenbergen, L.W., Schall, J.D. and Clopper, P.E. (2001b). "Hydraulic Engineering Circular No. 23 (HEC-23) – Bridge Scour and Stream Instability Countermeasures, Experience, Selection and Design Guidance," 2nd Ed., *Report No. FHWA-NHI-01-003*, Federal Highway Administration, Washington, D.C.

Lama, R.D. and Vutukuri, V.S. (1978). *Handbook on Mechanical Properties of Rocks*, Trans. Tech. Publications, Clausthal, Vol. 2.

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

Lee, K. L. and Singh, A. (1971). "Compaction of Granular Soils," *Proceedings,* 9th Engineering Geology and Soils Engineering Symposium, Idaho Department of Highways, Boise.

Liao, S.S.C. and Whitman, R.V. (1986). "Overburden Correction Factors for SPT in Sand," *Journal of the Geotechnical Engineering Division, ASCE,* Vol. 112, No. 3, pp. 373-377.

Mast, R., Marsh, L., Spry, C., Johnson, S., Griebenow, R., Guarre, J. and Wilson, W. (1996). "Seismic Design of Bridges," *Publication No. FHWA-SA-97-007*, Federal Highway Administration, Washington, D.C.

Mayne, P.W. and Kulhawy, F.H. (1982). "K_o-OCR Relationships in Soil," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 108, No. GT6, pp. 851-872.

Meyerhof, G.G. (1957). "The Ultimate Bearing Capacity of Foundations on Slopes," *Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering*, London, Vol. 1, pp. 384-386.

Moulton, L.K., Ganga Rao, H.V.S. and Halvorsen, G.T. (1982). "Tolerable Movement Criteria for Highway Bridges," *Publication No. FHWA-RD-81-162*, Federal Highway Administration, Washington, D.C.

Moulton, L.K., Ganga Rao, H.V.S. and Halvorsen, G.T. (1985). "Tolerable Movement Criteria for Highway Bridges," *Publication No. FHWA-RD-85-107*, Federal Highway Administration, Washington, D.C.

Munfakh, G., Arman, A., Collin, J.G., Hung, J.C.-J., and Brouillette, R.P. (2000). "Shallow Foundations Reference Manual," *Publication No. FHWA-NHI-01-023*, Federal Highway Administration, Washington, D.C.

NAVFAC (1986a). "Design Manual 7.1, *Soil Mechanics*," U.S. Department of the Navy, Naval Facilities Engineering Command, 200 Stovall Street, Alexandria, VA 22332.

NAVFAC (1986b). "Design Manual 7.2, *Foundations and Earth Structures*," U.S. Department of the Navy, Naval Facilities Engineering command, 200 Stovall Street, Alexandria, VA 22332.

Newmark, N.M. (1942). "Influence Charts for Computation of Stresses in Elastic Foundations," *University of Illinois Bulletin No. 338*.

Nowak, A.S. (1992). "Calibration of LRFD Bridge Design Code," *NCHRP Report No. 12-33*, prepared for the National Cooperative Highway Research Program by Department of Civil Engineering, University of Michigan, Ann Arbor, MI.

O'Neill, M.W., Townsend, F.C., Hassan, K.M., Buller, A. and Chan, P.S. (1996). "Load Transfer for Drilled Shafts in Intermediate Geomaterials," *Publication No. FHWA-RD-95-172*, Federal Highway Administration, Washington, D.C.

O'Neill, M.W. and Reese, L.C. (1999). "Drilled Shafts: Construction Procedures and Design Methods," *Publication No. FHWA-IF-99-025*, Federal Highway Administration, Washington, D.C.

Oweis, I.S. (1979). "Equivalent Linear Model for Predicting Settlements of Sand Bases," *Journal of Geotechnical Division*, ASCE, December.

Peck, R.B. (1969). "Advantages and Limitations of the Observational Method in Applied Soil Mechanics," *Geotechnique*, Vol. 19, No. 2, pp. 171-187. Reprinted in *Judgment in Geotechnical Engineering: The Professional Legacy of Ralph B. Peck* (1984), J. Dunnicliff and D.U. Deere (Eds.), Wiley, New York, pp. 122-127.

Peck, R.B. and Bazaraa, A.R.S. (1969). "Discussion of 'Settlement of Spread Footings on Sand,' by D'Appolonia, D'Appolonia and Brissette," *Journal of Soil Mechanics and Foundation Engineering, ASCE,* 96 (SM3), 905-909.

Peck, R.B., Hanson, W.E. and Thornburn, T.H. (1974). *Foundation Engineering*, 2nd Ed., John Wiley & Sons.

Perloff, W.H. (1975). "Pressure Distribution and Settlement," Chapter 4, *Foundation Engineering Handbook*, H.F. Winterkorn and H.T. Fang, Eds., Van Nostrand Reinhold Company, New York, pp. 148-196.

Poulos, H.G. and Davis E.H. (1974). *Elastic Solutions for Soil and Rock Mechanics*, New York, John Wiley & Sons.

Prandtl, L. (1920). "Uber die Harte Platischer Korper," Nach. Gesell. Wiss. Gottingen. Math-Phys. KI. pp. 74-85.

Reissner, H. (1924). "Zum erddruckproblem," *Proc. First Intem. Congress of Applied Mechanics*," Delft, pp. 295-311.

Riaund, J.-L., and Miran, J. (1992). "The Cone Penetrometer Test," *Report No. FHWA-SA-91-043*, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

Riaund, J.-L., and Miran, J. (1992). "The Flat Dilatometer Test," *Report No. FHWA-SA-91-044*, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

Richardson, E.V. and Davis, S.R. (1995). "Hydraulic Engineering Circular No. 18 (HEC-18) – Evaluating Scour at Bridges," 3rd Ed.," *Publication No. FHWA-IP-90-017*, Federal Highway Administration, Washington, D.C.

Sabatini, P.J., Pass, D.G., and Bachus, R.C. (1999). "Geotechnical Engineering Circular No. 4 – Ground Anchors and Anchored Systems," *Publication No. FHWA-SA-99-015*, Federal Highway Administration, Washington, D.C.

Schmertmann, J.H. (1970). "Static Cone to Compute Static Settlement Over Sand," *Journal of Soil Mechanics and Foundation Engineering (ASCE),* 96 (SM3), pp. 1011-1043.

SHB AGRA (1993). "Report: Geotechnical Investigation, Mount Rose Interchange (NDOT Porject 1-1949), Washoe County, Nevada," SHB AGRA, Inc., Sparks, NV.

Skempton, A.W. (1948). "Vane Tests in the Alluvial Plain of the River Forth Near Grangemouth," *Geotechnique,* Vol. 1, No. 2, pp. 111-124.

Spencer, E. (1967). "A Method of Analysis of the Stability of Embankments Assuming Parallel Interface Forces," *Geotechnique*, Vol. 17, No. 1, pp 11-26.

Terzaghi, K. and Peck, R. (1948). *Soil Mechanics in Engineering*, 1st Ed., John Wiley & Sons.

Terzaghi, K. and Peck, R. (1967). *Soil Mechanics in Engineering*, 2nd Ed., John Wiley & Sons.

TRB (1990). "Guide to Earthwork Construction," *State of the Art Report 8*, Transportation Research Board, National Research Council, Washington, D.C.

TRB (1996). "Landslides: Investigation and Mitigation," *Special Report No. 247*, Transportation Research Board, National Research Council, Washington, D.C.

Tschebotarioff, G.P. (1951). *Soil Mechanics, Foundations and Earth Structures*, McGraw-Hill, New York.

Urzua, A. (1993). "EMBANK: A Microcomputer Program to Determine One-Dimensional Compression Settlement Due to Embankment Loads," *Publication No. FHWA-SA-92-045*, Federal Highway Administration, Washington, D.C.

US Army Corps of Engineers (1988). "Arctic and Subarctic Construction: Calculation Methods for Determination of Depths of Freeze and Thaw in Soils," *Technical Manual TM5-852-6*.

Vesic, A.S. (1975). "Bearing Capacity of Shallow Foundations," *Foundation Engineering Handbook,* 2nd Ed., H.F. Winterkorn and H.T. Fang, Eds., Chapter 3, Van Nostrand Reinhold Company, New York, pp. 121-147.

Wahls, H. (1983). "Shallow Foundations for Highway Structures," *NCHRP Synthesis of Highway Practice 107*, Transportation Research Board, National Research Council, Washington, D.C.

Winterkorn, H.F. and Fang, H.-Y. (1975). *Foundation Engineering Handbook,* 2nd Ed., Van Nostrand Reinhold Co., New York.

Withiam, J.L. et al. (1998). "Load and Resistance Factor Design (LFRD) of Highway Bridge Structures," *Report No. FHWA HI-98-032*, NHI Course No. 13068, Federal Highway Administration, Washington, D.C.

Wyllie, D. and Mah, C.W. (1998). "Geotechnical and Foundation Engineering, Module 5 – Rock Slopes Reference Manual," *Publication No. FHWA NHI99-007,* NHI Course No. 13234, Federal Highway Administration, National Highway Institute, Arlington, Virginia.

Youd, T.L., et al. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of the Liquefaction Resistance of Soils," *Journal of Geotechnical and Geoenvironemental Engineering* (ASCE), Vol. 127, No. 10, pp. 817-833.

(This page intentionally blank)

APPENDIX A

STRUCTURAL FILL MATERIALS AND SPECIFICATIONS

FHWA SOILS AND FOUNDATIONS REFERENCE MANUAL

The following specification for select material is recommended in the *Soils and Foundations Workshop Reference Manual* (Cheney & Chassie, 2000). This specification should be used in conjunction with the suggested plan detail, Figure 9-1.

SELECT MATERIAL

<u>Description</u> This work shall consist of excavation, disposal, placement and compaction of all materials that are not provided for under another section of these specifications and shall be executed in conformance with payment lines, grades, thicknesses and typical sections specific in the contract documents.

<u>Materials – Tests and Control Methods</u> Materials tests and control methods pertaining to the item requirements and work of this section will be performed in conformance with the procedures used by the Department.

Materials furnished under these items shall conform to the following requirements:

1. Gradation. The material shall have the following gradation:

Sieve Size	Percent Passing (by weight)
100 mm	100
No. 40	0 to 70
No. 200	0 to 15

2. Soundness. The material shall be substantially free of shale or other soft, poor-durability particles. Where the State elects to test for this requirement, a material with a magnesium sulfate soundness loss exceeding 30 percent shall be rejected.

<u>Construction</u> The type of material to be used in filling and backfill at the structures, and payment lines, therefore, shall be in conformance with the details shown on the appropriate Standard Sheet or as noted on the plans or as ordered by the engineer.

Fill or backfill material at structures, culverts and pipes shall be deposited in horizontal layers not exceeding 150 mm in thickness prior to compaction. Compaction of each layer shall be as specified. A minimum of 100 percent of standard proctor maximum density will be required. When filling behind abutments and similar structures, all material shall be placed and compacted in front of the walls prior to placing fill behind the walls to a higher elevation. The limits to which this subsection will apply shall be in accordance with the Standard Sheets or as modified by the plans.

Measurement Quantities for this work shall be computed in cubic meters in the final compacted position. A deduction shall be made for pipes (based on nominal diameters) and other payment items when the combined cross-sectional area exceeds one 0.1 m^2 unless shown otherwise on the plans. No deduction will be made for the cross-sectional area of an existing facility.

Basis of Payment The unit price bid for all pay items of work encompassed by this section shall include the costs of furnishing all equipment, labor, and materials as necessary to complete the work of the item, except where specific costs are designated or included in another pay item of work. All incidental costs, such as acquisition of borrow pits or material outside of the right-of-way, rock drilling and blasting, compaction and special test requirements, stockpiling and re-handling of materials – precautionary measures to protect private property and utilities and to form and trim graded surfaces – shall be included in the unit price of the pay item where such costs are incurred.

WASHINGTON STATE DOT GRAVEL BORROW SPECIFICATION

The Washington State DOT uses the following materials specifications for embankments that will support bridge foundations:

Gravel Borrow

Aggregate for gravel borrow shall consist of granular material, either naturally occurring or processed, and shall meet the following requirements for grading and quality:

Sieve Size	Percent Passing
100 mm (4″)	100
50 mm (2″)	75 – 100
4.75 mm (#4)	50 – 80
0.425 mm (#40)	30 max.
0.075 mm (#200)	7.0 max.
Sand equivalent	42 min.

Method “C” compaction required (minimum of 95 percent maximum density according to AASHTO T-99 or WSDOT Test Method 606). Lift thickness limited to 200 mm (100 mm in top 0.6 m).

Note: WSDOT usually specifies Gravel Borrow for the entire embankment below the footing.

NEVADA DOT STRUCTURAL FILL SPECIFICATION

The following specification was used for the **Mt. Rose Highway Interchange Project (Nevada DOT)** described in Section 2.7.2:

Materials for the Structural Fill Under the Footings

Type 1 Class B Aggregate Base

Sieve Analysis:

Sieve Size	Percent Passing
37.5 mm	100
25 mm	80 – 100
4.75 mm	30 – 65
1.18 mm	15 – 40
75 µm	2 – 12

R-value: 70 Min.

Percentage of Wear (500 Rev.): 45% Max.

Liquid Limit: 35 Max.

Plasticity Index:

Percent Finer than 75 µm	PI
0.1 – 3.0	15 max
3.1 – 4.0	12 max
4.1 – 5.0	9 max
5.1 – 8.0	6 max
8.1 – 11.0	4 max
11.1 – 15.0	3 max

Fractured Faces: 15% Min.

Placement

Place in layers not exceeding 200 mm in thickness before compaction.

Compaction

Compact to not less than 95% of the maximum density as determined by Test Method No. Nev. T101. *(Note: T101 is based on the use of the Harvard Miniature Compaction Device.)*

MICHIGAN DOT STRUCTURAL FILL SPECIFICATION

The following specification was used for the **Cadillac Bypass Project, Structure S01 (Michigan DOT)** described in Section 2.7.3:

Materials for the Structural Fill Under the Footings

The material under the footings is specified as Structure Embankment. The structure embankment shall be constructed of Granular Material Class III.

Granular Material Class III

Sieve Analysis:

Sieve Size	Percent Passing
150 mm	100
75 mm	95 – 100

Loss by Washing: 0 – 15 percent

Placement

Granular materials, for backfill, shall be deposited and spread in layers not more than 250 mm in thickness.

Compaction

The compaction required will be 100 percent of its Maximum Unit Weight. *(Note: According to Greg Perry of MDOT, the Maximum Unit Weight is determined by the Modified Proctor Method.)*

APPENDIX B

DESIGN EXAMPLES

INTRODUCTION

This appendix provides worked design examples for bridge foundation support on shallow foundations in several applications using Service Load Design (SLD) methodology. The applications and design considerations for each are discussed briefly below. The examples follow the design process outlined in Table 6-1 and shown in the flow chart in Figure 6-1.

It should be noted that these examples are intended to convey one approach and process that can lead to a satisfactory design of bridge support on a shallow foundation. The examples should not be construed as representations of the *only* approach that can lead to a satisfactory design. Also note that the examples only check one or, at most, a few specific load cases. In practice, every complete design should consider all applicable load cases.

EXAMPLE 1: INTERIOR BRIDGE PIER ON A SPREAD FOOTING FOUNDED IN NATIVE SOIL

This example uses SLD Group I loads from service load design of an interior bridge column. The purpose of the example is to show how bearing capacity and settlement are considered together to calculate an allowable bearing capacity that controls the design of the footing.

EXAMPLE 2: INTEGRAL STUB ABUTMENT ON COMPACTED STRUCTURAL FILL

This example shows the successful and cost-effective application of a spread footing at a structurally integral abutment supported in a compacted structural fill over good foundation conditions. Since integral abutments are most frequently supported on pile foundations, this example was developed to illustrate the feasibility of using a spread footing in lieu of piles in order to realize the cost savings potential of shallow foundations. The load case is SLD Group I.

EXAMPLE 3: STUB SEAT-TYPE ABUTMENT FOOTING IN COMPACTED STRUCTURAL FILL

This example uses SLD Group IV loads to show the effect of shear loads transmitted through the bearing pads due to temperature and shrinkage, etc., at a seat-type abutment with a joint. The example also shows the effective use of engineered embankment design and stage construction to permit the use of a spread footing supported in an approach fill over embankment foundation conditions that are less than excellent.

EXAMPLE 4: FULL-HEIGHT SEAT ABUTMENT FOUNDED IN NATIVE SOIL

This example shows the design of a tall abutment founded on a spread footing in good ground conditions. The load case is SLD Group I. The example also shows design of a concrete cantilever retaining wall, since the intermediate construction stage of a backfilled stem without superstructure loads is also checked.

EXAMPLE 5: INTERIOR BRIDGE PIER ON A SPREAD FOOTING ON NATIVE ROCK

This example uses excellent foundation conditions and Group VII seismic loads to show an example of foundation design governed by considerations other than bearing capacity or settlement.

Note: Unless cited with a specific reference (e.g., AASHTO, 1996), the equation numbers in the examples refer to those in the main body of this Circular.

EXAMPLE 1: INTERIOR BRIDGE PIER ON SPREAD FOOTING

Given:

- Subsurface profile at the interior bridge pier is shown in Figure B1-1.
- There is no scour potential at this site.
- Frost penetration depth is 0.6 m (2 ft).
- Tolerable settlement is 38 mm (1.5 in).
- The sign conventions and loading directions are shown in Figure B1-2.

Find:

- Size the footing based on AASHTO Service Load Design, Group I loads (allowable stress design method) considering both the settlement and bearing capacity. Check for overturning and sliding.

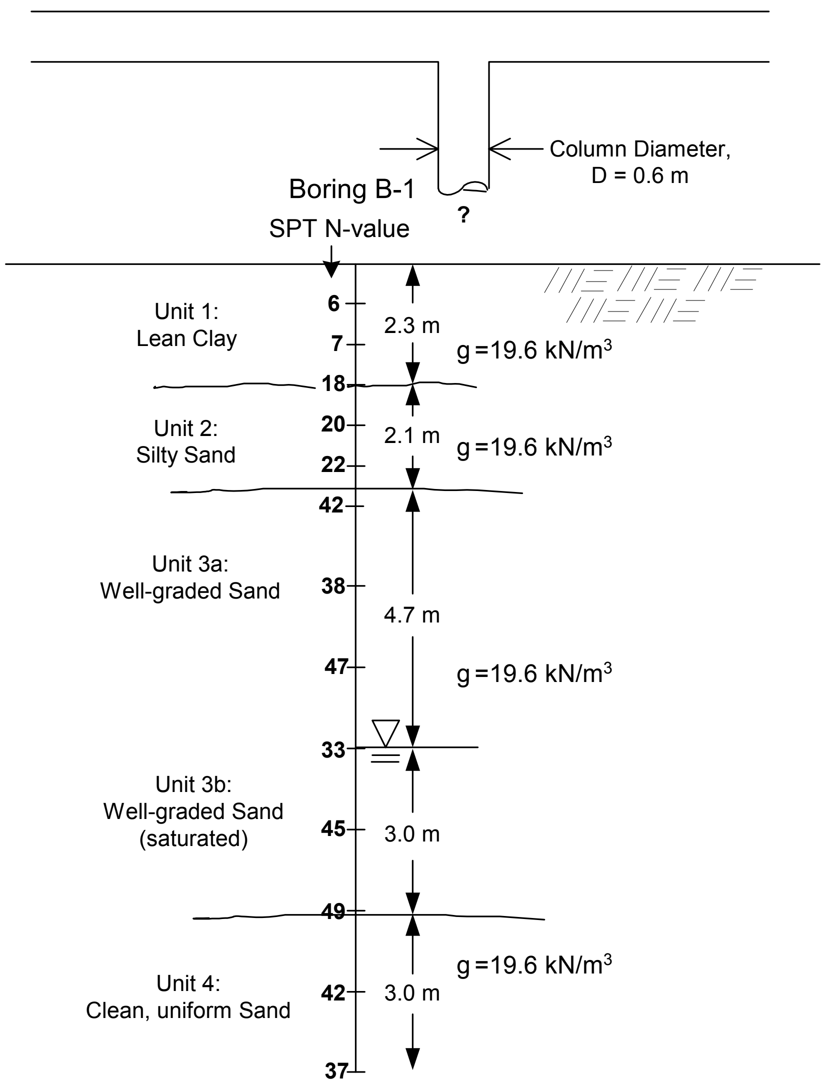


Figure B1-1: Subsurface Profile

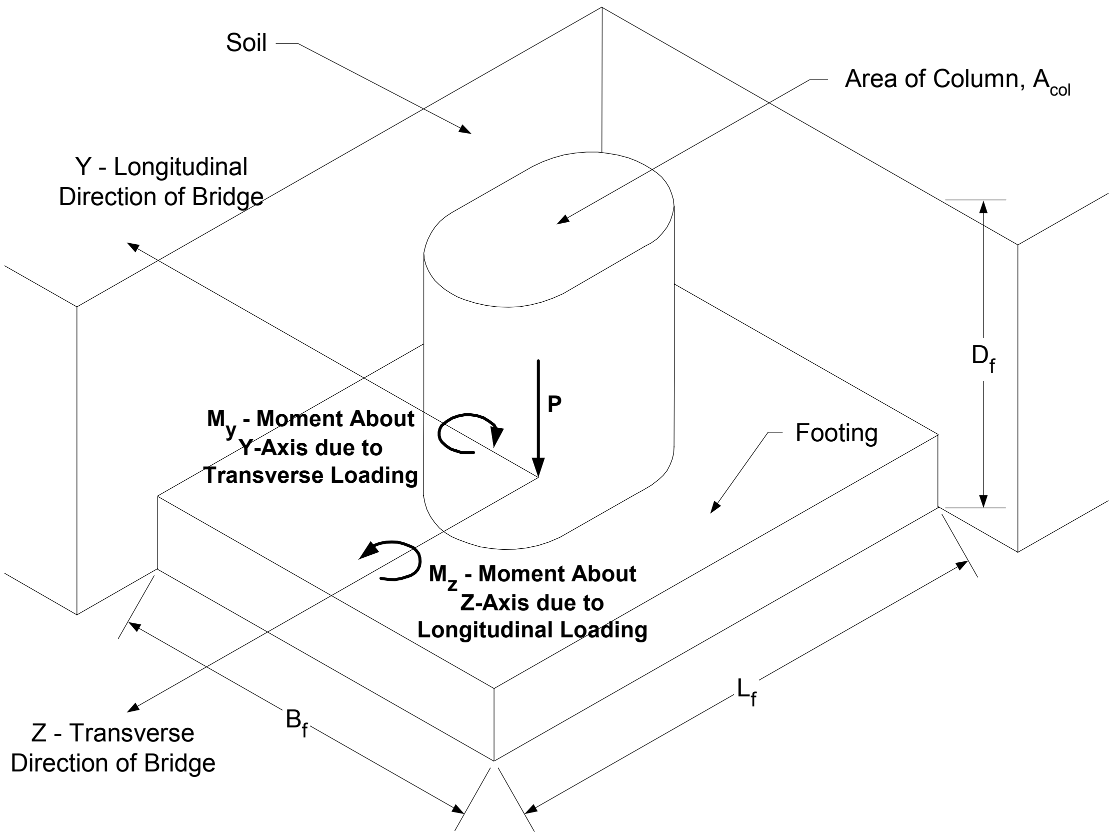


Figure B1-2: Sign Conventions and Loading Directions

<u>Solution:</u>

The solution can be obtained by following the step-by-step process shown in Figure 6-1, Design Process Flow Chart.

Steps 1 through 4 – Preliminary bridge layout, review of existing geologic and subsurface data, site reconnaissance, scour and frost potential:

The structural, hydraulics and geotechnical engineers completed these steps. The required design information is provided in the problem-given statement.

Step 5 – Determine the loads applied to the footing:

The structural engineer has calculated and provided the values shown in Table B1-1.

TABLE B1-1: LOADS AT COLUMN BASE

Load	Axial, P (kN)	Shear, V (kN)	Moment, M_Z (kN-m)	Moment, M_Y (kN-m)
Dead load (D)	6400	167	211	748
Live load (L)	1670	42	409	196.5
Impact (I)	315	8	77	37
Wind on structure (W)	884	49	89	226
Wind on live load (WL)	18	4	7	26
Earthquake (EQ)	1671	804	1675	5546

Service Load Design Loads for Group I:

The general equation is:

$$P_I = \gamma\,[\beta_D(D) + \beta_L\,(L + I) + \beta_C(CF) + \beta_E(E) + \beta_B\,(B) + \beta_S(SF) + \beta_W(W) + \beta_{WL}(WL) + \beta_L(LF) + \beta_R(R + S + T) + \beta_{EQ}(EQ) + \beta_{ICE}(ICE)]$$

(AASHTO, 1996, Eqn. 3-10)

where: γ = Load factor (always equal to 1.0 for SLD)

β_i = Coefficient for particular load, i, as found in Table 3.22.1A of AASHTO (1996).

Recall from Table 6-2, that SLD Group I is the load combination for normal vehicles using the bridge without wind, and that earthquake loads are only considered in Load Group VII. Therefore the coefficients for W, WL and EQ are all zero and the general load equation reduces to the following for this example:

$$P_I = \gamma\,[\beta_D(D) + \beta_L\,(L + I)]$$

The AASHTO provisions permit neglecting the impact load, I, for foundations and buried structures. Also, since the footing is relatively shallow, assume that the effect of footing self-weight is negligible. The factored loads are as follows:

Axial Loads:

$$\begin{aligned} P_I &= \gamma\,[\beta_D(D) + \beta_L\,(L)] \\ &= 1.0\,[1.0\,(6400^{kN}) + 1.0\,(1670^{kN})] \\ &= 8070^{kN} \end{aligned}$$

Shear:

$$
\begin{aligned}
V &= \gamma[\beta_D(D) + \beta_L(L)] \\
&= 1.0\,[1.0\,(167^{kN}) + 1.0\,(42^{kN})\,] \\
&= 209^{kN}
\end{aligned}
$$

Moment in the Z direction:

$$
\begin{aligned}
M_Z &= \gamma[\beta_D(D) + \beta_L(L)] \\
&= 1.0\,[1.0\,(211\ ^{kN\text{-}m}) + 1.0\,(409^{kN\text{-}m})\,] \\
&= 620^{kN\text{-}m}
\end{aligned}
$$

Moment in the Y direction:

$$
\begin{aligned}
M_Y &= \gamma[\beta_D(D) + \beta_L(L + I)] \\
&= 1.0\,[1.0\,(748\ ^{kN\text{-}m}) + 1.0\,(196.5^{kN\text{-}m})\,] \\
&= 944.5^{kN\text{-}m}
\end{aligned}
$$

Step 6 – Subsurface Exploration and Laboratory Testing:

This step is complete, and the subsurface data are shown in Figure B1-1. Laboratory testing was limited to soil classifications and moisture content due to the granular nature of the soils encountered in the boring. The foundation design will be based largely on design parameters correlated to the soil classifications and the field SPT N-values. Apply judgment and recognize that it is minimal excavation to remove all of the medium stiff Unit 1 lean clay to avoid any associated low strength or compressibility problems in this material. Therefore, select depth of footing, D_f, of 2.3 m to bear below Unit 1. Compute the initial vertical effective stresses at the midpoint of each soil layer below the footing:

Layer 2:

$$\sigma'_{vo_2} = 3.35\,\text{m}(19.6\,\frac{\text{kN}}{\text{m}^3}) = 65.7\,\text{kPa}$$

Layer 3a:

$$\sigma'_{vo_{3a}} = 6.75\,\text{m}(19.6\,\frac{\text{kN}}{\text{m}^3}) = 132\,\text{kPa}$$

Layer 3b:

$$\sigma'_{vo_{3b}} = 10.6\,\text{m}(19.6\,\frac{\text{kN}}{\text{m}^3}) - 1.5\,\text{m}(9.8\,\frac{\text{kN}}{\text{m}^3}) = 193\,\text{kPa}$$

Layer 4:

$$\sigma'_{vo_4} = 13.6\,\text{m}(19.6\,\tfrac{\text{kN}}{\text{m}^3}) - 4.5\,\text{m}(9.8\,\tfrac{\text{kN}}{\text{m}^3}) = 222\,\text{kPa}$$

The effective stress diagram for the profile at the pier location is plotted in Figure B1-3.

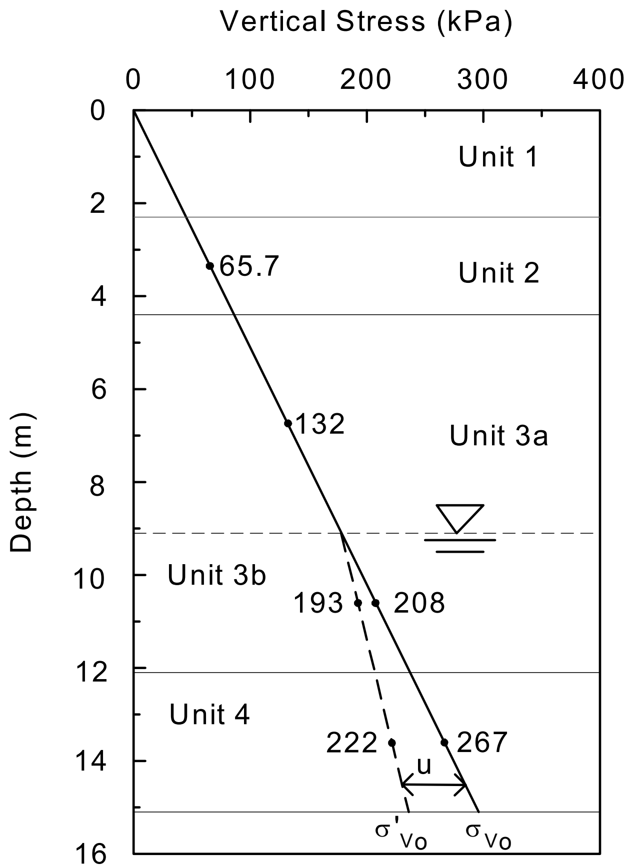


Figure B1-3: Effective Stress Diagram

Step 7 – Calculate allowable bearing capacity:

The geotechnical engineer will calculate the allowable bearing capacity, considering both the potential for bearing capacity failure (Section 5.1) and settlement (Section 5.2).

<u>Calculate ultimate bearing capacity</u>:

The general equation for bearing capacity is:

$$q_{ult} = cN_c s_c b_c + qN_q C_{W_q} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{W_\gamma} s_\gamma b_\gamma \qquad \text{(Eqn. 5-14)}$$

The first term goes to zero since the foundation materials are granular and cohesionless, so the cohesion term, c, is zero.

For the interior pier, we will assume that the footing is essentially rectangular (L/B < 5). Recall that the shape and inclination factors should not be applied together (see discussion in Section 5.2). Since the effect of the shape factors is significant for rectangular footings, and the inclination resulting from the relatively small shear load component (V) is small, we will only apply the shape factors as follows:

$$s_\gamma = 1 - 0.4\frac{B_f}{L_f}$$ Assume the effect of eccentricity is small for now, so $B_f \cong L_f$ (Table 5-2)

$$s_\gamma = 0.6$$

A design friction angle associated with Layer 2 is needed to calculate s_q. The geotechnical engineer has recommended that the footing be cast against the firm, undisturbed soil of Unit 2. Unit 2 is 2.1-m thick and overlies denser material at depth, so the properties of Unit 2 will govern the footing design. From Figure B1-1, the average uncorrected field SPT N-value for this layer is 20. From Figure B1-3, the average vertical effective stress in Layer 2 is 65.7 kPa (about 1.4 ksf). Entering Figure 4-1 with a blowcount of 20 and a vertical effective stress of 1.4 ksf, a relative density, D_R, of about 75 percent is read. Entering Figure 4-2 with this relative density for silty sand (Unified Soil Classification 'SM'), a friction angle of 35 is selected for computing bearing capacity.

(Table 5-2)

$$s_q = 1 + \frac{B_f}{L_f}\tan\phi$$

$$s_q = 1 + \frac{1}{1}\tan(35^o)$$

$$s_q = 1.7$$

Since the overburden materials above the footing are cohesive soils, the correction factor to account for shearing resistance of the material above the bearing elevation, d_q, should be set equal to 1.0:

$$d_q = 1.0$$ (Table 5-4)

Since this footing will be level, there is no effect of inclination of the base, so:

$$b_\gamma = b_q = 1.0$$ (Table 5-5)

Calculate the effect of the footing embedment, q. Apply judgment and recognize that it is minimal excavation to remove all of the Unit 1 medium stiff lean clay and avoid any associated low strength or compressibility characteristics in this material. Therefore select D_f = 2.3 m to bear below Unit 1:

In this case q is only a function of the depth of bearing.

$$q = \sigma'_{v_o} = \gamma' D_f \qquad \text{where:} \quad \gamma' = \gamma_{bulk} = 19.6\frac{kN}{m^3}$$

$$q = (19.6\frac{kN}{m^3})(2.3\,m)$$

$$q = 45.1\,kPa$$

Check ground water effect:

Depth of ground water table below the bearing depth:

D_f = Depth of footing = 2.3 m   Assumes a 0.9-m thick footing and 1.4-m cover
D_w = Depth of ground water below the ground surface (D_f) = 9.1 m

Conservatively estimate the footing width with an upper bound value of B_f = 6 m.

Calculate ground water correction factors, $C_{W\gamma}$ and C_{Wq}:

$$C_{W_\gamma} = 0.5 + 0.5\left(\frac{D_W}{1.5B_f + D_f}\right) \leq 1.0 \qquad \text{(Eqn. 5-10)}$$

$$C_{W_\gamma} = 0.5 + 0.5\left(\frac{9.1}{1.5(6) + 2.3}\right) = 0.9$$

$$C_{W_q} = 0.5 + 0.5\left(\frac{D_W}{D_f}\right) \leq 1.0 \qquad \text{(Eqn. 5-11)}$$

$$C_{W_q} = 0.5 + 0.5\left(\frac{9.1}{2.3}\right) = 2.48\ ; \ \text{Use } C_{Wq} = 1.0$$

From Table 5-1, the bearing capacity factors, N_γ and N_q, are 48.0 and 33.3, respectively, for a friction angle of 35 degrees in Layer 2.

So the equation for bearing capacity is:

$$q_{ult} = qN_q C_{W_q} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{W_\gamma} s_\gamma b_\gamma$$

$$q_{ult} = 45.1\,kPa\,(33.3)(1.0)(1.7)(1.0)(1.0) + 0.5\,(19.6\frac{kN}{m^3})(B_f)(48.0)(0.9)(0.6)(1.0)$$

$$q_{ult} = 2553\,kPa + 254B_f$$

Incorporating a factor of safety of 3.0:

$$q_{all} = q_{ult} \div 3.0$$
$$q_{all} = (2553\,\text{kPa} + 254B_f) \div 3.0$$
$$q_{all} = 851 + 85B_f \text{ kPa}$$

Estimate Footing Settlement

Calculate stress increase at the midpoint of each soil layer using the 2-on-1 stress distribution method for various footing widths – say, B_f = 3, 4.6 and 6.1 m:

$$\frac{\Delta\sigma_V}{q} = \frac{(B_f)(L_f)}{(B_f + Z)(L_f + Z)} \qquad \text{(Figure 5-12)}$$

but $B_f \cong L_f$, so:

$$\frac{\Delta\sigma_V}{q} = B_f^{\,2} / (B_f + Z)^2$$

Layer 2 with midpoint of layer @ Z = 1.05 m below base of footing:

$$\frac{\Delta\sigma_{V2}}{q} = 3^2/(3+1.05)^2 = 0.55$$

Layer 3a with midpoint of layer @ Z = 4.45 m below base of footing:

$$\frac{\Delta\sigma_{V3a}}{q} = 3^2/(3+4.45)^2 = 0.16$$

Layer 3b with midpoint of layer @ Z = 8.3 m below base of footing:

$$\frac{\Delta\sigma_{V3b}}{q} = 3^2/(3+8.3)^2 = 0.07$$

Layer 4 with midpoint of layer @ Z = 11.3 m below base of footing:

$$\frac{\Delta\sigma_{V4}}{q} = 3^2/(3+11.3)^2 = 0.04$$

Perform similar calculations for B_f = 4.6 m and B_f = 6.1 m and tabulate:

TABLE B1-2: STRESS INCREASE WITH DEPTH
AS FUNCTION OF FOOTING WIDTH

Soil Layer	Depth Range (m)	Stress Increase, $\Delta\sigma_v$		
		$B_f = 3$ m	$B_f = 4.6$ m	$B_f = 6.1$ m
2	2.3 – 4.4	0.55q	0.66q	0.73q
3a	4.4 – 9.1	0.16q	0.26q	0.33q
3b	9.1 – 12.1	0.07q	0.13q	0.18q
4	12.1 – 15.1	0.04q	0.08q	0.12q

Determine stress applied by footing to generate 38 mm of settlement for same range of footing widths, $B_f = 3$, 4.6 and 6.1 m, using the Hough method:

General equation:

$$\Delta H = H_o \frac{1}{C'} \log\left(\frac{\sigma'_{vo} + \Delta\sigma_{v_f}}{\sigma'_{vo}}\right) \qquad \text{(Eqn. 5-24)}$$

Correct SPT blowcounts to account for overburden pressure, N′, for each layer, and enter Figure 5-19 to obtain bearing capacity factor, C′:

Layer 2:

$N_{avg} = 20$
$N'/N = 1.2$ (from Figure 5-18)
$N' = 24$
$C' = 65$ (from Figure 5-19 for silty Sand)

Layer 3a:

$N_{avg} = 40$
$N'/N = 0.9$ (from Figure 5-18)
$N' = 36$
$C' = 120$ (from Figure 5-19 for well-graded Sand and Gravel)

Layer 3b:

$N_{avg} = 43$
$N'/N = 0.7$ (from Figure 5-18)
$N' = 30$
$C' = 102$ (from Figure 5-19 for well-graded Sand and Gravel)

Layer 4:

$N_{avg} = 40$
$N'/N = 0.66$ (from Figure 5-18)
$N' = 26$
$C' = 110$ (from Figure 5-19 for clean, uniform Sand)

Calculate settlement in each layer and sum the settlements to obtain the total settlement for a range of applied stresses and footing widths. Select applied stresses of q = 240 kPa, 290 kPa, 335 kPa and 380 kPa (corresponding to about 3, 4, 5 and 6 ksf) and nominal footing widths of 3, 4.6 and 6.1 m:

Layer 2 (B_f = 3 m):

$$\Delta H_2 = H_2(1/C')\log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}]$$
$$\Delta H_2 = 2.1\,\text{m}(1/65)\log[(65.7\,\text{kPa} + 0.55(240\,\text{kPa}))/65.7\,\text{kPa})]$$
$$\Delta H_2 = 0.015\,\text{m} = 15\,\text{mm}$$

$$\Delta H_{3a} = H_{3a}(1/C')\log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}]$$
$$\Delta H_{3a} = 4.7\,\text{m}(1/120)\log[(132\,\text{kPa} + 0.16(240\,\text{kPa}))/132\,\text{kPa})]$$
$$\Delta H_{3a} = 0.004\,\text{m} = 4\,\text{mm}$$

$$\Delta H_{3b} = H_3(1/C')\log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}]$$
$$\Delta H_{3b} = 3.0\,\text{m}(1/102)\log[(193\,\text{kPa} + 0.07(240\,\text{kPa}))/193\,\text{kPa})]$$
$$\Delta H_{3b} = 0.001\,\text{m} = 1\,\text{mm}$$

$$\Delta H_4 = H_4(1/C')\log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}]$$
$$\Delta H_4 = 3\,\text{m}(1/110)\log[(222\,\text{kPa} + 0.04(240\,\text{kPa}))/222\,\text{kPa})]$$
$$\Delta H_4 = 0.0005\,\text{m} \cong 1\,\text{mm}$$

$$\sum \Delta H_i = 15\,\text{mm} + 4\,\text{mm} + 1\,\text{mm} + 1\,\text{mm} = 21\,\text{mm}$$

Repeat and tabulate for various footing widths and applied stresses (a spreadsheet or computer tool can make these calculations rapidly):

TABLE B1-3: SETTLEMENT AS A FUNCTION OF APPLIED STRESS AND FOOTING WIDTH

Applied Stress (kPa)	Settlement (mm)		
	B_f = 3 m	B_f = 4.6 m	B_f = 6.1 m
240	21	28	31
290	25	31	35
335	28	34	38
380	30	37	41

Compare to allowable bearing capacities at same footing widths relative to shear failure, remembering that the maximum tolerable settlement is 38 mm:

For B_f = 3 m:

$$q_{all} = 851 + 85(3\,m)\,kPa = 1106\,kPa$$ >> 380 kPa (with 30 mm settlement)

For B_f = 4.6 m:

$$q_{all} = 851 + 85(4.6\,m)\,kPa = 1242\,kPa$$ >> 335 kPa (with 34 mm settlement)

For B_f = 6.1 m:

$$q_{all} = 851 + 85(6.1\,m)\,kPa = 1369\,kPa$$ >> 290 kPa (with 35 mm settlement)

Settlement will therefore limit the allowable bearing capacity for the design of this footing. Interpolating the values in Table B1-3, find ranges of footing widths that will limit settlement to 38 mm or less at allowable bearing capacities of 335 kPa and 380 kPa as shown in Table B1-4.

