National Cooperative Highway Research Program

NCHRP Synthesis 276

Geotechnical Related Development and Implementation of Load and Resistance Factor Design (LRFD) Methods

A Synthesis of Highway Practice

Transportation Research Board
National Research Council

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Geotechnical Related Development and Implementation of Load and Resistance Factor Design (LRFD) Methods

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PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

FOREWORD

By Staff
Transportation Research Board

This synthesis report will be of interest to geotechnical, structural, and bridge engineers, especially those involved in the development and implementation of the geotechnical aspects of the AASHTO Bridge Code. The synthesis documents a review of geotechnical related LRFD specifications and their development worldwide in order to compare them with the current AASHTO LRFD Bridge Code. Design procedures for foundations, earth retaining structures, and culverts are summarized and compared to methods specified by the AASHTO code.

Administrators, engineers, and researchers are continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

This report of the Transportation Research Board provides information to assist engineers in implementing the geotechnical features of LRFD methods. Information for the synthesis was collected by surveying U.S. and Canadian transportation agencies and by conducting a literature search using domestic and international sources. Interviews of selected international experts were also conducted. The limited available experience in the United States and information from international practice are discussed to understand the problems that have arisen so that solutions may be found. Based on a review of various LRFD codes, suggestions for further work are made.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the research in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

The derivation of existing load and resistance factors for geotechnical related design is not necessarily well understood or complete. This synthesis provides some background information on derivation, but limits on thoroughness were imposed by the scope of the study. For additional information, the interested reader may refer to the following resources: Appendix A of NCHRP Report 343 *Manuals for the Design of Bridge Foundations, Load and Resistance Factor Design (LRFD) of Highway Bridge Structures*, Reference Manual and Participant Workbook, FHWA-HI-98-032; and *Geotechnical Engineering Practices in Canada and Europe*, FHWA-PL-99-013. The former FHWA Manual and Workbook was developed for National Highway Course Number 13068 and the latter report is a result of a scanning tour conducted in March of 1998.

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GEOTECHNICAL RELATED DEVELOPMENT AND IMPLEMENTATION OF LOAD AND RESISTANCE FACTOR DESIGN (LRFD) METHODS

SUMMARY

The American Association of State Highway and Transportation Officials (AASHTO) adopted a load and resistance factor design (LRFD) code for bridges in 1994. This code was developed over a period of several years in a research project sponsored by the National Cooperative Highway Research Program (NCHRP). The primary effort in this project was devoted to the design of highway bridge superstructures, with less attention to the design of shallow and deep foundations, earth retention systems, and culverts, i.e. geotechnical facilities. The geotechnical portion of the code was developed in an additional NCHRP project. The implementation of this code is an ambitious effort because of the changes required, particularly in the geotechnical area where there was little experience with either the application of LRFD or of strongly prescriptive codes.

LRFD was brought quickly into practice in the United States with the adoption in 1963 of *The American Concrete Institute Building Design Code* after several years of experimentation. In the LRFD approach, the traditional factor of safety used in allowable stress design (ASD) was replaced by two types of factors, one type on the loads and one on the strength (resistance). They are intended to account separately for the variability of the particular loads and also the strength. At about the same time, LRFD was formulated and brought into practice for geotechnical applications by the Danish Geotechnical Institute under the name "limit states design."

In the late 1960s, researchers began to look at the use of probability theory to develop a rational basis for structural design. They proposed the use of the LRFD framework with the load and resistance factors generated by a probabilistic analysis of statistical data on loads and element strengths. This concept has been was widely accepted and is the basis for several design codes. Due to the lack of statistical data on element and system strength, it was not always possible to generate the resistance factors analytically. In those cases, resistance factors were determined by correlation with ASD. Resistance factors for the design of geotechnical facilities have been developed using analytical calibrations based on a probabilistic analysis in only a very few cases. Almost all geotechnical resistance factors have been selected based on ASD correlations.

Over the past 15 years there has been a general move toward the increased use of LRFD in the design of structures, including geotechnical facilities. New LRFD codes have been adopted in Canada, Australia, and the European Community (EC). The Ontario Ministry of Transportation was the leader in implementing LRFD for bridge design. A national highway bridge code has also been adopted in Canada. In Europe, the countries of the EC have undertaken the development of an LRFD standard for structural and geotechnical design and, after an extended effort, a document was adopted for geotechnical design in 1997. A major effort was devoted to the development of this code and for that reason it is a valuable resource. However, the geotechnical portion of this code was calibrated by comparison with

ASD rather than by probabilistic methods. LRFD for bridges was adopted in Australia in 1992 and more recently an LRFD code for deep foundations for non-transportation facility applications was adopted in Australia. All of these codes are discussed and reviewed here and are compared with the AASHTO LRFD Bridge Code.

A questionnaire was prepared for this Synthesis to determine to what extent the AASHTO LRFD Code has been implemented (1997) by state DOTs and the Canadian provinces. Questionnaire responses show that only a few states had already gained some experience, although more than half had plans for implementation. States experienced with the code reported the most difficulty with the portions on deep foundations. It should be noted that deep foundations are used more extensively than spread footings by most DOTs. In Canada, almost all provinces have implemented LRFD for geotechnical design. They also reported difficulty with the design of deep foundations and primarily with driven piles. The Canadian designers believed that the resulting LRFD-based designs for geotechnical facilities were somewhat more conservative than previous ASD-based designs.

In this synthesis, design procedures for foundations, earth retaining structures, and culverts were summarized and compared with the methods specified by the AASHTO LRFD Bridge Code. The fundamental design methods do not change when switching from ASD to LRFD, only the way safety margins are established. In some cases, differences were noted particularly for deep foundations. The design of foundations is quite similar to structural design in that load and resistance are clearly and simply separated. Considerable performance data are available concerning the ultimate geotechnical capacity of deep foundations and these data should be incorporated in a research effort to develop rational resistance factors by analytical probabilistic calibrations. The designs resulting from these resistance factors should be carefully compared with designs obtained from currently used ASD Codes.

Earth retaining systems present a very difficult problem in implementing LRFD. Both the load and the strength of the system contain soil properties. Thus, the system strength cannot be clearly separated from the loads. If the loads also contain structural loads, then the selection of load factors for structural loads and soil pressure must be consistent. Earth retaining system design has been based on experience and tradition. Since load and resistance are difficult to separate rationally, it will be difficult to develop rational load and resistance factors. A real solution to this problem will probably require a substantial research effort. In the interim, resistance factors will probably have to be determined and checked by calibration with existing practice.

The geotechnical engineer often confronts the situation where several test results are available from a single soil strata and these values vary over a wide range. How should soil properties be selected? A conservative selection versus an optimistic value can produce large differences in the strength and hence in the design. The Eurocode requires a specific procedure for making this decision. The current AASHTO LRFD Bridge Code does not make recommendations for the selection of soil properties, but perhaps some such recommendations should be considered.

The final proof of the usefulness and validity of a new code provision is to compare designs made by the old and the new provisions. This has been a common procedure for structural engineers and it needs to be done in the geotechnical area. To be effective, a large number of different project characteristics should be studied.

CHAPTER ONE

INTRODUCTION

NEED FOR THE SYNTHESIS

A load and resistance factor design (LRFD) code developed from National Cooperative Highway Research Program (NCHRP) Project 12-33 was adopted in 1994 by the AASHTO Highway Subcommittee on Bridges and Structures *(AASHTO 1994)*. Interim specifications have been adopted and the new design procedure is now being implemented into practice. In this report, the term "LRFD Bridge Code" will refer to the most recent available Interim issued in 1997 *(AASHTO 1994b)*. For structural elements, this change is less dramatic than for geotechnical features because LRFD has been in use for concrete buildings for some time and the approach has been taught to structural engineers in engineering educational institutions for three decades. LRFD also has been available for use in the design of several bridge superstructure types since 1977 *(AASHTO 1977)* under the name load factor design (LFD). However, for geotechnical design, implementation is more daunting than for structural design because there is little prior experience, little or no coverage in geotechnical engineering education, limited use of codes for geotechnical design, and less dependence on codes in design by geotechnical engineers than by structural engineers.

Of course, one can expect that the implementation of a radically different design process will be difficult. With time, experience, and familiarity with the process in the new code, the problems will gradually disappear. The larger impediment to LRFD implementation in geotechnical work is the intertwining of loads and resistances in geotechnical engineering. The resistance is often dependent on the loads, particularly in the case of earth retention structures. Further complicating the issue is the inherent variability of the materials themselves and the methods used to estimate the strengths and loads caused by geomaterials. The load and resistance factors must also account for the variability in the system, including the heterogeneity of soil, variability of soil sampling and testing methods, and the unreliability of the analysis methods. These problems make it necessary to adopt LRFD codes that will produce designs that are similar to current procedures.

In 1992, the AASHTO Subcommittee on Bridges and Structures adopted an updated, greatly changed, and improved geotechnical section in the Fifteenth Edition of the Standard Bridge Code *(AASHTO 1992)*. This geotechnical code modification was still completely based on allowable stress design methods. Many geotechnical designers have not implemented all of those changes into their design practice. Now these improved design procedures must be implemented together with the LRFD approach. In general, the use of codes is much better established for structural design than for geotechnical design, so the task for the bridge superstructure designer is just the implementation of a new code. However, many geotechnical designers must implement a new approach to design while simultaneously adopting a more rigorous and detailed code.

In the Sixteenth Edition of the *AASHTO Standard Specification for Highway Bridges* (AASHTO 1996), resistance factors were added in the geotechnical sections in addition to the factors of safety of the allowable stress design procedure. Thus, both ASD and LRFD procedures are available. These resistance factors are similar to those contained in the LRFD Bridge Code. However, the factored loads will be different because the load factors in the Sixteenth Edition of the Standard Bridge Code are quite different from those in the LRFD Bridge Code.

Geotechnical engineers involved in the implementation of the LRFD Bridge Code will be concerned that it will limit their creativity in design. Geotechnical design is more of an art than is structural design; therefore, it does not lend itself as easily to codification. In the present state of geotechnical practice, specific design methods are not uniformly accepted or appropriate; often, different methods are preferred in different localities or even by different engineers in the same locality. Many types of geotechnical systems are difficult or impossible to test with the same degree of realism as is possible for structural elements. This is particularly the case where both the load and the strength sides of the design relationship contain some of the same soil properties. For example, in the case of earth retention structures, both the load and the resistance are defined by some of the same soil material properties. Furthermore, most structural materials are manufactured products and much more uniform in their behavior than is a natural material such as soil or rock.

Prior to the adoption of the 1994 AASHTO LRFD Bridge Code, load factor design for bridge superstructures had been available as an option since 1977 *(AASHTO 1977)*. Load factors and load combinations were defined. In the design of reinforced concrete structural elements, a strength design approach with specified resistance factors was available. For structural steel elements, methods were specified for determining the ultimate strength. However, resistance factors were not specified or required.

In the design of foundations, it is necessary to determine the design load as a separate step if load factors are used for the superstructure, because factors of safety are specified to be used together with working loads. The need to obtain working load for foundation design from the factored loads used for superstructure design has been the case for 30 years in the design of building foundations. For earth-retention structures, to the extent that their design was codified, ASD was used. Problems arose in cases where structural loads were involved, such as abutments, or where earth loads were applied to structural elements such as retaining walls.

The LRFD Bridge Code unifies the design of structural and geotechnical aspects. It will require geotechnical designers to make many changes in their procedures, but clarity and uniformity can result. Particularly, when geotechnical engineers must work together with structural engineers, the design task will be improved by the use of LRFD. This Code change has now (1999) been available for 5 years and several agencies have studied it and attempted to use it. It is useful to review and evaluate their experiences. The process of the change to LRFD has been underway in other parts of the world and that experience can be particularly valuable to assist the implementation in the United States. Details of experience in both the United States and Canada are presented in this synthesis of information.

OBJECTIVES OF THE SYNTHESIS

The new LRFD Bridge Code requires that LRFD be used for the design of geotechnical facilities. As discussed above, the implementation of this new design process may be difficult for transportation agencies so it is desirable that the related experience of other design organizations be carefully considered. Also, the new code is in a state where a progression of changes will probably be required as indicated by the changes already contained in the 1997 Interim *(AASHTO 1997b)* and the comments obtained from the survey of state and provincial DOTs reported in this synthesis. These likely future changes will probably further complicate the implementation process.

In the geotechnical area, the AASHTO ASD Bridge Code *(AASHTO 1977)* was changed quite recently and an effort has been underway to implement those changes. In some areas, the changes have been of major consequence and they have not been fully implemented by many agencies. It may appear to many designers that the changes, coming from the earlier practice improvements, are the result of the conversion to LRFD.

The objective of this synthesis is to review LRFD geotechnical specifications and their development worldwide in order to compare them with the LRFD Bridge Code. It will provide information to assist engineers in implementing the geotechnical features of LRFD methods. The limited available experience in the United States was collected and examined to understand the problems that have arisen so that solutions may be found. Additional information from international practice has been collected to supplement the discussion of the various aspects of LRFD. Based on the review of these LRFD Codes, suggestions for further work are made in the conclusions in chapter 7.

TERMINOLOGY

When the LRFD design procedure is brought into use, it is necessary that the new and unfamiliar terms be clearly defined. This is particularly important in the case of LRFD for geotechnical applications because the design approach is new to geotechnical engineers. It is necessary that the terminology be the same as that used by structural engineers to facilitate communication between the two specialists.

Allowable Stress (or load)—a specified stress (or load) on an element that is not to be exceeded when the element is subjected to loads that can be expected to occur commonly during the life of the structure. The stress (or load) acting on the element is determined from the loads applied to the structure using a linear elastic analysis.

Allowable Stress Design (ASD)—a design method based on the requirement that the calculated stress (or load) not exceed the allowable stress. The calculated stress (or load) is obtained by a linear elastic analysis.

Calibration—the process that is used to determine the load and resistance factors for LRFD. Calibration may be performed using a probabilistic analysis of load and strength statistical data or it may be done by comparison with ASD for a selected live load/dead load ratio. Often a combination of these two methods is used.

Design Load—loads that are expected to occur commonly during the life of the structure.

Factor of Safety—the resistance of an element or system divided by the applied force, e.g., the ultimate strength of a pile divided by the design load.

Factored Load—in LRFD, a term that refers to the sum of the various specified applied loads times their individual load factors.

Factored Load Effect—this is the factored load that is carried by an individual element, e.g., the load carried by an individual pile of a group based on a linear elastic analysis of the structure subjected to the factored load.

Factored Resistance or Strength—the product of the resistance factor and the nominal resistance (or strength).

Limit State—a condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed.

Limit States Design—a design method that seeks to provide safety against a structure or structural element being rendered unfit for use. This method is commonly called load and resistance factor design (LRFD) in the United States.

Linear Elastic Analysis—a structural analysis method that assumes that the relationship between force and deformation for the structural system is linear. This analysis approach is commonly used by structural engineers to determine the forces and stresses in a structure.

Load Combinations—loads that are likely to act simultaneously for a given limit state.

Load Effect—the force in a member or element (axial force, shear force, bending, or torque) due to loading calculated by a linear elastic analysis.

Load Factor—a factor accounting for the variability of the loads, the lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads.

Load Factor Design (LFD)—a name for a design procedure adopted by AASHTO in 1977. In this design approach, factors are applied to the loads and in some cases also to the resistance.

LRFD Code Calibration—a process used to determine the load and resistance factors for use in the LFRD approach.

Nominal Resistance or Strength—calculated using a prescribed method to define the ultimate strength or limiting serviceability response of an element or system.

Resistance Factor—a factor accounting for the variability of material properties, structural dimensions, and workmanship, and the uncertainty in the prediction of resistance inherent in the nominal resistance evaluation method. Structural engineers have used phi (ϕ) to denote the resistance factor; geotechnical engineers will find this notation confusing since they use the same symbol to represent the angle of internal friction, a soil property.

Serviceability—a measure of performance, other than strength, that may cause the system behavior to be unsatisfactory (e.g., settlement).

THE LRFD METHOD

The term LRFD, in the strictest sense, refers to the use of factors (load factors) applied to the various types of loads and the associated resistance (resistance factors) in each of several combinations to 1. account for the variability of the load and resistance and 2. determine that the factored load effect does not exceed the resistance times the resistance factor for a given failure or serviceability mode. In this method, the probability of occurrence is the same for each of the load combinations. Thus, the appropriate load factor for the various load types is directly associated with the particular load type and load combination. In addition to the factored load combinations, the safety for the structure is assured to be within acceptable limits by also putting a factor on the strength. Thus, by the use of factors on the loads and resistance to deal with their variability and uncertainty, the safety of the structure is designed to be maintained within an acceptable risk. In traditional ASD, safety is achieved with a single factor of safety applied to the resistance to obtain an allowable stress (or load). The factor of safety is selected based on experience, judgment, and tradition.

Because there may be many different load types, the manner in which the loads are combined has sometimes been unclear in the traditional use of ASD. For instance, it is unlikely that the most extreme values of live load, wind load, stream load, and earthquake load will occur at the same time. LRFD provides a response to this problem by specifying several load combinations with load factors selected on a probabilistic basis. It should be noted that the AASHTO ASD Bridge Code, however, has offered specified load combinations for a considerable time.

Actually, the term LRFD implies more than just verifying the strength of the designed structure. In ASD, the limits on working stress often succeed, indirectly, in controlling problems of serviceability (e.g., deflections or vibrations). Limit states design is a more appropriate name than LRFD for the procedure and this name is sometimes used in other countries. *All of the possible conditions that may produce unsatisfactory performance are referred to as limit states and all of the various limit states must be accounted for in the design. However, the term LRFD will be used here to imply that all of the limit states are addressed.* In the United States, the term LRFD has come to imply the checking of all of the strength and serviceability limit states. It seems unnecessary to adopt the term limit states design in U.S. geotechnical practice since the name LRFD is so widely used by structural engineers. Kulicki *(1998)* has reported that during development of the LRFD Bridge Code the decision was made to continue using the term LRFD in reference to this approach to design.

Strength checking of the design is usually most important and it is emphasized in this section. Other limit states are dealt with using procedures similar to those used for the strength limit state. It is necessary that all applicable limit states be checked and one of the advantages of LRFD in design is that it emphasizes the importance of the examination of all of the limit states.