TABLE B1-4: SETTLEMENT-LIMITED ALLOWABLE BEARING CAPACITIES

Allowable Bearing Capacity (kPa)	Range of Footing Widths That Will Limit Settlement to < 38 mm
380	< 5.0 m
335	5.0 to 6.1 m

The geotechnical engineer should provide this table to the structural designer for sizing of the footing.

Step 8 – Calculate sliding and passive soil resistance:

The footing will bear in the Unit 2 soils. The friction angle, ϕ, of 35 degrees selected in Step 7 will be used to compute sliding resistance.

The general equation for sliding resistance on cohesionless soils is:

$$F_R = (W + P_v) \tan \delta \qquad \text{(Eqn. 5-38)}$$

The second term goes to zero since we are founding the footing on a cohesionless soil unit. For a concrete footing cast against the ground, $\delta = \phi$, so the ultimate sliding resistance is:

$$F_R = (W + P_v) \tan 35° = 0.7(W + P_v)$$

Since the sliding resistance will develop at strain levels much lower than those needed to develop passive pressures against the side of the footing, passive pressure will be ignored unless it is found that sliding resistance governs the design of the footing. The geotechnical engineer should provide this equation and the assumptions regarding use of passive pressures to the structural engineer for checking the footing's resistance to sliding.

Step 9 – Check global stability of the footing:

The geotechnical engineer has determined that since this is an interior pier with level ground conditions, global stability is not a concern.

Step 10 – Size the footing:

The structural designer should make a preliminary estimate of the size of the footing by dividing the Group I axial load by the largest allowable bearing pressure from Table B1-4:

$$A = P_I \div q_{all}$$
$$A = 8070^{kN} \div 380 \text{ kPa} = 21.2\,m^2$$
$$B_f = L_f = \sqrt{A} \cong 4.6\,m$$

Then calculate eccentricity and effective footing dimensions:

$$B'_f = B_f - 2e_y \qquad \text{(Eqn. 6-6)}$$
$$e_y = M_Z \div P$$
$$e_y = 620^{kN-m} \div 8070^{kN} = 0.077\,m$$
$$B'_f = 4.6\,m - 2(0.077\,m) = 4.45\,m$$

$$L'_f = L_f - 2e_z \qquad \text{(Eqn. 6-7)}$$
$$e_z = M_Y \div P$$
$$e_z = 944.5^{kN-m} \div 8070^{k} = 0.117\,m$$
$$L'_f = 4.6\,m - 2(0.117\,m) = 4.37\,m$$

$$A' = B'_f \cdot L'_f = (4.45\,m)(4.37\,m) = 19.4\,m^2$$

Check allowable bearing stress for effective footing area:

$$q_{applied} = P \div A' = 8070^{kN} \div 19.4\,m^2 = 416\,kPa$$

This is greater than q_{all} = 380 kPa for footings less than 5.0 m wide, according to Table B1-4, in order to limit settlement to less than 38 mm. Therefore the footing should be sized larger.

Try a 4.9 m-by-4.9 m footing. Calculate eccentricity and effective footing dimensions:

$$B'_f = B_f - 2e_y \qquad \text{(Eqn. 6-6)}$$

$$e_y = M_Z \div P$$

$$e_y = 620^{kN-m} \div 8070^{kN} = 0.077\,m$$

$$B'_f = 4.9\,m - 2(0.077\,m) = 4.75\,m$$

$$L'_f = L_f - 2e_z \qquad \text{(Eqn. 6-7)}$$

$$e_z = M_Y \div P$$

$$e_z = 944.5^{kN-m} \div 8070^{k} = 0.117\,m$$

$$L'_f = 4.9\,m - 2(0.117\,m) = 4.67\,m$$

$$A' = B'_f \cdot L'_f = (4.75\,m)(4.67\,m) = 22.2\,m^2$$

Check allowable bearing stress for effective footing area:

$$q_{applied} = P \div A' = 8070^{kN} \div 22.2\,m^2 = 364\,kPa$$

This is less than q_{all} = 380 kPa for footings less than 5.0 m wide, according to Table B1-4, in order to limit settlement to less than 38 mm.

The design is acceptable for bearing capacity and settlement considerations.

Step 11 – Check overturning (eccentricity) and sliding:

Overturning will be acceptable if eccentricity in any direction is less than one-sixth the footing dimension in that direction.

$$\frac{1}{6}B_f = \frac{1}{6}L_f = \frac{1}{6}(4.9\,m) = 0.82\,m > e_z > e_y = 0.077 \qquad \text{(OK)}$$

A minimum factor of safety of 1.5 should be provided against sliding:

$$FS = \frac{F_R}{F_{sliding}} = \frac{0.7(W + P_V)}{V} \qquad (F_R \text{ from Step 8})$$

$$FS = \frac{0.7(W + P_V)}{V}$$

Calculate the weight of cover over the footing and the weight of footing:

The footing is bearing 2.3 m below grade. The footing can be assumed to be 0.9 m thick.

The area of the column is:

$$A_{col} = \frac{\pi}{4}(0.6\text{m})^2 = 0.28\,\text{m}^2$$

The weight of the footing and the soil cover is then (assume unit weight of concrete, γ_{conc}, of 23.5 kN/m^3):

$$W_{ftg} = (0.9\,\text{m})(4.9\,\text{m})(4.9\,\text{m})(23.5\tfrac{\text{kN}}{\text{m}^3}) = 508^{\text{kN}}$$

$$W_{cover} = [(1.4\,\text{m})(4.9\,\text{m})(4.9\,\text{m}) - A_{col}(1.4\,\text{m})](19.6\tfrac{\text{kN}}{\text{m}^3}) = 651^{\text{kN}}$$

$$W = W_{ftg} + W_{cover} = 508^{\text{kN}} + 651^{\text{kN}} = 1159^{\text{kN}}$$

So:

$$FS = \frac{0.7(1159^{\text{kN}} + 8070^{\text{kN}})}{209^{\text{kN}}} = 31 > 1.5 \qquad \text{(OK)}$$

Step 12 – Complete structural design of footing:

The structural engineer completes this step.

EXAMPLE 2: INTEGRAL BRIDGE ABUTMENT ON STRUCTURAL FILL

Given:

- An integral abutment on a spread footing is proposed for a bridge.
- There is no scour potential at this site.
- The footing length is L = 25.0 m (82 ft); other abutment dimensions are shown in Figure B2-1.
- The approach embankment and the foundation soils are shown in Figure B2-1.
- The SPT hammer energy ratio is assumed to be ER = 60%.
- Frost penetration depth is 0.6 m (2 ft).
- Tolerable settlement is 25 mm (1 in).
- Abutment symbols are shown in Figure B2-2.

Find:

- Size the footing based on Service Load Design, Group I loads (allowable stress design method), considering both the settlement and bearing capacity. Check for overturning and sliding.

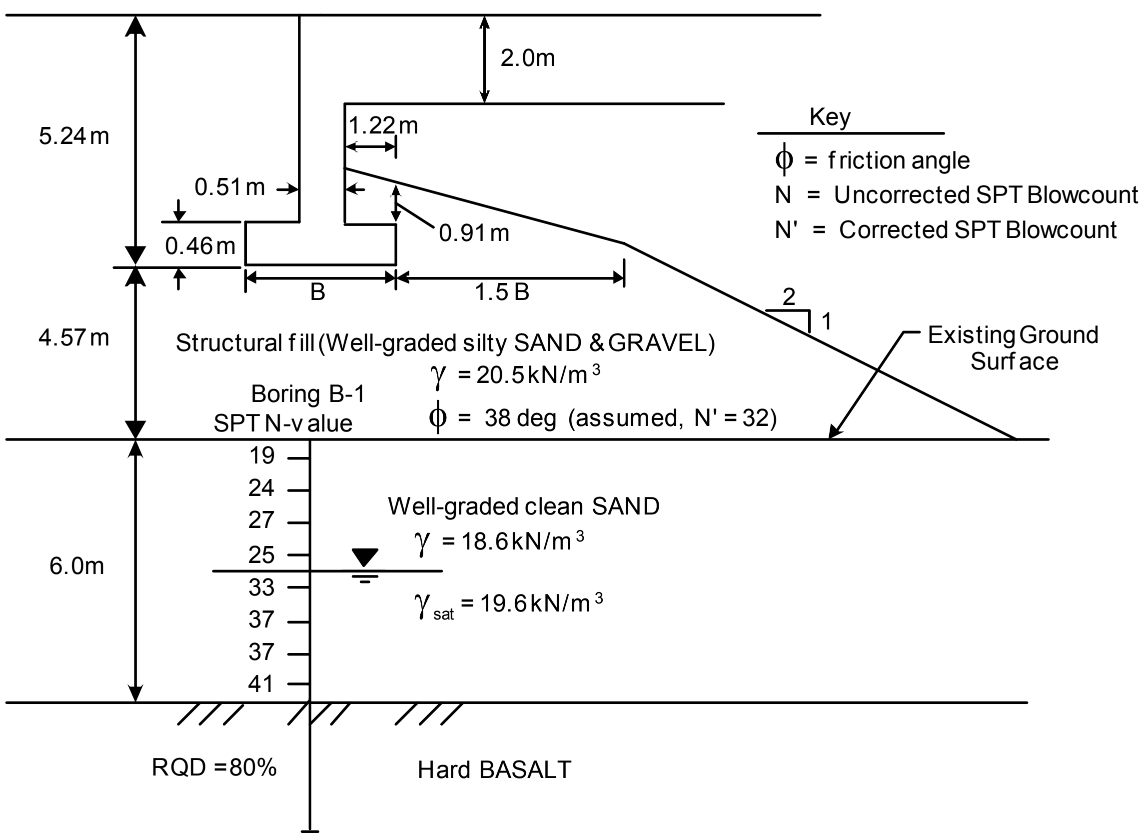


Figure B2-1: Approach Embankment and Foundation Soils

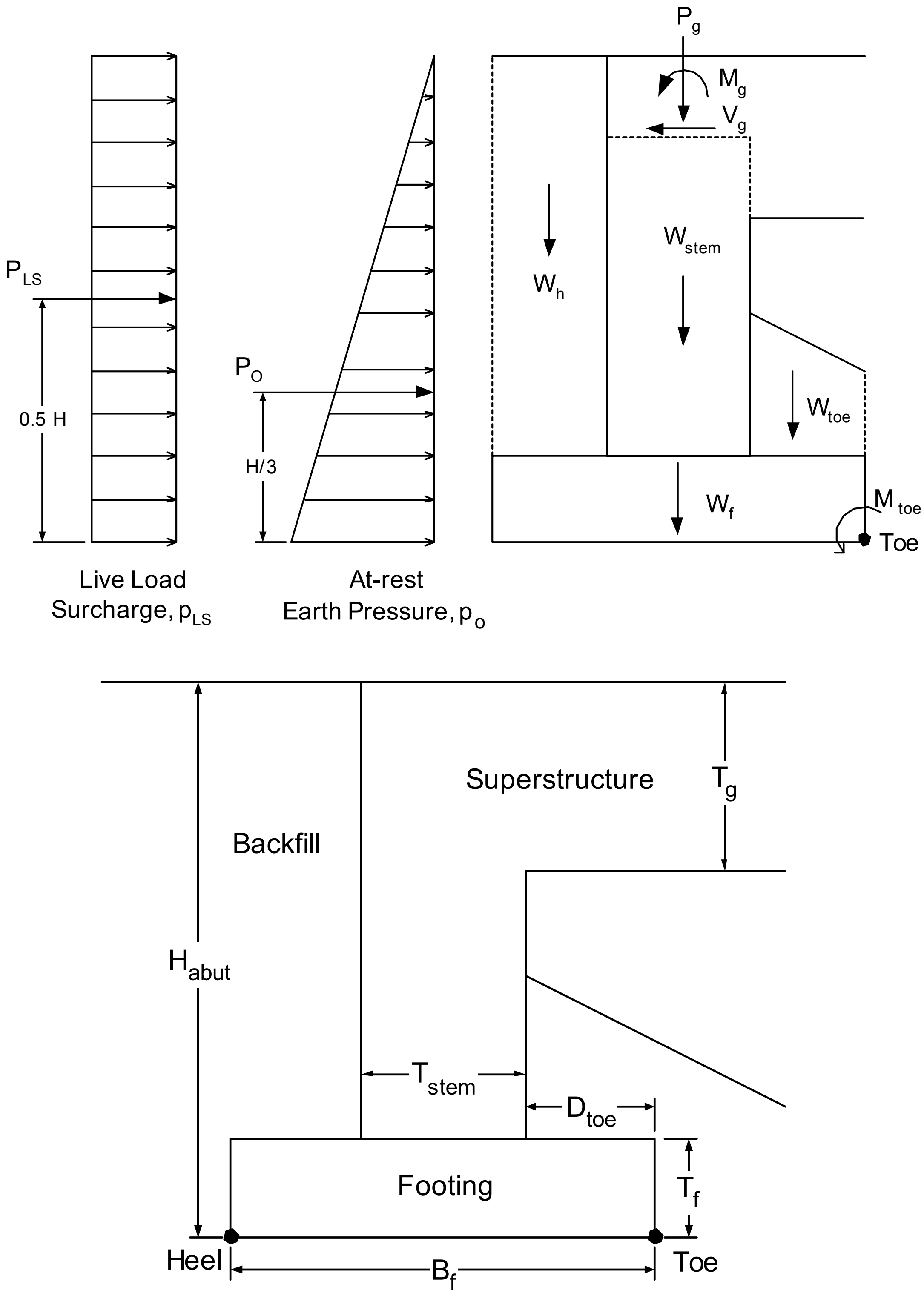


Figure B2-2: Integral Abutment Symbols

Solution:

Steps 1 through 4 – Preliminary bridge layout, review of existing geologic and subsurface data, site reconnaissance, scour and frost potential:

The structural, hydraulic and geotechnical engineers completed these steps. The required design information is provided in the problem-given statement.

Step 5 – Determine the loads applied to the footing:

The superstructure loads are transmitted to the abutment at the girder-stem connection as shown in Figure B2-2. The sense of the shear force is positive if it is in the direction from the toe to the heel (right to left) and negative if opposite. The sense of the moment of the couple is positive for resisting moments (counterclockwise) and negative for overturning moments (clockwise). Moments of these loads are taken about the toe of the footing. Their sense is determined in the same manner as for the couple M_g. Application of the superstructure loads is assumed at the top of the effective abutment stem, or one-half the girder height.

Loads from the superstructure (per m length of abutment wall):

The structural engineer has determined the following.

TABLE B2-1: LOADS FROM SUPERSTRUCTURE

Load	Vertical, P_g (kN/m)	Shear, V_g (kN/m)	Couple, M_g (kN•m/m)	Moment, M_{toe} (kN•m/m)
Dead load of components (DC)	169.22	42.61	-158.00	272.27
Dead load of wearing surfaces (DW)	19.99	16.64	-57.43	42.61
Vehicular live load (LL)	48.60	22.91	-65.65	103.17
Vehicular braking force (BR)	0.58	3.65	-10.02	6.31

Dead loads of the footing and earth fill (per m length of abutment wall):

Weight of the stem, W_{stem}:

$$\begin{aligned} W_{stem} &= \gamma_{conc} T_{stem} (H_{abut} - T_f - 0.5T_g) \\ &= (23.5\,kN/m^3)(0.51\,m)(5.24\,m - 0.460 - 0.5(2.0\,m)) \\ &= 45.30\,kN/m \end{aligned}$$

$$\begin{aligned} arm_{stem} &= D_{toe} + T_{stem}/2 \\ &= 1.22\,m + 0.51\,m/2 \\ &= 1.48\,m \end{aligned}$$

$$
\begin{aligned}
M_{toe} &= W_{stem}(arm_{stem}) \\
&= (45.30\,\text{kN/m})(1.48\,\text{m}) \\
&= 67.04\,\text{kN}\cdot\text{m/m}
\end{aligned}
$$

Weight of the footing, W_f (B_f in m):

$$
\begin{aligned}
W_f &= \gamma_{conc} T_f B_f \\
&= (23.5\,\text{kN/m}^3)(0.46\,\text{m})B_f \\
&= 10.81 B_f\ \text{kN/m}
\end{aligned}
$$

$$
arm_f = B_f / 2\,\text{m}
$$

$$
\begin{aligned}
M_{toe} &= W_f(arm_f) \\
&= 5.41 {B_f}^2\ \text{kN}\cdot\text{m/m}
\end{aligned}
$$

Weight of the soil over the toe, W_{toe}:

Assume granular backfill for toe and heel $(\gamma = 19.6\,\text{kN/m}^3,\ \phi' = 36°)$. Calculate the weight of the soil over the toe for the minimum cover.

$$
\begin{aligned}
W_{toe} &= \gamma_{soil}(D_{toe})(D_f - T_f) \\
&= (19.6\,\text{kN/m}^3)(1.22\,\text{m})(0.91\,\text{m}) \\
&= 21.76\,\text{kN/m}
\end{aligned}
$$

$$
\begin{aligned}
arm_{toe} &= D_{toe}/2 \\
&= 1.22\,\text{m}/2 \\
&= 0.61\,\text{m}
\end{aligned}
$$

$$
\begin{aligned}
M_{toe} &= W_{toe}(arm_{toe}) \\
&= (21.76\,\text{kN/m})(0.61\,\text{m}) \\
&= 13.27\text{kN}\cdot\text{m/m}
\end{aligned}
$$

Weight of the soil over the heel, W_h: B_f in m

$$
\begin{aligned}
W_h &= \gamma_{soil}(B_f - D_{toe} - T_{stem})(H_{abut} - T_f) \\
&= (19.6\,\text{kN/m}^3)(B_f - 1.22\,\text{m} - 0.51\,\text{m})(5.24\,\text{m} - 0.46\,\text{m}) \\
&= (93.69)(B_f - 1.73)\,\text{kN/m}
\end{aligned}
$$

$$\text{arm}_h = D_{toe} + T_{stem} + \frac{(B_f - D_{toe} - T_{stem})}{2}$$
$$= 1.22\,m + 0.51\,m + \frac{(B_f - 1.22\,m - 0.51\,m)}{2}$$
$$= 0.865 + 0.5B_f \text{ m}$$

$$M_{toe} = W_h(\text{arm}_h)$$
$$= (93.69)(B_f - 1.73)(0.865 + 0.5B_f) \text{ kN}\cdot\text{m/m}$$

Loads from the approach fill:

Earth Pressure Load: It is assumed that at-rest earth pressures will exist behind the wall because the wall is restrained from rotating sufficiently to prevent active earth pressures from developing. Assume granular backfill $(\gamma = 19.6\,\text{kN/m}^3,\ \phi' = 36°)$ behind the wall. The coefficient of earth pressure at rest is

$$K_o = 1 - \sin\phi'$$
$$= 1 - \sin 36°$$
$$= 0.41$$

$$P_o = -(0.5)K_o\gamma H_{abut}^2$$
$$= -(0.5)(0.41)(19.6\,\text{kN/m}^3)(5.24\,\text{m})^2$$
$$= -110.32\,\text{kN/m}$$

$$\text{arm}_{P_o} = H_{abut}/3$$
$$= 5.24\,\text{m}/3$$
$$= 1.75\,\text{m}$$

$$M_{toe} = P_o(\text{arm}_{P_o})$$
$$= (-110.32\,\text{kN/m})(1.75\,\text{m})$$
$$= -193.06\,\text{kN}\cdot\text{m/m}$$

Horizontal earth pressure from live load surcharge: Use H_{LS} = 0.60 m.

$$P_{LS} = -(K_o\gamma H_{LS})H_{abut}$$
$$= -(0.41)(19.6\,\text{kN/m}^3)(0.60\,\text{m})(5.24\,\text{m})$$
$$= -25.27\,\text{kN/m}$$

$$
\begin{aligned}
arm_{LS} &= (0.5)H_{abut} \\
&= (0.5)(5.24\,m) \\
&= 2.62\,m
\end{aligned}
$$

$$
\begin{aligned}
M_{toe} &= P_{LS}(arm_{LS}) \\
&= (-25.27\,kN/m)(2.62\,m) \\
&= -66.21\,kN \cdot m/m
\end{aligned}
$$

The above are summarized in Table B2-2.

TABLE B2-2: LOADS OF ABUTMENT COMPONENTS AND HORIZONTAL EARTH PRESSURES

Load	Vertical (kN/m)	Horizontal (kN/m)	Moment, M_{toe} (kN•m/m)
Weight of stem (W_{stem})	45.30	-	67.04
Weight of footing (W_f)	$10.81B_f$	-	$5.41B_f^2$
Weight of soil over toe (W_{toe})	21.76	-	13.27
Weight of soil over heel (W_h)	$(93.69)(B_f-1.73)$	-	$(93.69)(B_f-1.73)$ $*(0.865+0.5B_f)$
At-rest earth pressure load (P_o)	-	-110.32	-193.06
Horizontal earth load from live load surcharge (P_{LS})	-	-25.27	-66.21

Note: B_f = width of footing in m

Step 6 – Field exploration and laboratory testing:

This step is complete, and the subsurface data are shown in Figure B2-1. It was conservatively assumed that N′ (SPT blowcount corrected for overburden pressure) would not change after the construction of the approach embankment. For this example, the subsurface data includes the structural fill under the footing.

The initial vertical stresses are calculated using the layers shown in Figure B2-3. The initial effective vertical stresses are defined as the stresses that exist after the approach embankment is constructed up to the bearing elevation (bottom of the footing). Thus the initial vertical stresses at the midpoint of each layer are calculated as follows. It is assumed that the approach embankment is constructed to its full height concurrently with the abutment and superstructure construction.

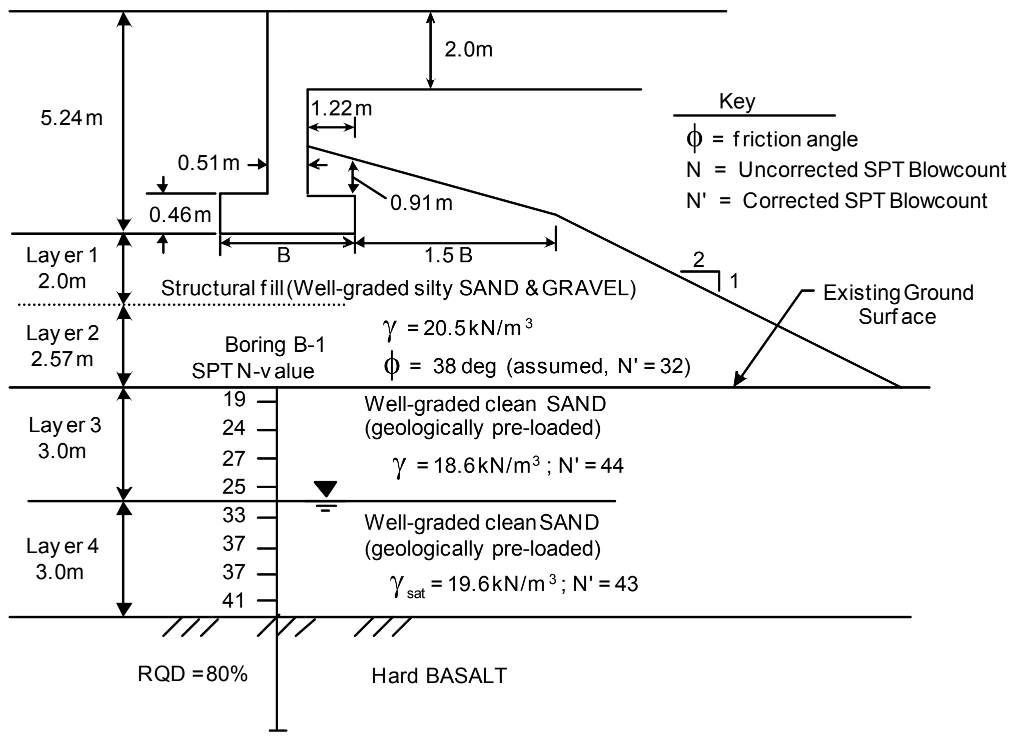


Figure B2-3: Layers Used in the Analysis

Compute the initial vertical effective stresses at the midpoints of each layer below the footing:

Layer 1: Structural fill ($H_1 = 2.0$ m)

$$\begin{aligned}\sigma'_{vo_1} &= \gamma_1 (H_1/2) \\ &= (20.5\,kN/m^3)(2.0\,m/2) \\ &= 20.5\,kPa\end{aligned}$$

Layer 2: Structural fill ($H_2 = 2.57$ m)

$$\begin{aligned}\sigma'_{vo_2} &= \gamma_1 (H_1) + \gamma_2 (H_2/2) \\ &= (20.5\,kN/m^3)(2.0\,m) + (20.5\,kN/m^3)(2.57\,m/2) \\ &= 67.3\,kPa\end{aligned}$$

Layer 3: Well-graded clean sand ($H_3 = 3.0$ m)

$$\begin{aligned}\sigma'_{vo_3} &= \gamma_1 (H_1) + \gamma_2 (H_2) + \gamma_3 (H_3/2) \\ &= (20.5\,kN/m^3)(2.0\,m) + (20.5\,kN/m^3)(2.57\,m) + (18.6\,kN/m^3)(3.0\,m/2) \\ &= 121.6\,kPa\end{aligned}$$

Layer 4: Saturated, well-graded clean sand ($H_4 = 3.0$ m)

$$\begin{aligned}\sigma'_{vo_4} &= \gamma_1(H_1) + \gamma_2(H_2) + \gamma_3(H_3) + \gamma'_4(H_4/2) \\ &= (20.5\,kN/m^3)(2.0\,m) + (20.5\,kN/m^3)(2.57\,m) + \\ &\qquad (18.6\,kN/m^3)(3.0\,m) + (19.6\,kN/m^3 - 9.81\,kN/m^3)(3.0\,m/2) \\ &= 164.1\,kPa\end{aligned}$$

Step 7 – Calculate allowable bearing capacity:

Calculate the allowable bearing capacity, considering both the bearing capacity failure and settlement.

Calculate q_{all} Based on Bearing Capacity Failure:

Use Equation 5-14.

$$q_{ult} = cN_c s_c b_c + qN_q C_{Wq} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{W\gamma} s_\gamma b_\gamma \qquad \text{(Eqn. 5-14)}$$

In our example, the soils are all cohesionless, and c = 0. So the first term becomes zero. Assume no cover on the footing, so the second term also becomes zero. Replace N_γ with $N_{\gamma q}$ from Figure 5-7 to account for slope effects.

Look at footing widths of B_f = 2, 3, and 4 m.

$B_f = 2$ m:

$$\frac{L_f}{B_f} = \frac{25.0\,m}{2.0\,m} = 12.5 > 5$$

This is greater than five, so the shape factor is

$$s_\gamma = 1 \qquad \text{(Table 5-2)}$$

The base of the footing is horizontal, so the base factor is

$$b_\gamma = 1 \qquad \text{(Table 5-5)}$$

The water table is 4.57 m + 3.0 m = 7.57 m below the base of the footing and is greater than 1.5 B_f = (1.5)(2.0 m) = 3.0 m. Therefore the ground water correction factor is

$$C_{W\gamma} = 1$$

A slope inclination angle of *i* = 26.6º (for 2H : 1V slope) will be used to obtain the bearing capacity factor, $N_{\gamma q}$. Although the upper portion of the slope is flatter, this will be more conservative.

$$D_f / B_f = 1.37\,m / 2.0\,m = 0.69$$

The drained friction angle of the structural fill is assumed to be $\phi' = 38°$. Using Figure 5-7c, for i = 26.6º and $\phi' = 38°$, the following are obtained:

For $D_f / B_f = 0$, $N_{\gamma q} \cong 17$.

For $D_f / B_f = 1$, $N_{\gamma q} \cong 80$.

By interpolation, for $D_f / B_f = 0.69$,

$$N_{\gamma q} = 17 + (0.69)(80 - 17) \cong 60$$

Thus

$$\begin{aligned} q_{ult} &= 0.5\gamma B_f N_{\gamma q} C_{W\gamma} s_\gamma b_\gamma \\ &= (0.5)(20.5\,\mathrm{kN/m^3})(2.0\,\mathrm{m})(60)(1)(1)(1) \\ &= 1230\,\mathrm{kPa} \end{aligned}$$

Using a factor of safety of 3, the allowable bearing capacity is

$$\begin{aligned} q_{all} &= q_{ult} / 3 \\ &= 1230\,\mathrm{kPa} / 3 \\ &= 410\,\mathrm{kPa} \end{aligned}$$

<u>B_f = 3 m:</u>

Proceeding in a similar manner, we obtain the following.

$$\frac{L_f}{B_f} = \frac{25.0\,\mathrm{m}}{3.0\,\mathrm{m}} = 8.33 > 5$$

$s_\gamma = 1$, $b_\gamma = 1$, and $C_{W\gamma} = 1$.

$D_f / B_f = 1.37\,\mathrm{m} / 3\,\mathrm{m} = 0.46$

By interpolation, for $D_f / B_f = 0.46$,

$$N_{\gamma q} = 17 + (0.46)(80 - 17) \cong 46$$

Thus

$$\begin{aligned} q_{ult} &= 0.5\gamma B_f N_{\gamma q} C_{W\gamma} s_\gamma b_\gamma \\ &= (0.5)(20.5\,\mathrm{kN/m^3})(3.0\,\mathrm{m})(46)(1)(1)(1) \\ &= 1414\,\mathrm{kPa} \end{aligned}$$

Using a factor of safety of 3, the allowable bearing capacity is

$$\begin{aligned} q_{all} &= q_{ult} / 3 \\ &= 1414\,\text{kPa} / 3 \\ &= 471\,\text{kPa} \end{aligned}$$

<u>$B_f = 4$ m:</u>

Proceeding in a similar manner, we obtain the following.

$$\frac{L_f}{B_f} = \frac{25.0\,\text{m}}{4.0\,\text{m}} = 6.25 > 5$$

$s_\gamma = 1$, $b_\gamma = 1$, and $C_{W\gamma} = 1$.

$$D_f / B_f = 1.37\,\text{m} / 4\,\text{m} = 0.34$$

When $B_f = 4$ m, the potential failure surface can extend into Layer 3 (well-graded sand). Check to determine if the design friction angle needs to be modified for the purpose of calculating bearing capacity. From Figure B2-3, the average uncorrected SPT N-value for Layer 3 is about 24. The effective vertical stress at the midpoint of Layer 3 at the time of drilling was:

$$\sigma'_{vo_{drilling}} = \gamma_3 (H_3 / 2) = (18.6\,\text{kN/m}^3)(3\,\text{m}/2) = 27.9\,\text{kPa} \quad (\cong 0.58\,\text{kips/ft}^2)$$

Entering Figure 4-1 with a blowcount of 24 and a vertical effective stress of 0.58 ksf, a relative density, D_R, of about 95 percent is read. Entering Figure 4-2 with this relative density for a well-graded sand (Unified Soil Classification 'SW'), a friction angle of 39 degrees is obtained. It is therefore conservative and acceptable to use ϕ' of $38°$ to obtain $N_{\gamma q}$.

By interpolation, for $D_f / B_f = 0.34$,

$$N_{\gamma q} = 17 + (0.34)(80 - 17) \cong 38$$

Also use a weighted average of the soil unit weights in Layers 2 and 3:

$$\gamma_{ave} = \frac{20.5\,\text{kN/m}^3 + 18.6\,\text{kN/m}^3}{2} = 19.6\,\text{kN/m}^3$$

The bearing capacity is therefore:

$$\begin{aligned} q_{ult} &= 0.5 \gamma_{ave} B_f N_{\gamma q} C_{W\gamma} s_\gamma b_\gamma \\ &= (0.5)(19.6\,\text{kN/m}^3)(4.0\,\text{m})(38)(1)(1)(1) \\ &= 1490\,\text{kPa} \end{aligned}$$

Using a factor of safety of 3, the allowable bearing capacity is

$$
\begin{aligned}
q_{all} &= q_{ult} / 3 \\
&= 1490\,\text{kPa} / 3 \\
&= 497\,\text{kPa}
\end{aligned}
$$

Calculate q_{all} based on allowable settlement:

To estimate the bridge loading to generate 25 mm of settlement, use both the Hough method and D'Appolonia's method. Use the Hough method to estimate the settlement in the structural fill Layers 1 and 2. However, because the native sand of Layers 3 and 4 is known to be geologically preloaded, use D'Appolonia's method to avoid overestimating the settlement in these layers. The total settlement can be calculated by summing the settlements in each layer.

The Hough method estimates settlements based on average changes in stress for the layer. D'Appolonia's method estimates settlements based on the applied stress at the top of the layer. Therefore, calculate the stress increase at the midpoint of Layers 1 and 2, and at the top of Layer 3 using the 2-on-1 stress distribution method for footing widths of $B_f = 2, 3, 4$ m.

$$
\frac{\Delta\sigma_v}{q} = \frac{B_f L_f}{(B_f + Z)(L_f + Z)} \qquad \text{(Eqn. 5-17)}
$$

where q = bridge loading per unit area at the base of the footing
Z = depth to the midpoint of a layer

Using this equation, $\Delta\sigma_v / q$, the stress increases due to bridge footing loads are summarized in Table B2-3.

TABLE B2-3: STRESS INCREASE DUE TO BRIDGE LOADING

Soil Layer	Effective Level of Loading	Depth to Effective Level (m)	Stress Increase, $\Delta\sigma_v / q$		
			B_f = 2 m	B_f = 3 m	B_f = 4 m
1	Middle of Layer	1.0	0.64	0.72	0.77
2	Middle of Layer	3.28	0.33	0.42	0.49
3,4	Top of Layer	4.57	0.26	0.34	0.39

Use the Hough method to estimate the settlement in Layers 1 and 2. The general equation is:

$$
\Delta H = H_o \frac{1}{C'} \log\left(\frac{\sigma'_{vo} + \Delta\sigma_{v_f}}{\sigma'_{vo}}\right) \qquad \text{(Eqn. 5-24)}
$$

Bearing capacity index (C') for the structural fill of layers 1 and 2 is obtained from Figure 5-19. For structural fill consisting of well-graded silty sand & gravel, use the recommended corrected

SPT N-value for compacted structural fill of 32. Entering Figure 5-19 with N′ = 32 for well-graded silty sand and gravel, a C′ of 110 is read.

For a footing pressure, q, of 100 kPa and a footing width of B_f = 2 m, the settlement in Layers 1 and 2 are calculated as follows.

Layer 1:

$$\Delta H_1 = H_1 \frac{1}{C_1'} \log\left(\frac{\sigma'_{vo_1} + \frac{\Delta\sigma_{v_1}}{q}(q)}{\sigma'_{vo_1}}\right)$$

$$= (2\,\text{m}) \frac{1}{110} \log\left(\frac{20.5\,\text{kPa} + (0.64)(100\,\text{kPa})}{20.5\,\text{kPa}}\right)$$

$$= 0.0112\,\text{m} = 11.2\,\text{mm}$$

Layer 2:

$$\Delta H_2 = H_2 \frac{1}{C_2'} \log\left(\frac{\sigma'_{vo_2} + \frac{\Delta\sigma_{v_2}}{q}(q)}{\sigma'_{vo_2}}\right)$$

$$= (2.57\,\text{m}) \frac{1}{110} \log\left(\frac{67.3\,\text{kPa} + (0.33)(100\,\text{kPa})}{67.3\,\text{kPa}}\right)$$

$$= 0.004\,\text{m} = 4\,\text{mm}$$

Use D'Appolonia's method to estimate the settlement in Layers 3 and 4. General equation is:

$$\Delta H = \left(\frac{\Delta\sigma_V B_f}{M}\right)\mu_o \mu_1 \qquad \text{(Eqn. 5-25)}$$

The modulus of compressibility of sand and gravel (M) for Layers 3 and 4 is obtained from Figure 5-21, and the correction factors (μ_o and μ_1) are obtained from Figure 5-20. Combining Layers 3 and 4 into one layer, the average uncorrected SPT N-value for the layer is 30. Entering Figure 5-21 with an average uncorrected blowcount of 30 and an M of 750 tsf, or 71820 kPa, is read for preloaded sand. Entering the upper chart of Figure 5-20 for D = 4.57 m and B = 6.57 to 8.57 m (based on the 2-on-1 stress distribution method and B_f = 2, 3, and 4 m), a μ_o of 0.74 is read. Entering the lower chart of Figure 5-20 for H = 6 m and B = 6.57 to 8.57 m, a μ_1 of 0.25 is read.

For a footing pressure of 100 kPa and a footing width of $B_f = 2$ m, the settlement in layers 3 and 4 are calculated as follows.

$$\Delta H_{3,4} = \left(\frac{\frac{\Delta\sigma_{v_{3,4}}}{q}(q)(B_f)}{M_{3,4}} \right) \mu_o \mu_1$$

$$= \left(\frac{0.26(100\,\text{kPa})(6.57\,\text{m})}{71820\text{kPa}} \right)(0.74)(0.25)$$

$$= 0.0004\,\text{m} = 0.4\,\text{mm}$$

The total settlement is therefore:

$$\sum \Delta H_i = 11.2 + 4.0 + 0.4 = 16\,\text{mm}$$

In a similar manner, the settlements for different widths and bridge loadings are obtained and are summarized as follows.

TABLE B2-4: SETTLEMENTS DUE TO VARIOUS FOOTING PRESSURES AND WIDTHS

Footing Pressure (kPa)	Settlement (mm)		
	$B_f = 2$ m	$B_f = 3$ m	$B_f = 4$ m
100	16	17	19
150	20	22	24
200	23	26	28
250	26	29	31

Table B2-4 shows that the maximum allowable footing pressure to limit the settlement to 25 mm decreases from about 233 kPa for $B_f = 2$ m to about 163 kPa for $B_f = 4$ m. These values are smaller than the q_{all} values based on bearing capacity failure that were calculated earlier.

Settlement will therefore limit the allowable bearing capacity for the design of this footing. Interpolating the values in Table B2-4, find ranges of footing widths that will limit settlement to 25 mm or less at nominal allowable bearing capacities of 150 kPa, 190 kPa and 230 kPa. These are summarized in Table B2-5.

TABLE B2-5: SETTLEMENT-LIMITED
ALLOWABLE BEARING CAPACITIES

Allowable Bearing Capacity (kPa)	Range of Footing Widths That Will Limit Settlement to 25 mm
230	≤ 2.0 m
190	2.0 to 3.0 m
150	3.0 to 4.0 m

The geotechnical engineer should provide this table to the structural designer for sizing of the footing.

Step 8 – Calculate sliding and passive soil resistance:

The footing concrete will be poured on compacted structural fill. Therefore, the friction angle, δ, to be used in the sliding analysis of the footing will be $\delta = \phi' = 38°$.