The LRFD method, as given in the AASHTO Code, may be stated in mathematical form as

$$\phi_k R_{nk} \geq \Sigma \eta_{ij} \gamma_{ij} Q_{ij}$$

where

ϕ_k	=	resistance factor for the kth failure mode or serviceability limit state,
R_{nk}	=	nominal strength or performance for the kth failure mode or serviceability limit state,
η_{ij}	=	factor to account for the ductility, redundancy, and operational importance of the element or system,
γ_{ij}	=	load factor for the ith load type in the load combination j under consideration, and
Q_{ij}	=	member load effect for the ith load type in the jth load combination.

It is useful to state Equation (1) in words. The left side of the equation defines the factored resistance. The nominal resistance is multiplied by the resistance factor, a value that is usually less than, or equal to, one. This product represents a resistance that has been reduced in magnitude to account for the reliability of the methods used to determine the nominal resistance. The right side of the equation represents the loads applied to the element under consideration (or more generally stated, the actions applied to the element, e.g., including shrinkage in a concrete structure). The loads are increased by a factor that is defined by the load type and the load combination. Thus, the load factor for dead load is quite small since it should be possible to determine the dead load quite reliably while the traffic load is multiplied by a much larger factor due to its great variability.

Load factors have already been defined in the LRFD Bridge Code *(AASHTO 1994)* after extensive study over the past several years as part of NCHRP Project 12-33, "A Calibration of LRFD Bridge Design Code" *(Nowak nd)*. The geotechnical designer is most concerned with the determination of the nominal strength, R_n, and the associated resistance factor, ϕ. In the general case, there may be several limit states of concern, each with their own resistance factor and nominal strength. For example, in designing a deep foundation, it is necessary to consider limit states associated with structural axial strength, soil strength, lateral load behavior, settlement, scour, ship impact response, and earthquake response. In each case, the strength or other response quantity must be determined and the associated resistance factor selected.

In the development of design codes in general and geotechnical codes specifically, it is necessary that appropriate load combinations be defined for each limit state. Most of this work has already been done and reported by Nowak. *Of primary concern is the development (or selection) of methods to determine the nominal resistance.* These procedures must determine the geotechnical system response of interest. They may be concerned with either strength or serviceability limit states. *In addition, the associated resistance factor must also be selected.* The resistance factors are dependent on both the limit state and the accuracy and reliability of the method used for determining the nominal resistance.

SYNTHESIS INVESTIGATION AND RESPONSE

One important task of this study was to determine the current state of practice in the application of LRFD in the design of geotechnical facilities. Because the implementation into U.S. highway practice is just beginning, a large experience base does not exist. However, it was useful to determine both existing experience and plans for future implementation. A questionnaire was circulated in July 1997, to transportation departments in all states and Canadian provinces. All of the responses were received by October 1997. A copy of the Questionnaire is included in Appendix A.

Thirty-eight responses were received from state DOTs and six from Canadian provinces. Some of the questionnaire responses from state DOTs can be easily tabulated and those results are given in Table 1. Additional tabular information is contained in Appendix A. More than 50 percent of the responding agencies had specific plans for either experimenting with or implementing LRFD. Six states, Arkansas, Colorado, Massachusetts, Michigan, Oklahoma, and Washington, had made substantial progress and had projects underway. Three states had committed to immediate implementation (Colorado, Oklahoma, and Pennsylvania) and four more implemented in 1998. Five states had research studies underway. It should be noted that the implementation of design code changes is usually controlled by the State Bridge Engineer. Superstructure design may occupy most of his or her attention, causing the structural implementation to be generally further advanced than geotechnical.

TABLE 1
QUESTIONNAIRE RESPONSES

States with projects underway or completed	6
No. of projects underway	~70
No. of states starting 1977 or earlier	3
No. of states planning to start in 1998	4
No. of states planning to start in 1999	3
No. of states planning to start in 2000 or later	17
No. of states currently implemented (Colorado, Oklahoma, and Pennsylvania	3
No. implementing in 1998	4
No. implementing in 1999	5
No. implementing 2000 or later	7
No specific implementing plans	18
No. of states responding to research	5

Fifteen responses contained useful written comments that do not lend themselves to tabulation. Many of the comments related to the current LRFD Bridge Code or dealt with similar topics. Some of them have been combined and paraphrased (Appendix A).

Replies were obtained from six Canadian provinces and all but one of them, British Columbia, implemented LRFD several years ago. British Columbia indicated that they have no plans to implement. The general tone of the comments from the other provinces was that they had relatively little difficulty in implementing the process. In general, they indicate that the resulting designs have become more conservative and also more costly. Specific comments are paraphrased in Appendix A.

The problems of implementation of LRFD for geotechnical facilities in the United States can be grouped into two categories. First, there is a general and sometimes very strong reluctance to change the design procedure. It is argued that little is to be gained by changing, the design process will be more difficult, and more design time will be required. The second reason for the reluctance is related to a variety of technical concerns for the current state of the specification. For example, several comments were concerned with the resistance factors used for driven pile design and the difficulty in matching them to modern design practice. These concerns are numerous and many are well founded. There were very few comments from either the United States or Canada regarding the use of LRFD for earth retaining structures, culverts, or slope stability. This may be due to the fact that designers had concerned themselves primarily with foundations in the early stages of implementation.

There is a serious, long-term problem in the implementation of LRFD in geotechnical engineering. The education of geotechnical engineers strongly emphasizes the evaluation of soil and rock properties. This is a natural approach because properties can be difficult to characterize. The result is that little time is spent on design and the design process does not receive the emphasis that it does in structural engineering education. The basic design concepts (e.g., ASD, LRFD, limit states) are not presented and the geotechnical engineering student will only receive this background if he takes structures courses.

There are two clear advantages to the use of LRFD. First, the uncertainty in performance evaluation is recognized to be made up of two parts, one concerned with loads and the other with strength (performance). Where the loads are defined by the structural engineer, it is inappropriate for the geotechnical engineer to define the total safety factor. LRFD is very effective in this case since the structural engineer will be concerned with the determination of the factored load for the various load combinations and the geotechnical engineer will define the resistance factor. In the case of deep foundations, earth retaining structures, and slope stability designs, the strength as it is currently determined for ASD is an ultimate strength and the geotechnical engineer does not have available a rationally determined allowable stress. Problems arise when the loads are defined by the soil pressures. It is then difficult to separate load factors from resistance factors so a calibration of the resulting design to current ASD must be used.

The second reason for using LRFD is that if the loads are defined by the structural engineer they will be factored loads, not working loads. Thus, for all cases where a structural design is involved it will be much simpler if LRFD is also used by the geotechnical engineer. For example, with the AASHTO LFD Code or the ACI Building Code the structural design is completed with factored loads but then the foundation design must be done with allowable stresses, or factors of safety.

In the immediate future resistance factors will be calibrated by comparison with factors of safety traditionally used in ASD. One would expect that this type of calibration, correctly used, would produce designs similar to the equivalent ASD-based design and of a similar cost. Therefore, it cannot be claimed that the use of LRFD will reduce cost. However, the LRFD system provides a rational, probability-based approach to improving future design codes and this may produce cost savings in the future.

CHAPTER TWO

HISTORICAL PERSPECTIVE

LRFD FOR STRUCTURAL DESIGN IN THE UNITED STATES

The earliest use in routine design practice of the methods that became known as LRFD was in the American Concrete Institute (ACI) "Building Code Requirements for Reinforced Concrete," adopted in 1956 by ACI Committee 318 *(ACI 1956)*. In that document, the LRFD approach was permitted for the design of reinforced concrete buildings as an alternate to working stress methods. At that time, the design method was called "Ultimate Strength Design," but it bore a strong similarity to the LRFD method. The document was very brief, fully contained in just over five pages of a 6" by 9" format. The Ultimate Strength portion of the 1956 ACI Code was not widely used.

The motivation for the ACI to change to Ultimate Strength Design came primarily from problems associated with the design of reinforced concrete elements in which reinforcing steel was subjected to compression (e.g. columns, and beams with compression reinforcement). In that case, with the assumption of a linear elastic stress distribution, the force in the reinforcement that is loaded in compression will be seriously undervalued. This problem arose because the stress in the compression steel increased due to shrinkage and creep in the concrete, i.e. time dependent, nonlinear effects. The ASD versions of the ACI Code had to use arbitrary and awkward procedures to attempt to achieve satisfactory results in this case. However, the ultimate strength of the section could be calculated using a simple rectangular compression stress block calibrated by available test data. But, this process required the use of ultimate loads rather than working loads.

In the 1956 ACI Code, resistance factors were not present, so all of the safety factor was embedded in the load factors. However, the load factors were different for different load types and also for different load combinations. The methods to determine the nominal section strength were specified and the strength was required to be greater than the factored load applied to the section.

The factored load was specified to be:

1. For structures located such that wind and earthquake loading could be ignored, the larger of:

$$1.2D + 2.4L$$
$$K(D + L)$$

2. For structures with wind and earthquake loading, the largest of:

$$1.2D + 2.4L + 0.6W$$
$$1.2D + 0.6L + 2.4W$$
$$K(D + L + 0.5W)$$
$$K(D + 0.5L + W)$$

where D is the dead load, L is the specified live load, and W refers to the specified wind or earthquake load. The constant K was specified as 2.0 for members subjected to combined bending and axial loads and 1.8 for members subjected to bending only. The effect of the difference in K is similar to a resistance factor since members subjected to bending only are allowed to have larger applied loads than columns.