The passive resistance of the soil in front of the footing will be ignored. The ultimate sliding resistance is therefore:

$$
\begin{aligned}
F_R &= (W + P_v)\tan\delta && \text{(Eqn. 5-38)} \\
&= (W + P_v)\tan(38°) \\
&= 0.78(W + P_v)
\end{aligned}
$$

Step 9 – Check global stability of the footing:

The geotechnical engineer has checked an abutment and approach embankment with the same dimensions and similar loading conditions but with a weaker foundation soil (silt with N′ = 10) and found that the minimum factor of safety was about 1.6 and satisfactory (see Example 3). Thus, with stronger foundation soils, the factor of safety should be greater than 1.6 and satisfactory.

Step 10 – Size the footing under full loading:

From Table B2-1, the total vertical load from the superstructure is

$$
\begin{aligned}
P_{v\ \text{structure}} &= DC + DW + LL + BR \\
&= 169.22\,\text{kN/m} + 19.99\,\text{kN/m} + 48.60\,\text{kN/m} + 0.58\,\text{kN/m} \\
&= 238.39\,\text{kN/m}
\end{aligned}
$$

Referring to Table B2-2, the additional load from the abutment components per 1-m length of the abutment wall for B_f = 3 m is

$$
\begin{aligned}
W_{stem} &= 45.30\,kN/m \\
W_f &= 10.81B_f = (10.81)(3) = 32.43\,kN/m \\
W_{toe} &= 21.76\,kN/m \\
W_h &= (93.69)(B_f - 1.73) = (93.69)(3 - 1.73) = 118.99\,kN/m \\
P_{v\ abut} &= 45.30 + 32.43 + 21.76 + 118.99 = 218.48\,kN/m
\end{aligned}
$$

Thus

$$
\begin{aligned}
P_v &= P_{v\ structure} + P_{v\ abut} \\
&= 238.39\,kN/m + 218.48\,kN/m \\
&= 457\,kN/m
\end{aligned}
$$

From Table B2-5, an allowable bearing pressure of 190 kPa may be applied by the footing if B_f = 2 to 3 m. Using a footing pressure of 190 kPa, the required footing width B_f is

$$
\begin{aligned}
B_f &= P_v/(190\,kPa) \\
&= (457\,kN/m)/(190\,kPa) \\
&= 2.40\,m
\end{aligned}
$$

This satisfies B_f = 2 to 3 m but appears to be too small to meet overturning requirements for H_{abut} = 5.24 m. A preliminary dimension for a footing width can be estimated as one-half the abutment height, or H_{abut} = (0.5)(5.24 m) = 2.62 m. Therefore, try B_f = 2.7 m.

Step 11 – Check overturning and sliding under full loading:

<u>Check overturning for B_f = 2.7 m:</u>

Analyze for 1 m of abutment wall length.

From Step 10 and Table B2-1, the loads and moments due to the superstructure are

$$
P_{v\ structure} = 238.39\,kN/m
$$

$$
\begin{aligned}
M_{toe\,structure} &= 272.27\,kN\cdot m/m + 42.61\,kN\cdot m/m + 103.17\,kN\cdot m/m + 6.31\,kN\cdot m/m \\
&= 424.36\,kN\cdot m/m
\end{aligned}
$$

Referring to Table B2-2, the loads and moments due to abutment components are

$$
\begin{aligned}
P_{v\ abut} &= 45.30 + 10.81B_f + 21.76 + (93.69)(B_f - 1.73) \\
&= 45.30 + (10.81)(2.7) + 21.76 + (93.69)(2.7 - 1.73) \\
&= 187.13\,kN/m
\end{aligned}
$$

$$
\begin{aligned}
M_{toe\ abut} &= 67.04 + 5.41 B_f^2 + 13.27 + (93.69)(B_f - 1.73)(0.865 + 0.5 B_f) \\
&\qquad -193.06 - 66.21 \\
&= 67.04 + (5.41)(2.7)^2 + 13.27 + (93.69)(2.7 - 1.73)(0.865 + (0.5)(2.7)) \\
&\qquad -193.06 - 66.21 \\
&= 61.78\,\text{kN} \cdot \text{m/m}
\end{aligned}
$$

The total loads and moments are:

$$
\begin{aligned}
P_v &= P_{v\ structure} + P_{v\ abut} \\
&= 238.39\,\text{kN/m} + 187.13\,\text{kN/m} \\
&= 425.52\,\text{kN/m}
\end{aligned}
$$

$$
\begin{aligned}
M_{toe} &= M_{toe\ structure} + M_{toe\ abut} \\
&= 424.36\,\text{kN} \cdot \text{m/m} + 61.78\,\text{kN} \cdot \text{m/m} \\
&= 486.14\,\text{kN} \cdot \text{m/m}
\end{aligned}
$$

The arm of the resultant is

$$
\begin{aligned}
arm_R &= M_{toe} / P_v \\
&= \frac{486.14\ \text{kN} \cdot \text{m/m}}{425.52\ \text{kN/m}} \\
&= 1.14\,\text{m}
\end{aligned}
$$

The eccentricity, e_y, is

$$
\begin{aligned}
e_y &= B_f / 2 - arm_R \\
&= 2.7\,\text{m}/2 - 1.14\,\text{m} \\
&= 1.35\,\text{m} - 1.14\,\text{m} \\
&= 0.21\,\text{m}
\end{aligned}
$$

For stability, the eccentricity should be less than one-sixth the footing width:

$$
e_y = 0.21\,\text{m} < B_f / 6 = 2.7\,\text{m}/6 = 0.45\,\text{m} \qquad \text{(OK)}
$$

<u>Check bearing stress for effective footing area:</u>

$$
\begin{aligned}
B'_f &= B_f - 2 e_y \\
&= 2.7\,\text{m} - (2)(0.21\,\text{m}) \\
&= 2.28\,\text{m}
\end{aligned}
$$

$$
\begin{aligned}
q_{footing} &= P_v / B'_f \\
&= (425.52\,kN/m)(2.28\,m) \\
&= 186.6 kPa
\end{aligned}
$$

From Table B2-5, the allowable bearing pressure for B'_f = 2 to 3 m is 190 kPa. Therefore, $q_{footing} < q_{all}$ and the bearing stress is smaller than the allowable value. (OK)

<u>Check sliding:</u>

In Table B2-1, the vehicular live load (LL) and the vehicular braking force (BR) act to resist sliding. To be conservative, these two loads will be ignored in the sliding analysis. The forces that tend to induce sliding are the sum of the following:

Shear from the dead load of components, DC (Table B2-1)

Shear from the dead load of wearing surfaces, DW (Table B2-1)

At-rest earth pressure load, P_o (Table B2-2)

Horizontal earth load from live load surcharge, P_{LS} (Table B2-2)

Analyzing per m length of the abutment wall, $F_{sliding}$ is

$$
\begin{aligned}
F_{sliding} &= DC + DW + P_o + P_{LS} \\
&= 42.61 + 16.64 - 110.32 - 25.27 \\
&= -76.34\,kN/m
\end{aligned}
$$

The force F_R available to resist sliding (from Step 8) is

$$F_R \quad = 0.78(W + P_v)$$

Neglecting the vehicular live load (LL) and vehicular breaking force (BR):

$$
\begin{aligned}
P_{v\ structure} &= DC + DW \\
&= 169.22 + 19.99\,kN/m = 189.21\,kN/m
\end{aligned}
$$

$$W = P_{v\ abut} \quad = 187.13\,kN/m$$

The factor of safety FS against sliding is

$$
\begin{aligned}
FS &= \frac{F_R}{F_{sliding}} \\
&= \frac{0.78(189.21\,kN/m + 187.13kN/m)}{76.34\,kN/m} \\
&= \frac{293.54\,kN/m}{76.34\,kN/m} \\
&= 3.8
\end{aligned}
$$

FS >1.5 (OK)

Step 12 – Complete structural design of the footing:

The structural engineer completes this step.

EXAMPLE 3: STUB SEAT-TYPE BRIDGE ABUTMENT ON STRUCTURAL FILL

Given:

- A seat-type abutment on a spread footing is proposed for a bridge.
- There is no scour potential at this site.
- The footing length is L = 25.0 m (82 ft); other dimensions are shown in Figure B3-1.
- The approach embankment and the foundation conditions are shown in Figure B3-1.
- The SPT hammer energy ratio is assumed to be ER = 60%.
- Frost penetration depth is 0.6 m (2 ft).
- Tolerable settlement is 38 mm (1.5 in).
- Abutment loading symbols are shown in Figure B3-2.

Find:

- Size the footing based on Service Load Design, Group IV loads (allowable stress design method), considering both settlement and bearing capacity. Check overturning and sliding.

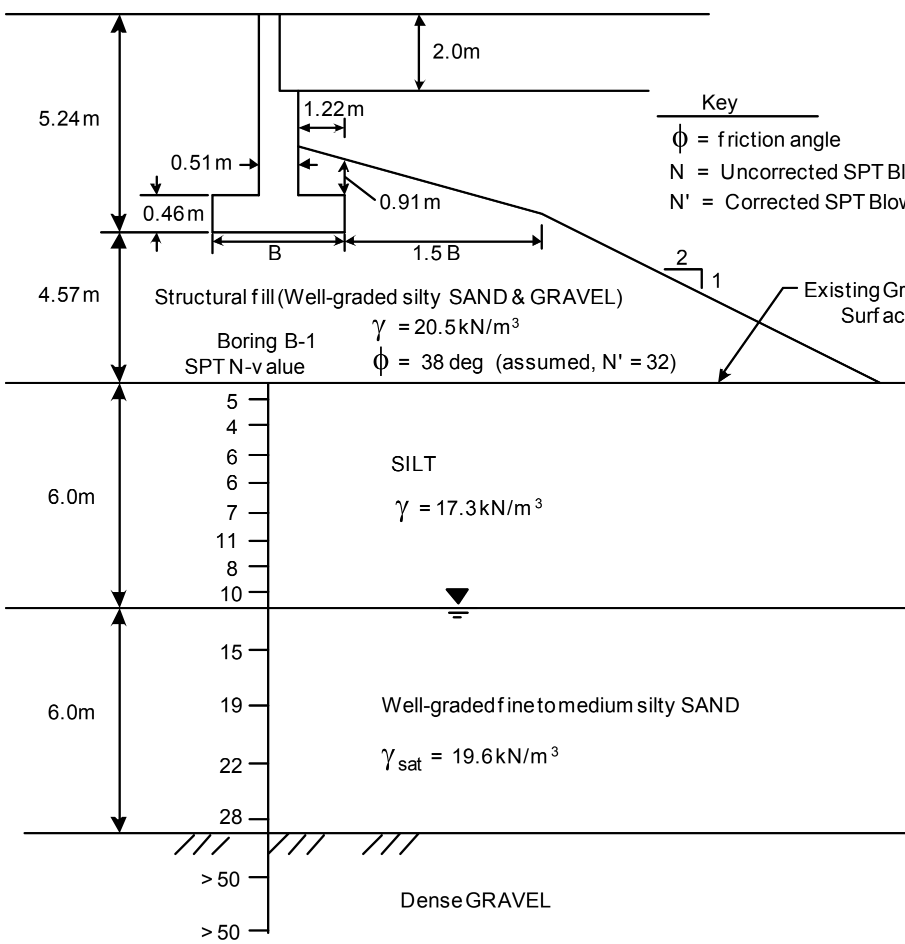


Figure B3-1: Approach Embankment and Foundation Soils

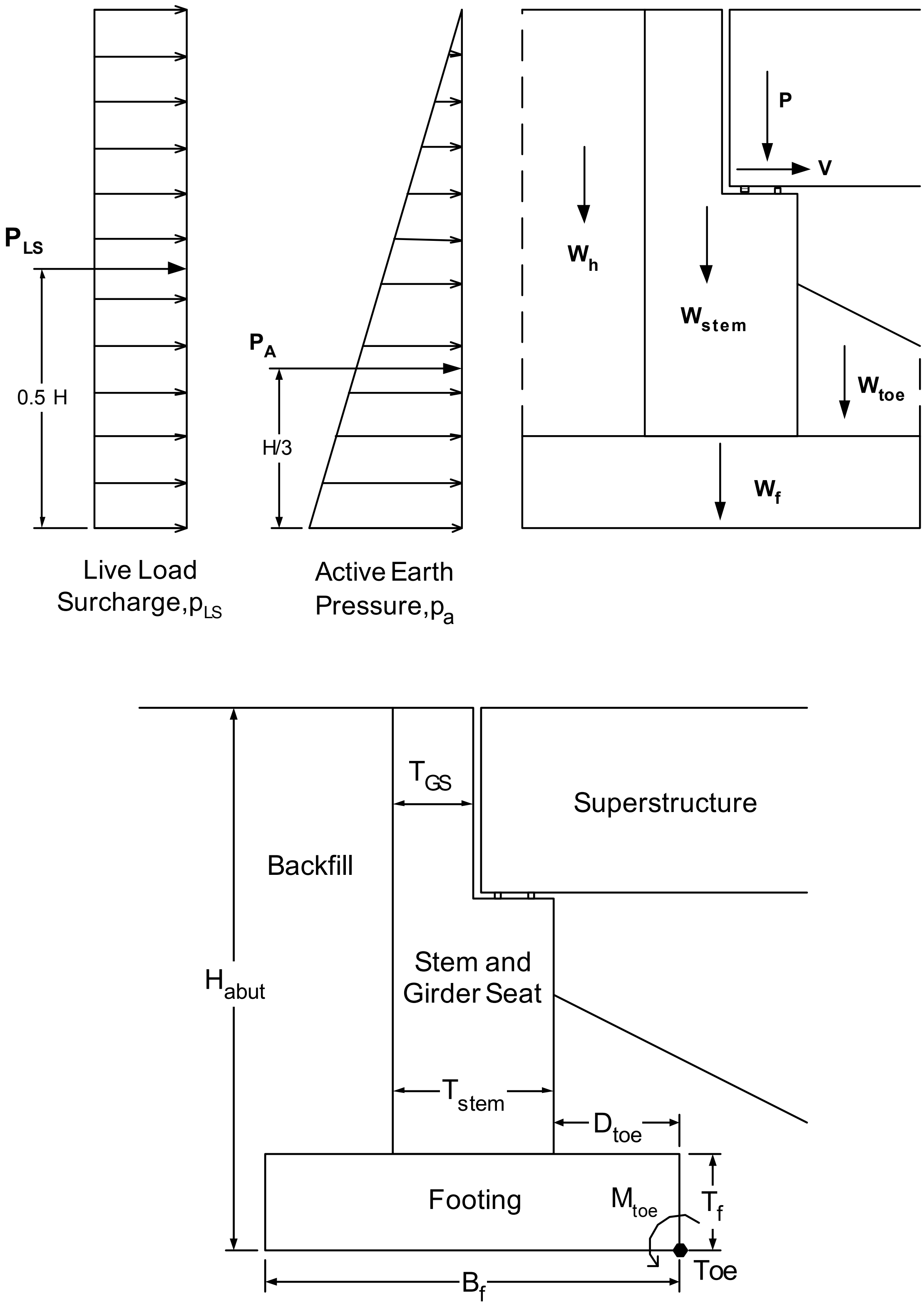


Figure B3-2: Abutment Loading Symbols

Solution:

Steps 1 through 4 – Preliminary bridge layout, review of existing geologic and subsurface data, site reconnaissance, scour and frost potential:

The structural, hydraulic and geotechnical engineers completed these steps. The required design information is provided in the problem-given statement.

Step 5 – Determine the loads applied to the footing:

Moments are taken about the front toe of the footing. The direction of the moment is positive for resisting moments (counterclockwise) and negative for overturning moments (clockwise).

<u>Loads from the girders</u>:

The structural engineer has provided Group IV loads as summarized in Table B3-1. Recall that Group IV is the SLD load combination with normal vehicles using the bridge without wind, and with consideration of long term shrinkage, shortening, and thermal movements.

TABLE B3-1: LOADS FROM GIRDERS

Load	Vertical, P (kN)	Shear, V (kN)	Moment, M_{toe} (kN•m)
Dead load of components (DC)	5233	-	7274
Dead load of wearing surfaces (DW)	446	-	620
Vehicular live load (LL)	1538	-	2138
Shear loads from bearing pads (V)	-	1047	-3392

Dead loads of the abutment components (per m of abutment wall, assume $\gamma_{concrete}$ = 23.5 kN/m^3)

Weight of the stem, W_{stem} (for simplicity, this computation does not subtract out the weight of the seat):

$$\begin{aligned} W_{stem} &= \gamma_{conc} T_{stem} (H_{abut} - T_f) \\ &= (23.5\,kN/m^3)(0.51\,m)(5.24\,m - 0.46\,m) \\ &= 57.29\,kN/m \end{aligned}$$

$$\begin{aligned} arm_{stem} &= D_{toe} + T_{stem}/2 \\ &= 1.22\,m + 0.51\,m/2 \\ &= 1.48\,m \end{aligned}$$

$$\begin{aligned} M_{toe} &= W_{stem}(arm_{stem}) \\ &= (57.29\,kN/m)(1.48\,m) \\ &= 84.79\,kN \cdot m/m \end{aligned}$$

Weight of the footing, W_f (B_f in m):

$$\begin{aligned} W_f &= \gamma_{conc} T_f B_f \\ &= (23.5\,kN/m^3)(0.46\,m)B_f \\ &= 10.81 B_f\ kN/m \end{aligned}$$

$$arm_f = B_f / 2\,m$$

$$\begin{aligned} M_{toe} &= W_f(arm_f) \\ &= 5.41 B_f^{\,2}\ kN \cdot m/m \end{aligned}$$

In this example, the weight of the soil over the toe, $W_{toe,}$ is ignored for design purposes because the soil may not get placed until after the bridge superstructure is constructed. Assume granular backfill for toe and heel $(\gamma = 19.6\,kN/m^3,\ \phi' = 36°)$.

Weight of the soil over the heel, W_h (B_f in m):

$$\begin{aligned} W_h &= \gamma_{soil}(B_f - D_{toe} - T_{stem})(H_{abut} - T_f) \\ &= (19.6\,kN/m^3)(B_f - 1.22\,m - 0.51\,m)(5.24\,m - 0.46\,m) \\ &= (93.69)(B_f - 1.73)\,kN/m \end{aligned}$$

$$\begin{aligned} arm_h &= D_{toe} + T_{stem} + \frac{(B_f - D_{toe} - T_{stem})}{2} \\ &= 1.22\,m + 0.51\,m + \frac{(B_f - 1.22\,m - 0.51\,m)}{2} \\ &= 0.865 + 0.5 B_f\ m \end{aligned}$$

$$\begin{aligned} M_{toe} &= W_h(arm_h) \\ &= (93.69)(B_f - 1.73)(0.865 + 0.5B_f)\ kN \cdot m/m \end{aligned}$$

<u>Loads from Approach Fill</u>:

Active earth pressure: Assume that the stem wall will rotate sufficiently to develop active earth pressures behind the wall. Use Rankine's active earth pressure coefficient. Assume granular backfill $(\gamma = 19.6\,kN/m^3,\ \phi' = 36°)$ behind the wall.

$$K_a = \tan^2\left(45° - \frac{\phi'}{2}\right) \quad \text{(Eqn. 6-3)}$$

$$= \tan^2\left(45° - \frac{36°}{2}\right)$$

$$= 0.26$$

$$P_A = (0.5)K_a \gamma H_{abut}^2$$

$$= (0.5)(0.26)(19.6\,kN/m^3)(5.24)^2$$

$$= 69.96\,kN/m$$

$$arm_{P_A} = H_{abut}/3$$

$$= 5.24\,m/3$$

$$= 1.75\,m$$

$$M_{toe} = -P_A(arm_{P_A})$$

$$= -(69.96\,kN/m)(1.75\,m)$$

$$= -122.43\,kN \cdot m/m$$

Horizontal earth pressure from live load surcharge (use $H_{LS} = 0.60$ m):

$$P_{LS} = (K_a \gamma H_{LS})H_{abut}$$

$$= (0.26)(19.6\,kN/m^3)(0.60\,m)(5.24\,m)$$

$$= 16.02\,kN/m$$

$$arm_{LS} = (0.5)H_{abut}$$

$$= (0.5)(5.24\,m)$$

$$= 2.62\,m$$

$$M_{toe} = -P_{LS}(arm_{LS})$$

$$= -(16.02\,kN/m)(2.62\,m)$$

$$= -41.97\,kN \cdot m/m$$

These loads are summarized in Table B3-2.

TABLE B3-2: LOADS OF ABUTMENT COMPONENTS
AND HORIZONTAL EARTH PRESSURES

Load	Vertical (kN/m)	Horizontal (kN/m)	Moment, M_{toe} (kN•m/m)
Weight of stem (W_{stem})	57.29	-	84.79
Weight of footing (W_f)	$10.81B_f$	-	$5.41B_f^2$
Weight of soil over toe (W_{toe})	-	-	-
Weight of soil over heel (W_h)	$(93.69)(B_f-1.73)$	-	$(93.69)(B_f-1.73)$ $*(0.865+0.5B_f)$
Active earth pressure load (P_A)	-	69.96	-122.43
Horizontal earth load from live load surcharge (P_{LS})	-	16.02	-41.97

Note: B_f = width of footing in m

Step 6 – Conduct subsurface exploration and laboratory testing:

This step is complete, and the subsurface data are shown in Figure B3-1. It was conservatively assumed that N′ (SPT blowcount corrected for overburden pressure) would not change after the construction of the approach embankment. For this example, the subsurface data include the structural fill under the footing.

The initial vertical stresses are calculated using the layers shown in Figure B3-3. The initial vertical stresses were calculated as the stresses that exist after the approach embankment is constructed to its full height. These are estimated by using the depth from the estimated ground surface of the approach embankment after its construction, as shown in Figure B3-3. The initial vertical stresses at the midpoint of each layer are calculated as follows.

Layer 1: Structural fill (H_1 = 2.0 m); note that the height of backfill is taken as the average height of the surcharge at the stem:

$$\begin{aligned}\sigma'_{vo_1} &= \gamma_{backfill}(3.5\,m) + \gamma_1(H_1/2)\\ &= (19.6\,kN/m^3)(3.5\,m) + (20.5\,kN/m^3)(2.0\,m/2)\\ &= 89.1\,kPa\end{aligned}$$

Layer 2: Structural fill (H_2 = 2.57 m)

$$\begin{aligned}\sigma'_{vo_2} &= \gamma_{backfill}(3.5\,m) + \gamma_1 H_1 + \gamma_2(H_2/2)\\ &= (19.6\,kN/m^3)(3.5\,m) + (20.5\,kN/m^3)(2.0\,m) + (20.5\,kN/m^3)(2.57\,m/2)\\ &= 135.9\,kPa\end{aligned}$$

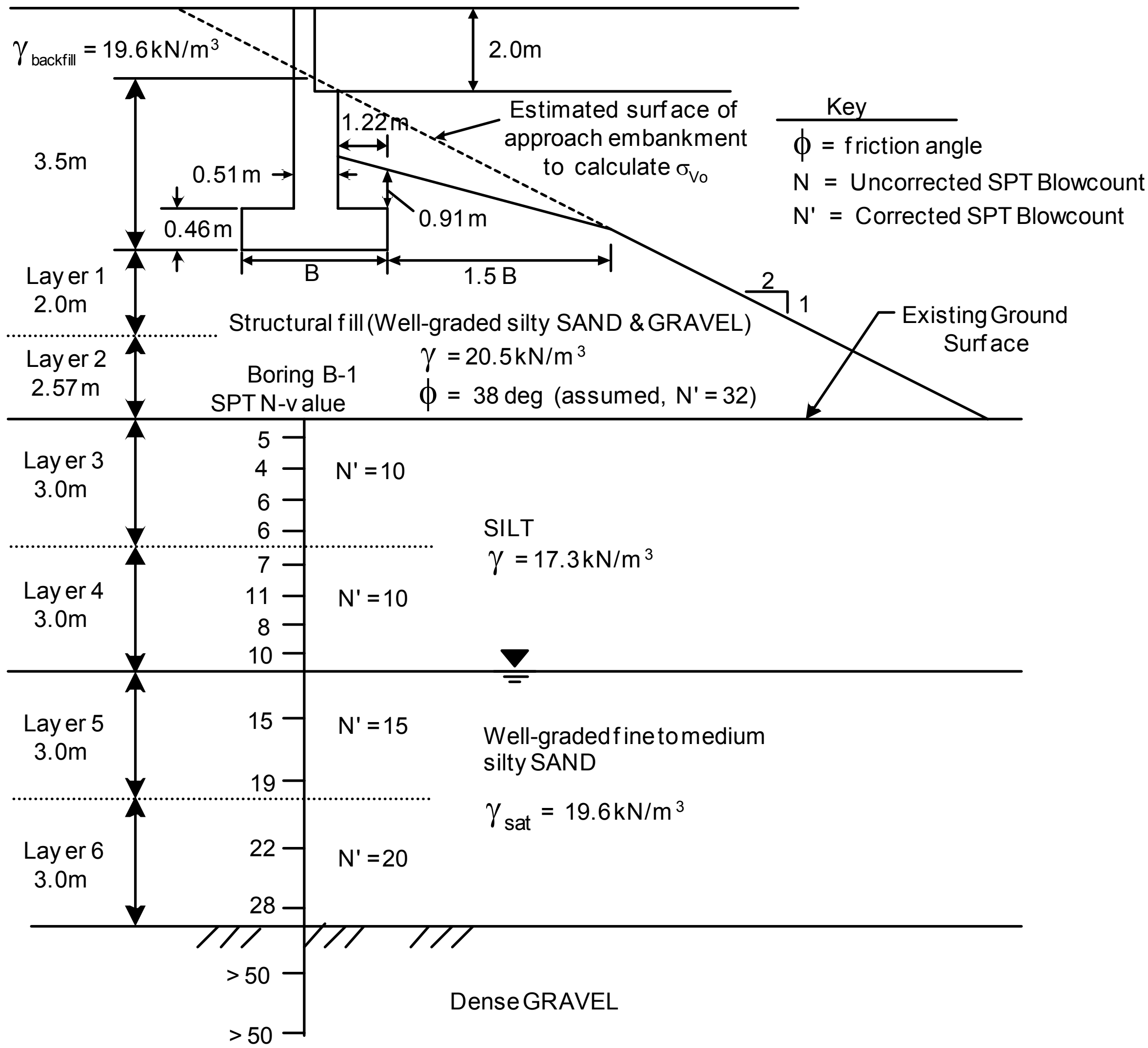


Figure B3-3: Layers Used in the Analysis

Layer 3: Silt (H_3 = 3.0 m)

$$\sigma'_{vo_3} = \gamma_{backfill}(3.5\,m) + \gamma_1 H_1 + \gamma_2 H_2 + \gamma_3 (H_3/2)$$
$$= (19.6\,kN/m^3)(3.5\,m) + (20.5\,kN/m^3)(2.0\,m) + (20.5\,kN/m^3)(2.57\,m)$$
$$+ (17.3\,kN/m^3)(3.0\,m/2)$$
$$= 188.2\,kPa$$

Layer 4: Silt (H_4 = 3.0 m)

$$\begin{aligned}\sigma'_{vo_4} &= \gamma_{backfill}(3.5\,m) + \gamma_1 H_1 + \gamma_2 H_2 + \gamma_3(H_3) + \gamma_4(H_4/2) \\ &= (19.6\,kN/m^3)(3.5\,m) + (20.5\,kN/m^3)(2.0\,m) + (20.5\,kN/m^3)(2.57\,m) \\ &\qquad +(17.3\,kN/m^3)(3.0\,m) + (17.3\,kN/m^3)(3.0\,m/2) \\ &= 240.1\,kPa\end{aligned}$$

Layer 5: Saturated silty sand (H_5 = 3.0 m)

$$\begin{aligned}\sigma'_{vo_5} &= \gamma_{backfill}(3.5\,m) + \gamma_1 H_1 + \gamma_2 H_2 + \gamma_3(H_3) + \gamma_4(H_4) + \gamma_5(H_5/2) \\ &= (19.6\,kN/m^3)(3.5\,m) + (20.5\,kN/m^3)(2.0\,m) + (20.5\,kN/m^3)(2.57\,m) \\ &\qquad +(17.3\,kN/m^3)(3.0\,m) + (17.3\,kN/m^3)(3.0\,m) \\ &\qquad +(19.6 - 9.81\,kN/m^3)(3.0\,m/2) \\ &= 280.7\,kPa\end{aligned}$$

Layer 6: Saturated silty sand (H_6 = 3.0 m)

$$\begin{aligned}\sigma'_{vo_6} &= \gamma_{backfill}(3.5\,m) + \gamma_1 H_1 + \gamma_2 H_2 + \gamma_3(H_3) + \gamma_4(H_4) + \gamma_5(H_5) + \gamma_6(H_6/2) \\ &= (19.6\,kN/m^3)(3.5\,m) + (20.5\,kN/m^3)(2.0\,m) + (20.5\,kN/m^3)(2.57\,m) \\ &\qquad +(17.3\,kN/m^3)(3.0\,m) + (17.3\,kN/m^3)(3.0\,m) \\ &\qquad +(19.6 - 9.81\,kN/m^3)(3.0\,m) + (19.6 - 9.81\,kN/m^3)(3.0\,m/2) \\ &= 310.1\,kPa\end{aligned}$$

Step 7 – Calculate allowable bearing capacity:

Calculate the allowable bearing capacity, considering both bearing capacity failure and settlement.

First calculate q_{all} using bearing capacity theory:

$$q_{ult} = cN_c s_c b_c + qN_q C_{Wq} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{W\gamma} s_\gamma b_\gamma \qquad \text{(Eqn. 5-14)}$$

In this example, the soils are all cohesionless, and c = 0. So the first term becomes zero. Assume no cover on the footing, so the second term also becomes zero. Replace N_γ with $N_{\gamma q}$ from Figure 5-7 to account for slope effects.

Look at footing widths of B_f = 3, 4 and 5 m.

$\underline{B_f = 3\text{ m}}$:

$$\frac{L_f}{B_f} = \frac{25.0\,\text{m}}{3.0\,\text{m}} = 8.33 > 5$$

This ratio is greater than five, so the shape factor is

$$s_\gamma = 1 \qquad \text{(Table 5-2)}$$

The base of the footing is horizontal, therefore the base factor is

$$b_\gamma = 1 \qquad \text{(Table 5-5)}$$

The water table is 4.57 m + 6.0 m = 10.57 m below the base of the footing and is greater than 1.5 B_f = (1.5)(3.0 m) = 4.5 m. Therefore, the ground water correction factor is

$$C_{W\gamma} = 1 \qquad \text{(Table 5-3)}$$

A slope inclination angle of i= 26.6º (for 2H : 1V slope) will be used to obtain the bearing capacity factor $N_{\gamma q}$. This will be conservative, even though the upper portion of the slope is flatter.

$$D_f / B_f = 1.37\,\text{m} / 3.0\,\text{m} = 0.46$$

The drained friction angle of the structural fill is assumed to be $\phi' = 38°$. Using Figure 5-7f, for i = 26.6º and $\phi' = 38°$, the following are obtained:

For $D_f / B_f = 0$, $N_{\gamma q} \cong 17$.

For $D_f / B_f = 1$, $N_{\gamma q} \cong 80$.

By interpolation, for $D_f / B_f = 0.46$,

$$N_{\gamma q} = 17 + (0.46)(80 - 17) \cong 46$$

The ultimate bearing capacity is

$$\begin{aligned} q_{ult} &= 0.5\gamma B_f N_{\gamma q} C_{W\gamma} s_\gamma b_\gamma \\ &= (0.5)(20.5\,\text{kN/m}^3)(3.0\,\text{m})(46)(1)(1)(1) \\ &= 1414\,\text{kPa} \end{aligned}$$

Using a factor of safety of 3, the allowable bearing capacity is

$$\begin{aligned} q_{all} &= q_{ult}/3 \\ &= 1414\,\text{kPa}/3 \\ &= 471\,\text{kPa} \end{aligned}$$

<u>B_f = 4 m</u>:

Proceeding in a similar manner,

$$\frac{L_f}{B_f} = \frac{25.0\,\text{m}}{4.0\,\text{m}} = 6.25 > 5$$

$$s_\gamma = 1,\ b_\gamma = 1, \text{ and } C_{W\gamma} = 1$$

$$D_f / B_f = 1.37\,\text{m} / 4\,\text{m} = 0.34$$

By interpolation, for $D_f / B_f = 0.34$,

When B_f = 4 m, the potential failure surface can extend into Unit 3 (silt). To determine how much of the failure surface will engage Unit 3, use the rule of thumb that the failure surface will extend to a maximum depth of about 1.5 times the footing width, or in this case 1.5(4 m) = 6 m. This extends about 1.5 m into Unit 3, which corresponds to about one-half of the failure surface passing through Unit 3 (visually estimated from Figure B3-1). Therefore, use a weighted average friction angle to obtain $N_{\gamma q}$. A design friction angle associated with Layer 3 is needed. From Figure B3-3, the average uncorrected SPT N-value for this layer is about 5.2. The effective vertical stress at the midpoint at the time of drilling was:

$$\sigma'_{v_{drilling}} = \gamma_3 (H_3 / 2) = (17.3\,\text{kN}/\text{m}^3)(3\,\text{m}/2) = 25.95\,\text{kPa} \cong 0.54\,\text{kips}/\text{ft}^2$$

Entering Figure 4-1 with a blowcount of 5.2 and a vertical effective stress of 0.54 ksf, a relative density, D_R, of about 50 percent is read. Entering Figure 4-2 with this relative density for lean silt (Unified Soil Classification 'ML'), a friction angle of 32º is obtained. Use a reduced ϕ' from $38°$ to $34°$ to obtain $N_{\gamma q}$. Using Figure 5-7, for *i* = 26.6º and $\phi' = 34°$, the following is obtained:

$$N_{\gamma q} = 12 + (0.34)(43 - 12) \cong 23$$

Use a weighted average of the soil unit weights in Units 2 and 3:

$$\gamma_{average} = \frac{20.5\,\text{kN}/\text{m}^3 + 17.3\,\text{kN}/\text{m}}{2} = 18.9\,\text{kN}/\text{m}^3$$

The ultimate bearing capacity is

$$\begin{aligned} q_{ult} &= 0.5\gamma B_f N_{\gamma q} C_{W\gamma} s_\gamma b_\gamma \\ &= (0.5)(18.9\,kN/m^3)(4.0\,m)(23)(1)(1)(1) \\ &= 869\,kPa \end{aligned}$$

Using a factor of safety of 3, the allowable bearing capacity is

$$\begin{aligned} q_{all} &= q_{ult}/3 \\ &= 869\,kPa/3 \\ &= 290\,kPa \end{aligned}$$

<u>B_f = 5 m</u>:

Proceeding in a similar manner,

$$\frac{L_f}{B_f} = \frac{25.0\,m}{5.0\,m} = 5$$

$s_\gamma = 1$, $b_\gamma = 1$, and $C_{W\gamma} = 1$.

$$D_f / B_f = 1.37\,m / 5\,m = 0.27$$

When B_f = 5 m, the potential failure surface can extend deeply into Unit 3 (silt). Therefore, conservatively use Unit 3 soil properties (γ = 17.3 kN/m^3, ϕ = 32 degrees) to compute bearing capacity:

For $D_f / B_f = 0$, $N_{\gamma q} \cong 7$.

For $D_f / B_f = 1$, $N_{\gamma q} \cong 32$.

By interpolation, for $D_f / B_f = 0.27$,

$$N_{\gamma q} = 7 + (0.27)(32 - 7) \cong 14$$

Thus

$$\begin{aligned} q_{ult} &= 0.5\gamma B_f N_{\gamma q} C_{W\gamma} s_\gamma b_\gamma \\ &= (0.5)(17.3\,kN/m^3)(5.0\,m)(14)(1)(1)(1) \\ &= 605\,kPa \end{aligned}$$

Using a factor of safety of 3, the allowable bearing capacity is

$$
\begin{aligned}
q_{all} &= q_{ult} / 3 \\
&= 605\,\text{kPa} / 3 \\
&= 202\,\text{kPa}
\end{aligned}
$$

Calculate q_{all} based on allowable settlement:

Since the embankment weight is so large, settlement due to approach embankment construction is, in general, much greater than the settlement due to bridge loading alone. Due to the presence of the compressible silt, the settlement due to the approach embankment alone was computed using the FHWA program EMBANK. The input parameters for soil unit weight and compression index shown below were estimated from the subsurface exploration SPT N-values per the Hough method. The EMBANK output is shown in Figure B3-4.

```
      ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration
            INCREMENT OF STRESSES BENEATH THE END OF FILL CONDITION

 Project Name    : Example 3              Client           : Shallow Fnd GEC
 File Name       : Ex3.dat                Project Manager  :
 Date            : 12/20/01               Computed by      : rek


            Settlement for X = 106.00 (ft)        Y =  64.00 (ft)

 Embank. slope, x direc. =    64.00 (ft)  Height of fill H     =   32.00 (ft)
                y direc. =    64.00 (ft)  Unit weight of fill =  130.00 (pcf)
 Embankment top width    =    82.00 (ft)  p load/unit area     = 4160.00 (psf)
 Embankment bottom width =   210.00 (ft)  Foundation Elev.     =  100.00 (ft)
 Ground Surface Elev.    =   100.00 (ft)
 Water table Elev.       =    80.00 (ft)  Unit weight of Wat. =   62.40 (pcf)


      LAYER              COMP.    RECOMP.   SWELL.     UNIT          Settlement
 No.  TYPE  THICK.      (1/C')    RATIO               WEIGHT
             (ft)                                      (pcf)             (in.)

  1   COMP.  20.0       0.033     0.003     0.000     110.00             5.73
  2   COMP.  10.0       0.020     0.002     0.000     125.00             0.93
  3   COMP.  10.0       0.017     0.002     0.000     125.00             0.65

                                               Total Settlement =       7.31


                SUBLAYER                   SOIL STRESSES
    No.  THICK.  ELEV.    INITIAL  INCREMENT  MAX.PAST PRESS.    SETTLEMENT
          (ft)    (ft)     (psf)     (psf)        (psf)             (in.)

     1  10.00    95.00    550.00   4073.24      550.00             3.66
     2  10.00    85.00   1650.00   3846.32     1650.00             2.07
     3  10.00    75.00   2513.00   3614.73     2513.00             0.93
     4  10.00    65.00   3139.00   3373.09     3139.00             0.65

                                           Total Settlement =       7.31 (in.)
                                                            =       186 mm
      Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu
```

Figure B3-4: Summary Output of EMBANK Settlement Analysis

The settlement was computed to be 186 mm. This is excessive for the bridge. It will therefore be necessary to mitigate the settlement by surcharging the site prior to construction of the abutment foundation. The contract will specify that the embankment must be built full height and to full dimensions and settlement monitored until complete, before excavating back down to the bearing elevation and constructing the footing. Since the settlement potential is mostly due to the inelastic silt layer, the settlement should occur relatively quickly, and the potential for post-construction settlement due to secondary consolidation (creep) will be low. Therefore, a spread footing is still a cost-effective foundation choice for this site.

The design of the abutment footing will therefore need to consider only the potential settlement due to the structure loads, as follows:

Calculate the stress increase at the midpoint of each soil layer using the 2-on-1 stress distribution method for footing widths of $B_f = 3, 4, 5$ m.

$$\frac{\Delta\sigma_v}{q} = \frac{B_f L_f}{(B_f + Z)(L_f + Z)} \quad \text{(Eqn. 5-17)}$$

Using this equation, $\Delta\sigma_v / q$ ratios are tabulated in Table B3-3.

TABLE B3-3: STRESS INCREASE DUE TO BRIDGE LOADING

Soil Layer	Depth to Midpoint (m)	Stress Increase, $\Delta\sigma_v / q$		
		$B_f = 3$ m	$B_f = 4$ m	$B_f = 5$ m
1	1.0	0.72	0.77	0.80
2	3.28	0.42	0.49	0.53
3	6.07	0.27	0.32	0.36
4	9.07	0.18	0.22	0.26
5	12.07	0.13	0.17	0.20
6	15.07	0.10	0.13	0.16

Use the Hough method to estimate the bridge loading to generate 38 mm of settlement.