In the next version of the ACI 318 Code *(ACI 1963)*, a complete LRFD format was used, including resistance factors, and LRFD was put on an equal footing with working stress methods. The design method was still known as Ultimate Strength Design but in format it was identical with LRFD. The LRFD portion of the document was complete and was brought into practice very quickly as a result of a major educational program organized by the Portland Cement Association. Most structural engineering firms involved with building design in the United States had implemented the new code by 1965.

Resistance factors were included on the strength side of the basic design expression in 1963 with values that are essentially the same as those used by the ACI Code today. In the 1969 version of the ACI Code *(ACI 1969)*, separate considerations were included for deflections and cracking and they were based on an analysis at the service load condition. Thus, the final form of LRFD was reached, including serviceability limit states. The old allowable stress design requirements were relegated to an appendix and were later dropped completely. It is important to understand that both the load and resistance factors were selected by the intuition and judgment of ACI Committee 318. No formal analysis was reported but extensive comparative designs were completed.

A landmark paper was published by Cornell in 1969 *(Cornell 1969)* proposing that probability theory be used as the basis for a design code. The ideas contained in this paper had grown from the work of Freudenthal on the application of reliability theory in structural analysis (see for example *Freudenthal et al. 1966*). Cornell outlined the framework of a code and described procedures that could be used to determine the required factors. While some changes were made in the code format, this paper is still an excellent reference for the engineer who is interested in gaining a deeper knowledge of the theoretical basis for LRFD.

A major study was performed by the National Bureau of Standards (now National Institute of Standards and Technology) and reported in NBS Report 577 *(Ellingwood et al. 1980)*. This study emphasized buildings and building loads. A set of load combinations and load factors were developed that were different from those used by the ACI Code. These factors were developed based on a probabilistic analysis and the report outlined how the specific factors were obtained for the design of buildings of both structural steel and concrete. This study is one of the most useful references available for the review of the fundamental probabilistic basis of LRFD. All of the basic theoretical concepts for the application of probability theory in design were presented in this report. The current plethora of papers that describe in abundant detail the fundamental theoretical basis for LRFD have contributed little additional basic information of significance to the topic.

The NBS Report 577 load combinations and factors were used by the American Institute of Steel Construction (AISC) when they adopted the LRFD format in 1986 *(AISC 1986)*. Before preparation of the AISC Code, however, an additional extensive calibration study was completed to determine values for resistance factors of the various steel structural elements *(Ravindra et al. 1978; Yura et al. 1978; Bjorhovde et al. 1978; Cooper et al. 1978; Hansell et al. 1978; Galambos et al. 1978)*. Many of the resistance factors were generated rationally by probabilistic analysis of available statistical data, providing a strong basis for the factors selected for use in the code. These papers provide an excellent guide to the general direction that should be taken in code development and illustrate the use of statistical data in determining rational resistance factors. The statistical data on steel element behavior was available for use in generating the resistance factors for the LRFD Bridge Code.

The AISC Code is a very concise, clearly written document. Beginning with earlier versions of the AISC Code, a systematic method of checking the code requirements, as developed by Goel and Fenves, "Computer--Aided Processing of Design Specification" *(Goel et al. 1971)* was applied. This procedure uses decision tables to check that all steps in the design process are covered by the specification and that there are no ambiguities. It may not be necessary to actually use formalized and automated methods but the concept may be usefully applied on a manual basis. With this type of check, requirements that overlap are found and conditions that are not covered can also be identified. It seems that these tools have not been used in geotechnical code developments.

The ACI Code of 1963, once adopted, was quickly implemented into practice. The column section was completely revised. On the other hand, the AISC LRFD Code of 1986 has been resisted by practicing engineers. Its usage is still not dominant after more than a decade of availability despite being taught in institutions of higher education since its adoption. Why has there been such difficulty in implementing the AISC Code compared with the dramatic success with the implementation of the ACI Code of 1963? It is hypothesized that the substantial improvement in strength calculation associated with the "Ultimate Strength Design" ACI Code may have been a major driving force favoring its use.

Today we have the undesirable condition that the two most commonly used codes for building structures in the United States use different load factors. A steel building on concrete footings would have the steel structure designed with different load factors than the concrete footings. If NBS Report 577 *(Ellingwood et al. 1980)* load factors were adopted for concrete buildings, the resistance factors would have to change from the currently used values. The most recent ACI Code *(ACI 1995)* allows the use of the NBS (AISC) load factors for structures with mixed materials (i.e. steel structures with concrete footings) and includes a set of resistance factors for concrete structural elements when using the AISC load factors. Eventually the same load factors will probably be adopted for the design of buildings regardless of the material type.

All of the code developments by ACI and AISC discussed above ignored foundation design requirements. Thus, it was necessary for the geotechnical aspects of the foundation design to be performed based on ASD while the structural aspects were based on LRFD. For example, in designing a spread footing, it is sized using allowable bearing pressures but designed structurally using LRFD. This condition has continued for 30 years in concrete building design.

The bridge design code adopted by AASHTO in 1977 contained a design procedure that was called load factor design (LFD). This was the only code so titled and it also contained ASD. Either method could be used in design. The ASD portion continued the code that had been used previously (with annual updates). The LFD portion was contained in the document together with the ASD. Both working loads and factored loads were included. In the LFD load combination IA, dead plus live plus impact, the dead load factor was 1.3 and the factor on live load plus impact was 2.86. Allowable stresses were specified in the concrete design section and resistance factors were also included. The resistance factors had the same values as specified in the ACI Code (*ACI 1995*). The section on prestressed concrete specifies that strength shall be calculated based on LFD loads and "behavior at service conditions" on ASD loads.

Kulicki described the procedures used in the preparation of the LRFD Bridge Code in "Development of Comprehensive Bridge Specification and Commentary" *(Kulicki 1998)*. This paper is useful because it presents the procedures used in the code development in considerable detail. A byproduct of this presentation is a discussion of probability analysis used to generate both load and resistance factors with emphasis on the particular problems of dealing with traffic loads. The method used for the LRFD Bridge Code is basically the same used in the development of the Ontario Bridge Code. This approach was developed by Nowak and Lind *(Nowak et al. 1979)*.

DANISH DEVELOPMENTS

At about the same time that the ACI Building Code with an LRFD basis came into being, a limit states code for geotechnical applications was being investigated at the Danish Geotechnical Institute. The concept was first suggested by Hansen *(Hansen 1953, 1956)*. This approach was used informally until 1966 *(Hansen 1966)* when it was adopted by the Danish Engineering Association. The resulting code recognized the existence of multiple conditions that must be considered in design and referred to them as limit states. Apparently, the name and the concept came from this effort, although the ACI Code had recognized the problem of multiple limiting design conditions by about the same time. The Danish Code used factors on both the load and the resistance. In the Danish application the resistance factors were applied to the soil properties rather than directly to the resistance, as has been done in the United States. These factors were derived from previous Danish practice and were adjusted when problems or failures were encountered and also when the total absence of problems indicated excessive conservatism. The Danish experience is very important in the development of, what is called here, LRFD.

Mortensen discussed the entire Danish development *(Mortensen 1983)* and concluded that "limit state design represents a logical calculation principal *(sic)*. It is not in itself a radically new method compared to earlier design practice, but presents a clearer formulation of some widely accepted principals *(sic)*." He further concluded, "Within geotechnical and foundation engineering, even our best methods for obtaining the necessary geotechnical data, and our best calculation methods, are inadequate to the point that our factors of safety act, to some extent, as correction factors. For that very reason the best way of determining our design criteria is a combination of experience and back-calculation of successful foundation constructions. This also applies if limit state design is used with or without the partial coefficient system. *If this fundamental fact is neglected, and design criteria are based on purely theoretical considerations, then there is a risk that the benefits of extensive practical experience gained within foundation engineering will be lost"* (emphasis added).

In summary, the Danish approach has been to use rather small factors on the load side of Equation 1. The resistance side of Equation 1 is assumed to be determined based on soil properties applied with some analysis model. The resistance factors are applied directly to the soil properties rather than to the nominal resistance, as shown in Equation 1.

Because of the extended time that LRFD has been used in Denmark the experience should be carefully considered. The topic is reviewed more extensively in Chapters 3, 4, 5, and 6 dealing with loads and particular types of geotechnical systems.

DEVELOPMENT OF THE THEORETICAL BASIS FOR LRFD

Beginning in the late 1960s, a theoretical basis for LRFD was developed in the United States based on probability theory. The fundamental concept held that, because neither the loads nor the strength (performance) are deterministic, it was appropriate to treat both load and strength as random variables and to develop an approach to the evaluation of structures based on probability theory. The concept is illustrated in Figure 1 where probability density functions are shown for both the load effect, Q, and the resistance, R. (Load effect refers to the load calculated to act on the particular element in question.) The area under the curve between two points on the abscissa represents the probability that the resistance will have a magnitude between the two values. This is represented in Figure 1 where the area A is the probability that the resistance will be between a and b. Probability-based design is founded on the concept that the design be selected so that the probability of failure is equal to, or less than, some prescribed value. The required failure probability is based on the analysis of the failure probability of successful elements and systems.

The load effect in Figure 1 has been shown much narrower than the resistance for illustrative purposes, indicating that, in this case, the load effect has less variability. This variability is measured by the standard deviation of the distribution. (The standard deviation is not shown here.) In Figure 1, the mean values are denoted by $\overline{Q}$ and $\overline{R}$. The

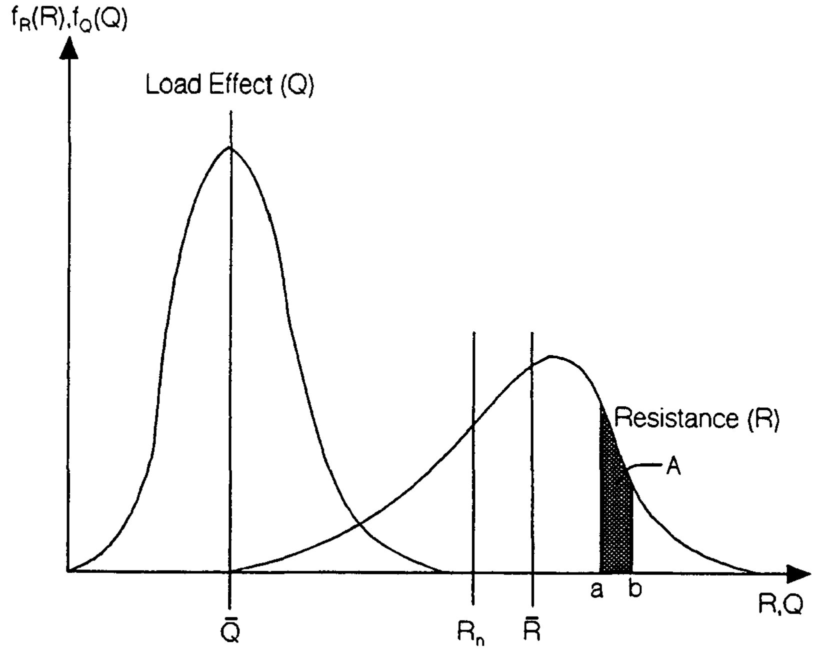


FIGURE 1 An illustration of probability density functions for load effect and resistance.

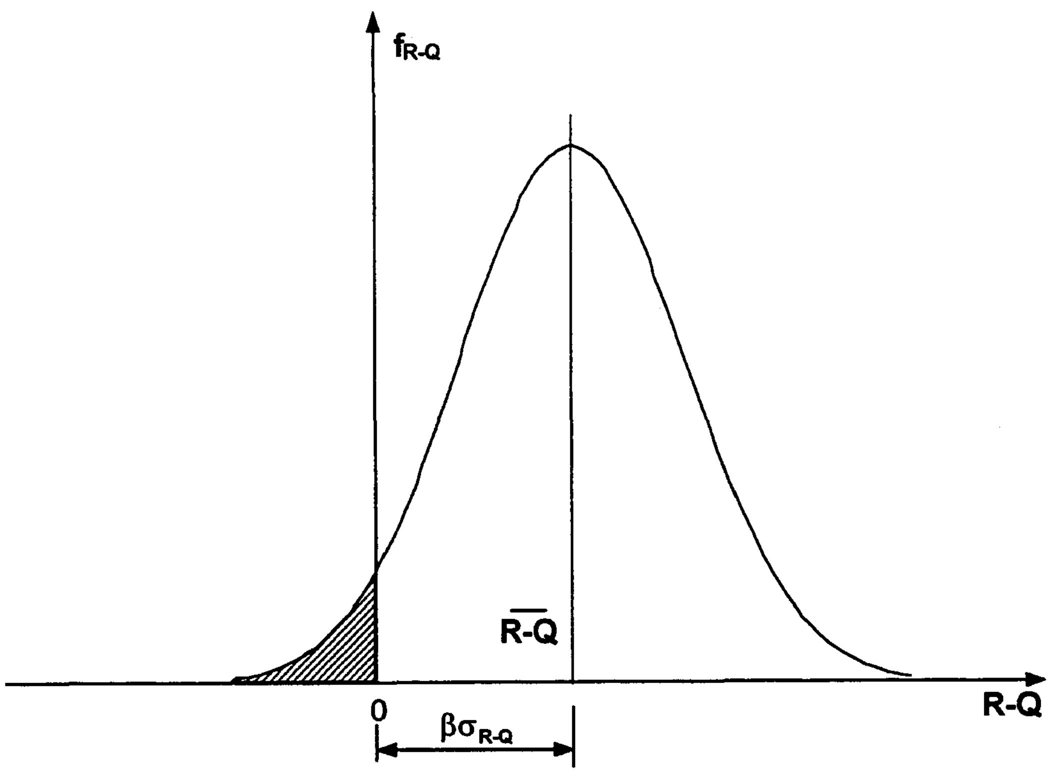


FIGURE 2 An illustration of a probability density function for R - Q.

nominal strength, R_n as shown is not necessarily the same as the mean strength illustrated in Figure 1 but is, rather, the strength determined by the prescribed method.

If distributions are available for both the load effect and the resistance, then the probability of failure can be determined directly. One approach that has been used is to consider the combined probability density function for R - Q and this is illustrated in Figure 2. Failure is defined when R - Q is less than zero and the region is shaded in Figure 2. The basis for design can be to require that the mean of the distribution, $\overline{R - Q}$, be greater than the value of R - Q = 0. Usually the mean value is used to define the reference value and the distance of the mean above zero is

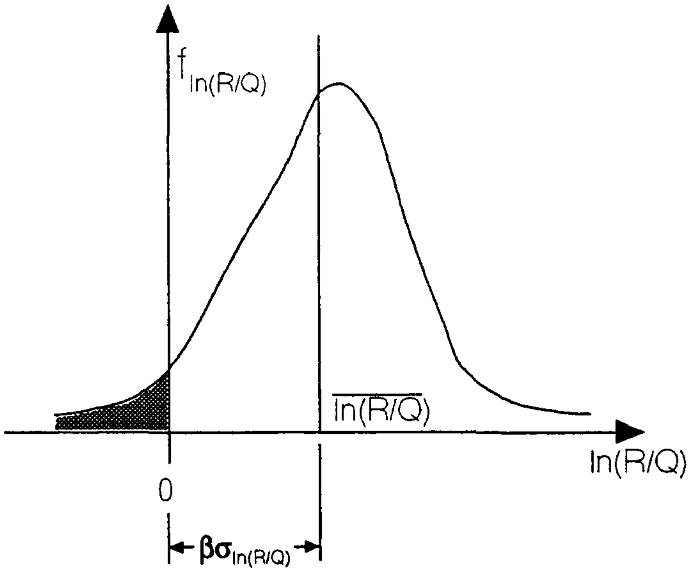


FIGURE 3 An illustration of probability density function for ln R / Q.

taken as a multiple of the standard deviation of the distribution. The multiple of the standard deviation, shown here as β, is called the safety index or reliability index. In this report the term safety index will be used.

Another approach, illustrated in Figure 3, is to use the ln R/Q and then the limiting failure condition occurs when $R/Q = 1$, or ln $R/Q = 0$. The safety index is defined as before.

The values for the load and resistance factors are selected so that the safety index has a specified value. The calibration of a particular LRFD code requires that existing, traditionally executed designs be analyzed to determine the safety index. With extensive analysis of existing structures it is possible to characterize values of the safety index for successful structures and those studies have shown that common safety indices, β, are typically 3.0 to 3.5.

Values for load factors were selected to represent the differing variability of load types. Load factors have ranged from about 0.5 to 2.1 depending on the load type and the load combination. With the load factors established, resistance factors were selected to satisfy the requirements for proper safety indices. The structural load factors and load combinations have been established for the LRFD Bridge Code after an extensive study, completed as part of NCHRP Project 12-33 *(Kulicki 1998, Nowak nd)* and it can be anticipated that those factors and combinations will not change to any substantial and fundamental degree. The methods used to generate load and resistance factors have been discussed by Kulicki (*Kulicki 1998*). Resistance factors must be used that will give reasonable safety indices consistent with current practice. It can be anticipated that additional investigations may be necessary in this area.

Those interested in a more thorough discussion of probabilistic methods applied to the development of load and resistance factors are referred to NCHRP *Research Results Digest 198* (*Kulicki 1998*). The development of the load and resistance factors for the LRFD Bridge Code is discussed in some detail. For further information and a somewhat more complete theoretical presentation, the reader is referred to Nowak and Lind (*1979*) and Ellingwood et al. (*1980*).

Load and resistance factors can also be calibrated based on a direct comparison with existing design practice. Since load factors have been selected for the LRFD Bridge Code, resistance factors can be developed to result in designs similar to those of the current ASD Code. This process is illustrated in Figure 4, where resistance factors are shown for various factors of safety as a function of live load to dead load ratios. Of course, the designs will not *all* be identical but may be more or less conservative depending on the live load to dead load ratio. This approach requires knowledge of the load factors to determine resistance factors for particular live load to dead load ratios.