General equation is:

$$\Delta H = H_o \frac{1}{C'} \log\left(\frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vo}}\right) \quad \text{(Eqn. 5-24)}$$

Bearing capacity index (C′) of the layers are obtained from Figure 5-19 as follows. This assumes N, and therefore N′ is unchanged after pre-loading, which is conservative.

<u>Layers 1 and 2</u>: Structural fill consisting of well-graded silty sand & gravel

Use the recommended assumed corrected SPT N-value for compacted structural fill:

$$N' = 32$$

From Figure 5-19, $C'_{1,2} = 110$.

Layer 3: Alluvium consisting of inorganic silt

$N_{ave} = 5.2$ (Figure B3-3)

The effective vertical pressure at the time of drilling was

$$\sigma'_{vo_{drilling3}} = \gamma_3 (H_3 / 2) = (17.3\,kN/m^3)(3\,m/2) = 25.9\,kPa$$

Thus $N'/N = 1.92$ (Figure 5-18).

$N' = 10$

$C'_3 = 30$ (Figure 5-19)

Layer 4: Alluvium consisting of inorganic silt

$N_{ave} = 9.0$ (Figure B3-3)

$$\begin{aligned}\sigma'_{vo_{drilling4}} &= \sigma'_{vo_{drilling3}} + \gamma_3 (H_3/2) + \gamma_4 (H_4/2) \\ &= 25.95\,kPa + (17.3\,kN/m^3)(3\,m/2) + (17.3\,kN/m^3)(3\,m/2) \\ &= 25.95\,kPa + 25.95\,kPa + 25.95\,kPa \\ &= 77.8\,kPa\end{aligned}$$

$N'/N = 1.11$ (Figure 5-18)

$N' = 10$

$C'_4 = 30$ (Figure 5-19)

Layer 5: Alluvium consisting of well-graded fine to medium sand

$N_{ave} = 17.0$ (Figure B3-3)

$$\begin{aligned}\sigma'_{vo_{drilling5}} &= \sigma'_{vo_{drilling4}} + \gamma_4 (H_4/2) + \gamma'_5 (H_5/2) \\ &= 77.85\,kPa + (17.3\,kN/m^3)(3\,m/2) + (19.6 - 9.81\,kN/m^3 (3\,m/2) \\ &= 77.85\,kPa + 25.95\,kPa + 14.69\,kPa \\ &= 118.5\,kPa\end{aligned}$$

$N'/N = 0.9$ (Figure 5-18)

$N' = 15$

$C'_5 = 50$ (Figure 5-19)

Layer 6: Alluvium consisting of well-graded fine to medium sand

$N_{ave} = 25.0$ (Figure B3-3)

$$\begin{aligned}
\sigma'_{vo_{drilling6}} &= \sigma'_{vo_{drilling5}} + \gamma'_5(H_5/2) + \gamma'_6(H_6/2) \\
&= 118.49\,\text{kPa} + (19.6 - 9.81\,\text{kN/m}^3(3\,\text{m}/2) + (19.6 - 9.81\,\text{kN/m}^3(3\,\text{m}/2) \\
&= 118.49\,\text{kPa} + 14.69\,\text{kPa} + 14.69\,\text{kPa} \\
&= 147.9\,\text{kPa}
\end{aligned}$$

$N'/N = 0.805$ (Figure 5-18)

$N' = 20$

$C'_6 = 58$ (Figure 5-19)

The total settlement can be calculated by summing the settlements in each layer. For a range of bridge loadings from, say, q = 100 kPa to 300 kPa and footing widths of B_f = 3 m to 5 m, the settlements are calculated as follows:

Layer 1:

$$\begin{aligned}
\Delta H_1 &= H_1 \frac{1}{C'_1} \log\left(\frac{\sigma'_{vo_1} + \Delta\sigma_{v_1}}{\sigma'_{vo_1}}\right) \\
&= 2\,\text{m}\left(\frac{1}{110}\right)\log\left(\frac{89.1\,\text{kPa} + (0.72)(100\,\text{kPa})}{89.1\,\text{kPa}}\right) \\
&= 0.0047\,\text{m} \cong 5\,\text{mm}
\end{aligned}$$

Layer 2:

$$\begin{aligned}
\Delta H_2 &= H_2 \frac{1}{C'_2} \log\left(\frac{\sigma'_{vo_2} + \Delta\sigma_{v_2}}{\sigma'_{vo_2}}\right) \\
&= 2.57\,\text{m}\left(\frac{1}{110}\right)\log\left(\frac{135.9\,\text{kPa} + (0.42)(100\,\text{kPa})}{135.9\,\text{kPa}}\right) \\
&= 0.0027\,\text{m} \cong 3\,\text{mm}
\end{aligned}$$

Layer 3:

$$\Delta H_3 = H_3 \frac{1}{C'_3} \log\left(\frac{\sigma'_{vo_3} + \Delta\sigma_{v_3}}{\sigma'_{vo_3}}\right)$$

$$= 3.0\,\mathrm{m}\left(\frac{1}{30}\right)\log\left(\frac{188.2\,\mathrm{kPa} + (0.27)(100\,\mathrm{kPa})}{188.2\,\mathrm{kPa}}\right)$$

$$= 0.0058\,\mathrm{m} \cong 6\,\mathrm{mm}$$

Layer 4:

$$\Delta H_4 = H_4 \frac{1}{C'_4} \log\left(\frac{\sigma'_{vo_4} + \Delta\sigma_{v_4}}{\sigma'_{vo_4}}\right)$$

$$= 3.0\,\mathrm{m}\left(\frac{1}{30}\right)\log\left(\frac{240.1\,\mathrm{kPa} + (0.18)(100\,\mathrm{kPa})}{240.1\,\mathrm{kPa}}\right)$$

$$= 0.0031\,\mathrm{m} \cong 3\,\mathrm{mm}$$

Layer 5:

$$\Delta H_5 = H_5 \frac{1}{C'_5} \log\left(\frac{\sigma'_{vo_5} + \Delta\sigma_{v_5}}{\sigma'_{vo_5}}\right)$$

$$= 3.0\,\mathrm{m}\left(\frac{1}{50}\right)\log\left(\frac{280.7\,\mathrm{kPa} + (0.13)(100\,\mathrm{kPa})}{280.7\,\mathrm{kPa}}\right)$$

$$= 0.0012\,\mathrm{m} \cong 1\,\mathrm{mm}$$

Layer 6:

$$\Delta H_6 = H_6 \frac{1}{C'_6} \log\left(\frac{\sigma'_{vo_6} + \Delta\sigma_{v_6}}{\sigma'_{vo_6}}\right)$$

$$= 3.0\,\mathrm{m}\left(\frac{1}{58}\right)\log\left(\frac{310.1\,\mathrm{kPa} + (0.10)(100\,\mathrm{kPa})}{310.1\,\mathrm{kPa}}\right)$$

$$= 0.0007\,\mathrm{m} \cong 1\,\mathrm{mm}$$

The total settlement is:

$$\sum \Delta H_i = 5 + 3 + 6 + 3 + 1 + 1 = 19\,\mathrm{mm}$$

In a similar manner, the settlements for different footing widths and bridge loadings are calculated and summarized in Table B3-4.

TABLE B3-4: SETTLEMENTS DUE TO BRIDGE LOADING

Bridge Loading q (kPa)	Settlement (mm)		
	$B_f = 3$ m	$B_f = 4$ m	$B_f = 5$ m
100	19	21	24
200	33	38	42
300	46	53	59

Table B3-4 shows that the maximum allowable q to limit the settlement to 38 mm decreases from about 238 kPa for $B_f = 3$ m to about 178 kPa for $B_f = 5$ m. These values are smaller than the q_{all} values based on bearing capacity failure that were calculated earlier. Therefore, the settlement will limit the allowable bearing capacity of the footings.

The allowable bearing pressure is summarized in Figure B3-5.

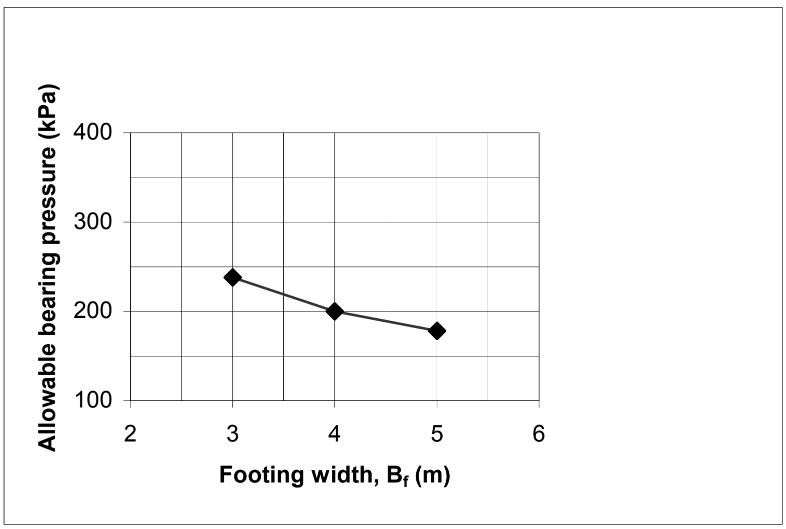


Figure B3-5: Allowable Bearing Pressure as a Function of Footing Width

Step 8 – Calculate sliding and passive soil resistance:

The footing concrete will be poured on compacted structural fill. Thus the friction angle, δ, to be used in the sliding analysis of the footing will be $\delta = \phi' = 38°$.

The passive resistance of the soil in front of the footing will be ignored. The ultimate sliding resistance is therefore:

$$F_R = (W + P_v)\tan\delta \qquad \text{(Eqn. 5-38)}$$

$$= (W + P_v)\tan(38^o)$$

$$= 0.78(W + P_v)$$

Step 9 – Check global stability of the footing:

The slope stability program XSTABL™ was used to evaluate the global stability of the approach embankment with the abutment and the bridge loading applied. The program uses Bishop's method of slices to compute a global stability factor of safety for the slope. A footing width of $B_f = 3.5$ m was assumed. The footing loads and the abutment backfill were modeled as applied vertical surcharge loads at the footing bearing elevation. The applied footing stress corresponded to the allowable bearing pressure for this footing width (e.g., 240 kPa from Figure B3-4). At this stress, the critical failure surface was just behind the heel of the footing. The minimum factor of safety was about 1.6 and satisfactory. The potential failure arc is shown in Figure B3-6.

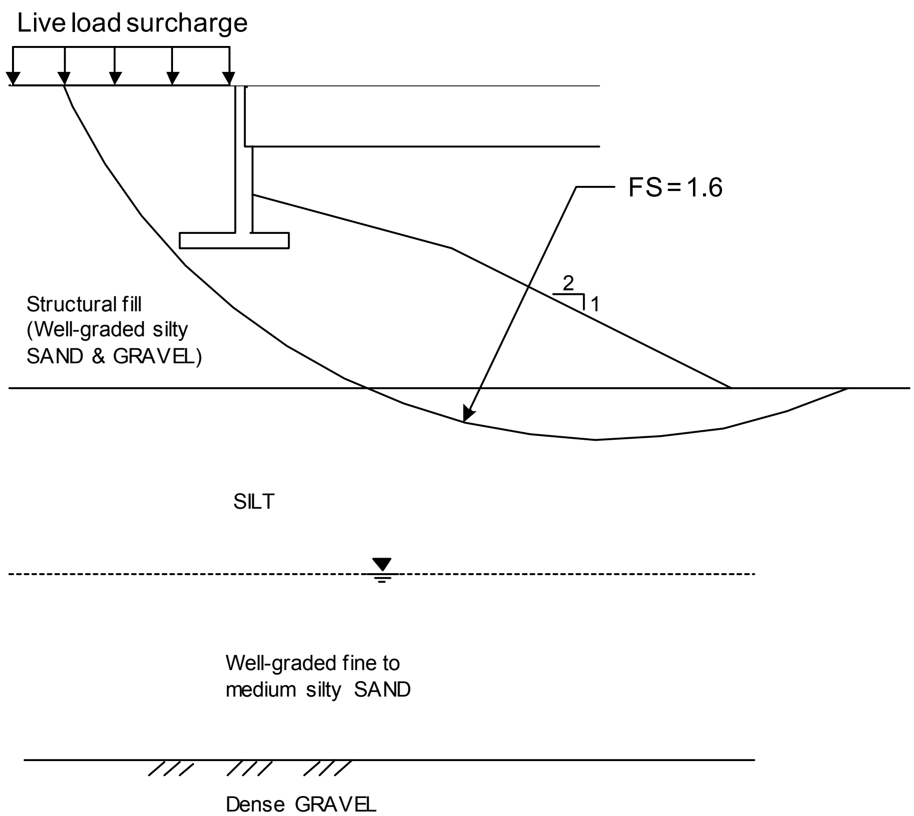


Figure B3-6: Global Stability

Step 10 – Size the footing under full loading:

From Table B3-1, the total vertical load from the bridge girders is

$$\begin{aligned} P \quad &= DC + DW + LL \\ &= 5233\,\text{kN} + 446\,\text{kN} + 1538\,\text{kN} \\ &= 7217\,\text{kN} \end{aligned}$$

Using $L_f = 25$ m,

$$P_{v\ girders} = \frac{P}{L_f}$$
$$= \frac{7217\,kN}{25\,m}$$
$$= 288.7\,kN/m$$

The additional load from the abutment components per 1-m length of the abutment wall is calculated as follows for $B_f = 3$ m.

$$W_{stem} = 57.29\,kN/m$$
$$W_f = 10.81 B_f = (10.81)(3) = 32.43\,kN/m$$
$$W_h = (93.69)(B_f - 1.73)$$
$$= (93.69)(3 - 1.73)$$
$$= 118.99\,kN/m$$
$$P_{v\ abut} = 57.29 + 32.43 + 118.99 = 208.71\,kN/m$$

Thus

$$P_v = P_{v\ girders} + P_{v\ abut}$$
$$= 288.7\,kN/m + 208.7\,kN/m$$
$$\cong 497\,kN/m$$

From Figure B3-4, a bearing pressure of up to 230 kPa may be applied by the footing. Using q = 230 kPa, the required footing width B_f is

$$B_f = P_v / q$$
$$= (497\,kN/m)/(230\,kPa)$$
$$= 2.16\,m$$

This appears to be too small for $H_{abut} = 5.24$ m. A preliminary dimension for a footing width can be estimated as one-half the abutment height, or $H_{abut} = (0.5)(5.24\text{ m}) = 2.62$ m. Therefore, try $B_f = 2.7$ m.

Step 11a – Check overturning (eccentricity), bearing pressure and sliding under full loading without P_{DW} and P_{LL}:

Check overturning for $B_f = 2.7$ m:

Analyze for 1 m of abutment length.

From Table B3-1, the loads and moments due to girders are

$$
\begin{aligned}
P_{v\ girders} &= \frac{5233\,\text{kN}}{L_f} \\
&= \frac{5233\,\text{kN}}{25\,\text{m}} \\
&= 209.3\,\text{kN/m}
\end{aligned}
$$

$$
\begin{aligned}
M_{toe\ girders} &= \frac{(7274-3392)\,\text{kN}\cdot\text{m}}{L_f} \\
&= \frac{3882\,\text{kN}\cdot\text{m}}{25\,\text{m}} \\
&= 155.3\,\text{kN}\cdot\text{m/m}
\end{aligned}
$$

From Table B3-2, the loads and moments due to abutment components are

$$
\begin{aligned}
P_{v\ abut} &= 57.29 + 10.81B_f + (93.69)(B_f - 1.73)\ \text{kN/m} \\
&= 57.29 + (10.81)(2.7) + (93.69)(2.7 - 1.73)\ \text{kN/m} \\
&= 177.4\,\text{kN/m}
\end{aligned}
$$

$$
\begin{aligned}
M_{toe\ abut} &= 84.79 + 5.41{B_f}^2 + (93.69)(B_f - 1.73)(0.865 + 0.5B_f) - 122.43 - 41.97 \\
&= 84.79 + (5.41)(2.7)^2 + (93.69)(2.7\,\text{m} - 1.73)(0.865 + (0.5)(2.7\,\text{m})) \\
&\qquad -122.43 - 41.97 \\
&= 161.1\text{kN}\cdot\text{m/m}
\end{aligned}
$$

The total loads are

$$
\begin{aligned}
P_v &= P_{v\ girders} + P_{v\ abut} \\
&= 209.3\,\text{kN/m} + 177.4\,\text{kN/m} \\
&= 386.7\,\text{kN/m}
\end{aligned}
$$

$$
\begin{aligned}
M_{toe} &= M_{toe\ girders} + M_{toe\ abut} \\
&= 155.3\,\text{kN}\cdot\text{m/m} + 161.1\,\text{kN}\cdot\text{m/m} \\
&= 316.4\,\text{kN}\cdot\text{m/m}
\end{aligned}
$$

The arm of the resultant is

$$
\begin{aligned}
arm_R &= M_{toe} / P_v \\
&= (316.4\,\text{kN}\cdot\text{m/m})/(386.7\,\text{kN/m}) \\
&= 0.82\,\text{m}
\end{aligned}
$$

The eccentricity, e_y, is

$$\begin{aligned} e_y &= B_f / 2 - arm_R \\ &= 2.7\,m / 2 - 0.82\,m \\ &= 1.35\,m - 0.82\,m \\ &= 0.53\,m \end{aligned}$$

For stability, the eccentricity should be less than one-sixth the footing width:

$$e_y = 0.53\,m < B_f / 6 = 2.7\,m / 6 = 0.45\,m \qquad \text{(No Good)}$$

Try increasing B_f to 3.2 m.

<u>Check overturning for B_f = 3.2 m:</u>

Analyze for 1 m of abutment length.

From the previous calculations, the loads and moments due to girders are

$$P_{v\ girders} = 209.3\,kN/m$$

$$M_{toe\ girders} = 155.3\,kN \cdot m/m$$

From Table B3-2, the loads and moments due to abutment components are

$$\begin{aligned} P_{v\ abut} &= 57.29 + 10.81 B_f + (93.69)(B_f - 1.73)\ kN/m \\ &= 57.29 + (10.81)(3.2) + (93.69)(3.2 - 1.73)\ kN/m \\ &= 229.6\,kN/m \end{aligned}$$

$$\begin{aligned} M_{toe\ abut} &= 84.79 + 5.41 B_f^2 + (93.69)(B_f - 1.73)(0.865 + 0.5 B_f) - 122.43 - 41.97 \\ &= 84.79 + (5.41)(3.2)^2 + (93.69)(3.2\,m - 1.73)(0.865 + (0.5)(3.2\,m)) \\ &\qquad -122.43 - 41.97 \\ &= 315.3 kN \cdot m/m \end{aligned}$$

The total loads are

$$\begin{aligned} P_v &= P_{v\ griders} + P_{v\ abut} \\ &= 209.3\,kN/m + 229.6\,kN/m \\ &= 438.9\,kN/m \end{aligned}$$

$$
\begin{aligned}
M_{toe} &= M_{toe\ girders} + M_{toe\ abut} \\
&= 155.3\,kN \cdot m/m + 315.3\,kN \cdot m/m \\
&= 470.6\,kN \cdot m/m
\end{aligned}
$$

The arm of the resultant is

$$
\begin{aligned}
arm_R &= M_{toe} / P_v \\
&= (470.6\,kN \cdot m/m)/(438.9\,kN/m) \\
&= 1.07\,m
\end{aligned}
$$

The eccentricity, e_y, is

$$
\begin{aligned}
e_y &= B_f / 2 - arm_R \\
&= 3.2\,m/2 - 1.07\,m \\
&= 1.6\,m - 1.07\,m \\
&= 0.527\,m
\end{aligned}
$$

For stability, the eccentricity should be less than one-sixth the footing width:

$$
e_y = 0.527\,m < B_f / 6 = 3.2\,m/6 = 0.533\,m \quad \text{(OK)}
$$

<u>Check bearing stress for effective footing area</u>:

$$
\begin{aligned}
B'_f &= B_f - 2e_y \qquad \text{(Eqn. 6-6)} \\
&= 3.2\,m - (2)(0.53\,m) \\
&= 2.14\,m
\end{aligned}
$$

$$
\begin{aligned}
q &= P_v / B'_f \\
&= (438.9\,kN/m)/(2.14\,m) \\
&= 205.1\,kPa
\end{aligned}
$$

By extrapolating the values in Figure B3-4, the allowable bearing pressure for $B'_f = 2.14$ m to limit settlement to less than 38 mm is about

$$
q_{all} = 275\,kPa
$$

Under Group IV loads, the 25 percent overstress allowance can be applied to this allowable bearing pressure. By inspection, however, this computation is not necessary since $q < q_{all}$, and the bearing stress is smaller than the allowable value. (OK)

<u>Check sliding</u>:

The forces, $F_{sliding}$, that tend to induce sliding are the sum of the following:

Shear loads from bearing pads, V (Table B3-1)

Active earth pressure load, P_A (Table B3-2)

Horizontal earth load from live load surcharge, P_{LS} (Table B3-2)

Analyzing per m of the abutment wall, $F_{sliding}$ is

$$
\begin{aligned}
F_{sliding} &= V/L_f + P_A + P_{LS} \\
&= 1047\,kN/25\,m + 69.96\,kN/m + 16.02\,kN/m \\
&= 127.9\,kN/m
\end{aligned}
$$

The force, F_R, available to resist sliding (from Step 8) is

$$F_R = 0.78(W + P_v)$$

The vertical force, P_v computed above includes the self-weight of the footing, W. The live load and weight of the wearing surface should not be included in this calculation. Thus the factor of safety FS against sliding is

$$FS_{sliding} = \frac{F_R}{F_{sliding}} \geq 1.5 \qquad \text{(Eqn. 5-40)}$$

$$
\begin{aligned}
&= \frac{0.78(438.9\,kN/m)}{127.9\,kN/m} \\
&= \frac{342.3\,kN/m}{127.9\,kN/m} \\
&= 2.7
\end{aligned}
$$

Step 11b – Check overturning, bearing pressure and sliding under full loading:

<u>Check overturning for B_f = 3.2 m</u>:

Analyze for 1 m of abutment length.

From Table B3-1, the loads and moments due to girders are

$$
\begin{aligned}
P_{v\ girders} &= \frac{(5233 + 446 + 1538)\,kN}{L_f} \\
&= \frac{7217\,kN}{25\,m} \\
&= 288.7\,kN/m
\end{aligned}
$$

$$M_{toe\ girders} = \frac{(7274 + 620 + 2138 - 3392)\,kN \cdot m}{L_f}$$
$$= \frac{6640\,kN \cdot m}{25\,m}$$
$$= 265.6\,kN \cdot m / m$$

From Step 11a, the loads and moments due to abutment components are

$$P_{v\ abut} = 229.6\,kN / m$$

$$M_{toe\ abut} = 315.3 kN \cdot m / m$$

The total loads are

$$P_v = P_{v\ girders} + P_{v\ abut}$$
$$= 288.7\,kN / m + 229.6\,kN / m$$
$$= 518.3\,kN / m$$

$$M_{toe} = M_{toe\ girders} + M_{toe\ abut}$$
$$= 265.6\,kN \cdot m / m + 315.3\,kN \cdot m / m$$
$$= 580.9\,kN \cdot m / m$$

The arm of the resultant is

$$arm_R = M_{toe} / P_v$$
$$= (580.9\,kN \cdot m / m) / (518.3\,kN / m)$$
$$= 1.12\,m$$

The eccentricity, e_y, is

$$e_y = B_f / 2 - arm_R$$
$$= 3.2\,m / 2 - 1.12\,m$$
$$= 1.6\,m - 1.12\,m$$
$$= 0.48\,m$$

For stability, the eccentricity should be less than one-sixth the footing width:

$$e_y = 0.48\,m < B_f / 6 = 3.2\,m / 6 = 0.53\,m \qquad \text{(OK)}$$

Check bearing stress for effective footing area:

$$B'_f = B_f - 2e_y \qquad \text{(Eqn. 6-6)}$$
$$= 3.2\,m - (2)(0.48\,m)$$
$$= 2.24\,m$$

$$q = P_v / B'_f$$
$$= (518.3\,kN/m)/(2.24\,m)$$
$$= 231.4\,kPa$$

By extrapolating the values in Figure B3-4, the allowable bearing pressure for $B'_f = 2.24$ m to limit settlement to less than 38 mm is about

$$q_{all} = 270\,kPa$$

Under Group IV loads, the 25 percent overstress allowance can be applied to this allowable bearing pressure. By inspection, however, this computation is not necessary since $q < q_{all}$, and the bearing stress is smaller than the allowable value. (OK)

Check sliding:

The forces, $F_{sliding}$, that tend to induce sliding are the sum of the following:

Shear loads from bearing pads, V (Table B3-1)

Active earth pressure load, P_A (Table B3-2)

Horizontal earth load from live load surcharge, P_{LS} (Table B3-2)

Analyzing per m of the abutment wall, $F_{sliding}$ is

$$F_{sliding} = V/L_f + P_A + P_{LS}$$
$$= 1047\,kN/25\,m + 69.96\,kN/m + 16.02\,kN/m$$
$$= 127.9\,kN/m$$

The force, F_R, available to resist sliding (from Step 8) is

$$F_R = 0.78(W + P_v)$$

The vertical force, P_v, computed above includes the self-weight of the footing, W. The live load should not be included in this calculation. Thus the factor of safety FS against sliding is

$$FS_{sliding} = \frac{F_R}{F_{sliding}} \geq 1.5 \qquad \text{(Eqn. 5-40)}$$

$$= \frac{0.78(518.3\,\text{kN/m} - 1538\,\text{kN}/25\,\text{m})}{127.9\,\text{kN/m}}$$

$$= \frac{356.3\,\text{kN/m}}{127.9\,\text{kN/m}}$$

$$= 2.8$$

Again, the 25 percent overstress allowance could be applied, but since FS > 1.5, this is not necessary. (OK)

Step 11c – Check overturning, bearing pressure, and sliding without the bridge loading from the girders:

Consider the loads of abutment components only, without the horizontal earth load from live load surcharge, P_{LS}. *Note that this condition would not occur in the consideration of Group IV loads because the superstructure must be in place for the temputeraturе and shrinkage loads to be applied to the footing. This check is normally done when considering Group I loads but is included here for completeness.*

Check overturning (eccentricity) for B_f = 3.2 m:

Referring to Step 11b above and Table B3-2, the following are obtained without the contribution from P_{LS}.

$$P_{v\ abut} = 229.6\,\text{kN/m}$$

$$M_{toe\ abut} = 315.3\,\text{kN}\cdot\text{m/m} + 41.97\,\text{kN}\cdot\text{m/m} = 357.3\,\text{kN}\cdot\text{m/m}$$

The arm of the resultant is

$$\begin{aligned} arm_R &= M_{toe\ abut} / P_{v\ abut} \\ &= (357.3\,\text{kN}\cdot\text{m/m})/(229.6\,\text{kN/m}) \\ &= 1.56\,\text{m} \end{aligned}$$

The eccentricity, e_y, is

$$\begin{aligned} e_y &= B_f/2 - arm_R \\ &= 3.2\,\text{m}/2 - 1.56\,\text{m} \\ &= 1.6\,\text{m} - 1.56\,\text{m} \\ &= 0.04\,\text{m} \end{aligned}$$

For stability, the eccentricity should be less than one-sixth the footing width:

$$e_y = 0.04\,\text{m} < B_f/6 = 3.2\,\text{m}/6 = 0.53\,\text{m} \qquad \text{(OK)}$$

Check bearing stress for effective footing area:

$$B'_f = B_f - 2e_y$$
$$= 3.2\,m - (2)(0.04\,m)$$
$$= 3.12\,m$$

$$q = P_{v\ abut} / B'_f$$
$$= (229.6\,kN/m)/(3.12\,m)$$
$$= 73.6\,kPa$$

From Figure B3-4, the allowable bearing pressure for B'_f = 3.12 m to limit settlement to less than 35 mm is about

$$q_{all} = 235\,kPa$$

The applied bearing stress is smaller than the allowable value. (OK)

Check sliding:

The force, $F_{sliding}$, that tends to induce sliding consists only of the active earth pressure load, P_A (Table B3-2):

$$F_{sliding} = P_A = 69.96\,kN/m$$

The force F_R available to resist sliding (from Step 8) is

$$F_R = 0.78(W + P_v)$$

The factor of safety FS against sliding is:

$$FS_{sliding} = \frac{F_R}{F_{sliding}} \geq 1.5 \qquad \text{(Eqn. 5-40)}$$

$$= \frac{0.78(229.6\,kN/m)}{69.96\,kN/m}$$
$$= \frac{179.4\,kN/m}{69.96\,kN/m}$$
$$= 2.6$$

This should be > 1.5. (OK)

Step 12 – Complete structural design of the footing:

The structural engineer completes this step.

(This page intentionally blank)

EXAMPLE 4: FULL-HEIGHT BRIDGE ABUTMENT WITH LATERAL EARTH PRESSURE

Given:

- Subsurface profile at the abutment is shown in Figure B4-1.
- Geometry of the abutment is shown in Figure B4-2.
- Abutment stem is located $B_f/3$ from the footing toe.
- T_{stem} = 1.0 m, T_{GS} = 0.5 m, and T_f = 0.9 m.
- The height of the abutment, H_{abut}, is 8.5 m, and the length is 25 m.
- There is no scour potential at this site.
- Frost penetration depth is 0.6 m (2 ft).
- Tolerable settlement is 25 mm (1.0 in).

Find:

- Size the footing based on Service Load Design, Group I loads (allowable stress design method), considering both the settlement and bearing capacity. Check for overturning and sliding.

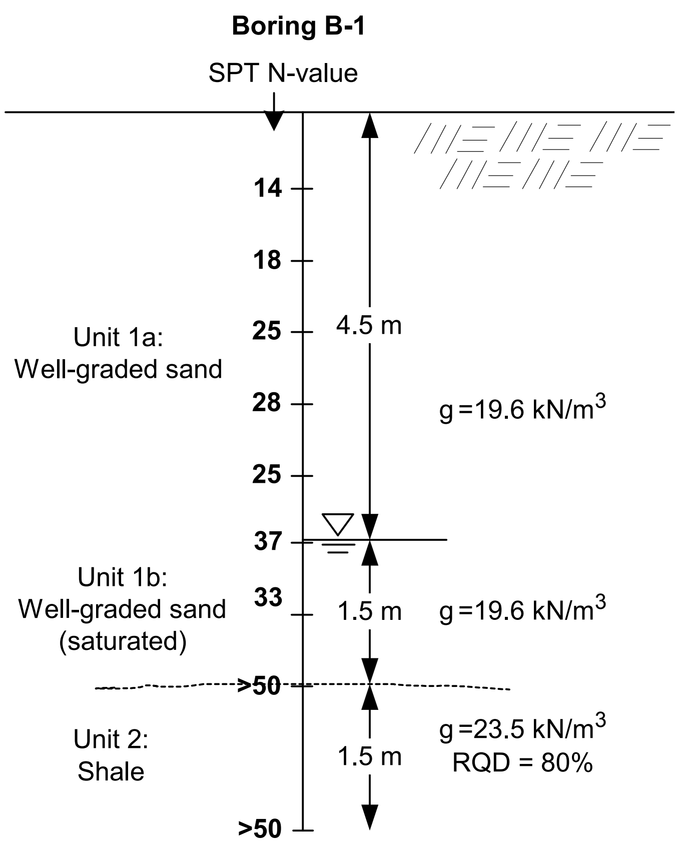


Figure B4-1: Subsurface Profile

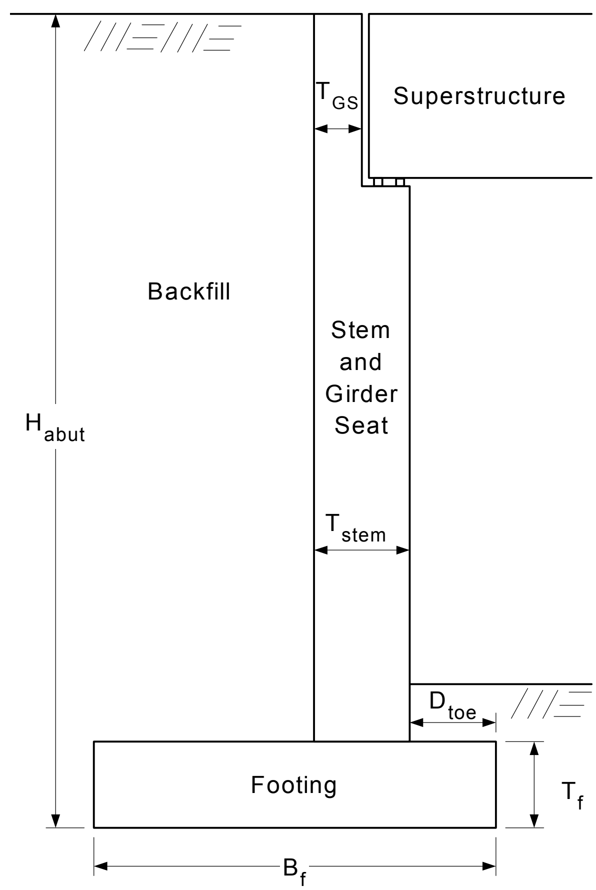


Figure B4-2: Abutment Geometry

Solution:

The solution can be obtained by following the step-by-step process shown in Figure 6-1, Design Process Flow Chart.

Steps 1 through 4 – Preliminary bridge layout, review of existing geologic and subsurface data, site reconnaissance, scour and frost potential:

The structural, hydraulics and geotechnical engineers completed these steps. The required design information is provided in the problem-given statement.

Step 5 – Determine the loads applied to the footing:

The structural engineer has calculated the following values for loading along the abutment wall:

Loads at base of the superstructure (applied through the bearings at the girder seat) are summarized on Table B4-1.

TABLE B4-1: LOADS AT SUPERSTRUCTURE BASE

Load	Axial, P (kN/m)	Shear, V (kN/m)
Dead load (D)	227	41.9
Live load (L)	61.6	0

The shear load shown in Table B4-1 is the shear transferred through the bearings due to temperature, shrinkage and rib shortening. Under Group I loads, the load factor for this shear component is zero and will therefore not be included in the design loads for this example.

Final Service Load Design Loads for Group I:

The general equation is (refer to Example B-1 for discussion of variables):

$$\begin{aligned} P_I = \gamma\,[&\beta_D(D) + \beta_L\,(L+I) + \beta_C(CF) + \beta_E(E) + \beta_B\,(B) \\ &+ \beta_S(SF) + \beta_W(W) + \beta_{WL}(WL) + \beta_L(LF) + \beta_R\,(R+S+T) \\ &+ \beta_{EQ}(EQ) + \beta_{ICE}(ICE)] \end{aligned}$$

(AASHTO, 1996, Eqn. 3-10)

This reduces to the following for the loads under consideration for Load Group I in this example:

$$P_I = \gamma\,[\beta_D(D) + \beta_L\,(L)]$$

So the factored loads are as follows:

Axial Loads:

$$\begin{aligned} P_I &= \gamma\,[\beta_D(D) + \beta_L\,(L)] \\ &= 1.0\,[1.0\,(227^{kN/m}) + 1.0\,(61.6^{kN/m})] \\ &= 289^{kN/m} \end{aligned}$$

Shear:

$$\begin{aligned} V &= \gamma\,[\beta_D(D)] \\ &= 1.0\,[1.0\,(41.9^{kN/m})] \\ &= 41.9^{kN/m} \end{aligned}$$

Step 6 – Conduct field exploration and laboratory testing:

This step is complete, and the subsurface data are shown in drawing at pier location, Figure B4-1. Laboratory testing was limited to soil classifications and moisture content due to the granular nature of the soils encountered in the boring. The foundation design will be based largely on design parameters correlated to the soil classifications and the SPT N-values. Compute the initial vertical effective stresses at the midpoint of each layer:

Layer 1a:

$$\sigma'_{v1a} = 2.25\,m(19.6\,\frac{kN}{m^3}) = 44.1\,kPa$$

Layer 1b:

$$\sigma'_{v1b} = 5.25\,m(19.6\,\frac{kN}{m^3}) - 0.75\,m(9.8\,\frac{kN}{m^3}) = 95.6\,kPa$$

The effective stress diagram for the profile at the abutment is plotted in Figure B4-3.

Step 7 – Calculate allowable bearing capacity:

The geotechnical engineer will calculate the limiting allowable bearing capacity, considering both the potential for bearing capacity failure (Section 5.2) and settlement (Section 5.3).

Calculate Ultimate Bearing Capacity:

The general equation for bearing capacity is:

$$q_{ult} = cN_c s_c b_c + qN_q C_{W_q} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{W_\gamma} s_\gamma b_\gamma \qquad \text{(Eqn. 5-14)}$$

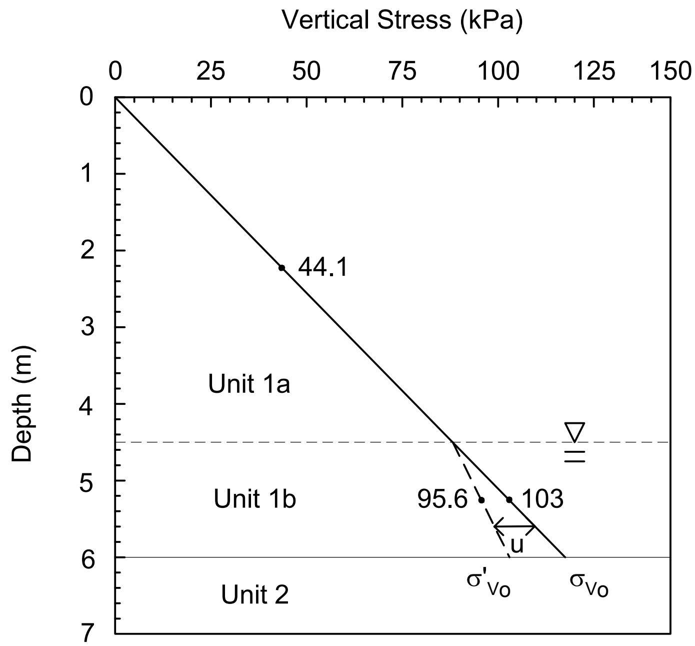


Figure B4-3: Effective Stress Diagram

The first term goes to zero since the foundation materials are granular and cohesionless, so the cohesion term, c, is zero.

The footing length (L_f) is 25 m. For footing widths (B_f) less than 5 m, the ratio of footing length, L_f, to width is greater than 5. This footing classifies as a strip footing, and the shape factors should be set equal to 1.0:

$$s_\gamma = s_q = 1.0 \qquad \text{(Table 5-2)}$$

Since this footing will be level, there is no effect of inclination of the base, so:

$$b_\gamma = b_q = 1.0 \qquad \text{(Table 5-5)}$$

Calculate the effect of the footing embedment, q. Assuming a 0.9-m-thick footing and 0.6-m cover, D_f = 1.5 m:

In this case, q is only a function of the depth of bearing.

$$q = \sigma'_{vo} = \gamma' D_f \qquad \text{where: } \gamma' = \gamma_{bulk} = 19.6\frac{kN}{m^3}$$

$$q = (19.6\frac{kN}{m^3})(1.5\,m)$$

$$q = 29.4\,kPa$$

Since the ratio of the footing depth, D_f, to the footing width, B_f, will be less than one, the correction factor to account for shearing resistance of the material above the bearing elevation, d_q, should be set equal to 1.0:

$$d_q = 1.0 \qquad \text{(Table 5-4)}$$

Check ground water effect:

Depth of ground water table below the bearing depth:

D_f = Depth of footing = 1.5 m — Assumes a 0.9-m-thick footing and 0.6-m cover

D_w = Depth of ground water below the ground surface = 4.5 m

Conservatively estimate the footing width for ground water correction factors with an upper bound value of B_f = 6 m.