Because different load factors are used for dead and live load, the equivalent ASD safety factor is dependent on the ratio of live to dead load. In this figure, the load factors used are those for the limit state, Strength I in Section 3, Table 3.4.1-1 of the LRFD Bridge Code. Only the load factor 1.25 was used for dead load and 1.75 for live load and other load types in the Strength I case were ignored. This simplification makes it possible to show the relationships easily. For example, for a live to dead load ratio of 0.2, a safety factor of 2.0 is equivalent to a resistance factor of 0.67. Using this approach, resistance factors can be compared with current practice for a range of factors of safety and live load to dead load ratios. The example shown here was included only as an illustration and the curves of Figure 4 should not be used without careful study of the particular case in question.

Another approach to code calibration is to perform comparative designs. Particular conditions are selected and a design is completed using the current ASD code and proposed new code. In this way the behavior of the proposed code can be most realistically evaluated (*see Mortensen 1983*).

MODERN GEOTECHNICAL IMPLEMENTATIONS OF LRFD

Over the past two or three decades there has been a general, worldwide move toward the use of LRFD in the design of structures, including foundations and earth retention systems. In the United States, this development has been instigated by structural engineers but in some

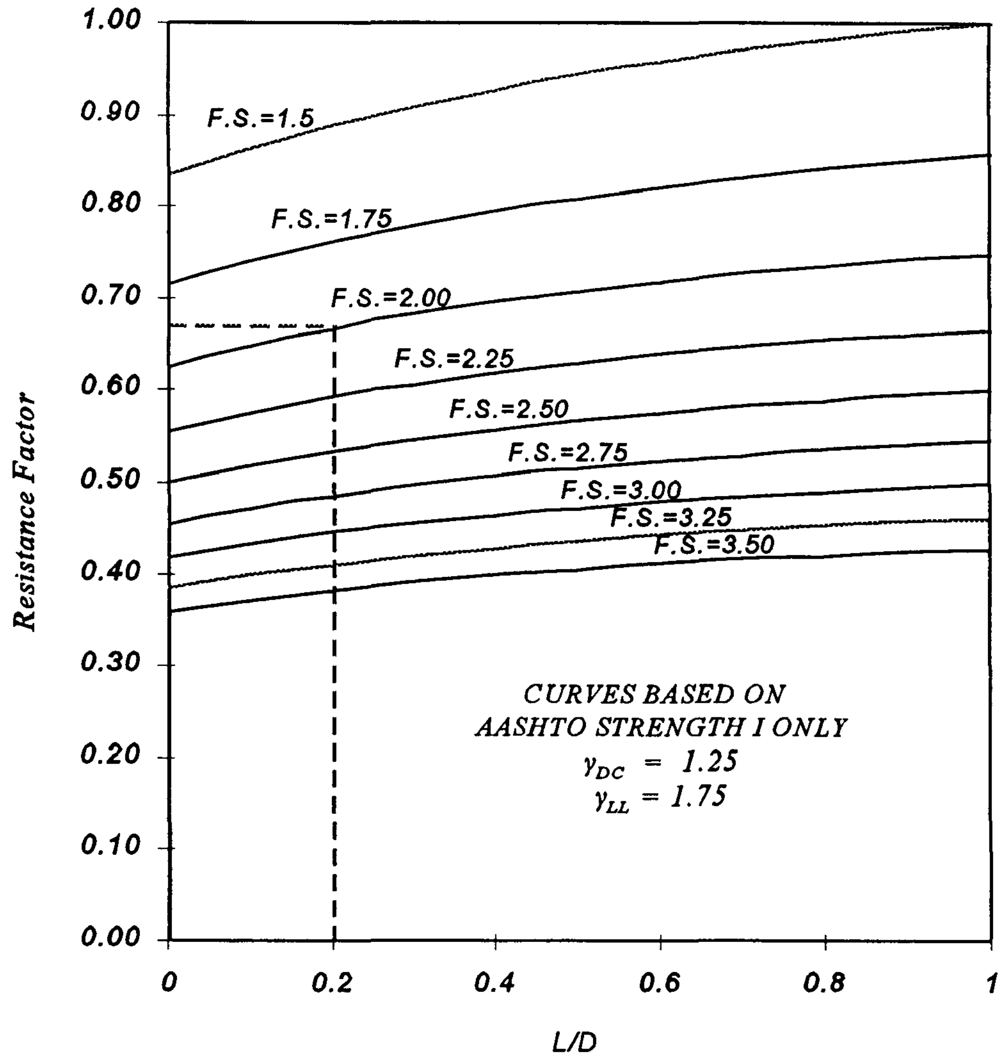


FIGURE 4 Resistance factors for different factors of safety as a function of live load-dead load ratio. (This example is only intended to be illustrative and should not be used in design.)

other countries the impetus has come from the geotechnical area as indicated in the preceding section. The development of various codes pertaining to geotechnical design is summarized briefly here. The detailed aspects of these codes will be discussed in subsequent chapters in the discussion of the various design elements.

Ontario Bridge Code

The province of Ontario adopted LRFD for bridge design in 1979 with the publication of *Ontario Highway Bridge Design Code and Commentary*, after having traditionally used the AASHTO Bridge Code. In Canada, the term limit states design (LSD) is normally used instead of LRFD. This development was part of a much larger change in bridge design and operation philosophy. Ontario made a major effort to modernize bridge design, including important changes in allowable truck weights based on studies of existing trucks using the Ontario highways. The 1979 Code with Commentary was not used extensively *(personal communication, Ontario bridge engineers, March 1998)*.

In 1983, the second edition of the LSD Code with Commentary was adopted in Ontario *(Ontario 1983)* and its use became mandatory. This Code was developed based on a safety index of 3.5 at least for superstructure elements. The results of the usage in the geotechnical area were not encouraging in that foundation elements generally became larger, implying that the designs were more conservative.

The third edition of the Ontario Bridge Code was adopted in 1992 *(Ontario 1992)* with Commentary and its use indicates that the designs seem to be more reasonable but probably still more conservative than the previous AASHTO-based designs using ASD. In the opinion of the Ontario bridge engineers, the designs are better balanced and the structures are designed to higher standards. This illustrates the point that, in comparing the load and resistance factors in different design codes it would be desirable if design load magnitudes relative to actual load were also compared. In bridge design, this is very difficult since the truck loads actually crossing bridges are dependent on the permit load policies and even on truck load enforcement in the appropriate governmental jurisdictions.

When the third edition of the Ontario Bridge Code was adopted, a 3-day seminar was held to educate designers in its use. In 1998, a new Bridge Code was to be adopted and implemented. More extensive educational activities are planned to assist the designers in its implementation.

Canadian Bridge Code

Most of the other Canadian provinces have gradually adopted the Canadian National Bridge Code *(Canada Standards Association 1992)* over the past 10 years. This code is written in an LRFD format and was adapted from the Ontario Bridge Code. However, there are substantial differences between the two documents. The questionnaires indicated that most Canadian provinces have gradually implemented the method into their practice with varying degrees of difficulty. Extensive studies were performed and committees were constituted to review the results to assure that the best possible end result was achieved. Responses to the questionnaire indicated that changes have occurred in the Code since it has been introduced and further changes will probably occur.

The geotechnical part of this document is quite brief, consisting of about 20 pages of code. However, it also includes a commentary.

Canadian National Building Code

The structural design part of the Canadian National Building Code is LRFD-based (*National Research Council of Canada 1995*). This code covers all aspects of building design such as fire protection, plumbing, safety measures, etc. and structural design including foundations is only a small part. The structural design part does include loads, load combinations, and load factors. It also includes load combinations for ASD. Resistance factors are not included, but in some cases, reference is made to other, materials-oriented design specifications that include resistance factors. No reference could be found to a geotechnical design specification, however, designers use the Canadian Foundation Engineering Manual (*Canadian Geotechnical Society 1992*), an extensive document covering all aspects of concern in foundation design.

Florida Department of Transportation

The Florida Department of Transportation has prepared modifications to LRFD Bridge Code *(Florida DOT 1997)* to deal with problems that they encountered in practice. They performed calibrations of their current ASD practice to the LRFD format using the new AASHTO load combinations and load factors. This document makes an important contribution since it represents the thoughts of a progressive geotechnical organization in a transportation agency and it deals with their response to problems they saw with the LRFD Bridge Code.

Eurocode

The countries of the European Community have joined together to prepare a standard design code for the construction (buildings and transportation) that takes place in those countries. A document has been adopted for geotechnical design *(European Committee for Standardization 1994)* but the extent that it has been implemented seems to vary widely. This has been a major effort with contributions from some of the leading geotechnical engineers in Europe.

Eurocode 7, Part 1 consists of more than 100 pages of requirements that are stated in a general form. In only a few cases are specific requirements included. It is written without a Commentary, although much of the information in the code would appear in a Commentary in North American practice. Further design and construction requirements are given in Part 2, Design by Laboratory Testing and Part 3, Design by Field Testing. Other documents have been developed to meet the needs of particular systems, such as sheet piles and concrete piles. Eurocode 3 contains a section concerned with the design of steel piling *(ECS 1994)* and loads and load factors are contained in Eurocode 1 *(ESC 1995)*. In total, the document is very large and a complete evaluation is beyond the scope of this Synthesis. The Eurocode 7 resistance factors were not generated from a reliability analysis but were selected to fit current practice.