Calculate ground water correction factors, $C_{W\gamma}$ and C_{Wq}:

$$C_{W_\gamma} = 0.5 + 0.5\left(\frac{D_W}{1.5B_f + D_f}\right) \leq 1.0 \qquad \text{(Eqn. 5-9)}$$

$$C_{W_\gamma} = 0.5 + 0.5\left(\frac{4.5}{1.5(6) + 1.5}\right) = 0.7$$

$$C_{W_q} = 0.5 + 0.5\left(\frac{D_W}{D_f}\right) \leq 1.0 \qquad \text{(Eqn. 5-10)}$$

$$C_{W_q} = 0.5 + 0.5\left(\frac{4.5}{1.5}\right) = 2.0 \text{ ; Use } C_{Wq} = 1.0$$

A design friction angle associated with Unit 1a is needed to calculate the bearing capacity factors. From Figure B4-1, the average uncorrected SPT N-value for this unit is 22. From Figure B4-3, the average vertical effective stress in Unit 1 is 44.1 kPa (about 0.9 ksf). Entering Figure 4-1 with a blowcount of 22 and a vertical effective stress of 0.9 ksf, a relative density, D_R, of about 85 percent is read. Entering Figure 4-2 with this relative density for well-graded sand (Unified Soil Classification 'SW'), a friction angle of 38 degrees is selected for computing bearing capacity.

From Table 5-1, the bearing capacity factors, N_γ and N_q, are 78.0 and 48.9, respectively, for a friction angle of 38 degrees. So the equation for bearing capacity is:

$$q_{ult} = 29.4\,\text{kPa}\,(48.9)(1.0)(1.0)(1.0)(1.0)$$
$$+0.5\,(19.6\,\tfrac{\text{kN}}{\text{m}^3})(B_f)(78.0)(0.7)(1.0)(1.0)$$
$$q_{ult} = 1438\,\text{kPa} + 535 B_f$$

Incorporating a factor of safety of 3.0:

$$q_{all} = q_{ult} \div 3.0$$
$$q_{all} = (1438 + 535 B_f\ \text{kPa}) \div 3.0$$
$$q_{all} = 479 + 178 B_f\ \text{kPa}$$

<u>Estimate Footing Settlement</u>:

First, estimate the settlement in the sand layers, 1a and 1b, assuming that settlement in the shale, Layer 2, will be negligible. Calculate stress increase at the midpoint of each soil layer using the 2-on-1 stress distribution method for various footing widths – say, B_f = 3.0, 4.3, 5.5 and 6.7 m:

$$\frac{\Delta\sigma_V}{q} = B_f\,/(B_f + Z) \qquad \text{(for strip footing)} \qquad \text{(Figure 5-12)}$$

Layer 1a with midpoint of layer @ Z = 1.5 m below base of footing:

$$\frac{\Delta\sigma_{V1a}}{q} = 3.0\,/(3.0 + 1.5) = 0.67$$

Layer 1b with midpoint of layer @ Z = 3.75 m below base of footing:

$$\frac{\Delta\sigma_{V1b}}{q} = 3.0\,/(3.0 + 3.75) = 0.44$$

Perform similar calculations for B_f = 3.0, 4.3, 5.5 and 6.7 m, then tabulate as shown in Table B4-2.

TABLE B4-2: STRESS INCREASE WITH DEPTH
AS FUNCTION OF FOOTING WIDTH

Soil Layer	Depth Range (m)	Stress Increase, $\Delta\sigma_v$			
		B_f = 3.0 m	B_f = 4.3 m	B_f = 5.5 m	B_f = 6.7 m
1a	1.5 – 4.5	0.67q	0.74q	0.79q	0.82q
1b	4.5 – 6.0	0.44q	0.53q	0.59q	0.64q

Determine stress applied by footing to generate 25 mm of settlement for the same range of footing widths, B_f = 3.0, 4.3, 5.5 and 6.7 m, using the Hough method:

General equation:

$$\Delta H = H_o (1/C') \log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}] \qquad \text{(Eqn. 5-24)}$$

Correct SPT blowcounts to account for overburden pressure, N′, for each layer, and enter Figure 5-19 to obtain bearing capacity factor, C′:

Layer 1a:

N_{avg} = 22
N′/N = 1.47 (from Figure 5-18)
N′ = 32
C′ = 108 (from Figure 5-19 for well-graded sand and gravel)

Layer 1b:

N_{avg} = 35
N′/N = 1.0 (from Figure 5-18)
N′ = 35
C′ = 115 (from Figure 5-19 for well-graded sand and gravel)

Calculate settlement in each layer and sum the settlements to obtain the total settlement under a range of applied stresses of q = 240, 290 and 335 kPa and a range of footing widths from 3.0 to 6.7 m:

Layer 1a:

$$\Delta H_{1a} = H_{1a}(1/C')\log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}]$$
$$\Delta H_{1a} = 3\,\text{m}(1/108)\log[(58.8\,\text{kPa} + 0.67(240\,\text{kPa}))/58.8\,\text{kPa})]$$
$$\Delta H_{1a} = 0.0160\,\text{m} = 16\,\text{mm}$$

Layer 1b:

$$\Delta H_{1b} = H_{1b}(1/C')\log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}]$$
$$\Delta H_{1b} = 1.5\,\text{m}(1/115)\log[(95.6\,\text{kPa} + 0.44(240\,\text{kPa}))/95.6\,\text{kPa})]$$
$$\Delta H_{1b} = 0.004\,\text{m} = 4\,\text{mm}$$

$$\sum \Delta H_i = 16\,\text{mm} + 4\,\text{mm} = 20\,\text{mm}$$

Repeat and tabulate for various footing widths and applied stresses (a spreadsheet or other computer tool can make these calculations rapidly):

TABLE B4-3: SETTLEMENT AS A FUNCTION OF APPLIED STRESS AND FOOTING WIDTH

Applied Stress (kPa)	Settlement (mm)			
	B_f = 3.0 m	B_f = 4.3 m	B_f = 5.5 m	B_f = 6.7 m
240	20	22	23	23
290	22	24	25	26
335	24	26	27	28

Check the estimated settlement in the shale, Layer 2, to verify that this settlement is negligible.

For settlement analysis, the shale can be modeled as elastic half-space. From elastic theory:

$$\delta_v = \frac{C_d \Delta\sigma_V B_f (1-\nu^2)}{\alpha_E E_{IR}} \qquad \text{(Eqn. 5-36)}$$

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \qquad \text{(Eqn. 5-37)}$$

From Table 5-12 for a rectangle with L/B ratio of 5, C_d = 1.83. For a shale with RQD = 80%, ν = 0.14 and E_{IR} = 9.8 x 10^6 kPa, from Tables 5-13 and 5-14. For a conservative footing width of 5.5 m and a bearing pressure of 290 kPa:

$$B = B_f + Z - D_f = 5.5\,m + 6.0\,m - 1.5\,m = 10\,m$$

$$\Delta\sigma_V = q(B_f / B) = 290\,kPa(5.5\,m/10\,m) = 159\,kPa$$

$$\alpha_E = 0.0231(80) - 1.32 = 0.53$$

$$\delta_v = \frac{(1.83)(159\,kPa)(10\,m)(1-0.14^2)}{(0.53)(9800000\,kPa)} = 0.0005\,m = 0.5\,mm$$

This settlement is negligible.

Compare the values in Table B4-3 to allowable bearing capacities at the same footing widths relative to shear failure. Recall that the water table correction factor was conservatively computed near the upper bound of footing widths at B_f = 6 m. This would be revisited only if bearing capacity failure is found to limit the allowable bearing capacity.

For B_f = 3.0 m:

$q_{all} = 479 + 178(3.0\,m)\,kPa = 1013\,kPa$ >> 335 kPa (with 24 mm settlement)

For B_f = 4.3 m:

$$q_{all} = 479 + 178(4.3\,\text{m})\,\text{kPa} = 1244\,\text{kPa}$$ >> 290 kPa (with 24 mm settlement)

For B_f = 5.5 m:

$$q_{all} = 479 + 178(5.5\,\text{m})\,\text{kPa} = 1458\,\text{kPa}$$ >> 290 kPa (with 25 mm settlement)

For B_f = 6.7 m:

$$q_{all} = 479 + 178(6.7\,\text{m})\,\text{kPa} = 1672\,\text{kPa}$$ >> 240 kPa (with 23 mm settlement)

Settlement will therefore limit the allowable bearing capacity for the design of this footing. Interpolating the values in Table B4-3, find ranges of footing widths that will limit settlement to 25 mm or less at allowable bearing capacities of 240, 290 and 335 kPa:

TABLE B4-4: SETTLEMENT-LIMITED
ALLOWABLE BEARING CAPACITIES

Allowable Bearing Capacity (kPa)	Range of Footing Widths That Will Limit Settlement to ≤ 25 mm
335	≤ 3.0 m
290	3.0 to 5.5 m
240	5.5 to 6.7 m

The geotechnical engineer should provide this table to the structural designer for sizing of the footing.

Step 8 – Calculate sliding and passive soil resistance:

The geotechnical engineer has recommended that the footing be cast against the firm, undisturbed soil of Unit 1a. The friction angle of 38 degrees selected in Step 7 will be used to compute sliding resistance.

The general equation for sliding resistance is:

$$F_R = (W + P_v) \tan \delta \qquad \text{(Eqn. 5-38)}$$

The second term goes to zero since we are founding the footing on a cohesionless soil unit. For a concrete footing cast against the ground, $\delta = \phi$, so the ultimate sliding resistance is:

$$F_R = (W + P_v) \tan 38° = 0.78(W + P_v)$$

Since the sliding resistance will develop at strain levels much lower than those needed to develop passive pressures against the side of the footing, passive pressure will be ignored unless it is found that sliding resistance governs the design of the footing. The geotechnical engineer should provide this equation and the assumptions regarding use of passive pressures to the structural engineer for checking the footing's resistance to sliding.

Step 9 – Check global stability of the footing:

The global stability of the abutment was checked using Bishop's simplified method of slices using the slope stability computer program XSTABL™. For this analysis, the actual footing was modeled as a very strong material, thus forcing the critical failure surface behind the heel of the footing (i.e., failure surfaces that passed through the footing were not permitted). The output is shown in Figure B4-4.

The calculated factor of safety for the specified surface was 2.6, which is larger than 1.5. (OK)

Step 10a – Size the footing:

The footing should be designed based on three loading conditions: (1) Intermediate Construction Step 1, after construction and backfill of the abutment and before erection of the superstructure (Figure B4-5); (2) Intermediate Construction Step 2, after erection of the superstructure and before live loads are applied (Figure B4-6); and (3) Final Loading Conditions (Figure B4-7).

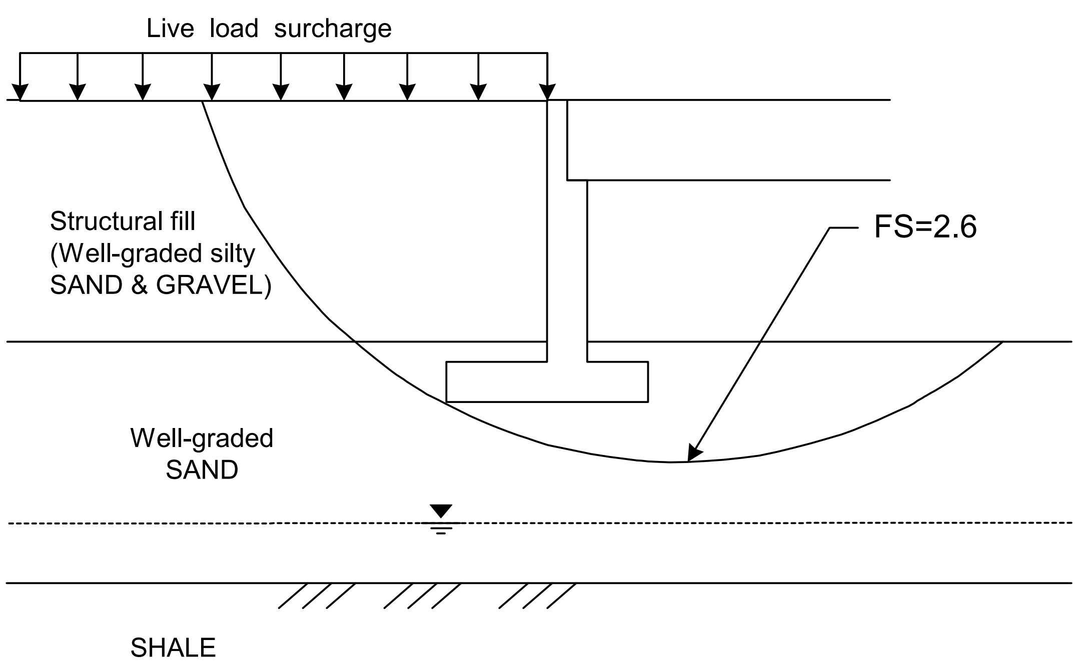


Figure B4-4: Global Stability Analysis Results

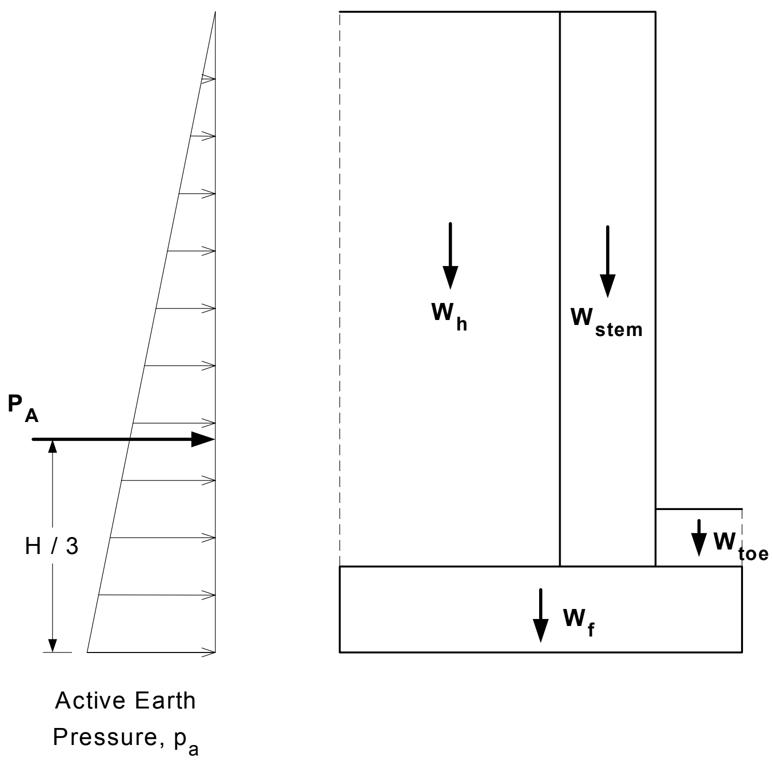


Figure B4-5: Intermediate Construction Step 1 Loading Conditions

Perform Steps 10 and 11 for the first intermediate construction step loading conditions:

Try a 5.2-m footing width:

Given that the superstructure will be erected before the girder seat is constructed, assume that the top of the backfill is at the base of the girder seat. The height of the girder seat is approximately 1.8 m.

Calculate earth pressures and weights:

$$P_A = (1/2)K_a \gamma H^2 \qquad \text{(Eqn. 6-1)}$$

$$K_a = \tan^2(45 - \phi'/2) \qquad \text{(Eqn. 6-3)}$$

$$K_a = \tan^2(45 - 38/2) = 0.24$$

$$P_A = (1/2)(0.24)(19.6\frac{kN}{m^3})(8.5m - 1.8m)^2 = 106^{kN}$$

AASHTO recommends using a minimum K_a of 0.3. However, because this material will be a specified compacted select granular material, we will use the calculated K_a value of 0.24 for design.

$$W_{stem} = (1.0m)(8.5m - 1.8m - 0.9m)(23.5\frac{kN}{m^3}) = 136^{kN} \qquad \text{assume } \gamma_{concrete} = 23.5\frac{kN}{m^3}$$

$$W_f = (5.2m)(0.9m)(23.5\frac{kN}{m^3}) = 110^{kN}$$

$$W_h = (5.2\,m\left(\frac{2}{3}\right) - 0.5\,m)(8.5m - 1.8m - 0.9m)(19.6\frac{kN}{m^3}) = 338^{kN}$$

$$W_{toe} = (5.2\,m\left(\frac{1}{3}\right) - 0.5\,m)(0.6m)(19.6\frac{kN}{m^3}) = 14.5^{kN}$$

Calculate moment arms about toe:

$$arm_{stem} = 5.2\,m/3 = 1.73\,m$$

$$arm_f = 5.2\,m/2 = 2.6\,m$$

$$arm_h = 0.5(5.2\,m + [(5.2\,m/3) + 0.5\,m] = 3.72\,m$$

$$arm_{toe} = 0.5(5.2\,m/3 - 0.5\,m) = 0.62\,m$$

$$arm_{P_A} = (8.5\,m - 1.8\,m)/3 = 2.23\,m$$

Calculate eccentricity:

$$e = (B_f/2) - (M_{toe}/P_R)$$

$$P_R = W + P_v = W_{stem} + W_f + W_h + W_{toe}$$
$$P_R = 136^{kN} + 110^{kN} + 338^{kN} + 14.5^{kN} = 599^{kN}$$

$$M_{toe} = W_{stem}(arm_{stem}) + W_f(arm_f) + W_h(arm_h) + W_{toe}(arm_{toe}) - P_A(arm_{P_A})$$
$$M_{toe} = (136^{kN})(1.73m) + (110^{kN})(2.6m) + (338^{kN})(3.72m) + (14.5^{kN})(0.62m)$$
$$- (106^{kN})(2.23m)$$
$$M_{toe} = 1787^{kN-m} - 236^{kN-m} = 1551^{kN-m}$$

$$e = (5.2m/2) - (1551^{kN-m}/599^{kN}) = 0.01\,m$$

Calculate effective footing dimensions:

$$B'_f = B_f - 2e \qquad \text{(Eqn. 6-6)}$$

$$B'_f = 5.2\,\text{m} - 2(0.01\,\text{m}) = 5.18\,\text{m}$$

Check allowable bearing stress for effective footing area:

$$q_{applied} = P \div B'_f = 599^{kN} \div 5.18 = 116\text{kPa}$$

This is less than q_{all} = 290 kPa for footings between 3.0 and 5.5 m wide, according to Table B4-4, to limit settlement to less than 25 mm. (OK)

Step 11a – Check overturning and sliding:

Overturning will be acceptable if eccentricity in any direction is less than one-sixth the footing dimension in that direction.

$$\frac{1}{6}B_f = \frac{1}{6}(5.2\text{m}) = 0.87\,\text{m} > e = 0.01 \qquad \text{(OK)}$$

The loads resisting sliding are:

$$W + P_v = P_R = 599^{kN}$$

The force, $F_{sliding}$, that tends to induce sliding consists only of the active earth pressure load, P_A:

$$\begin{aligned} F_{sliding} &= P_A \\ &= 106\,\text{kN} \end{aligned}$$

The force, F_R, available to resist sliding (from Step 8) is

$$F_R = 0.78(W + P_v)$$

The factor of safety FS against sliding is

$$FS_{sliding} = \frac{F_R}{F_{sliding}} \geq 1.5 \qquad \text{(Eqn. 5-40)}$$

$$\begin{aligned} &= \frac{0.78(599\,\text{kN})}{106\,\text{kN}} \\ &= \frac{467\,\text{kN}}{106\,\text{kN}} \\ &= 4.4 \end{aligned}$$

The factor of safety should be > 1.5. (OK)

Repeat Steps 10 and 11 for the second intermediate construction step loading conditions as shown in Figure B4-6:

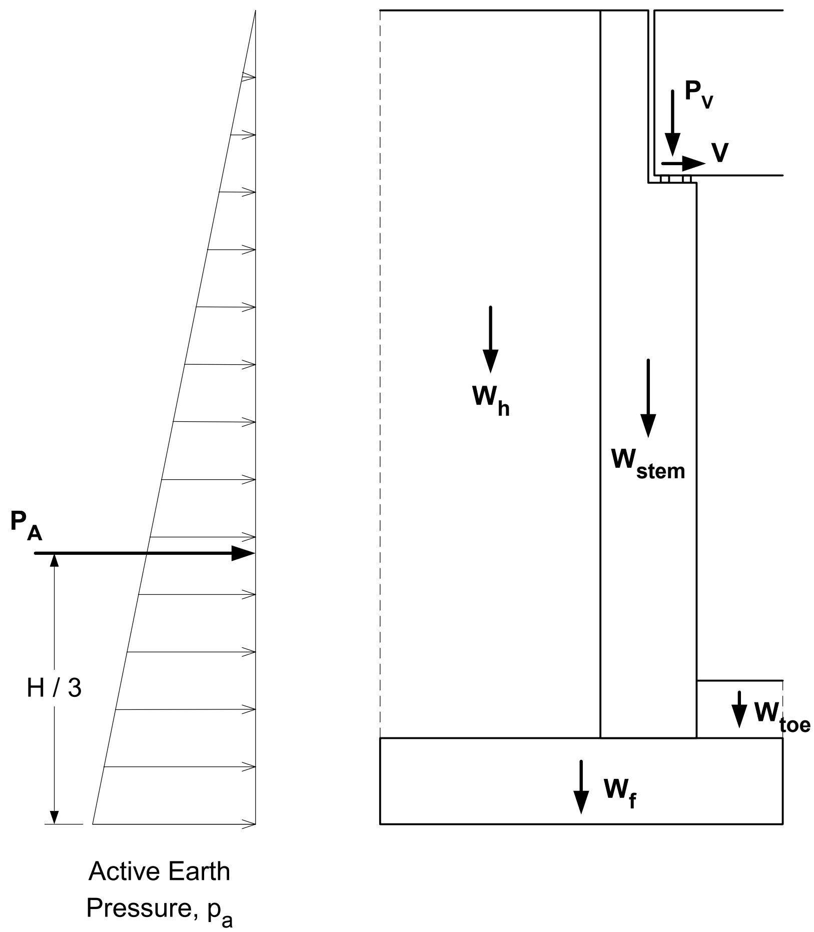


Figure B4-6: Intermediate Construction Step 2 Loading Conditions

Step 10b – Size the footing:

For a 5.2-m footing width:

Calculate earth pressures and weights:

$$P_A = \frac{1}{2} K_a \gamma H^2 \qquad \text{(Eqn. 6-1)}$$

$$K_a = \tan^2 (45 - \phi'/2) \qquad \text{(Eqn. 6-3)}$$

$$K_a = \tan^2(45 - 38/2) = 0.24$$

$$P_A = \frac{1}{2}(0.24)(19.6\tfrac{kN}{m^3})(8.5m)^2 = 170^{kN}$$

$$W_{stem} = [(1.0m)(8.5m - 0.9m) - (0.5m)(1.8m)](23.5\frac{kN}{m^3}) = 157^{kN}$$

$$W_f = (5.2m)(0.9m)(23.5\frac{kN}{m^3}) = 110^{kN}$$

$$W_h = (5.2\,m\left(\frac{2}{3}\right) - 0.5\,m)(8.5m - 0.9m)(19.6\tfrac{kN}{m^3}) = 442^{kN}$$

$$W_{toe} = (5.2\,m\left(\frac{1}{3}\right) - 0.5\,m)(0.6m)(19.6\tfrac{kN}{m^3}) = 14.5^{kN}$$

Calculate moment arms about toe:

$$arm_{stem} = \frac{(5.2\,m/3)(8.5\,m - 0.9\,m - 1.8\,m)(1\,m) + [(5.2\,m/3) + 0.25\,m](1.8\,m)(0.5\,m)}{(8.5\,m - 0.9\,m)(1\,m) - (1.8\,m)(0.5\,m)}$$

$$= 1.77\,m$$

$$arm_f = 5.2\,m/2 = 2.6\,m$$

$$arm_h = 0.5(5.2\,m + [(5.2\,m/3) + 0.5\,m] = 3.72\,m$$

$$arm_{toe} = 0.5(5.2\,m/3 - 0.5\,m) = 0.62\,m$$

$$arm_{P_v} = (5.2\,m/3 - 0.25\,m) = 1.48\,m$$

$$arm_{P_A} = 8.5\,m/3 = 2.83\,m$$

$$arm_v = 8.5\,m - 1.8\,m = 6.7\,m$$

Calculate eccentricity:

$$e = (B_f/2) - (M_{toe}/P_R)$$

$$P_R = W + P_v = W_{stem} + W_f + W_h + W_{toe} + P_v$$

$$P_R = 157^{kN} + 110^{kN} + 442^{kN} + 14.5^{kN} + 227^{kN} = 951^{kN}$$

$$M_{toe} = W_{stem}(arm_{stem}) + W_f(arm_f) + W_h(arm_h) + W_{toe}(arm_{toe}) + P_v(arm_{P_v})$$
$$- P_A(arm_{P_A}) - V(arm_V)$$
$$M_{toe} = (157^{kN})(1.77m) + (110^{kN})(2.6m) + (442^{kN})(3.72m) + (14.5^{kN})(0.62m)$$
$$+ (227^{kN})(1.48m) - (170^{kN})(2.83m) - (41.9^{kN})(6.7m)$$
$$M_{toe} = 2553^{kN-m} - 762^{kN-m} = 1791^{kN-m}$$

$$e = (5.2m/2) - (1791^{kN-m}/951^{kN}) = 0.72\,m$$

Calculate effective footing dimensions:

$$B'_f = B_f - 2e \qquad \text{(Eqn. 6-6)}$$

$$B'_f = 5.2\,m - 2(0.72\,m) = 3.76\,m$$

Check allowable bearing stress for effective footing area:

$$q_{applied} = P \div B'_f = 951^{kN} \div 3.76\,m = 253kPa$$

This is less than q_{all} = 290 kPa for footings between 3.0 and 5.5 m wide, according to Table B4-4, to limit settlement to less than 25 mm. (OK)

Step 11b – Check overturning and sliding:

Overturning will be acceptable if eccentricity in any direction is less than 1/6 the footing dimension in that direction.

$$\frac{1}{6}B_f = \frac{1}{6}(5.2m) = 0.87\,m > e = 0.72 \qquad \text{(OK)}$$

The loads resisting sliding are:

$$W + P_v = P_R = 951^{kN}$$

The force, $F_{sliding}$, that tends to induce sliding consists of the active earth pressure load, P_A and the shear load, V (Table B4-1):

$$F_{sliding} = P_A + V$$
$$= 170\,kN + 41.9\,kN$$
$$= 212\,kN$$

The force, F_R, available to resist sliding (from Step 8) is

$$F_R \quad = 0.78(W + P_v)$$

The factor of safety FS against sliding is

$$FS_{sliding} = \frac{F_R}{F_{sliding}} \geq 1.5 \qquad \text{(Eqn. 5-40)}$$

$$= \frac{0.78(951\,kN/m)}{212\,kN/m}$$

$$= \frac{742\,kN/m}{212\,kN/m}$$

$$= 3.5$$

The factor of safety should be > 1.5. (OK)

Repeat Steps 10 and 11 for the final loading conditions:

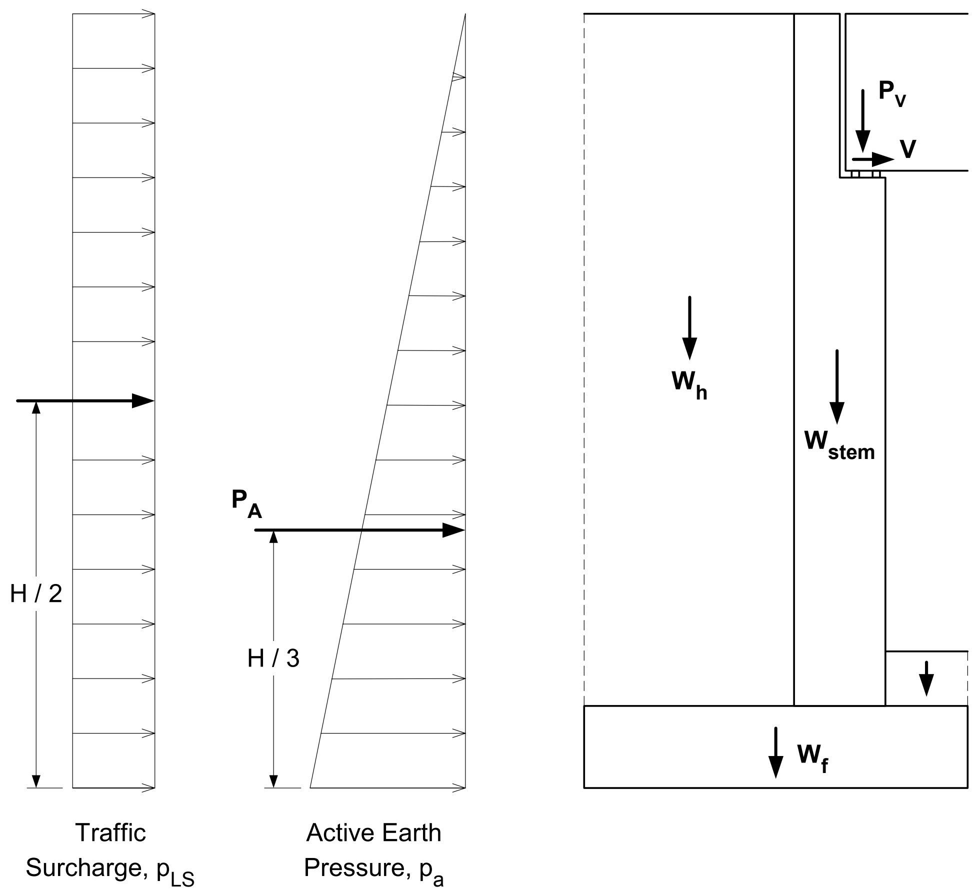


Figure B4-7: Final Loading Conditions

Step 10c – Size the footing:

For a 5.2-m footing width:

From Step 10b:

$$P_{LS} = K_a \gamma H_{LS} H_{abut}$$
$$P_{LS} = (0.24)(19.6\tfrac{kN}{m^3})(0.6m)(8.5m) = 24.0^{kN}$$

$$arm_{P_{LS}} = 8.5\,m / 2 = 4.25\,m$$

Calculate eccentricity:

$$e = (B_f / 2) - (M_{toe} / P_R)$$

$$P_R = W + P_v = W_{stem} + W_f + W_h + W_{toe} + P_v$$
$$P_R = 157^{kN} + 110^{kN} + 442^{kN} + 14.5^{kN} + 227^{kN} + 61.6^{kN} = 1013^{kN}$$

$$M_{toe} = W_{stem}(arm_{stem}) + W_f(arm_f) + W_h(arm_h) + W_{toe}(arm_{toe}) + P_v(arm_{P_v})$$
$$- P_A(arm_{P_A}) - P_{LS}(arm_{LS}) - V(arm_V)$$
$$M_{toe} = (157^{kN})(1.77m) + (110^{kN})(2.6m) + (442^{kN})(3.72m) + (14.5^{kN})(0.62m)$$
$$+ (227^{kN} + 61.6^{kN})(1.48m) - (170^{kN})(2.83m) - (24.0^{kN})(4.25m) - (41.9^{kN})(6.7m)$$
$$M_{toe} = 2645^{kN-m} - 864^{kN-m} = 1781^{kN-m}$$

$$e = (5.2m / 2) - (1781^{kN-m} / 1013^{kN}) = 0.84\,m$$

Calculate effective footing dimensions:

$$B'_f = B_f - 2e \qquad \text{(Eqn. 6-6)}$$

$$B'_f = 5.2\,m - 2(0.84\,m) = 3.52\,m$$

Check allowable bearing stress for effective footing area:

$$q_{applied} = P \div B'_f = 1013^{kN} \div 3.52\,m = 288Pa$$

This is less than q_{all} = 290 kPa for footings between 3.0 and 5.5 m wide, according to Table B4-4, to limit settlement to less than 25 mm. (OK)

Step 11c – Check overturning and sliding:

Overturning will be acceptable if eccentricity in any direction is less than one-sixth the footing dimension in that direction.

$$\frac{1}{6}B_f = \frac{1}{6}(5.2\text{m}) = 0.87\,\text{m} > e = 0.84 \qquad \text{OK}$$

The loads resisting sliding are:

$$W + P_v = P_R = 1013^{\text{kN}}$$

The force, $F_{sliding}$, that tends to induce sliding consists of the active earth pressure load, P_A and the shear load, V (Table B3-2):

$$\begin{aligned} F_{sliding} &= P_A + V \\ &= 170\,\text{kN} + 41.9\,\text{kN} \\ &= 212\,\text{kN} \end{aligned}$$

The force F_R available to resist sliding (from Step 8) is

$$F_R = 0.78(W + P_v)$$

The factor of safety FS against sliding is

$$FS_{sliding} = \frac{F_R}{F_{sliding}} \geq 1.5 \qquad \text{(Eqn. 5-40)}$$

$$\begin{aligned} &= \frac{0.78(1013\,\text{kN/m})}{212\,\text{kN/m}} \\ &= \frac{790\,\text{kN/m}}{212\,\text{kN/m}} \\ &= 3.7 \end{aligned}$$

This should be > 1.5. (OK)

Step 12 – Complete structural design of footing:

The structural engineer completes this step.

(This page intentionally blank)

EXAMPLE 5: INTERIOR BRIDGE PIER ON IN-SITU ROCK

Given:

- Subsurface profile at the interior bridge pier is shown in Figure B5-1.
- There is no scour potential at this site.
- Tolerable settlement is 38 mm (1.5 in).
- The direction conventions are shown in Figure B5-2.

Find:

- Size the footing based on Service Load Design, Group VII (seismic) loads (allowable stress design method), considering both the settlement and bearing capacity. Check for overturning and sliding.

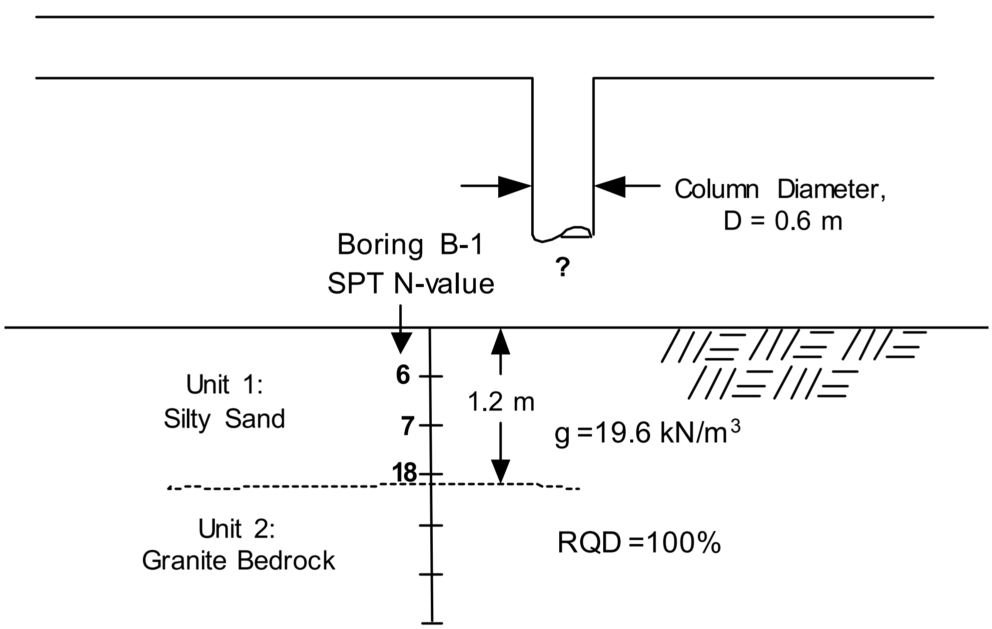


Figure B5-1: Subsurface Profile

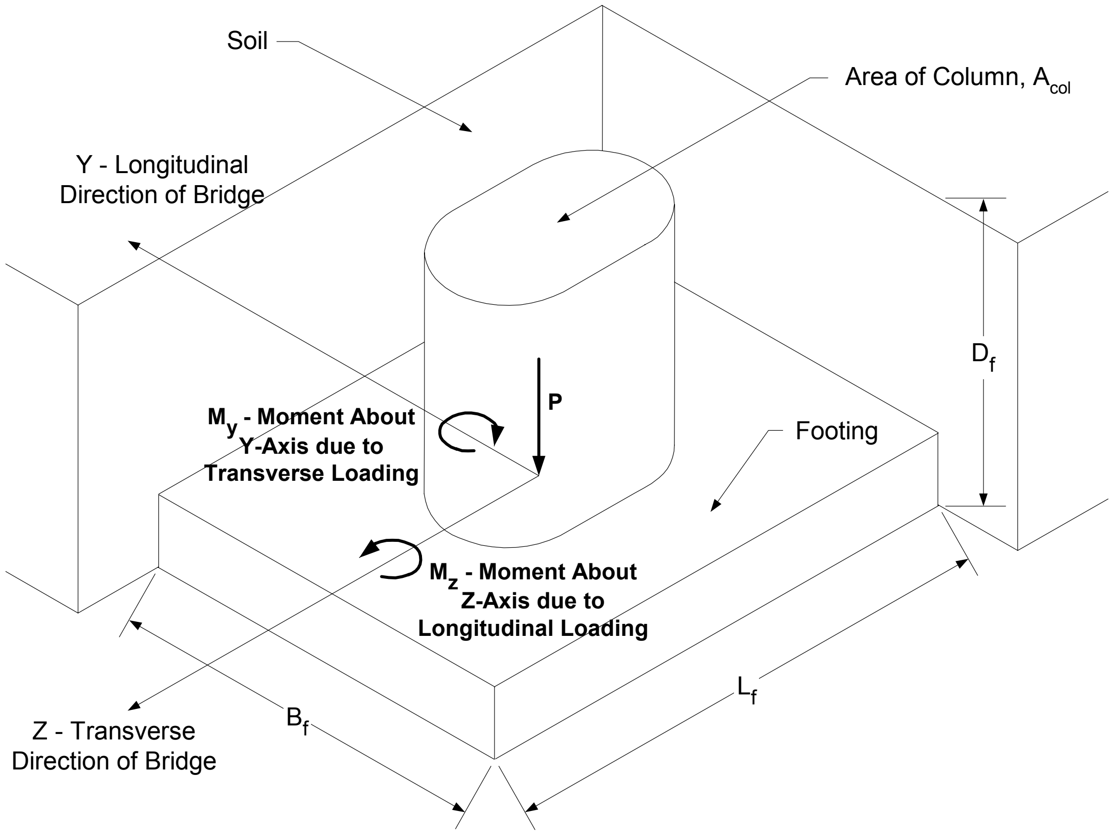


Figure B5-2: Sign Conventions and Loading Directions

Solution:

The solution can be obtained by following the step-by-step process shown in Figure 6-1, Design Process Flow Chart.

Steps 1 through 4 – Preliminary bridge layout, review of existing geologic and subsurface data, site reconnaissance, scour and frost potential:

The structural, hydraulics and geotechnical engineers completed these steps. The required design information is provided in the problem given statement.

Step 5 – Determine the loads applied to the footing:

The structural engineer has provided the loads applied at the base of the column as shown in Table B5-1.