The Eurocode 7 has been developed to allow a gradual implementation by the various member countries. In the beginning, the framework will be used by members with specific requirements defined by the individual countries. For example, the constants required in the specific code limitations are bracketed, indicating that specific values can be established by the national codes. Probably, the national codes will continue to take precedence but they will eventually begin to converge on the Eurocode somewhat like in the United States where individual state DOTs control their bridge codes and may make some changes from the AASHTO Code.

There is, however, a strong resistance to the use of the Eurocode for foundations (*Stocker 1997*). There are many well-developed but different geotechnical practices in

Europe, each developed to meet the needs of particular countries. A great deal can be learned by a careful study of this total document.

Danish Code of Practice for Foundation Engineering

This Danish Code *(Danish Geotechnical Institute 1985)* is the successor to the original code developed at the Danish Geotechnical Institute (*Hansen 1966*). It deals with the design of both shallow and pile foundations and, in addition, specifies procedures for establishing earth pressures. Structures are divided into three foundation classes—low, normal, and high—based on the nature and size of the structure, conditions with regard to adjacent structures, soil conditions, and groundwater conditions. Procedures to be used in subsurface investigations and design are outlined. Methods to be used for establishing soil properties are discussed and ranges of values are stated. In addition, presumptive values of properties to be used in preliminary design studies are provided.

This design code is of particular interest because it has been in use for an extended period of time and probably has been modified to deal with problems encountered in practice. It should be noted that the soils in Denmark are of limited variety compared with conditions in the United States.

1992 AUSTROADS Bridge Design Code

The AUSTROADS Bridge Design Code *(AUSTROADS 1992)* governs the design of bridges for all of Australia. AUSTROADS is an association of state, territory, and federal road and traffic authorities in Australia that seems to be similar to AASHTO in the United States. This Code is written in an LRFD format and contains a foundations section that also includes the design requirements for abutments, retaining walls, culverts, and anchorages.

Australian Code

A design standard was adopted (*Standards Association of Australia 1995*) for all geotechnical facilities except highway work. This standard is LRFD based and concisely written. When development of this standard began, the intention was to generally follow the Eurocode but as the writing effort proceeded the final style differed substantially. This code has now been broadly implemented in Australia. Future versions of the AUSTROADS Code will use the requirements of the Standards Association of Australia Code by reference for geotechnical facilities *(personal communication, Julian Seidel, Professor, Monash University, Melbourne, Australia)*.

American Petroleum Institute

An LRFD-based code was adopted for the design and construction of offshore platforms by the American Petroleum Institute (API) in 1993 *(API 1993)*. This standard generally follows the LRFD approach with load factors that are similar in magnitude to values used in other design codes. Considerable attention is devoted to loads characteristic of the ocean environment (e.g. waves, wind). On the resistance side the primary implementation in the geotechnical area is for driven piles since almost all offshore platforms are supported on piles.

Transmission Tower Reliability-Based Design Method

The Electric Power Research Institute (EPRI) developed a transmission tower design code for drilled shaft foundations primarily by contract with Cornell University in a project directed by Dr. Fred Kulhawy *(Phoon et al. 1995)*. Due to changes in the operation of EPRI the code is now a proprietary document and is available only at high cost. It was not acquired for the purposes of this study. However, Kulhawy's work that is published in the open literature was reviewed and it will be discussed in the appropriate sections of this report.

CHAPTER THREE

LOADS, LOAD FACTORS, AND SOIL PROPERTIES

INTRODUCTION

The load and resistance factors of LRFD have replaced the safety factors that geotechnical engineers have traditionally used in ASD. In structural design by ASD, allowable stresses were normally used instead of a safety factor. These stresses were selected and codified based on tradition and experience, and the factor of safety was not visible to the designer. By comparison, allowable stresses were not established for most aspects of geotechnical design. Instead, strengths were determined and a factor of safety was applied. Thus, in the geotechnical area, there has been a tradition of evaluating the acceptability of a design based on a calculated strength together with a factor of safety. When the conversion is made to LRFD, it is desirable that the design process produce designs that are similar to those produced by ASD using a safety factor. This implies that the sum of the influence of the LRFD load and resistance factors should be equivalent to the ASD factor of safety.

When loads originating from the geotechnical aspects of the design are involved the problem is not as clear. Loads coming from soil pressure are determined from the soil properties but so is the nominal resistance. It then becomes difficult to select the load factors as a separate exercise from the resistance factors. For instance, did the failure occur in an earth retaining structure because the loads were incorrectly estimated or because the strength was inadequate?

The load factors and load combinations associated with structural loads for the LRFD Bridge Code have been established and reported in NCHRP 12-33A *(Nowak nd)*. The loads and load factors for earth pressures have also been established and the basis for their selection was presented in NCHRP Report 343 (*Barker et al. 1991*). The probabilistic analysis used by Nowak in NCHRP Project 12-33A has been based on the load combination that most often controls the design. This combination is the one that considers the action of gravity on the structure and its primary functional loads, i.e. dead load plus live load. In a building, the loads that most frequently control the design are the structural dead load combined with the live loads coming from the building function; for a bridge the critical load combination is usually dead load plus the traffic load, the combination known as Strength I in the LRFD Bridge Code.

During the examination of several different LRFD codes it was observed that some of the resistance factors for a particular limit state had values varying over a considerable range among the different codes. Some of this variability may have its source in differences in the load factors between codes. As an example, the current ACI code *(ACI 1995)* specifies a different set of resistance factors for the two different sets of load factors that are permitted by the code.

To make the comparison of resistance factors easier, the load factors and load combinations from several codes have been collected. These load sets are included here as Appendix B. The most important and commonly limiting load combination is dead plus live load. The load factors for this combination are given below for the codes that were reviewed. Of course, the specified structural loads themselves may be different in different codes. What is the relationship between building live loads and highway traffic loads? What is the relationship between traffic loads, for example, in Ontario and the United States? Are they more or less conservatively defined?

LOAD FACTOR SUMMARY

For purposes of comparison among the structural loads for the various codes, the primary strength load combination for each of the codes that were reviewed are summarized below.

Comparison of Strength Loads by Design Code

Code	Load
AASHTO LRFD Bridge Code[1]	1.25D + 1.75L
ACI 318-95	1.4D + 1.7L
AISC and NBS 577	1.2D + 1.6L
Ontario Bridge Code[1]	1.2D + 1.4L
Canadian Bridge Code	1.2D + 1.6L
Eurocode[2]	1.35D + 1.5L
Danish Foundations Code	1.0D + 1.3L
Australian Code	1.25D + 1.5L
API Code[3]	1.3D + 1.5L

[1]There is a variety of load factors for the various dead load types.
[2]This is a considerable simplification of the Eurocode specification on loads and load factors. The individual countries retain control of some of the specific design requirements. In addition, the definition of loads, load combinations, and load factors is quite complex.
[3]The code of the American Petroleum Institute is concerned with the design of offshore petroleum recovery platforms. As such, the primary emphasis in the loads section is placed on wind and wave loadings.

The summary table shows the rather large variation of load factors now in use in the LRFD codes that were reviewed. The load factor on dead load ranges from 1.0 in

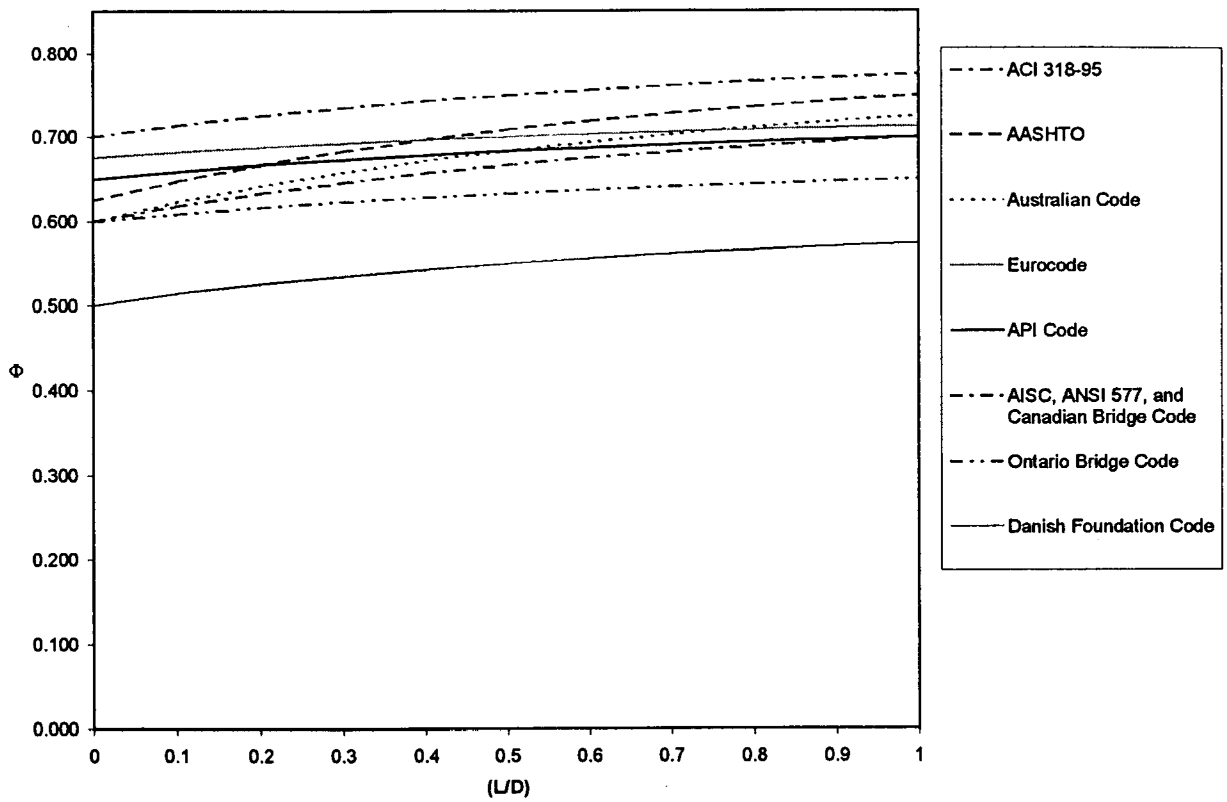


FIGURE 5 Resistance factors as a function of live load-dead load ratio for a factor of safety of 2.0 for the codes reviewed. (This summary of load and resistance factor relationships for a specific factor of safety is only intended to be illustrative and should not be used in design.)