TABLE B5-1: LOADS AT COLUMN BASE

Load	Axial, P (kN)	Shear, V (kN)	Moment, M_Z (kN-m)	Moment, M_Y (kN-m)
Deadload (D)	6400	167	211	748
Live load (L)	1670	42	409	196.5
Impact (I)	315	8	77	37
Wind on Structure (W)	884	49	89	226
Wind on Live Load (WL)	18	4	7	26
Earthquake (EQ)	1671	804	1675	5546

<u>Service Load Design Loads for Group VII</u>:

The general equation is (refer to Example B-1 for discussion of variables):

$$\begin{aligned} P_I = \gamma\,[&\beta_D(D) + \beta_L\,(L+I) + \beta_C(CF) + \beta_E(E) + \beta_B\,(B) \\ &+ \beta_S(SF) + \beta_W(W) + \beta_{WL}(WL) + \beta_L(LF) + \beta_R\,(R+S+T) \\ &+ \beta_{EQ}(EQ) + \beta_{ICE}(ICE)] \end{aligned}$$

(AASHTO, 1996, Eqn. 3-10)

Because the coefficients, β , for live load (L), impact (I), wind (W) and wind on live load (WL) are all zero for Load Group VII, this reduces to the following:

$$P_{VII} = \gamma\,[\beta_D(D) + \beta_E\,(E) + \beta_B\,(B) + \beta_S\,(SF) + \beta_{EQ}(EQ)]$$

Also, because there are no earth (E), buoyancy (B) or stream force (SF) loads in Table B5-1, the equation further reduces to:

$$P_{VII} = \gamma\,[\beta_D(D) + \beta_{EQ}(EQ)]$$

The axial load due to earthquake can act in either the positive or negative direction. For checking bearing stress, the load will be considered to act downward. For checking overturning (eccentricity), the load will be considered to act upward. So the factored loads are as follows:

Axial loads for checking bearing stress:

$$\begin{aligned} P_{V_{bearing}} &= \gamma\,[\beta_D(D) + \beta_{EQ}(EQ)] \\ &= 1.0\,[1.0\,(6400^{kN}) + 1.0\,(1671^{kN})] \\ &= 8071^{kN} \end{aligned}$$

Axial loads for checking overturning:

$$P_{v_{OT}} = \gamma\,[\beta_D(D) + \beta_{EQ}(EQ)]$$
$$= 1.0\,[1.0\,(6400^{kN}) + 1.0\,(-1671^{kN})]$$
$$= 4729^{kN}$$

Shear:

$$V = \gamma\,[\beta_D(D) + \beta_{EQ}(EQ)]$$
$$= 1.0\,[1.0\,(167^{kN}) + 1.0\,(804^{kN})]$$
$$= 971^{kN}$$

Moment about the Z-axis:

$$M_Z = \gamma\,[\beta_D(D) + \beta_{EQ}(EQ)]$$
$$= 1.0\,[1.0\,(211^{kN\text{-}m}) + 1.0\,(1675^{kN\text{-}m})]$$
$$= 1886^{kN\text{-}m}$$

Moment about the Y-axis:

$$M_Y = \gamma\,[\beta_D(D) + \beta_{EQ}(EQ)]$$
$$= 1.0\,[1.0\,(748^{\ kN\text{-}m}) + 1.0\,(5546^{kN\text{-}m})]$$
$$= 6294^{kN\text{-}m}$$

Step 6 – Conduct field exploration and laboratory testing:

This step is complete, and the subsurface data are shown in Figure B5-1. The information indicates that the granite bedrock is hard, sound rock with discontinuities less than 2 mm wide, spaced more than 6 m apart. Granite RQD = 100 percent.

Step 7 – Calculate allowable bearing capacity:

Allowable bearing capacities for footings on competent rock can be determined using presumptive bearing pressures. From Table 5-7, the recommended allowable bearing capacity for sound granite is 7.7 MPa. According to AASHTO (1996) Table 3.22.1A for earthquake loadings, the allowable bearing capacity can be increased by one-third, resulting in an overstress allowable bearing capacity under earthquake loads of 10.2 MPa; therefore, use q_{all} = 10,200 kPa.

In general practice, settlement is not checked for Group VII loads because seismic loads are transient and temporary in nature. However, for completeness, we will check settlement in Step 10 based on elastic theory for the actual footing dimensions and bearing stress.

Step 8 – Calculate sliding and passive soil resistance:

The geotechnical engineer has recommended that the footing be cast against unweathered granite rock in Unit 2.

The general equation for sliding resistance is:

$$F_R = (W + P_v) \tan \delta \qquad \text{(Eqn. 5-38)}$$

The second term goes to zero since we are not founding the footing on a cohesive soil unit. For a concrete footing cast against the ground, assume an interface friction angle of $\delta = 35$ degrees (Table 5-15, after NAVFAC (1986b)). In reality, this is a very conservative assumption for mass concrete poured on a rough, monolithic rock surface. However, sliding is not usually the controlling condition for interior bridge foundations, so we will use this conservative assumption unless it is found to control the design of the footing. The ultimate sliding resistance is therefore:

$$F_{sliding} = (W + P_v)\tan 35° = 0.7(W + P_v)$$

Since the sliding resistance will develop at strain levels much lower than those needed to develop passive pressures against the side of the footing, passive pressure will be ignored unless it is found that sliding resistance governs the design of the footing. The geotechnical engineer should provide this equation and the assumptions regarding use of passive pressures to the structural engineer for checking the footing's resistance to sliding.

Step 9 – Check global stability of the footing:

The geotechnical engineer has determined that since this is an interior pier with level ground conditions global stability is not a concern.

Step 10 – Size the footing:

The moments applied to the footing will create large eccentric loadings. First determine the minimum effective footing area necessary to provide sufficient bearing capacity. Then size the footing to account for the eccentric loading applied by the moment.

The minimum effective footing area is calculated as follows using the maximum axial load:

$$A' = P_{v_{bearing}} \div q_{all}$$

$$A' = 8071^{kN} \div 10{,}200 \text{ kPa} = 0.79\,m^2$$

Due to the large differences in applied moments, make the footing rectangular to optimize the size. The relative dimensions of the footing should be the same as the relative magnitudes of the moments.

$$B'_f = (L'_f)\frac{M_Z}{M_Y} = (L'_f)\frac{1886}{6294} = 0.3(L'_f)$$

Determine the effective footing dimensions:

$$A' = B_f' \times L_f'$$
$$A' = 0.3L_f' \times L_f'$$
$$L_f' = \sqrt{\frac{A'}{0.3}} = \sqrt{\frac{0.79\text{m}^2}{0.3}} = 1.62\,\text{m}$$
$$B_f' = 0.3(L_f') = 0.3(1.62\text{m}) = 0.49\,\text{m}$$

The actual footing dimensions are influenced by the eccentricity. Calculate the eccentricity using the minimum axial load.

$$e_y = M_Z \div P_{v_{OT}}$$
$$e_y = 1886^{\text{kN}-\text{m}} \div 4729^{\text{kN}} = 0.399\,\text{m}$$

$$e_z = M_Y \div P_{v_{OT}}$$
$$e_z = 6294^{\text{kN}-\text{m}} \div 4729^{\text{k}} = 1.33\,\text{m}$$

Determine the footing dimensions necessary to resist the moments by rearranging Equations 6-6 and 6-7:

$$L_f = L_f' + 2e_z = 1.62\,\text{m} + 2(1.33\,\text{m}) = 4.28\,\text{m}$$

$$B_f = B_f' + 2e_y = 0.49\text{m} + 2(0.399\text{m}) = 1.29\,\text{m}$$

Perform a quick check to ensure that overturning is also acceptable. Overturning will be acceptable if eccentricity in any direction is less than one-fourth the footing dimension in that direction for footings on rock, as discussed in Section 6.4.1.

$$\frac{1}{4}B_f = \frac{1}{4}(1.3\,\text{m}) = 0.33\,\text{m} < e_y = 0.399\,\text{m} \qquad (\underline{\text{not}}\ \text{OK})$$

$$\frac{1}{4}L_f = \frac{1}{4}(4.3\,\text{m}) = 1.01\,\text{m} < e_z = 1.33\,\text{m} \qquad (\underline{\text{not}}\ \text{OK})$$

Therefore, use a minimum footing width and length of 4 times e_y and e_z, respectively.

$$B_f = 4e_y = 4(0.399\,\text{m}) = 1.6\,\text{m}$$

$$L_f = 4e_z = 4(1.33\,\text{m}) = 5.32\,\text{m}$$

Therefore, a footing with dimensions $L_f = 5.4$ m by $B_f = 1.6$ m will be stable and adequate for bearing capacity. Typically, the minimum footing width should be at least 3 column diameters [i.e., 3(0.6 m) = 1.8 m] to provide the required development length for the reinforcement of the column-footing connection. The final dimensions of the footing will be controlled by the structural design of the footing, but the footing dimension should be no smaller than L_f = 5.4 m and B_f = 1.8 m.

Check the assumption regarding settlement:

Using elastic theory:

$$\delta_v = \frac{C_d \Delta\sigma_V B_f (1-\nu^2)}{\alpha_E E_{IR}} \qquad \text{(Eqn. 5-36)}$$

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \qquad \text{(Eqn. 5-37)}$$

From Table 5-12 for a rectangle with L/B ratio of 3, C_d = 1.78. For a granite with RQD = 100%, ν = 0.2 and E_{IR} = 52 x 10^6 kPa, from Tables 5-13 and 5-14. The applied stress will be equal to the allowable bearing stress since the footing bears directly on the rock.

$$\alpha_E = 0.0231(100) - 1.32 = 0.99$$

So:

$$\delta_v = \frac{(1.78)(10200\text{kPa})(1\,\text{m})(1-0.20^2)}{(0.99)(52000000\text{kPa})} = 0.0003\,\text{m} = 0.3\,\text{mm}$$

This settlement is negligible.

The design is acceptable for bearing capacity and settlement considerations.

Step 11 – Check overturning and sliding:

Overturning was already checked during the sizing of the footing in Step 10.

The footing will be stable against sliding if:

$$FS = \frac{F_R}{F_{sliding}} \geq 1.5$$

From Step 8:

$$F_R = 0.7(W + P_v)$$

Calculate the weight of cover over the footing and the weight of the footing:

The footing is bearing 1.2 m below grade and can be assumed to be 1 m thick.

The area of the column is:

$$A_{col} = \frac{\pi}{4}(0.6\text{m})^2 = 0.28\,\text{m}^2$$

The weight of the footing and the soil cover is then:

$$W_{ftg} = (1.0\,\text{m})(1.8\text{m})(5.4\text{m})(23.5\tfrac{\text{kN}}{\text{m}^3}) = 228.4^{\text{kN}}$$

$$W_{cover} = [(0.2\,\text{m})(1.8\text{m})(5.4\text{m}) - (0.2\text{m})(0.28\,\text{m}^2)](19.6\tfrac{\text{kN}}{\text{m}^3}) = 37.0^{\text{kN}}$$

So, resistance to sliding is:

$$F_R = 0.7(W + P_{v_{OT}})$$

$$F_R = 0.7(W_{ftg} + W_{cover} + P_{v_{OT}})$$
$$= 0.7(228.4^{\text{kN}} + 37^{\text{kN}} + 4729^{\text{kN}}) = 3496^{\text{kN}}$$

The forces acting to cause the footing to slide are only due to the shear force, V, so

$$F_{sliding} = V = 971\text{kN}$$

Then:

$$FS = \frac{F_R}{F_{sliding}} = \frac{3496}{971} = 3.6$$

This should be greater than about 1.0 to 1.15 for seismic loads. (OK)

Step 12 – Complete structural design of footing

The structural engineer completes this step.

APPENDIX C

LOAD AND RESISTANCE FACTOR DESIGN

This appendix provides a discussion of the design of shallow foundations for bridges using Load and Resistance Factor Design (LRFD) methodology, and is based on the AASHTO LRFD Bridge Design Specifications (1998). Design examples B-1 and B-3 are re-worked using LRFD methods and principles. The purpose of these examples is to show that designs completed using LRFD methodology are not radically different from those completed by SLD.

It should be noted that these examples are intended to convey one approach and process that can lead to a satisfactory design of bridge support on a shallow foundation. The examples should not be construed as representations of the *only* approach that can lead to a satisfactory design. Also note that the examples only check one or, at most, a few specific load cases. In practice, every complete design should consider all applicable load cases.

EXAMPLE C-1: INTERIOR BRIDGE PIER ON A SPREAD FOOTING FOUNDED IN NATIVE SOIL

This example sizes the footing for an interior pier considering the LRFD Service I, Strength I and Extreme I limit states. The loads and ground conditions are the same as used in Example B-1.

EXAMPLE C-2: STUB SEAT-TYPE ABUTMENT FOOTING IN COMPACTED STRUCTURAL FILL

This example sizes an abutment footing considering the LRFD Service I and Strength I limit states. The loads and ground conditions are the same as used in Example B-3.

Note: Unless cited with a specific reference (e.g., AASHTO, 1996), the equation numbers in the examples refer to those in the main body of this Circular.

C.1 INTRODUCTION

This section compares and contrasts the differences and similarities of Load and Resistance Factor Design (LRFD) methodology with Service Load Design (SLD) and explains how the geotechnical engineer must modify the design recommendations to make them usable for the structural engineer. It is not intended to provide comprehensive coverage on the background and application of LRFD to bridge design.

The reader should keep in mind that the fundamental theories and procedures used to calculate bearing capacity and settlement for shallow foundations are not different between LRFD and SLD. The differences come in the way the geotechnical engineer presents the design recommendations and the way the structural engineer compares them to the factored load combinations.

Recall that the "Standard Specifications" (AASHTO, 1996) were based on Service Level Design (SLD). In this design method, the stresses caused in members by design loads are compared to an allowable stress for the member. The allowable stress is determined by taking the nominal strength of the material (e.g., yield stress for steel) and applying a factor of safety. The allowable stress is also limited for serviceability considerations such as settlement. The following equation demonstrates the basic principle of the SLD method:

$$\sum Q_i = \frac{R_n}{FS} \qquad \text{(Eqn. C-1)}$$

where: Q_i is an applied load or stress
R_n is the resistance (e.g., strength or yield stress) of the member or material
FS is the applied factor of safety

The allowable stress includes considerations for strength, fatigue, serviceability and settlement. The factors of safety used in the allowable stress method are derived from industry code writing groups such as ACI and AISC. Factors of safety for geotechnical engineering are derived from industry practice and performance observation. Typically, the factors of safety for structural materials are in the range of 1.7, while the geotechnical factors of safety for foundations are from 2 to 3. These factors of safety are not derived based on probability theory. They are based on observation, full-scale tests, theory and judgment. Factors of safety adopted in the design codes are then agreed upon by committee consensus.

The allowable stress method has some limitations in that the variability in the strengths of the materials and the variability in the different loads are all combined into a single safety factor. For example, truck live loads have a much greater variability than the dead load of the structure, but a single factor of safety implies that the variability of all loads are similar.

The goal of LRFD methodology is to statistically account for the variability of the individual loads and resistances computed for design. Most of the AASHTO specification was calibrated based on the data available through NCHRP Project 12-33 (Nowak, 1992). Because not much data existed for calibrating the foundation (geotechnical) portion of the specification, some of the resistance factors for foundations and most of the resistance factors for walls were back-fit to match the existing SLD levels of reliability. The basic equation for the LRFD method is:

$$\sum \eta_i \gamma_i Q_i \leq \varphi R_n = R_r \qquad \text{(Eqn. C-2)}$$

where: η_i is a factor that includes the effects of ductility, redundancy and importance
γ_i is the load factor for a particular load
Q_i is a service level load
φ is the resistance factor
R_n is the nominal resistance
R_r is the factored resistance

Note that the load and resistance factors in the AASHTO (1998) LRFD Specification were calibrated with η_i equal to 1.0. The use of η_i other than 1.0 will therefore result in factored loads that are not consistent with the calibration, and should be done with caution or in accordance with the policies and procedures of the owner agency.

C.2 LIMIT STATES

A "limit state," as defined by the AASHTO 1998 specification, is a condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed. To meet the limit states applied to a design, the above equation must be satisfied for each limit state. The limit states are described in Table C-1. The descriptions of these limit states and the design conditions they consider can be compared with the load groups used in SLD (see Table 6-2 in the main text).

TABLE C-1: LRFD LIMIT STATES

Limit State	Description
Strength	The structure is checked for enough strength and stability to resist the loads imparted on it.
Strength I	Load combination with normal vehicles using the bridge without wind
Strength II	Load combination with permit vehicles using the bridge without wind
Strength III	Load combination with maximum winds and no vehicles
Strength IV	In this load case, large spans create large dead-to-live load effects. This load case need be used only where the dead-to-live load ratio is above about 7.0.
Strength V	Load combination with normal vehicles using the bridge with wind
Extreme Event	Load combinations relating to ice load, collision by vessels and vehicles, and to certain hydraulic events and earthquakes, whose recurrence interval exceeds the design life of the structure. The structure needs to survive these events. Some of these events need to be checked in the scoured condition.
Extreme Event I	Load combination for seismic loading
Extreme Event II	Load combination relating to ice load, collision by vessels and vehicles, and certain hydraulic events
Service	This is a check so that crack widths, deflections and other conditions do not jeopardize the life of the structure.
Service I	Load combination with normal vehicles and wind
Service II	Load combination for slip-critical steel connections
Service III	Load combination to control cracks in prestressed concrete structures
Fatigue	Fatigue and fracture load combination using a special fatigue vehicle

The AASHTO (1998) LRFD specification is very explicit regarding the different limit states that have to be considered in design. The LRFD strength limit states are similar to the AASHTO (1996) "Standard Specifications" for factored Load Groups I through VI, and the extreme limit states are similar to the factored Load Groups VII through IX. However, the "Standard Specifications" (AASHTO, 1996) do not include the explicit service (deformation) checks for all design elements that are included throughout the LRFD specifications (AASHTO, 1998).

SLD		**LRFD**
Deformations	⟺	Service
Capacity	⟺	Strength

C.3 LOADS AND LOAD COMBINATIONS

The AASHTO LRFD specifications prescribe the procedures used to compute loads and detail how to factor and combine the loads for comparing to the factored resistance. The load combinations and load factors per Table 3.4.1-1 (AASHTO, 1998) are shown in Table C-2.

TABLE C-2: LRFD LOAD COMBINATIONS AND LOAD FACTORS (AASHTO, 1998)

Load Combination Limit State	DC DD DW EH EV ES	LL IM CE BR PL LS EL	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time EQ	 IC	 CT	 CV
Strength I (unless noted)	γ_P	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Strength II	γ_P	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Strength III	γ_P	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Strength IV EH, EV, ES, DW DC only	γ_P 1.50	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
Strength V	γ_P	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Extreme Event I	γ_P	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
Extreme Event II	γ_P	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Service II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
Service III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Fatigue LL, IM & CE only	-	0.75	-	-	-	-	-	-	-	-	-	-	-

For permanent loads:

- DD = downdrag
- DC = dead load of structural components and nonstructural attachments
- DW = dead load of wearing surfaces and utilities
- EH = horizontal earth pressure load
- EL = accumulated locked-in effects resulting from the construction process (not usually important for foundation design)
- ES = earth surcharge load
- EV = vertical pressure from dead load of earth fill

For transient loads:

- BR = vehicular braking force
- CE = vehicular centrifugal force
- CR = creep
- CT = vehicular collision force (not usually important for foundation design)
- CV = vessel collision force (only important over navigable waterways)
- EQ = earthquake
- FR = friction (usually applies only with respect to forces from bearing assemblies)
- IC = ice load
- IM = vehicular dynamic load allowance (This is zero for the buried portions of structures. So for most foundations, this is zero.)
- LL = vehicular live load
- LS = live load surcharge
- PL = pedestrian live load
- SE = settlement
- SH = shrinkage
- TG = temperature gradient (not usually a factor for foundations)
- TU = uniform temperature
- WA = water load and stream pressure
- WL = wind on live load
- WS = wind load on structure

Per the specification, the larger of the two load factor values for TU, SH and CR should be used for deformations, and the smaller values for all other effects.

Table C-3 shows the load factors for permanent loads (γ_P) per Table 3.4.1-2 (AASHTO, 1998).

TABLE C-3: LOAD FACTORS FOR PERMANENT LOADS

Type of Load	Load Factor - γ_P	
	Maximum	Minimum
DC: Component and Attachment	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
EH: Horizontal Earth Pressure		
• Active	1.50	0.90
• At-Rest	1.35	0.90
EL: Locked-in Erection Stresses	1.00	1.00
EV: Vertical Earth Pressure		
• Retaining Walls and Abutments	1.35	1.00
• Rigid Buried Structure	1.30	0.90
• Rigid Frames	1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
• Flexible Metal Box Culverts	1.50	0.90
ES: Earth Surcharge	1.50	0.75

γ_{TG} and γ_{SE} are to be determined on a project-by-project basis. Traditionally, they are usually taken as zero for the design of open girders and multiple steel box girders. However, the specification proposes to use 0.0 at the strength and extreme limit states, 1.0 at the service limit state without live load, and 0.5 at the service limit state when live load is considered.

γ_{EQ} is a load factor applied to live load effects when considering earthquake loads and is supposed to be determined on a project specific basis. Traditionally this has been taken as zero. However, commentary to the specification suggests that 0.5 may be more appropriate when considering the possibility of a partial live load combined with seismic loads.

For foundation design not all of the above load cases will apply. The following load cases will generally apply to the design of foundations:

- Strength I
- Strength II (although with the new live loading being applied, this load case may not be necessary)
- Strength III
- Strength IV (this will only control for long-span structures where creep, shrinkage and temperature effects become significant)
- Strength V
- Extreme I
- Service I

Note that maximum and minimum load factors can be applied for any given foundation. Usually, the maximum load factors will be used to find the maximum soil pressures, while the minimum load factors will be used to check stability of a foundation.

Individual loads also have numerous different maximum and minimum effects. A good example of this is live load (LL). Live load effects on a foundation will include a loading case with maximum vertical load and two loading cases with maximum moments in the orthogonal directions. These all need to be checked when sizing and designing a shallow foundation. There are also configurations for which minimum live load effects can cause uplift at foundations. These also need to be included in the design of the foundation.

C.3.1 Live Load Surcharge

The live load surcharge under the LRFD methodology is also modeled as an equivalent soil surcharge behind the wall or abutment. However, the equivalent height of soil to use in calculating the live load surcharge varies as a function of the height of the wall and can be computed according Table C-4. The lateral load due to live load surcharge is then calculated and applied according to Section 6.3.

TABLE C-4: LIVE LOAD SURCHARGE

Wall Height (m)	h_{eq} (m)
1.5	1.2
3.0	0.9
6 or higher	0.6

C.4.1 Active Lateral Earth Pressures

Under LRFD methodology, the application of active lateral earth pressures on abutments and retaining walls is at four-tenths the height of the abutment (0.4 x H_{abut}) to account for backfill compaction effects.

C.4 GEOTECHNICAL DESIGN

Computations of bearing capacity and settlement of shallow foundations under LRFD methodology use the same procedures and equations as used in SLD. These are described in Chapter 5.

C.4.1 Service Limit State Bearing Stress

The service limit state checks in LRFD ensure that deflections and movements under normal bridge loads are within tolerable limits. For shallow foundations, service limit state considerations will be controlled by settlement computations. The procedures in Section 5.3 should be used to estimate settlement of footings as a function of the effective footing width, B'_f. This information can be plotted as the nominal bearing resistance for the service limit state on a graph, or tabulated, for use by the structural engineer in sizing the footing.

C.4.2 Strength and Extreme Limit State Bearing Resistance

For shallow foundations, the nominal bearing resistance at the strength and extreme limit states is usually taken as the ultimate bearing capacity computed using the procedures in Section 5.2. It is generally convenient to plot or tabulate the computed values as a function of effective footing width, B'_f. The geotechnical engineer should also recommend the resistance factor to apply to the nominal bearing resistance for performing the strength limit state checks.

It is not unusual for geotechnical engineers to initially feel uncomfortable with the magnitude of the ultimate bearing capacity that is computed. These bearing capacities are generally much larger than the allowable bearing capacities associated with Service Load Design. The geotechnical engineer should be aware that applying too much conservative judgment at this point could result in overconservative designs. Good communication with the structural engineer will help prevent designs being unnecessarily controlled by the strength limit state due to an overly conservative nominal bearing resistance. The geotechnical engineer should keep in mind that the load combinations in the strength limit state are multiplied by load factors that are larger than 1.0.

Note that presumptive bearing capacities include combined consideration of settlement and bearing capacity. For normally consolidated soils (e.g., not IGMs or rock), use of presumptive values for strength and extreme limit state checks will frequently result in grossly over-conservative designs, since they are generally controlled by settlement, which is a service limit state consideration. There is no rational approach to obtaining service and extreme limit state nominal bearing resistance. Increasing the presumptive values with arbitrary factors for use in strength and extreme limit state design checks is unwise and could be unsafe.

C.4.3 Resistance Factors

Resistance factors for the service and extreme limit states are nearly always taken as 1.0. Resistance factors for the strength limit state are less than 1.0 and are a function of the method used to compute the nominal (ultimate) bearing capacity.

Table C-5 presents the strength limit state resistance factors recommended by AASHTO for shallow foundations.

C.4.5 Global Stability

When the location of a shallow foundation can contribute to the instability of a slope, such as an abutment constructed in an approach fill, the overall, or global, stability of the slope should be checked. It is important to note that limit equilibrium methods are by far the most expedient method for evaluating slope stability. An important prerequisite of limit equilibrium methods is that the solutions are based on failure, or ultimate, stresses. It is recommended that this be done at the Service I limit state using unfactored loads and resistances. That is, the loads applied by the footing that are acting to destabilize the slope should be unfactored working loads. Classic limit equilibrium methods, as described in Section 5.5, should be used to calculate the factor of safety of the slope. All slopes supporting footings should have a global factor of safety of at least 1.5.

C.5 Shallow Foundation Design

The structural engineer will use the nominal bearing resistances provided by the geotechnical engineer to size the footing and perform all the necessary limit state checks.

C.5.1 Sizing the Footing

Since settlement considerations frequently control the size and design of shallow foundations, it is generally expedient to size the footing at the service limit state and then check to see that the nominal bearing resistances are below the factored resistance at the strength and extreme limit state.

C.5.2 Bearing Stresses

Once an initial sizing of the footing is made, all applicable limit states should be checked to ensure that the applied bearing stress is below the factored resistance at that limit state.

TABLE C-5: RESISTANCE FACTORS FOR STRENGTH LIMIT STATE FOR SHALLOW FOUNDATIONS (MODIFIED AFTER TABLE 10.5.5-1, AASHTO, 1998)

Method/Soil/Condition			Resistance Factor
Bearing Capacity		Sand	
		- *Semiempirical procedure using SPT data*	0.45
		- Semiempirical 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
		Clay	
		- Semiempirical procedure using CPT Data	0.50
		- Rational Method:	
		--using shear resistance measured in lab tests	0.60
		--using shear resistance measured in field vane tests	0.60
		--using shear resistance estimated from CPT data	0.50
		Rock	
		Semiempirical procedure, Carter and Kulhawy (1988)	0.60
		Plate Load Test	0.55
Sliding	φ_T	Precast concrete placed on sand	
		--using φ_f estimated from SPT data	0.90
		--using φ_f estimated from CPT data	0.90
		Concrete cast-in-place on sand	
		--using φ_f estimated from SPT data	0.80
		--using φ_f estimated from CPT data	0.80
		Clay (where shear resistance is less than 0.5 times normal pressure)	
		--using shear resistance measured in lab tests	0.85
		--using shear resistance measured in field tests	0.85
		--using shear resistance estimated form CPT data	0.80
		Clay (where the resistance is greater than 0.5 times normal pressure)	0.85
		Soil on soil	1.0
Passive Pressure	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

C.5.3 Eccentricity

The check for overturning potential is also performed by determining the eccentricity of the resultant load under maximum overturning conditions. This usually occurs using the minimum load factors for resisting loads. The maximum eccentricity of the loads determined with the minimum load factors should be less than one-fourth of the corresponding footing dimension, B or L, on soil and less than three-eighths of the corresponding footing dimension, B or L, on rock. Note that these permissible ranges differ from those used in Service Load Design.

Eccentricity is computed using the procedure and equations in Section 6.4.

C.5.4 Sliding

Sliding of the footing is checked to make sure that lateral loads do not make the footing move. The lateral capacity of the footing due to lateral loads is:

$$Q_R = \varphi_T Q_T + \varphi_{ep} Q_{ep} \qquad \text{(Eqn. C-3, after AASHTO 1998 Eqn. 10.6.3.3-1)}$$

where: Q_R is the factored resistance
φ_T is the resistance factor for shear resistance between the footing base and the soil
Q_T is the nominal shear resistance between soil and foundation
φ_{ep} is the resistance factor for passive soil resistance
Q_{ep} is the nominal passive resistance

As in Service Load Design, it is customary to neglect or reduce the passive resistance for the same reasons discussed in Section 5.4.

For cohesionless soils in which the concrete is cast against the soil:

$$Q_T = P_v \tan \phi \qquad \text{(Eqn. C-4, after AASHTO 1998 Eqn. 10.6.3.3-2)}$$

For cohesionless soils where the concrete is precast use:

$$Q_T = P_v(0.8)\tan \phi \qquad \text{(Eqn. C-5, after AASHTO 1998 Eqn. 10.6.3.3-2)}$$

where: Q_T = Nominal sliding resistance
ϕ = Internal friction angle of soil
P_v = Applied vertical force

Note that these equations inherently account for the lower sliding resistance resulting from precast (formed) concrete. The associated resistance factors in Table C-5 must therefore be used with these equations. For a footing bearing directly on cohesive soils the sliding resistance is equal to the undrained shear strength of the soil or one-half the vertical stress, whichever is less.

C.6 Design Process Flow Chart

The LRFD design process for a bridge supported on shallow foundations includes the steps shown in Table C-6 and in the LRFD design process flow chart, Figure C-1.

TABLE C-6: STEPS IN LRFD DESIGN PROCESS FOR BRIDGE SUPPORTED ON SHALLOW FOUNDATIONS

Step	LRFD Design Activity	Responsible Disciplines
1.	Develop **preliminary bridge layout**. The desired bridge type, size and location will be established. Span lengths and pier locations will be defined, considering geometrical and environmental constraints.	Structural, in coordination with general civil and environmental considerations and geotechnical for approach stability
2.	Determine the shallow foundation feasibility based on **review of existing geologic and subsurface data**. Competent bearing material must be present within a reasonable distance from the ground surface. A preliminary assessment of approach embankment stability should be conducted to evaluate potential impacts to abutment locations and span lengths. (Section 4.1).	Geotechnical, in coordination with structural, general and environmental
3.	A **site reconnaissance** with the structural and general civil engineer should be completed at this stage to evaluate constructability of foundation types (Section 4.2).	Geotechnical, in coordination with structural, general and environmental
4.	Determine the **depth of the footing** so that it will not be susceptible to **scour potential** or **frost** (Section 6.2).	Hydraulic, with geologic input from geotechnical
5.	Determine the **loads** applied to the footing (Section 6.3).	Structural
6.	Determine the design soil properties from the **subsurface exploration** and **laboratory testing** program (Sections 4.3 and 4.4).	Geotechnical
7.	Calculate the **nominal bearing resistance**, based on effective footing width, B'_f (Section 5.2) at the **strength and extreme** limit states.	Geotechnical
8.	Calculate the **nominal bearing resistance** based on effective footing dimensions at the **service limit state** (Section 5.3).	Geotechnical
9.	Calculate the **sliding** and **passive soil resistance** at the **strength and extreme** limit state (Section 5.4).	Geotechnical
10.	When overall stability of the footing may govern the design (e.g., footings on or near slopes), perform a **global stability analysis** of the footing using service (unfactored) loads (Section 5.4).	Geotechnical
11.	Size the **footing dimensions** at the service limit state (Section 6.4.1).	Structural
12.	Check the **bearing pressure**, maximum **eccentricity** and **sliding** at the **strength** limit state (Sections 6.4.3 and 6.4.4).	Structural
13.	Check the **bearing pressure**, maximum **eccentricity** and **sliding** at the **extreme** limit state (Sections 6.4.3 and 6.4.4).	Structural
14.	Complete the **structural design** of the footing using factored loads according to the concrete section of the specification (AASHTO, 1998).	Structural

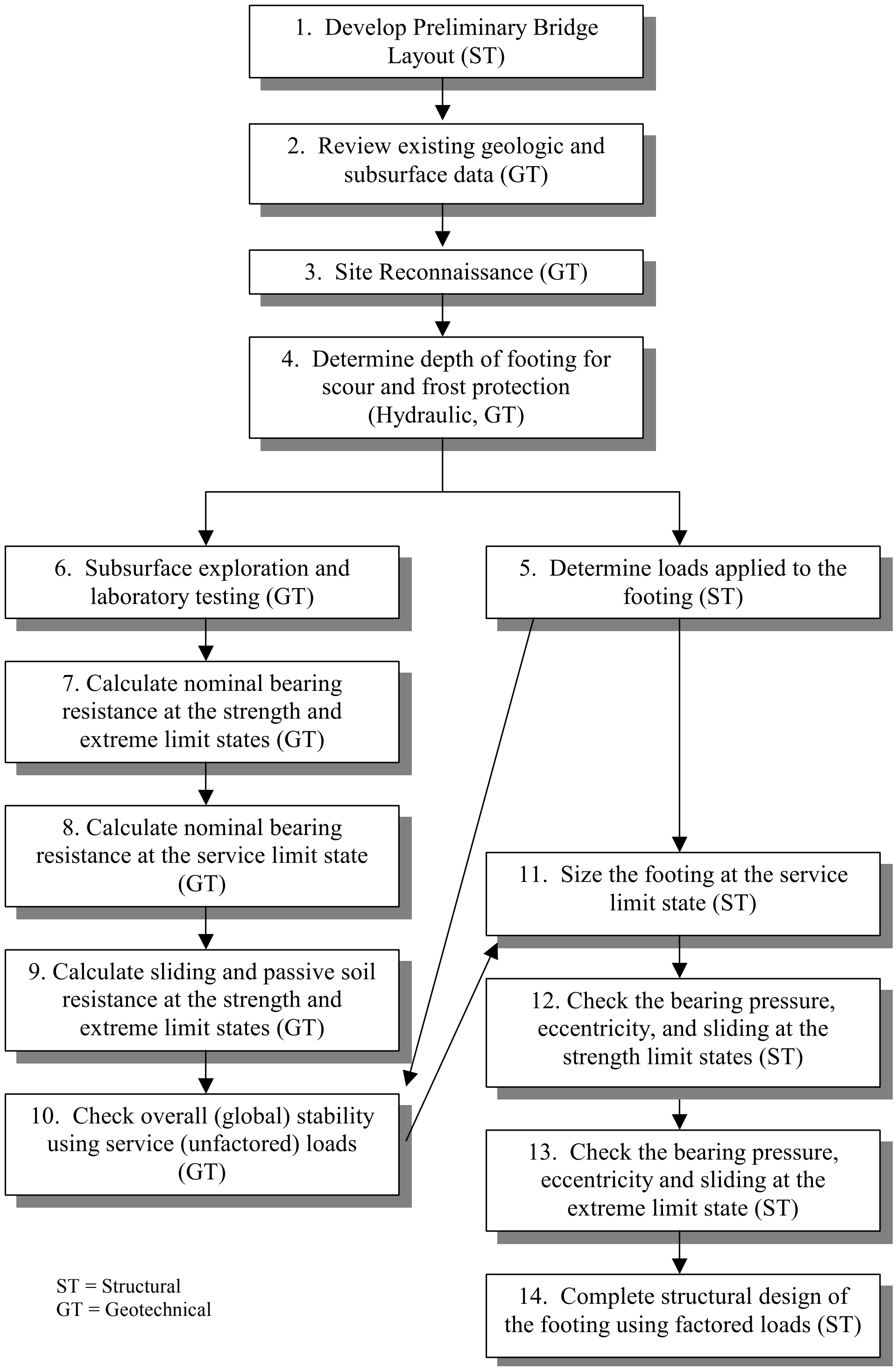


Figure C-1: LRFD Design Process Flow Chart – Bridge Shallow Foundations

LRFD EXAMPLE C1: INTERIOR BRIDGE PIER ON SPREAD FOOTING

(Note: This example uses the same given geometry, loading and subsurface conditions as Example 1 in Appendix B, which was worked using Service Load Design criteria.)

Given:

- Subsurface profile at the interior bridge pier is shown in Figure C1-1.
- There is no scour potential at this site.
- Frost penetration depth is 0.6 m (2 ft).
- Tolerable settlement is 38 mm (1.5 in).
- The direction conventions are shown in Figure C1-2.

Find:

- Size the footing based on Load and Resistance Factor Design at the Service I, Strength I and Extreme I limit states (Table C-2). Assume factor, η, = 1.0.

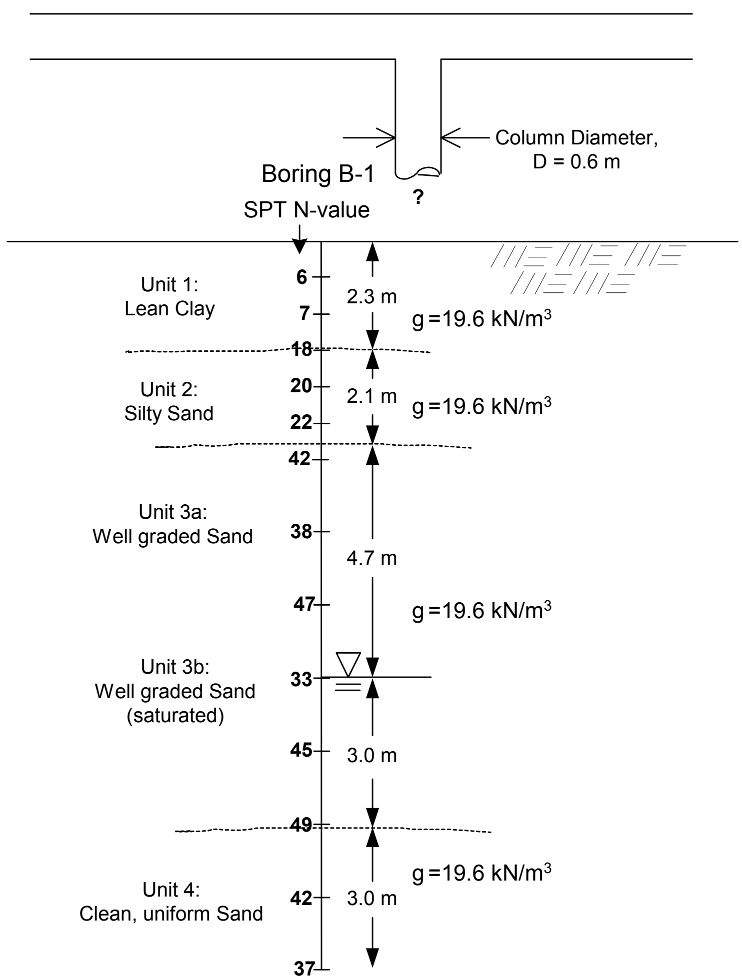


Figure C1-1: Subsurface Profile

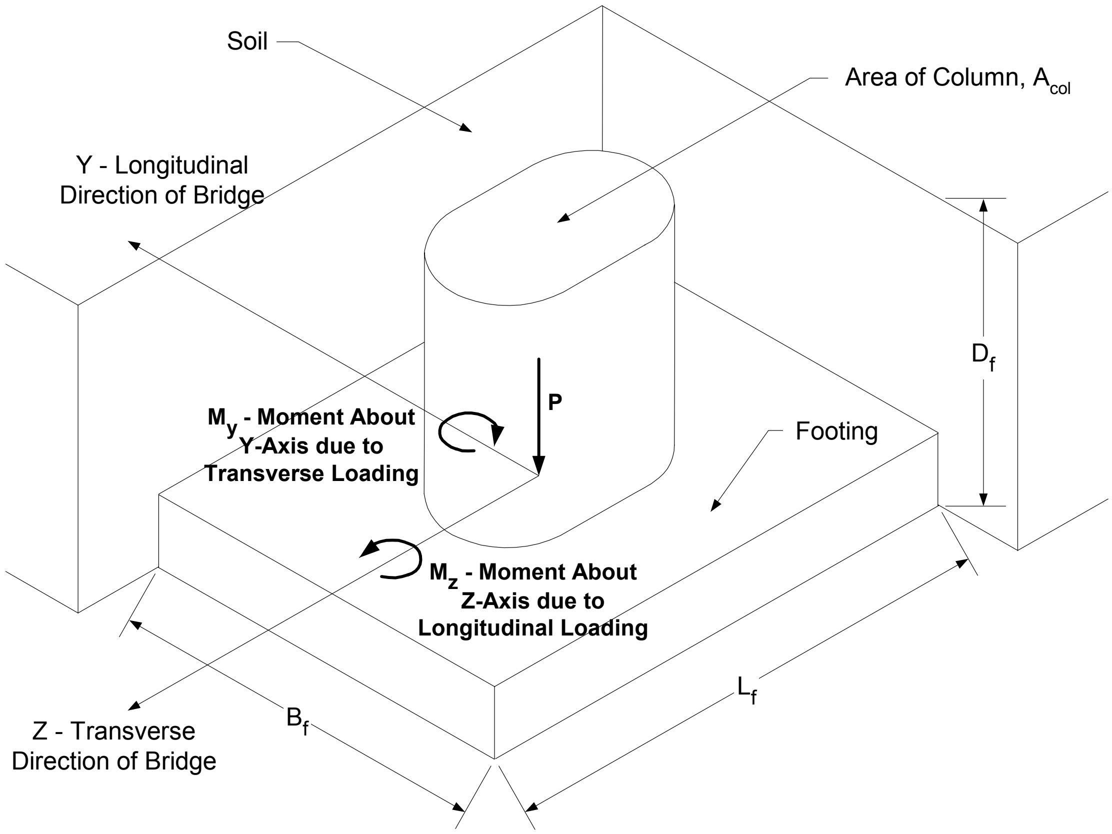


Figure C1-2: Sign Conventions and Loading Directions

<u>Solution:</u>

The solution can be obtained by following the step-by-step process shown in Figure C-1, Design Process Flow Chart.

Steps 1 through 4 – Preliminary bridge layout, review of existing geologic and subsurface data, site reconnaissance, scour and frost potential:

The structural, hydraulics and geotechnical engineers completed these steps. The required design information is provided in the problem given statement.

Step 5 – Determine the loads applied to the footing:

The structural engineer has calculated and provided the values shown in Table C1-1.

TABLE C1-1: LOADS AT COLUMN BASE

Load	Axial, P kN	Shear, V kN	Moment, M_Z kN-m	Moment, M_Y kN-m
Dead load components (DW)	6400	167	211	748
Live load (LL)	1670	42	409	196.5
Vehicular dynamic load allowance (IM)	315	8	77	37
Wind on structure (WS)	884	49	89	226
Wind on live load (WL)	18	4	7	26
Earthquake (EQ)	1671	804	1675	5546

Loads for Service I Limit State:

The general equation for this set of loads and limit state is:

$$\sum \eta_i \gamma_i Q_i = \eta[\gamma_P (DC) + \gamma_{LL} (LL) + \gamma_{IM} (IM) + \gamma_{WS} (WS) + \gamma_{WL} (WL) + \gamma_{EQ} (EQ)]$$

(after Eqn. 3.4.1-1, AASHTO, 1998)

At the Service I limit state, all load factors are 1.0 except for wind on the structure (WS), which is taken as 0.3, and earthquake, which is an extreme limit state load and is not included in service limit state checks (Table 3.4.1-1, AASHTO, 1998). Also note that the AASHTO specifications provide for taking the vehicular dynamic load allowance (impact) as zero for design of substructures. We will also take the factor, η, to be 1.0. Assume the effect of footing self-weight is negligible. So the Service I limit state loads are as follows:

Axial loads:

$$P = \eta[\gamma_P (DC) + \gamma_{LL} (LL) + \gamma_{IM} (IM) + \gamma_{WS} (WS) + \gamma_{WL} (WL) + \gamma_{EQ} (EQ)]$$

$$P = 1.0[1.0 (6400^{kN}) + 1.0 (1670^{kN}) + 1.0 (0^{kN}) + 0.3 (884^{kN}) + 1.0 (18^{kN}) + 0 (1671^{kN})]$$

$$P = 8353^{kN}$$

Shear:

$$V = \eta[\gamma_P (DC) + \gamma_{LL} (LL) + \gamma_{IM} (IM) + \gamma_{WS} (WS) + \gamma_{WL} (WL) + \gamma_{EQ} (EQ)]$$

$$V = 1.0[1.0 (167^{kN}) + 1.0 (42^{kN}) + 1.0 (0^{kN}) + 0.3 (49^{kN}) + 1.0 (4^{kN}) + 0 (804^{kN})]$$

$$V = 228^{kN}$$

Moment in the Z direction:

$$M_Z = \eta[\gamma_P\,(DC) + \gamma_{LL}\,(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}\,(WL) + \gamma_{EQ}(EQ)]$$

$$M_Z = 1.0[1.0\,(211^{kN-m}) + 1.0\,(409^{kN-m}) + 1.0(0^{kN-m}) + 0.3(89^{kN-m})$$
$$+1.0\,(7^{kN-m}) + 0(1675^{kN-m})]$$

$$M_Z = 654^{kN-m}$$

Moment in the Y direction:

$$M_Y = \eta[\gamma_P\,(DC) + \gamma_{LL}\,(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}\,(WL) + \gamma_{EQ}(EQ)]$$

$$M_Y = 1.0[1.0\,(748^{kN-m}) + 1.0\,(196.5^{kN-m}) + 1.0(0^{kN-m}) + 0.3(226^{kN-m})$$
$$+1.0\,(26^{kN-m}) + 0(5546^{kN-m})]$$

$$M_Y = 1038^{kN-m}$$

<u>Loads for Strength I Limit State:</u>

At the strength limit state, all load factors for the dead load components will be taken as minimums or maximums, depending on whether the load acts to stabilize or destabilize the footing. For this set of loads, the following load factors, γ_P, will be applied for the various design checks:

For checking bearing resistance: $\gamma_P = \gamma_{max} = 1.25$

For checking sliding: $\gamma_P = \gamma_{min} = 0.9$

For checking eccentricity: $\gamma_P = \gamma_{min} = 0.9$

Note that the AASHTO specifications provide for taking the vehicular dynamic load allowance (impact) as zero for design of substructures. Also assume that the effect of footing self-weight is negligible. The minimum load cases will conservatively ignore live load. So the Strength I limit state loads are as follows:

Axial loads:

$$P_{max} = \eta[\gamma_P\,(DC) + \gamma_{LL}\,(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}\,(WL) + \gamma_{EQ}(EQ)]$$

$$P_{max} = 1.0[1.25\,(6400^{kN}) + 1.75\,(1670^{kN}) + 1.0(0^{kN}) + 0(884^{kN}) + 0\,(18^{kN})$$
$$+0(1671^{kN})]$$

$$P_{max} = 10923^{kN}$$

$$P_{min} = 1.0[0.9\,(6400^{kN}) + 1.75\,(0^{kN}) + 1.0(0^{kN}) + 0(884^{kN}) + 0\,(18^{kN}) + 0(1671^{kN})]$$
$$P_{min} = 5760^{kN}$$

Shear:

$$V_{max} = \eta[\gamma_P\,(DC) + \gamma_{LL}\,(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}\,(WL) + \gamma_{EQ}(EQ)]$$
$$V_{max} = 1.0[1.25\,(167^{kN}) + 1.75\,(42^{kN}) + 1.0(0^{kN}) + 0(49^{kN}) + 0\,(4^{kN}) + 0(804^{kN})]$$
$$V_{max} = 282^{kN}$$

$$V_{min} = 1.0[0.9\,(167^{kN}) + 1.75\,(0^{kN}) + 1.0(0^{kN}) + 0(49^{kN}) + 0\,(4^{kN}) + 0(804^{kN})]$$
$$V_{min} = 150^{kN}$$

Moment in the Z direction:

$$M_{Z\,max} = \eta[\gamma_P\,(DC) + \gamma_{LL}\,(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}\,(WL) + \gamma_{EQ}(EQ)]$$
$$M_{Z\,max} = 1.0[1.25\,(211^{kN-m}) + 1.75\,(409^{kN-m}) + 1.0(0^{kN-m}) + 0(89^{kN-m}) + 0\,(7^{kN-m}) + 0(1675^{kN-m})]$$
$$M_{Z\,max} = 980^{kN-m}$$

$$M_{Z\,min} = 1.0[0.9\,(211^{kN-m}) + 1.75\,(0^{kN-m}) + 1.0(0^{kN-m}) + 0(89^{kN-m}) + 0\,(7^{kN-m}) + 0(1675^{kN-m})]$$
$$M_{Z\,min} = 190^{kN-m}$$

Moment in the Y direction:

$$M_{Y\,max} = \eta[\gamma_P\,(DC) + \gamma_{LL}\,(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}\,(WL) + \gamma_{EQ}(EQ)]$$
$$M_{Y\,max} = 1.0[1.25\,(748^{kN-m}) + 1.75\,(196.5^{kN-m}) + 1.0(0^{kN-m}) + 0(226^{kN-m}) + 0\,(26^{kN-m}) + 0(5546^{kN-m})]$$
$$M_{Y\,max} = 1279^{kN-m}$$

$$M_{Y\,min} = 1.0[0.9\,(748^{kN-m}) + 1.75\,(0^{kN-m}) + 1.0(0^{kN-m}) + 0(226^{kN-m}) + 0\,(26^{kN-m}) + 0(5546^{kN-m})]$$
$$M_{Y\,min} = 673^{kN-m}$$

Loads for Extreme I Limit State:

At the extreme limit state, all load factors for the dead load components will be taken as maximums, since the dynamic model used maximum factored loads to compute the earthquake forces. The designer should be consistent with respect to application of load factors. A separate check using earthquake loads computed using minimum load factors should also be performed. For the set of loads in this example, the following load factors, γ_P, will be applied for the various design checks:

For checking bearing resistance: $\gamma_P = \gamma_{max} = 1.25$

For checking sliding: $\gamma_P = \gamma_{max} = 1.25$

For checking eccentricity: $\gamma_P = \gamma_{max} = 1.25$

The AASHTO commentary suggests that the load factor for live load at the extreme limit state, γ_{EQ}, should be taken as 0.5. This does not reflect current practice; therefore, γ_{EQ} for live load will be taken as zero.

Note that the AASHTO specifications provide for taking the vehicular dynamic load allowance (impact) as zero for design of substructures. Also assume that the effect of footing self-weight is negligible. So the Extreme I limit state loads are as follows:

Axial loads for evaluating bearing stress (seismic force acting downward):

$$P_{E_{max}} = \eta[\gamma_P\,(DC) + \gamma_{LL}\,(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}\,(WL) + \gamma_{EQ}(EQ)]$$
$$P_{E_{max}} = 1.0[1.25\,(6400^{kN}) + 0\,(1670^{kN}) + 1.0(0^{kN}) + 0(884^{kN}) + 0\,(18^{kN}) + 1.0(1671^{kN})]$$
$$P_{E_{max}} = 9671^{kN}$$

Axial loads for evaluating eccentricity and sliding (seismic force acting upward):

$$P_{E_{min}} = 1.0[1.25\,(6400^{kN}) + 0\,(1670^{kN}) + 1.0(0^{kN}) + 0(884^{kN}) + 0\,(18^{kN}) - 1.0(1671^{kN})]$$
$$P_{E_{min}} = 6329^{kN}$$

Shear:

$$V_E = \eta[\gamma_P\,(DC) + \gamma_{LL}\,(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}\,(WL) + \gamma_{EQ}(EQ)]$$
$$V_E = 1.0[1.25\,(167^{kN}) + 0\,(42^{kN}) + 1.0(0^{kN}) + 0(49^{kN}) + 0\,(4^{kN}) + 1.0(804^{kN})]$$
$$V_E = 1013^{kN}$$

Moment in the Z direction:

$$M_{Z_E} = \eta[\gamma_P\,(DC) + \gamma_{LL}\,(LL) + \gamma_{IM}(IM) + \gamma_{WS}(WS) + \gamma_{WL}\,(WL) + \gamma_{EQ}(EQ)]$$

$$M_{Z_E} = 1.0[1.25\,(211^{kN-m}) + 0\,(409^{kN-m}) + 1.0(0^{kN-m}) + 0(89^{kN-m})$$
$$+0\,(7^{kN-m}) + 1.0(1675^{kN-m})]$$

$$M_{Z_E} = 1939^{kN-m}$$

Moment in the Y direction:

$$M_{Y_E} = 1.0[1.25\,(748^{kN-m}) + 0\,(196.5^{kN-m}) + 1.0(0^{kN-m}) + 0(226^{kN-m})$$
$$+0\,(26^{kN-m}) + 1.0(5546^{kN-m})]$$

$$M_{Y_E} = 6481^{kN-m}$$

Step 6 – Conduct Field Exploration and Laboratory Testing:

This step is complete, and the subsurface data are shown in a drawing at the pier location, Figure C1-1. Because of the granular nature of the soils encountered in the boring, laboratory testing was limited to soil classifications and moisture content. The foundation design will be based largely on design parameters correlated to the soil classifications and the SPT N-values. Compute the initial vertical effective stresses at the midpoint of each layer:

Layer 2:

$$\sigma'_{vo_2} = 3.35\,m(19.6\,\frac{kN}{m^3}) = 65.7\,kPa$$

Layer 3a:

$$\sigma'_{vo_{3a}} = 6.75\,m(19.6\,\frac{kN}{m^3}) = 132\,kPa$$

Layer 3b:

$$\sigma'_{vo_{3b}} = 10.6\,m(19.6\,\frac{kN}{m^3}) - 1.5\,m(9.8\,\frac{kN}{m^3}) = 193\,kPa$$

Layer 4:

$$\sigma'_{vo_4} = 13.6\,m(19.6\,\frac{kN}{m^3}) - 4.5\,m(9.8\,\frac{kN}{m^3}) = 222\,kPa$$

The effective stress diagram for the profile at the pier location is plotted in Figure C1-3.

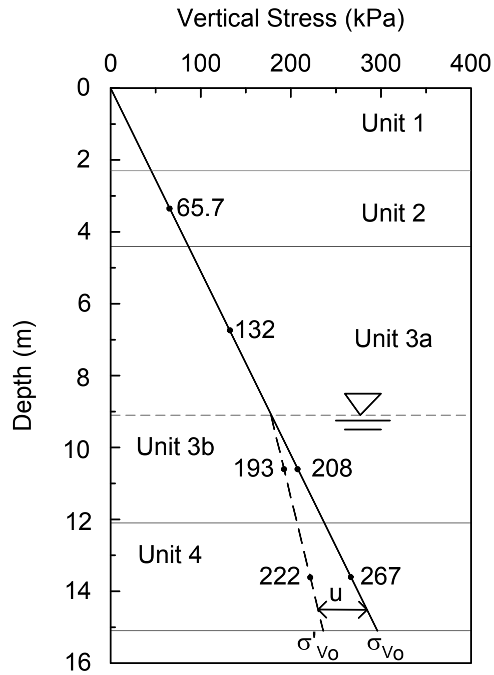


Figure C1-3: Effective Stress Diagram

Step 7 – Calculate nominal bearing resistance at the strength and extreme limit state:

The general equation for bearing capacity is:

$$q_{ult} = cN_c s_c b_c + qN_q C_{W_q} s_q b_q d_q + 0.5\gamma B_f N_\gamma C_{W_\gamma} s_\gamma b_\gamma \qquad \text{(Eqn. 5-14)}$$

The first term goes to zero since the foundation materials are granular and noncohesive, so the cohesion term, c, is zero.

For the interior pier, we will assume that the footing is essentially rectangular (L/B < 5). Recall that the shape and inclination factors should not be applied together (see discussion in Section 5.2). Since the effect of the shape factors is significant for rectangular footings, and considering the inclination resulting from the relatively small shear load component (V), we will only apply the shape factors as follows:

$$s_\gamma = 1 - 0.4\frac{B_f}{L_f} \qquad \text{(Table 5-2)}$$

$s_\gamma = 0.6$ Assume the effect of eccentricity is small for now, so $B_f \cong L_f$

A design friction angle associated with Layer 2 is needed to calculate s_q. The geotechnical engineer has recommended that the footing be cast against the firm, undisturbed soil of Unit 2. Unit 2 is 2.1 m thick and overlies denser material at depth, so the properties of Unit 2 will govern the footing design. From Figure C1-1, the average uncorrected SPT N-value for this layer is 20. From Figure C1-3, the average vertical effective stress in Layer 2 is 65.7 kPa (about 1.4 ksf). Entering Figure 4-1 with a blowcount of 20 and a vertical effective stress of 1.4 ksf, a relative density, D_R, of about 75 percent is read. Entering Figure 4-2 with this relative density for silty sand (Unified Soil Classification 'SM'), a friction angle of 35 degrees is selected for computing bearing capacity.

$$s_q = 1 + \frac{B_f}{L_f}\tan\phi \qquad \text{(Table 5-2)}$$

$$s_q = 1 + \frac{1}{1}\tan(35°)$$

$$s_q = 1.7$$

Since this footing will be level, there is no effect of inclination of the base, so:

$$b_\gamma = b_q = 1.0 \qquad \text{(Table 5-5)}$$

Since the overburden materials above the footing are cohesive soils, the correction factor to account for shearing resistance of the material above the bearing elevation, d_q, should be set equal to 1.0:

$$d_q = 1.0 \qquad \text{(Table 5-4)}$$

Calculate the effect of the footing embedment, q. Apply judgment; recognize that minimal excavation is necessary to remove all of the medium stiff, lean clay; thus avoiding any low strength or compressibility characteristics associated with this material. Therefore, select D_f = 2.3 m to bear below Unit 1:

In this case, q is only a function of the depth of bearing.

$$q = \sigma'_{v_o} = \gamma' D_f \qquad \text{where:} \quad \gamma' = \gamma_{bulk} = 19.6\frac{kN}{m^3}$$

$$q = (19.6\frac{kN}{m^3})(2.3\,m)$$

$$q = 45.1\ kPa$$

Check ground water effect:

Depth of ground water table below the bearing depth:

D_f = Depth of footing = 2.3 m — Assumes a 0.9-m-thick footing and 1.4-m cover

D_w = Depth of ground water below the ground surface (D_f) = 9.1 m

Conservatively estimate the footing width with an upper bound value of $B_f = 6$ m.

Calculate ground water correction factors, $C_{W\gamma}$ and C_{Wq}:

$$C_{W_\gamma} = 0.5 + 0.5\left(\frac{D_W}{1.5B_f + D_f}\right) \leq 1.0 \qquad \text{(Eqn. 5-9)}$$

$$C_{W_\gamma} = 0.5 + 0.5\left(\frac{9.1}{1.5(6) + 2.3}\right) = 0.9$$

$$C_{W_q} = 0.5 + 0.5\left(\frac{D_W}{D_f}\right) \leq 1.0 \qquad \text{(Eqn. 5-10)}$$

$$C_{W_q} = 0.5 + 0.5\left(\frac{6.8}{2.3}\right) = 2.0 \;;\; \text{Use } C_{Wq} = 1.0$$

From Table 5-1, the bearing capacity factors, N_γ and N_q, are 48.0 and 33.3, respectively, for a friction angle of 35 degrees in Layer 2.

So the equation for nominal bearing resistance at the strength and extreme limit states is:

$$q_{ult} = 0.5\gamma B_f N_\gamma C_{W_\gamma} s_\gamma b_\gamma + qN_q C_{W_q} s_q b_q d_q$$
$$q_{ult} = 0.5\,(19.6\,\tfrac{kN}{m^3})(B_f)(48.0)(0.9)(0.6)(1.0) + 45.1\,kPa\,(33.3)(1.0)(1.7)(1.0)(1.0)$$
$$q_{ult} = 254B_f + 2553\,kPa$$

Since the nominal bearing resistance was computed using a soil friction angle correlated to SPT N-values, a resistance factor of $\varphi = 0.35$ should be applied to resistances computed using this equation when checking strength limit states per Table C-5 (from AASHTO (1998) Table 10.5.5-1).

Step 8 – Calculate the nominal bearing resistance at the service limit state:

Calculate stress increase at the midpoint of each soil layer using the 2-on-1 stress distribution method for various footing widths; say $B_f = 3$, 4.6 and 6.1 m, as a function of the stress applied by the footing, q:

$$\frac{\Delta\sigma_V}{q} = B_f \cdot L_f /(B_f + Z)(L_f + Z) \qquad \text{(Figure 5-12)}$$

but $B_f \cong L_f$, so:

$$\frac{\Delta\sigma_V}{q} = B_f^{\,2} /(B_f + Z)^2$$

Layer 2 with midpoint of layer @ Z = 1.05 m below base of footing:

$$\frac{\Delta\sigma_{V2}}{q} = 3^2/(3+1.05)^2 = 0.55$$

Layer 3a with midpoint of layer @ Z = 4.45 m below base of footing:

$$\frac{\Delta\sigma_{V3a}}{q} = 3^2/(3+4.45)^2 = 0.16$$

Layer 3b with midpoint of layer @ Z = 8.3 m below base of footing:

$$\frac{\Delta\sigma_{V3b}}{q} = 3^2/(3+8.3)^2 = 0.07$$

Layer 4 with midpoint of layer @ Z = 11.3 m below base of footing:

$$\frac{\Delta\sigma_{V4}}{q} = 3^2/(3+11.3)^2 = 0.04$$

Perform similar calculations for B_f = 4.6 m and B_f = 6.1 m and tabulate:

TABLE C1-2: STRESS INCREASE WITH DEPTH AS FUNCTION OF FOOTING WIDTH AND STRESS APPLIED BY THE FOOTING (q)

Soil Layer	Depth Range (m)	Stress Increase, $\Delta\sigma_v$		
		B_f = 3 m	B_f = 4.6 m	B_f = 6.1 m
2	2.3 – 4.4	0.55q	0.66q	0.73q
3a	4.4 – 9.1	0.16q	0.26q	0.33q
3b	9.1 – 12.1	0.07q	0.13q	0.18q
4	12.1 – 15.1	0.04q	0.08q	0.12q

For these footing widths, determine the stress that would need to be applied by footing in order to generate 38 mm of settlement using the Hough method. These stresses will be plotted as a function of footing width to generate the nominal bearing resistance at the service limit state for a deformation (settlement) criterion of 38 mm.

General equation:

$$\Delta H = H(1/C')\log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}] \qquad \text{(Eqn. 5-24)}$$

Correct SPT blowcounts to account for overburden pressure, N′, for each layer, and enter Figure 5-19 to obtain bearing capacity factor, C′:

Layer 2:

$N_{avg} = 20$
$N'/N = 1.2$ (from Figure 5-18)
$N' = 24$
$C' = 65$ (from Figure 5-19 for silty Sand)

Layer 3a:

$N_{avg} = 40$
$N'/N = 0.9$ (from Figure 5-18)
$N' = 36$
$C' = 120$ (from Figure 5-19 for well-graded Sand)

Layer 3b:

$N_{avg} = 43$
$N'/N = 0.7$ (from Figure 5-18)
$N' = 30$
$C' = 102$ (from Figure 5-19 for well-graded Sand)

Layer 4:

$N_{avg} = 40$
$N'/N = 0.66$ (from Figure 5-18)
$N' = 26$
$C' =110$ (from Figure 5-19 for clean, uniform Sand)

Calculate settlement in each layer and sum the settlements to find the applied stress required to yield a total settlement of 38 mm at a footing width of 3 m. This is an iterative process.

Layer 2:

$$\Delta H_2 = H_2(1/C')\log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}]$$
$$\Delta H_2 = 2.1\,\text{m}(1/65)\log[(65.7\,\text{kPa} + 0.55(600\,\text{kPa}))/65.7\,\text{kPa})]$$
$$\Delta H_2 = 0.025\,\text{m} = 25\,\text{mm}$$

Layer 3a:

$$\Delta H_{3a} = H_{3a}(1/C')\log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}]$$
$$\Delta H_{3a} = 4.7\,\text{m}(1/120)\log[(132\,\text{kPa} + 0.16(600\,\text{kPa}))/132\,\text{kPa})]$$
$$\Delta H_{3a} = 0.009\,\text{m} = 9\,\text{mm}$$

Layer 3b:

$$\Delta H_{3b} = H_3(1/C')\log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}]$$
$$\Delta H_{3b} = 3.0\,\text{m}(1/102)\log[(193\,\text{kPa} + 0.07(600\,\text{kPa}))/193\,\text{kPa})]$$
$$\Delta H_{3b} = 0.003\,\text{m} = 3\,\text{mm}$$

Layer 4:

$$\Delta H_4 = H_4 (1/C') \log[(\sigma'_{v_o} + \Delta\sigma_v)/\sigma'_{v_o}]$$
$$\Delta H_4 = 3\,\mathrm{m}(1/110)\log[(222\,\mathrm{kPa} + 0.04(600\,\mathrm{kPa}))/222\,\mathrm{kPa})]$$
$$\Delta H_4 = 0.001\,\mathrm{m} = 1\,\mathrm{mm}$$

$$\sum \Delta H_i = 25\,\mathrm{mm} + 9\,\mathrm{mm} + 3\,\mathrm{mm} + 1\,\mathrm{mm} = 38\,\mathrm{mm}$$

Repeat this process for several footing widths over the expected range of footing dimensions – say, 1.5 to 7 m (a spreadsheet or computer tool can make these iterative calculations rapidly). Plot the results as a function of effective footing width as shown in Figure C1-4.

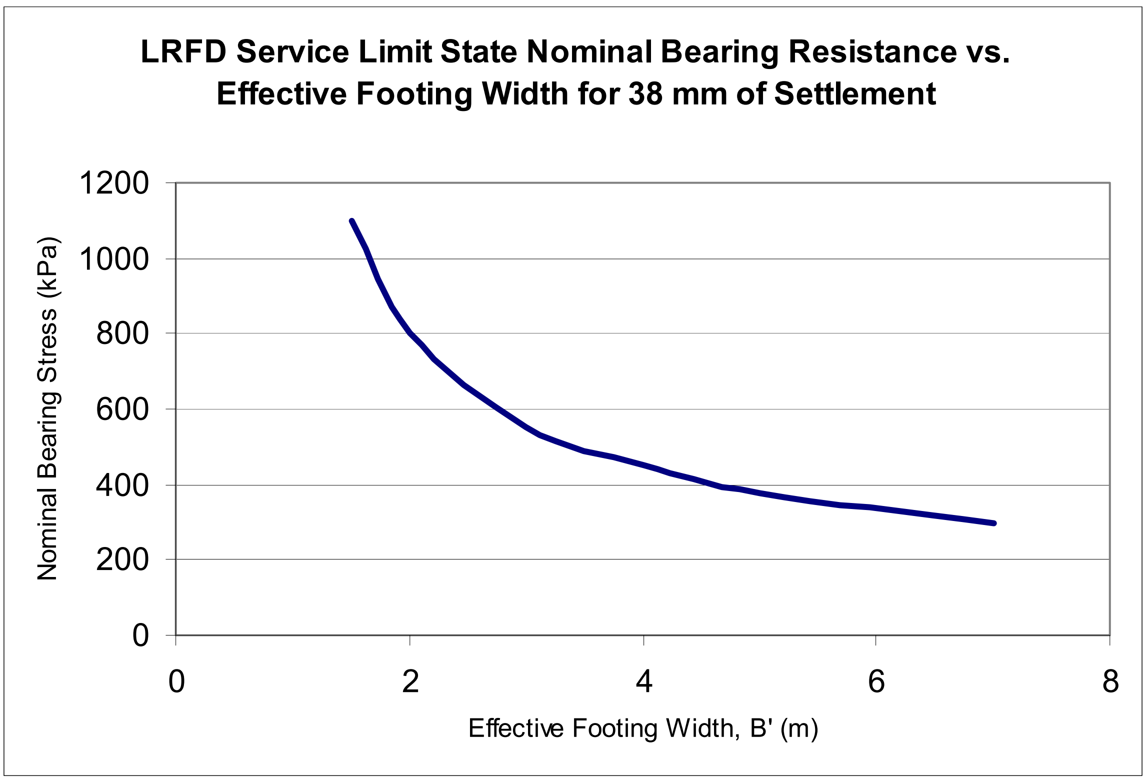


Figure C1-4: Nominal Bearing Resistance vs. Effective Footing Width at Service-I Limit State

The geotechnical engineer should provide this chart and the equation for computing nominal resistances at the strength limit state, with the corresponding resistance factor (Step 7), to the structural designer for sizing of the footing. Note that the nominal resistance shown in Figure C1-4 is only applicable for service limit state design checks because it was developed with a serviceability limitation (e.g., maximum permissible settlement of 38 mm). The resistance factor associated with nominal resistances obtained from Figure C1-4 is 1.0.

Step 9 – Calculate nominal sliding and passive soil resistance at the strength and extreme limit state:

The footing will bear in the Unit 2 soils. The friction angle of 35 degrees selected in Step 7 will be used to compute sliding resistance.

The equation for calculating sliding resistance of concrete footings cast against the soil is:

$$Q_T = P_v \tan\phi \qquad \text{(Eqn. C-4)}$$

The ultimate, or nominal, sliding resistance is:

$$Q_T = P_v \tan 35° = 0.7(P_v)$$

The resistance factor associated with strength limit state checks is $\varphi = 0.8$ (Table 10.5.5-1, AASHTO, 1998).

Since the sliding resistance will develop at strain levels much lower than that needed to develop passive pressures against the side of the footing, passive pressure will be ignored unless it is found that sliding resistance governs the design of the footing. The geotechnical engineer should provide this equation and the assumptions regarding use of passive pressures to the structural engineer for checking the footing's sliding resistance at the strength and extreme limit states.

Step 10 – Check global stability of the footing:

The geotechnical engineer has determined that since this is an interior pier with level ground conditions, global stability is not a concern.

Step 11 – Size the footing at the service limit state:

First determine the eccentricity of the loads applied to the footing in both directions. This will be used to find the effective footing dimensions, B'_f and L'_f.

$$e_y = M_Z \div P$$

$$e_y = 654^{kN-m} \div 8353^{kN} = 0.078\,m$$

$$e_z = M_Y \div P$$

$$e_z = 1038^{kN-m} \div 8353^{k} = 0.124\,m$$

Estimate trial footing dimensions. Try 5-m-by-5-m footing and calculate effective footing dimensions. The smaller will be used to enter Figure C1-4 to determine the nominal bearing resistance at the service limit state.

$$B'_f = B_f - 2e_y \qquad \text{(Eqn. 6-6)}$$

$$B'_f = 5\,m - 2(0.078\,m) = 4.844\,m$$

$$L'_f = L_f - 2e_z$$
$$L'_f = 5\,m - 2(0.124\,m) = 4.752\,m \qquad \text{(Eqn. 6-7)}$$

Since L'_f is less than B'_f, use L'_f = 4.752 m to check nominal bearing resistance at the service limit state. From Figure C1-4, the nominal bearing resistance for this effective footing dimension is about 390 kPa. Compute applied bearing stress over effective footing area, A′:

$$A' = B'_f \cdot L'_f = (4.844\,m)(4.752\,m) = 23.0\,m^2$$

$$q_{applied} = P \div A' = 8353^{kN} \div 23.0\,m^2 = 363\,kPa$$

This is less than 390 kPa from Figure C1-4 for settlement less than 38 mm. Use 5-m-by-5-m footing to perform the rest of the design checks.

Check assumption to neglect self-weight of footing, assuming a footing thickness of 1 m:

$$W_{ftg} = V_{ftg}(\gamma_{concrete} - \gamma_{soil})$$
$$W_{ftg} = (5\,m)(5\,m)(1\,m)(23.5^{kN/m^3} - 19.6^{kN/m^3}) = 98^{kN}$$

This should be less than about 1 to 2 percent of the axial load applied to the footing, or:

$$2\%P = 0.02(8353^{kN}) = 167^{kN} \qquad \text{(OK)}$$

Step 12 – Check the bearing pressure, maximum eccentricity and sliding at the strength limit state:

Bearing Pressure:

Compute effective footing dimensions and bearing area using maximum load factors applied to the dead load components:

$$e_y = M_{Z\,max} \div P_{max}$$
$$e_y = 980^{kN-m} \div 10923^{kN} = 0.089\,m$$

$$e_z = M_{Y\,max} \div P_{max}$$
$$e_z = 1279^{kN-m} \div 10923^{k} = 0.117\,m$$

$$B'_f = B_f - 2e_y$$
$$B'_f = 5\,m - 2(0.089\,m) = 4.82\,m \qquad \text{(Eqn. 6-6)}$$

$$L'_f = L_f - 2e_z$$
$$L'_f = 5\,m - 2(0.117\,m) = 4.766\,m \qquad \text{(Eqn. 6-7)}$$

$$A' = B'_f \cdot L'_f = (4.82\,m)(4.766\,m) = 23.0\,m^2$$

$$q_{applied} = P \div A' = 10923^{kN} \div 23.0\,m^2 = 475\,kPa$$

Nominal bearing resistance at this effective footing width (use L'_f as minimum value):

$$q_{ult} = 254B'_f + 2553\,kPa$$

$$q_{ult} = 254(4.766\,m) + 2553\,kPa = 3764\,kPa$$

The factored resistance at the strength limit state is then:

$$Q_R = \varphi(q_{ult}) = 0.35(3764\,kPa)$$
$$Q_R = 1317\,kPa$$

This is greater than $q_{applied} = 475$ kPa (OK)

Maximum eccentricity:

The maximum eccentricity in any direction should be less than one-fourth of the actual footing dimension (AASHTO, 1998). The maximum eccentricity is evaluated using the minimum load factor combination:

$$e_y = M_{Z\,min} \div P_{min}$$

$$e_y = 190^{kN-m} \div 5760^{kN} = 0.033\,m$$

$$e_z = M_{Y\,min} \div P_{min}$$

$$e_z = 673^{kN-m} \div 5760^{k} = 0.117\,m$$

$$\frac{1}{4}B_f = \frac{1}{4}L_f = \frac{1}{4}(5\,m) = 1.25\,m$$

$$e_y < e_z < 1.25\,m \qquad \text{(OK)}$$

Sliding:

The factored horizontal shear force must be less than the factored nominal sliding resistance. This is evaluated using minimum load factors. The equation for Q_T is provided in Step 9. Passive resistance is ignored, so Q_{ep} is zero.

$$Q_R = \varphi_T Q_T + \varphi_{ep} Q_{ep} \qquad \text{(Eqn. C-3)}$$

$$Q_R = 0.8(0.7(5760^{kN})) = 3226^{kN}$$

$V_{min} = 224^{kN} < 3226^{kN}$ (OK)

Step 13 – Check the bearing pressure, maximum eccentricity and sliding at the extreme limit state:

Bearing Pressure:

Compute effective footing dimensions and bearing area using maximum load factors applied to the dead load components:

$$e_y = M_{Z_E} \div P_{E_{max}}$$
$$e_y = 1939^{kN-m} \div 9671^{kN} = 0.200\,m$$

$$e_z = M_{Y_E} \div P_{E_{max}}$$
$$e_z = 6481^{kN-m} \div 9671^{k} = 0.670\,m$$

$$B'_f = B_f - 2e_y$$
$$B'_f = 5\,m - 2(0.200\,m) = 4.60\,m \qquad \text{(Eqn. 6-6)}$$

$$L'_f = L_f - 2e_z$$
$$L'_f = 5\,m - 2(0.670\,m) = 3.66\,m \qquad \text{(Eqn. 6-7)}$$

$$A' = B'_f \cdot L'_f = (4.60\,m)(3.66\,m) = 16.8\,m^2$$

$$q_{applied} = P_{E_{max}} \div A' = 9671^{kN} \div 16.8\,m^2 = 576\,kPa$$

Nominal bearing resistance at this effective footing width (use either B'_f or L'_f as minimum value):

$$q_{ult} = 254B'_f + 2553\,kPa$$

$$q_{ult} = 254(3.66\,m) + 2553\,kPa = 3483\,kPa$$

The factored resistance at the extreme limit state is then:

$$Q_R = \varphi(q_{ult}) = 1.0(3483\,kPa)$$
$$Q_R = 3483\,kPa$$

This is greater than $q_{applied}$ = 576 kPa (OK)

Maximum eccentricity:

The maximum eccentricity in any direction should be less than one-fourth of the actual footing dimension. The maximum eccentricity is evaluated using the minimum load factor combination:

$$e_y = M_{Z_E} \div P_{E_{min}}$$

$$e_y = 1939^{kN-m} \div 6329^{kN} = 0.306\,m$$

$$e_z = M_{Y_E} \div P_{E_{min}}$$

$$e_z = 6481^{kN-m} \div 6329^{k} = 1.024\,m$$

$$\frac{1}{4}B_f = \frac{1}{4}L_f = \frac{1}{4}(5\,m) = 1.25\,m$$

$$e_y < e_z < 1.25\,m \qquad \text{(OK)}$$

Sliding:

The factored horizontal shear force must be less than the factored nominal sliding resistance. This is evaluated using minimum load factors:

$$Q_R = \varphi_T Q_T + \varphi_{ep} Q_{ep} \qquad \text{(Eqn. C-3)}$$

$$Q_R = 1.0(0.7(6329^{kN})) = 4430^{kN}$$

$$V_E = 1013^{kN} < 4430^{kN} \qquad \text{(OK)}$$

Step 14 – Complete structural design of footing:

The structural engineer completes this step.

LRFD EXAMPLE 2: STUB SEAT-TYPE BRIDGE ABUTMENT ON STRUCTURAL FILL

(Note: This example uses the same given geometry, loading and subsurface conditions as Example 3 in Appendix B, which was solved using Service Load Design criteria.)

Given:

- A seat-type abutment on a spread footing is proposed for a bridge.
- The footing length is L = 25.0 m (82 ft).
- The foundation soils and abutment dimensions are shown in Figure C2-1.
- Frost penetration depth is 0.6 m (2 ft).
- Tolerable settlement is 38 mm (1.5 in).
- Abutment loading symbols are shown in Figure C2-2.

Find:

- Size the footing based on Load and Resistance Factor Design at the Service I and Strength I states. Assume that the importance factor, η, = 1.0. Check for overturning and sliding.

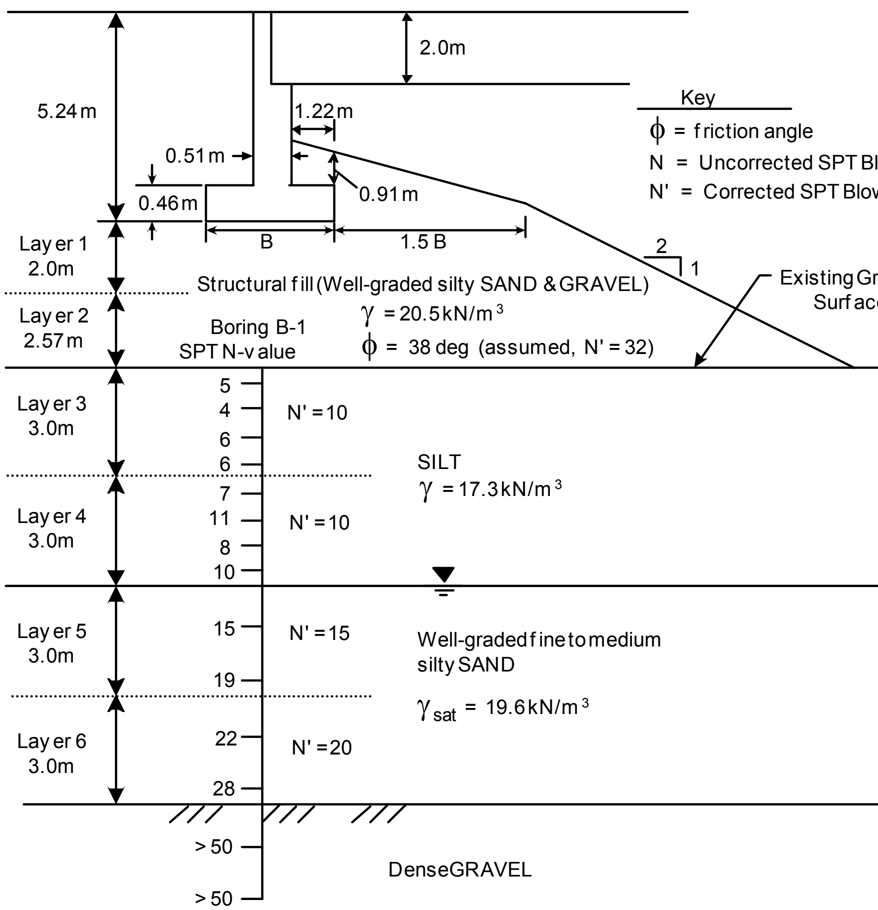


Figure C2-1: Approach Embankment and Foundation

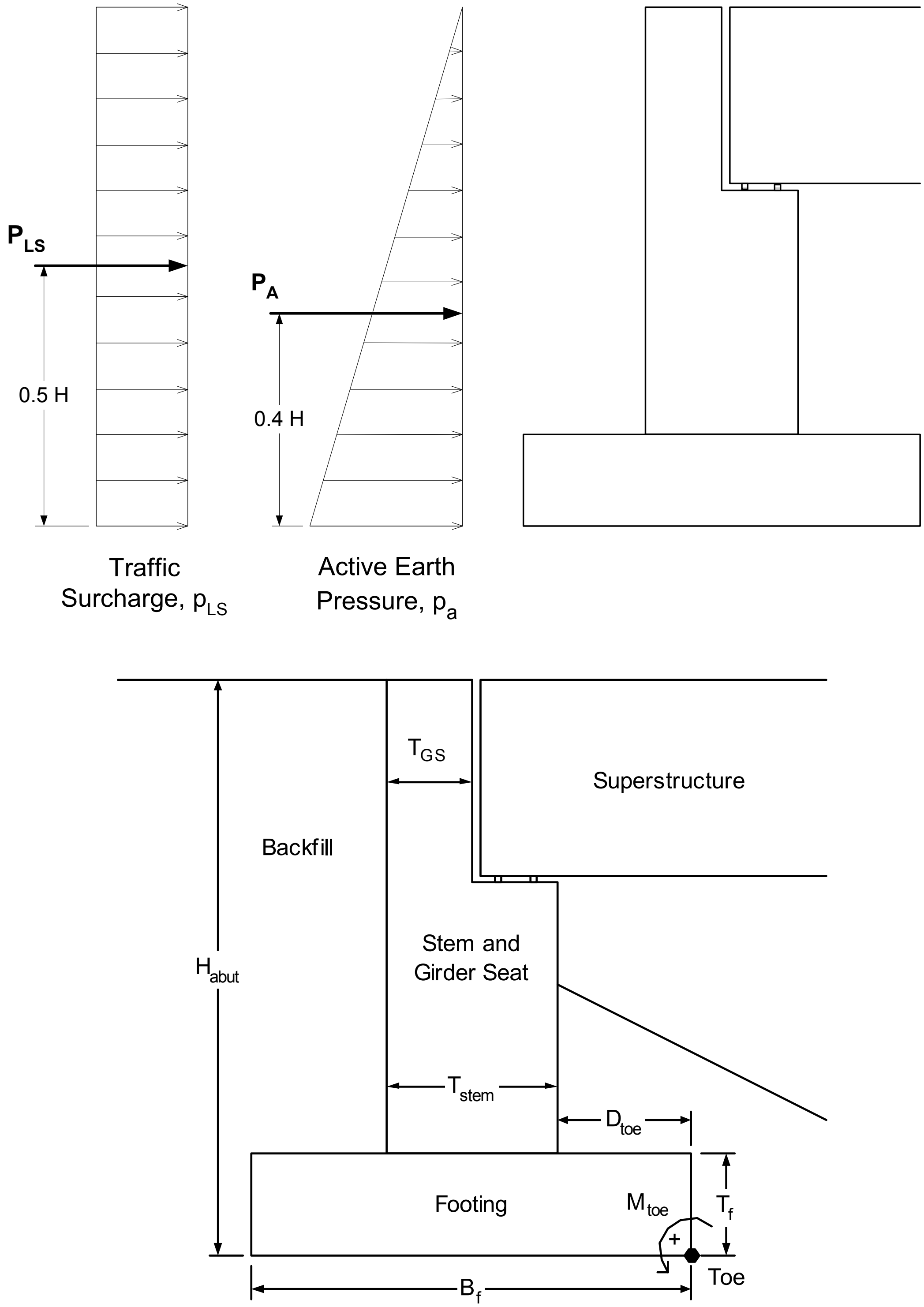


Figure C2-2: Direction and Load Conventions per LRFD Methodology

Solution:

Steps 1 through 4 – Preliminary bridge layout, review of existing geologic and subsurface data, site reconnaissance, scour and frost potential:

The structural, hydraulic and geotechnical engineers completed these steps. The required design information is provided in the problem-given statement.

Step 5 – Determine the loads:

Moments are taken about the toe of the footing. The sense of the moment is positive if counterclockwise (resisting moments) and negative if clockwise (overturning moments).

Loads from the girders:

The structural engineer has calculated and provided the values shown in Table C2-1.

TABLE C2-1: LOADS FROM GIRDERS

Load	Axial, P (kN)	Shear, V (kN)	Moment, M_Z (kN•m)
Dead load of components, P (DC)	5233	-	7274
Dead load of wearing surfaces, P (DW)	446	-	620
Vehicular live load, P (LL)	1538	-	2138
Shear loads from bearing pads, V (TU+CR+SH)	-	1047	-3392

The shear load is computed as the force transmitted through the bearings due to rib shortening, shrinkage and temperature and is considered in both Strength I and Service I states. The moments calculated are due to the vertical and shear loads and are taken about the toe of the footing.

Dead loads of the abutment components (per m of abutment wall):

Dead loads of the abutment components, and the moments caused by these loads about the toe of the footing, were calculated as part of Service Load Design Example 3 (Appendix B). These values are valid for the unfactored loads of the LRFD design and are listed in Table C2-2.

Loads from approach fill:

Loads from the approach fill, and the moments caused by these loads about the toe of the footing, were calculated as part of Service Load Design Example 3 (Appendix B). These values remain valid for the LRFD solution, with the exception of the horizontal earth pressure for the live load surcharge, PLS, and the moment due to active lateral earth pressure (M_{toe} due to EH).

Horizontal earth pressure from live load surcharge:

From Table C-4, h_{eq} = 0.7 m for a wall height of 5.24 m, and:

$$
\begin{aligned}
P_{LS} &= (K_a \gamma h_{eq}) H_{abut} \\
&= (0.26)(19.6\,\text{kN/m}^3)(0.70\,\text{m})(5.24\,\text{m}) \\
&= 18.69\,\text{kN/m}
\end{aligned}
$$

$$
\begin{aligned}
arm_{LS} &= (0.5)H_{abut} \\
&= (0.5)(5.24\,\text{m}) \\
&= 2.62\,\text{m}
\end{aligned}
$$

$$
\begin{aligned}
M_z &= -P_{LS} \cdot arm_{LS} \\
&= -(18.69\,\text{kN/m})(2.62\,\text{m}) \\
&= -48.97\,\text{kN} \cdot \text{m/m}
\end{aligned}
$$

Moment due to active lateral earth pressure:

The code (AASHTO, 1998) specifies application of the resultant due to lateral earth pressure at 0.4 times the height of the abutment, H_{abut}, due to the effects of backfill compaction. Therefore:

$$
\begin{aligned}
M_{toe} &= 0.4(P_A)(H_{abut}) \\
&= 0.4(-69.96\,\text{kN/m})(5.24\ \text{m}) \\
&= -146.6\,\text{kN} \cdot \text{m/m}
\end{aligned}
$$

The loads and moments computed above are summarized in Table C2-2, along with values that are unchanged from Service Load Design Example 3 (Appendix B).

TABLE C2-2: LOADS OF ABUTMENT COMPONENTS AND HORIZONTAL EARTH PRESSURES

Load	Vertical (kN/m)	Horizontal (kN/m)	Moment, M_{toe} (kN•m/m)
Weight of stem, W_{stem} (DC)	57.29	-	84.79
Weight of footing, W_f (DC)	$10.81B_f$	-	$5.41B_f^2$
Weight of soil over toe, W_{toe} (EV)	-	-	-
Weight of soil over heel, W_h (EV)	$(93.69)(B_f-1.73)$	-	$(93.69)(B_f-1.73)$ $*(0.865+0.5B_f)$
Active earth pressure load, P_A (EH)	-	69.96	-146.6
Horizontal earth load from live load surcharge, P_{LS} (LS)	-	18.69	-48.97

Note: B_f = width of footing in meters

Loads for Service I Limit State:

The general equation for this set of loads and limit states is:

$$\sum \eta_i \gamma_i Q_i = \eta[\gamma_P (DC) + \gamma_P (DW) + \gamma_P (EV) + \gamma_P (EH) + \gamma_{LL} (LL) + \gamma_{LL} (LS) + \gamma_{TU} (TU) + \gamma_{CR} (CR) + \gamma_{SH} (SH)]$$

At the service limit state, all load factors except those for uniform temperature, creep and shrinkage are 1.0. At the service limit state, the load factors γ_{TU}, γ_{CR} and γ_{SH} for force effects are all equal to 1.0.

Loads for Strength I Limit State:

At the strength limit state, all load factors for the dead load components will be taken as minimums or maximums, depending on whether the load acts to stabilize or destabilize the footing. At the strength limit state, the load factors γ_{TU}, γ_{CR} and γ_{SH} for force effects are all equal to 0.5.

For this set of loads, the following load factors, γ_P, will be applied for the various design checks:

For checking bearing resistance:

$\gamma_{P\text{ for DC}} = \gamma_{max} = 1.25$
$\gamma_{P\text{ for DW}} = \gamma_{max} = 1.50$
$\gamma_{P\text{ for EV}} = \gamma_{max} = 1.35$
$\gamma_{LL} = 1.75$
$\gamma_{LS} = 1.75$

For checking sliding:

$\gamma_{P\text{ for DC}} = \gamma_{min} = 0.90$
$\gamma_{P\text{ for DW}} = \gamma_{min} = 0$
$\gamma_{P\text{ for EV}} = \gamma_{min} = 1.00$
$\gamma_{P\text{ for EH}} = \gamma_{max} = 1.50$
$\gamma_{LL} = 0$
$\gamma_{LS} = 1.75$
$\gamma_{TU,CR,SH} = \gamma_{max} = 0.5$

For checking eccentricity:

$\gamma_{P\text{ for DC}} = \gamma_{min} = 0.90$
$\gamma_{P\text{ for DW}} = \gamma_{min} = 0$
$\gamma_{P\text{ for EV}} = \gamma_{min} = 1.00$
$\gamma_{P\text{ for EH}} = \gamma_{max} = 1.50$
$\gamma_{LL} = 0$
$\gamma_{LS} = 1.75$
$\gamma_{TU,CR,SH} = \gamma_{max} = 0.5$

Step 6 – Conduct field exploration and laboratory testing:

This step is complete and the subsurface data are shown in Figure C2-1. The initial vertical effective stresses are the same as computed for Service Load Design Example 3 (Appendix B) and are not repeated here.

Step 7 – Calculate nominal bearing resistance at the strength limit state:

The ultimate bearing capacity was calculated for this abutment in Service Load Design Example 3 (Appendix B). The ultimate bearing capacity for effective footing widths of 3, 4, and 5 m were 1414 kPa, 869 kPa, and 605 kPa. Due to the eccentric load effects, it will be useful to plot a few more values for smaller effective footing widths. Using the same methods used in the Service Load Design Example 3 (Appendix B) for effective footing widths of 2 and 2.5 meters:

$$\frac{L_f}{B_f} = \frac{25.0\,\mathrm{m}}{2.0\,\mathrm{m}} = 12.5 > 5$$

$s_\gamma = 1$, $b_\gamma = 1$, and $C_{W\gamma} = 1$

$$D_f / B_f = 1.37\,\mathrm{m} / 2\,\mathrm{m} = 0.685$$

By interpolation, for $D_f / B_f = 0.685$,

$$N_{\gamma q} = 17 + (0.685)(80 - 17) \cong 60$$

And:

$$\begin{aligned} q_{ult} &= 0.5\gamma B_f N_{\gamma q} C_{W\gamma} s_\gamma b_\gamma \\ &= (0.5)(20.5\,\mathrm{kN/m^3})(2.0)(60)(1)(1)(1) \\ &= 1230\,\mathrm{kPa} \end{aligned}$$

Similarly, for 2.5 m:

$$D_f / B_f = 1.37\,\mathrm{m} / 2.5\,\mathrm{m} = 0.548$$

By interpolation, for $D_f / B_f = 0.548$,

$$N_{\gamma q} = 17 + (0.548)(80 - 17) \cong 51.5$$

And:

$$
\begin{aligned}
q_{ult} &= 0.5\gamma B_f N_{\gamma q} C_{W\gamma} s_\gamma b_\gamma \\
&= (0.5)(20.5\,kN/m^3)(2.5)(51.5)(1)(1)(1) \\
&= 1320\,kPa
\end{aligned}
$$

The results are plotted in Figure C2-3. As discussed in Example 3, the pressure bulb for effective footing widths greater than about 3 m begins to extend into the upper foundation silt layer, reducing the bearing capacity, and producing the shape of curve shown in Figure C2-3.

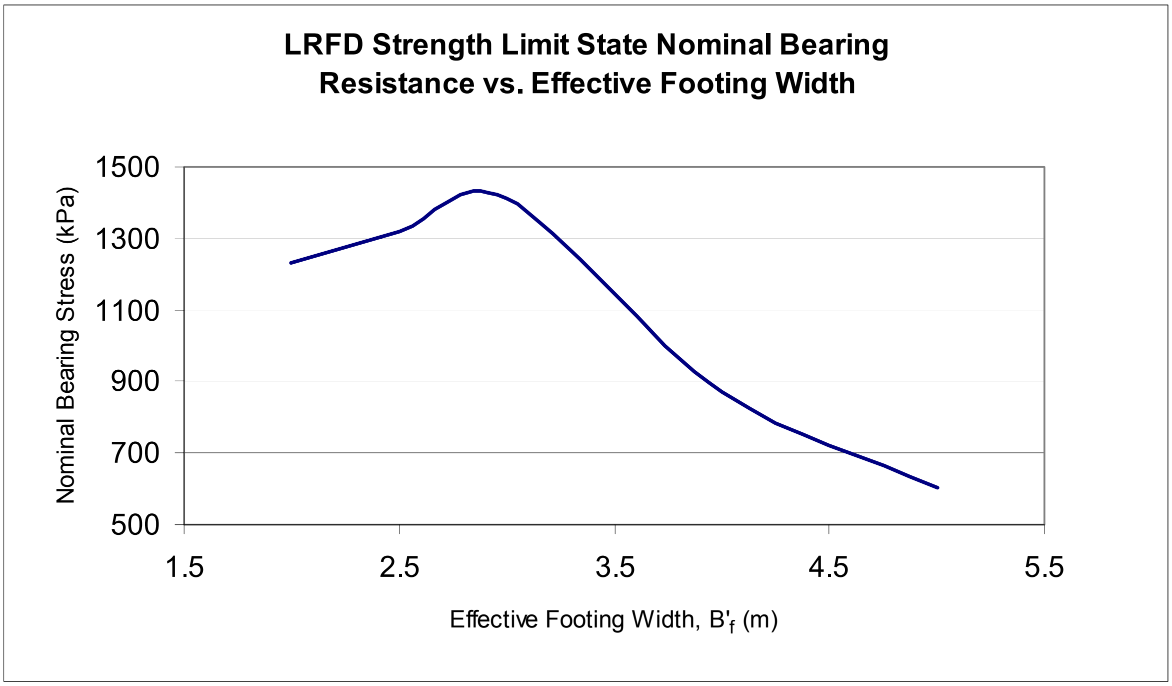


Figure C2-3: Nominal Stress versus Effective Footing Width at Strength 1 Limit State

Since the nominal bearing resistance was computed using a soil friction angle correlated to SPT N-values, a resistance factor of $\varphi = 0.35$ should be applied to resistances from Figure C2-3 when checking strength limit states.

Step 8 – Calculate the nominal bearing resistance at the service limit state:

Settlement estimates were made for effective footing widths of 3 m, 4 m, and 5 m, as part of the Service Load Design Example 3. The estimated settlement for various combinations of footing widths and applied load were used to estimate the bearing pressure resulting in 38 mm of settlement for each footing width. The process was repeated for several footing widths over the expected range of footing dimensions (a spreadsheet or other computer tool can make these calculations rapidly). The results are plotted as a function of effective footing width, as shown in Figure C2-4.

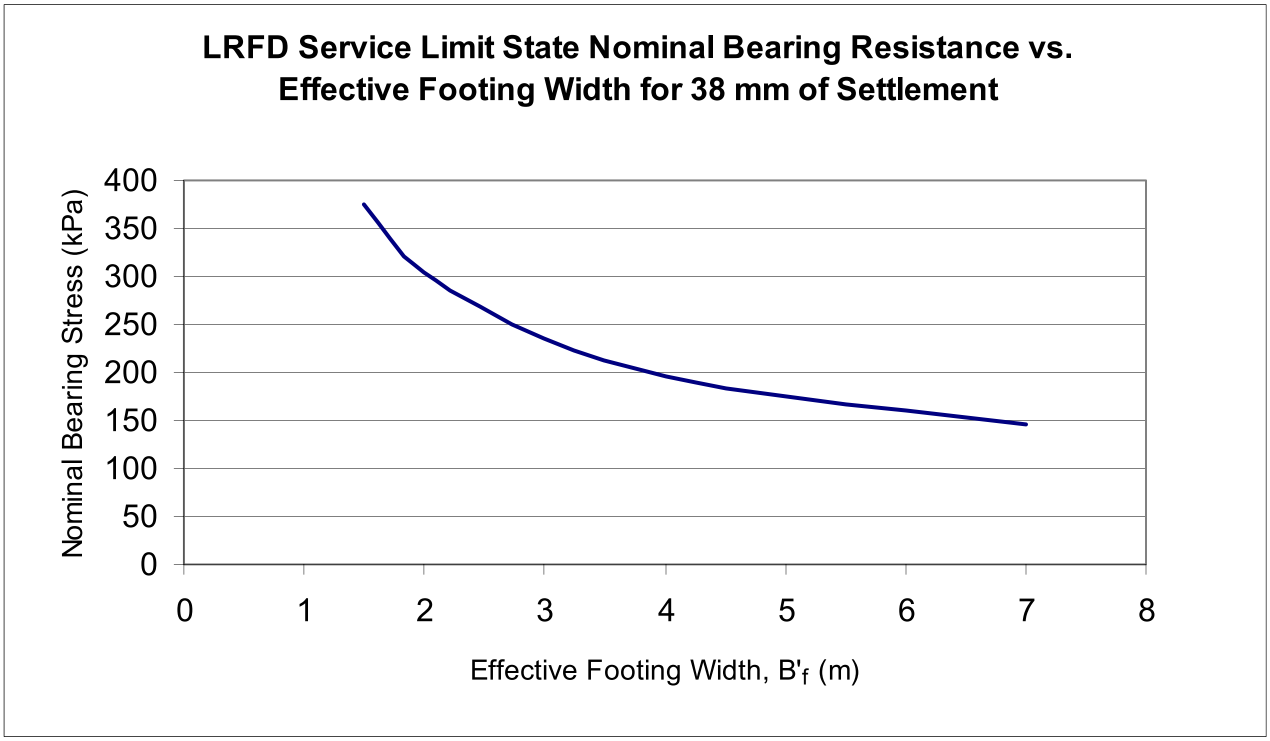


Figure C2-4: Nominal Bearing Resistance versus Effective Footing Width at Service-1 Limit State

The geotechnical engineer should provide both charts of nominal resistances at the strength and service limit states (e.g. Figures C2-3 and C2-4) to the structural designer for sizing of the footing.

Step 9 – Calculate nominal sliding and passive soil resistance at the strength limit state:

The footing concrete will be poured on compacted structural fill. Thus the friction angle δ to be used in the sliding analysis of the footing will be:

$$\delta = \phi' = 38°$$

The equation for calculating sliding resistance of concrete footings cast against the soil is:

$$Q_T = P_v \tan\phi$$

The ultimate, or nominal, sliding resistance is:

$$Q_T = 0.78(P_v)$$

The resistance factor associated with strength limit state checks is $\varphi = 0.8$ (Table 10.5.5-1, AASHTO, 1998).

The passive resistance of the soil in front of the footing will be ignored.

Step 10 – Check global stability of the footing:

The geotechnical engineer has determined that the global stability is satisfactory (see Service Load Design Example 3).

Step 11 – Size the footing at the service limit state under full loading:

Select a trial footing width, say $B_f = 3.2$ m.

<u>Check bearing resistance for $B_f = 3.2$ m:</u>

Analyze for 1 m of abutment width. Under the service limit state the load factors are taken as 1.0.

From Table C2-1, the loads and moments due to the loads of the superstructure are:

$$\begin{aligned}
P_{v\,girders} &= \eta[\gamma_P\,(DC) + \gamma_P\,(DW) + \gamma_{LL}\,(LL)] \\
&= 1.0[1.0(DC) + 1.0(DW) + 1.0(LL)] \\
&= DC + DW + LL \\
&= \frac{(5233 + 446 + 1538)\,kN}{25\,m} \\
&= 288.7\ kN/m
\end{aligned}$$

$$\begin{aligned}
M_{toe\ girders} &= \eta[\gamma_P\,(DC) + \gamma_P\,(DW) + \gamma_{LL}\,(LL) + 1.0(TU) + 1.0(CR) + 1.0(SH)] \\
&= 1.0[1.0(DC) + 1.0(DW) + 1.0(LL) + 1.0(TU + CR + SH)] \\
&= \frac{(7274 + 620 + 2138)kN \cdot m + 1.0(-3392)\,kN \cdot m}{25\,m} \\
&= 265.6\ kN \cdot m/m
\end{aligned}$$

From Table C2-2, the loads and moments due to abutment components are:

$$\begin{aligned}
P_{v\,abut} &= \eta[\gamma_P\,(DC) + \gamma_P\,(EV)] \\
&= 1.0[1.0(DC) + 1.0(EV)] \\
&= 57.29 + 10.81 B_f + (93.69)(B_f - 1.73) \\
&= 57.29 + (10.81)(3.2) + (93.69)(3.2 - 1.73) \\
&= 229.6\ kN/m
\end{aligned}$$

$$
\begin{aligned}
M_{\text{toe abut}} &= \eta[\gamma_P\,(DC) + \gamma_P\,(EV) + \gamma_P\,(EH) + \gamma_{LS}(LS)] \\
&= 1.0[1.0(DC) + 1.0(EV) + 1.0(EH) + 1.0(LS)] \\
&= 84.79 + 5.41{B_f}^2 + (93.69)(B_f - 1.73)(0.865 + 0.5B_f) - 146.6 - 48.97 \\
&= 84.79 + (5.41)(3.2)^2 + (93.69)(3.2 - 1.73)(0.865 + (0.5)(3.2)) - 146.6 - 48.97 \\
&= 284.1\,\text{kN}\cdot\text{m}/\text{m}
\end{aligned}
$$

The total loads and moments are:

$$
\begin{aligned}
P_v &= P_{v\ \text{girders}} + P_{v\ \text{abut}} \\
&= 288.7\,\text{kN}/\text{m} + 229.6\,\text{kN}/\text{m} \\
&= 518.3\,\text{kN}\cdot\text{m}/\text{m}
\end{aligned}
$$

$$
\begin{aligned}
M_{\text{toe}} &= M_{\text{toe girders}} + M_{\text{toe abut}} \\
&= 265.6\,\text{kN}\cdot\text{m}/\text{m} + 284.1\,\text{kN}\cdot\text{m}/\text{m} \\
&= 549.7\,\text{kN}\cdot\text{m}/\text{m}
\end{aligned}
$$

The arm of the resultant is:

$$
\begin{aligned}
\text{arm}_R &= M_{\text{toe}}/P_v \\
&= (549.7\,\text{kN}\cdot\text{m}/\text{m})/(518.3\,\text{kN}/\text{m}) \\
&= 1.06\,\text{m}
\end{aligned}
$$

The eccentricity, e_y, is

$$
\begin{aligned}
e_y &= B_f/2 - \text{arm}_R \\
&= 3.2\,\text{m}/2 - 1.06\,\text{m} \\
&= 1.6\,\text{m} - 1.06\,\text{m} \\
&= 0.54\,\text{m}
\end{aligned}
$$

<u>Check bearing stress for effective footing area:</u>

$$
\begin{aligned}
B'_f &= B_f - 2e_y \\
&= 3.2\,\text{m} - (2)(0.54\,\text{m}) \\
&= 2.12\,\text{m}
\end{aligned}
$$

$$
\begin{aligned}
q_{applied} &= P_v / B'_f \\
&= (518.3\,kN/m)/(2.12\,m) \\
&= 246.8\,kPa
\end{aligned}
$$

This is less than 295 kPa from Figure C2-4 for settlement less than 38 mm and $B'_f = 2.12$ m. Use 3.2 m footing to perform the rest of the design checks.

Step 12a – Check the bearing pressure, maximum eccentricity and sliding at the Strength I limit state under full loading:

Check maximum eccentricity for $B_f = 3.2$ m:

From Table C2-1, resisting loads are multiplied by minimum factors and driving loads are multiplied by maximum factors. At the strength limit state, the load factors γ_{TU}, γ_{CR} and γ_{SH} for force effects are all equal to 0.5.

The loads and moments due to the superstructure are:

$$
\begin{aligned}
P_{v\,girders} &= \eta[\gamma_{P\,min}(DC) + \gamma_{P\,min}(DW) + \gamma_{LL}(LL)] \\
&= 1.0[0.9(DC) + 0(DW) + 0(LL)] \\
&= 0.9(DC) + 0(DW) + 0(LL) \\
&= \frac{0.9(5233) + 0(446) + 0(1538)\,kN}{25\,m} \\
&= 188.4\ kN/m
\end{aligned}
$$

$$
\begin{aligned}
M_{toe\ girders} &= \eta[\gamma_{P\,min}(DC) + \gamma_{P\,min}(DW) + \gamma_{LL}(LL) + 0.5(TU) + 0.5(CR) + 0.5(SH)] \\
&= 1.0[0.9(DC) + 0(DW) + 0(LL) + 0.5(TU + CR + SH)] \\
&= \frac{0.9(7274) + 0(620) + 0(2138) + 0.5(-3392)\,kN \cdot m}{25\,m} \\
&= 194.0\,kN \cdot m/m
\end{aligned}
$$

From Table C2-2, the loads and moments due to abutment components are:

$$
\begin{aligned}
P_{v\,abut} &= \eta[\gamma_{P\,min}(DC) + \gamma_{P\,min}(EV)] \\
&= 1.0[0.9(DC) + 1.00(EV)] \\
&= 0.9(57.29 + 10.81B_f) + (93.69)(B_f - 1.73) \\
&= 0.9(57.29) + 0.9(10.81)(3.2) + (93.69)(3.2 - 1.73) \\
&= 220.4\,kN/m
\end{aligned}
$$

$$
\begin{aligned}
M_{toe\ abut} &= \eta[\gamma_{P\min}(DC) + \gamma_{P\min}(EV) + \gamma_{P\max}(EH) + \gamma_{LS}(LS)] \\
&= 1.0[0.9(DC) + 1.0(EV) + 1.5(EH) + 1.75(LS)] \\
&= 0.9(84.79 + 5.41 B_f{}^2) + 1.0[(93.69)(B_f - 1.73)(0.865 + (0.5)(B_f))] \\
&\qquad -1.5(146.6) - 1.75(48.97) \\
&= 0.9(84.79) + 0.9(5.41)(3.2)^2 + (93.69)(3.2 - 1.73)(0.865 + (0.5)(3.2)) \\
&\qquad -1.5(146.6) - 1.75(48.97) \\
&= 160.1\ \text{kN} \cdot \text{m/m}
\end{aligned}
$$

The total loads and moments are:

$$
\begin{aligned}
P_v &= P_{v\ girders} + P_{v\ abut} \\
&= 188.4\ \text{kN/m} + 220.4\ \text{kN/m} \\
&= 408.8\ \text{kN} \cdot \text{m/m}
\end{aligned}
$$

$$
\begin{aligned}
M_{toe} &= M_{toe\ girders} + M_{toe\ abut} \\
&= 194.0\ \text{kN} \cdot \text{m/m} + 160.1\ \text{kN} \cdot \text{m/m} \\
&= 354.1\ \text{kN} \cdot \text{m/m}
\end{aligned}
$$

The arm of the resultant is:

$$
\begin{aligned}
arm_R &= M_{toe} / P_v \\
&= (354.1\ \text{kN} \cdot \text{m/m}) / (408.8\ \text{kN/m}) \\
&= 0.866\ \text{m}
\end{aligned}
$$

And the eccentricity, e_y, is:

$$
\begin{aligned}
e_y &= B_f / 2 - arm_R \\
&= 3.2\text{m}/2 - 0.866\text{m} \\
&= 1.6\text{m} - 0.866\text{m} \\
&= 0.73\text{m}
\end{aligned}
$$

The eccentricity should be in the middle half of the footing, or:

$$
B_f / 4 = (3.2\text{m})/4 = 0.8\text{m}
$$

Since $e_y < B_f / 4$, $B_f = 3.2$ m satisfies the eccentricity (overturning) requirement.

Check bearing stress for effective footing area:

Compute B'_f using maximum load factors. At the strength limit state, the load factors γ_{TU}, γ_{CR} and γ_{SH} for force effects are all equal to 0.5.

$$
\begin{aligned}
P_{v\,girders} &= \eta[\gamma_{P\,max}(DC) + \gamma_{P\,max}(DW) + \gamma_{LL}(LL)] \\
&= 1.0[1.25(DC) + 1.5(DW) + 1.75(LL)] \\
&= \frac{1.25(5233) + 1.5(446) + 1.75(1538)\,kN}{25\,m} \\
&= 396.1\ kN/m
\end{aligned}
$$

$$
\begin{aligned}
M_{toe\ girders} &= \eta[\gamma_{P\,max}(DC) + \gamma_{P\,max}(DW) + \gamma_{LL}(LL) + 0.5(TU) + 0.5(CR) + 0.5(SH)] \\
&= 1.0[1.25(DC) + 1.5(DW) + 1.75(LL) + 0.5(TU + CR + SH)] \\
&= \frac{1.25(7274) + 1.5(620) + 1.75(2138) + 0.5(-3392)\,kN \cdot m}{25\,m} \\
&= 482.7\ kN \cdot m/m
\end{aligned}
$$

$$
\begin{aligned}
P_{v\,abut} &= \eta[\gamma_{P\,max}(DC) + \gamma_{P\,max}(EV)] \\
&= 1.0[1.25(DC) + 1.35(EV)] \\
&= 1.25(57.29 + 10.81B_f) + 1.35(93.69)(B_f - 1.73) \\
&= 1.25(57.29) + 1.25(10.81)(3.2) + 1.35(93.69)(3.2 - 1.73) \\
&= 300.8\,kN/m
\end{aligned}
$$

$$
\begin{aligned}
M_{toe\ abut} &= \eta[\gamma_{P\,min}(DC) + \gamma_{P\,min}(EV) + \gamma_{P\,max}(EH) + \gamma_{LS}(LS)] \\
&= 1.0[1.25(DC) + 1.35(EV) + 1.5(EH) + 1.75(LS)] \\
&= 1.25(84.79 + 5.41B_f^{\,2}) + 1.35[(93.69)(B_f - 1.73)(0.865 + (0.5)(B_f))] \\
&\qquad -1.5(146.6) - 1.75(48.97) \\
&= 1.25(84.79) + 1.25(5.41)(3.2)^2 \\
&\qquad +1.35[(93.69)(3.2 - 1.73)(0.865 + (0.5)(3.2))] - 1.5(146.6) \\
&\qquad -1.75(48.97) \\
&= 328.0\ kN \cdot m/m
\end{aligned}
$$

The total loads and moments are:

$$
\begin{aligned}
P_v &= P_{v\ girders} + P_{v\ abut} \\
&= 396.1\ kN/m + 300.8\ kN/m \\
&= 696.9\ kN \cdot m/m
\end{aligned}
$$

$$
\begin{aligned}
M_{toe} &= M_{toe\ girders} + M_{toe\ abut} \\
&= 482.7\ kN \cdot m/m + 328.0\ kN \cdot m/m \\
&= 810.7\ kN \cdot m/m
\end{aligned}
$$

The arm of the resultant is:

$$
\begin{aligned}
arm_R &= M_{toe} / P_v \\
&= (810.7\ kN \cdot m/m)/(696.9\ kN/m) \\
&= 1.163\ m
\end{aligned}
$$

The eccentricity, e_y, is:

$$
\begin{aligned}
e_y &= B_f / 2 - arm_R \\
&= 3.2\,m/2 - 1.163\,m \\
&= 1.6\,m - 1.163\,m \\
&= 0.44\,m
\end{aligned}
$$

And the effective footing width is:

$$
\begin{aligned}
B'_f &= B_f - 2e_y \\
&= 3.2\,m - 2(0.44\,m) \\
&= 3.2\,m - 0.88\,m \\
&= 2.32\,m
\end{aligned}
$$

So the applied stress is:

$$
\begin{aligned}
q_{applied} &= P_v / B'_f \\
&= 696.9\,kN/2.32\,m \\
&= 300\,kPa
\end{aligned}
$$

From Figure C2-3, the nominal bearing resistance at the strength limit state for an effective footing width of 2.32 m is about 1300 kPa. Applying a resistance factor of 0.35, the factored resistance at the strength limit state is:

$$Q_R = \varphi(q_{ult}) = 0.35(1300\text{ kPa}) = 455\text{ kPa} \geq q_{applied} \qquad \text{(OK)}$$

Check sliding:

The factored horizontal shear force must be less than the factored nominal sliding resistance. This is evaluated using minimum load factors:

$$Q_R = \varphi_T Q_T + \varphi_{ep} Q_{ep} \qquad \text{(Eqn. C-3)}$$

$$Q_R = 0.8(0.78(408.8\text{kN})) = 255.1\text{kN}$$

$$V_{max} = 1.5(EH) + 1.75(LS) + 0.5(TU + SH + CR)$$

$$= 1.5(69.96) + 1.75(18.69) + 0.5(1047/25) = 158.6\text{kN} < 255.1\text{kN} \qquad \text{(OK)}$$

Step 12b – Check the bearing pressure, maximum eccentricity and sliding at the Strength I limit state without the bridge loading from the girders:

Check maximum eccentricity for B_f = 3.2 m:

From Table C2-2, considering that resisting loads are multiplied by minimum factors and driving loads are multiplied by maximum factors, the loads and moments due to abutment components are:

$$\begin{aligned} P_{v\,abut} &= \eta[\gamma_{P\,min}(DC) + \gamma_{P\,min}(EV)] \\ &= 1.0[0.9(DC) + 1.00(EV)] \\ &= 0.9(57.29 + 10.81B_f) + (93.69)(B_f - 1.73) \\ &= 0.9(57.29) + 0.9(10.81)(3.2) + (93.69)(3.2 - 1.73) \\ &= 220.4\text{kN/m} \end{aligned}$$

$$\begin{aligned} M_{toe\ abut} &= \eta[\gamma_{P\,min}(DC) + \gamma_{P\,min}(EV) + \gamma_{P\,max}(EH)] \\ &= 1.0[0.9(DC) + 1.0(EV) + 1.5(EH)] \\ &= 0.9(84.79 + 5.41{B_f}^2) + 1.0(93.69)(B_f - 1.73)(0.865 + 0.5B_f) - 1.5(146.6) \\ &= 0.9(84.79) + 0.9(5.41)(3.2)^2 + (93.69)(3.2 - 1.73)(0.865 + 0.5(3.2)) - 1.5(146.6) \\ &= 245.8\text{ kN}\cdot\text{m/m} \end{aligned}$$

The arm of the resultant is:

$$
\begin{aligned}
arm_R &= M_{toe\ abut} / P_{v\ abut} \\
&= (245.8\,kN \cdot m/m)/(220.4\,kN/m) \\
&= 1.12\,m
\end{aligned}
$$

The eccentricity, e_y, is:

$$
\begin{aligned}
e_y &= B_f/2 - arm_R \\
&= 3.2\,m/2 - 1.12\,m \\
&= 1.6\,m - 1.12\,m \\
&= 0.48\,m
\end{aligned}
$$

The eccentricity should be in the middle half of the footing, or:

$$B_f/4 = (3.2\,m)/4 = 0.8\,m$$

Since $e_y < B_f/4$, B_f = 3.2 m satisfies the eccentricity (overturning) requirement.

<u>Check bearing stress for effective footing area:</u>

$$
\begin{aligned}
P_{v\,abut} &= \eta[\gamma_{P\,max}(DC) + \gamma_{P\,max}(EV)] \\
&= 1.0[1.25(DC) + 1.35(EV)] \\
&= 1.25(57.29 + 10.81B_f) + 1.35(93.69)(B_f - 1.73) \\
&= 1.25(57.29) + 1.25(10.81)(3.2) + 1.35(93.69)(3.2 - 1.73) \\
&= 300.8\,kN/m
\end{aligned}
$$

$$
\begin{aligned}
M_{toe\ abut} &= \eta[\gamma_{P\,min}(DC) + \gamma_{P\,min}(EV) + \gamma_{P\,max}(EH) + \gamma_{LS}(LS)] \\
&= 1.0[1.25(DC) + 1.35(EV) + 1.5(EH) + 1.75(LS)] \\
&= 1.25(84.79 + 5.41{B_f}^2) + 1.35[(93.69)(B_f - 1.73)(0.865 + (0.5)(B_f))] \\
&\qquad -1.5(146.6) - 1.75(48.97) \\
&= 1.25(84.79) + 1.25(5.41)(3.2)^2 + 1.35[(93.69)(3.2 - 1.73)(0.865 + (0.5)(3.2))] \\
&\qquad -1.5(146.6) - 1.75(48.97) \\
&= 328.0\ kN \cdot m/m
\end{aligned}
$$

The arm of the resultant is:

$$\begin{aligned} arm_R &= M_{toe\ abut} / P_{v\ abut} \\ &= (328.0\ kN \cdot m/m)/(300.8\ kN/m) \\ &= 1.090\ m \end{aligned}$$

The eccentricity, e_y, is:

$$\begin{aligned} e_y &= B_f / 2 - arm_R \\ &= 3.2\,m/2 - 1.09\,m \\ &= 1.6\,m - 1.09\,m \\ &= 0.51\,m \end{aligned}$$

And the effective footing width is:

$$\begin{aligned} B'_f &= B_f - 2e_y \\ &= 3.2\,m - 2(0.51\,m) \\ &= 3.2\,m - 1.02\,m \\ &= 2.18\,m \end{aligned}$$

So the applied stress is:

$$\begin{aligned} q_{applied} &= P_v / B'_f \\ &= 300.8\,kN / 2.18\,m \\ &= 138\,kPa \end{aligned}$$

From Figure C2-3, the nominal bearing resistance at the strength limit state for an effective footing width of 2.18 m is about 1260 kPa. Applying a resistance factor of 0.35, the factored resistance at the strength limit state is:

$$Q_R = \varphi(q_{ult}) = 0.35(1260\ kPa) = 441\ kPa \geq q_{applied} \qquad \text{(OK)}$$

Check sliding:

The factored horizontal shear force must be less than the factored nominal sliding resistance. This is evaluated using minimum load factors:

$$Q_R = \varphi_T Q_T + \varphi_{ep} Q_{ep} \qquad \text{(Eqn. C-3)}$$

$$Q_R = 0.8(0.78(220.4\text{kN})) = 137.5\text{kN}$$

$$V_{max} = 1.5(EH)$$

$$= 1.5(69.96) = 104.9\,\text{kN} < 137.5\,\text{kN} \quad \text{(OK)}$$

Step 13 Check the bearing pressure, eccentricity and sliding at the extreme limit state:

This step is skipped in this example. See Example C-1 for an extreme limit state design check.

Step 14 Perform the structural design of the footing:

The structural engineer performs this step.