the Danish Code to 1.4 in the ACI Code. The most common value is 1.2 but the Eurocode uses 1.35, almost as large as the ACI Code. The range of values for load factors on live load extends from 1.3 in the Danish Code to 1.75 in the LRFD Bridge Code. Curves are shown in Figure 5 of the dead load and live load combination for each of the codes reviewed. This figure shows the relationship between live load-dead load ratio and resistance factor for a factor of safety of 2.0. It can be seen that the difference in resistance factor for a factor of safety of 2.0 can be as much as 40 percent (between ACI 318-95 and the Danish Code).

It is quite understandable that load factors for live loads could be quite different among the various codes since the live loads could be more variable in one application than another. The same conclusion cannot be reached with regard to structural dead load. The range of values of the load factors applied to dead load should be very narrow since the structural dead load should have nearly the same variability in all cases. A difference of 1.0 to 1.4 for the Danish and the ACI Code is quite large.

LOAD FACTORS DISCUSSED

The loads and load factors for the *AASHTO LRFD Bridge Design Specifications (AASHTO 1994)* were developed during the course of NCHRP Project 12-33. The load factors were finalized by the Calibration Task Group under the chairmanship of Professor Andrezj Nowak. This work used probability analysis to determine load factors for the most common load combinations using a safety index, β, of 3.5 (*Nowak nd*). As part of the same study, resistance factors for several structural failure modes were also found for common bridge girder types. The procedure used for generating both load and resistance factors is discussed in NCHRP RRD 198 (*Kulicki 1998*). These load factors remained essentially unchanged in the 1997 Interim of the AASHTO LRFD Bridge Code and that information is presented in Appendix B. It is reasonable to assume that the structural load factors and load combinations are now essentially finalized. However, in the Suggested Research Section of the Nowak Committee Report, it was recommended that the statistical data be verified and *the calibration be performed for substructure design.*

One of the primary geotechnical serviceability conditions is settlement. The determination of settlement for noncohesive soils is discussed and example problems are solved in the *FHWA Driven Pile Design Manual (Hannigan et al. 1996)*. These settlements should be calculated for a realistic combination of loads including selected live loads because the settlement response is quite rapid in these soils and it may be increased by vibration due to traffic. However, since the response is rapid and the magnitude usually small, settlement calculation is often less critical than for cohesive soils. Because settlement of cohesionless soils occurs immediately as the load is applied, minor changes can be made in the elevations of the structure during construction that compensate for some of these rapidly developing settlements. It may be useful to study examples of typical structures on soils of this type to evaluate the importance of settlement in noncohesive soils and it may be possible to ignore the calculation of settlements in cohesionless soils in some cases. There are cases where settlement is critical in noncohesive soils so the problem must be considered. The LRFD Bridge Code does not include a load combination to deal with this case.

In cohesive soils, settlement occurs much more slowly and is often of larger magnitude than in noncohesive soils. Because settlement is slow, only permanently applied loads need to be considered in settlement calculations. Section 10.6.2.2.2 of the LRFD Bridge Code notes that "Time dependent settlements in cohesive soils may be determined by using the permanent loads only." However, there is no serviceability load combination for evaluation of settlement in cohesive soils among those of Table 1-B in Appendix B.

It is of interest to note that downdrag load on piles (negative skin friction) is contained in all of the Strength and Extreme Event load combinations and a load factor of 1.8 is specified in Table 2-B of Appendix B, taken from the LRFD Bridge Code. This load factor seems to be quite large compared with current practice. Further study is appropriate to review the load combinations contained in the LRFD Bridge Code with particular regard for foundation loads and also the load factors in these combinations, particularly the downdrag case.

EARTH PRESSURE LOADS AND LOAD FACTORS

The loads for earth retaining structures are defined quite specifically in the LRFD Bridge Code. In addition to loads coming from structural dead and live loads that might be typical of bridge abutments, there will be earth pressure loads. Sections 3.11.1 to 3.11.8 of the LRFD Bridge Code present specific methods for calculating earth pressures for the common possible cases that must be dealt with. These pressures should be design pressures since a load factor is applied to them. Thus the methods that have been commonly used for calculating earth pressures can be used with appropriate load factors.

The load factors for earth retaining structures from the LRFD Bridge Code are given in Table 2-B and the load combinations in Table 1-B. They are different from the LFD values of the earlier LFD AASHTO Code *(AASHTO 1977)*. They were developed in the study by Barker et al., reported in NCHRP Report 343 *(Barker et al. 1991)*.

SOIL PROPERTIES

The definition of soil properties for use in design has traditionally been left up to the designer in United States practice. Based on information from subsurface exploration it is common that several different sets of test results may be available from either laboratory or in situ tests all coming from the same strata. The LRFD Bridge Code does not give the designer any guidance as to the approach that should be used in selecting the design value. In some cases, there may be a considerable range in the available data so the resulting loads or resistance's could vary considerably based on the judgment of the engineer selecting the design soil properties.

Eurocode 7 makes a specific suggestion for dealing with this problem. In Section 2.4.3 *(ECS 1994)* it states "If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of a limit state is not greater than 5%." The other LRFD Codes that were reviewed did not make recommendations regarding procedures to be used in selecting material properties. Geotechnical engineers may argue that such decisions should be left to the judgment of the engineer. On the other hand, some guidance in this area would be useful to engineers to aid in producing designs that have a common basis. If load and resistance factors are developed based on the assumption that mean values of soil properties are used by the engineer when in fact conservative values are actually selected, then excessively conservative designs could result. If the opposite were the case the resulting design could even be unsafe. Perhaps in the development of geotechnical codes this issue should be studied.

The Danish Foundation Code specifies in Section 4.3 that water pressures be selected so that the assumed value would not be exceeded with a 98 percent probability in one year.

CHAPTER FOUR

LRFD FOR FOUNDATIONS

INTRODUCTION

Chapter 2 presented the history, framework, and philosophical basis of LRFD and the procedures used to determine the necessary factors were described. Chapter 3 discussed the loads emphasizing some of the details of load factor selection and gave a summary of the load factors used in various LRFD design codes. In this chapter, the specific details for LRFD applied to foundations will be discussed, with emphasis on the resistance side of the design expression of Equation (1). A discussion of the methods for determining nominal response (e.g. strength and serviceability) and selecting the associated resistance factors for foundations for the various limit states will be presented. Also, the geotechnical portions of the LRFD Bridge Code for foundations will be reviewed and compared with other such documents.

FOUNDATION DESIGN PROCEDURE

The LRFD process for foundations must be based on a clear understanding of the total foundation design and construction monitoring procedure. *The basic design approach will be the same regardless of whether ASD or LRFD is used, i.e. the same performance characteristics, such as strength and serviceability, must be determined in either approach and they are usually determined in the same ways.* The foundation design process has been studied and reported in "Design and Construction of Driven Pile Foundations, Workshop Manual" *(Hannigan et al. 1996).* The process can be summarized by the flow chart shown in Figure 6.

In Blocks 1-4, the design requirements are determined and the geotechnical information is collected. At the completion of this effort, the preliminary design requirements, including the structural and other superimposed loads, have been established and the subsurface information at the site has been collected. The decision must then be made regarding the type of foundation system, shallow or deep. In this section, the design of shallow foundations will be discussed so it is assumed that a shallow foundation is appropriate. Further steps in the design process for deep foundations will be discussed in those sections dealing with the individual deep foundation types.

SHALLOW FOUNDATION DESIGN PROCEDURE

The design procedure for shallow foundations is shown in Figure 6. Having selected a shallow foundation, the next step in the design process in Block SF2 of Figure 6 is to determine the resistance factor. It may depend on the method of capacity determination. If that is the case, when the capacity determination method is selected the resistance factor is known. Some design organizations may choose capacity determination methods that are not included in the code. Associated resistance factors will then have to be determined. The nominal bearing pressure is also found in Block SF2 of Figure 6. As previously defined, the nominal pressure (strength) is the pressure, calculated according to a specified or accepted procedure, that would result in failure (or exceeding a serviceability limit). The normal design process is to make the first footing size selection based on strength or the engineer's judgment so the nominal bearing pressure is determined first and then, from the nominal strength, footing dimensions can be selected. The procedures for determining allowable stresses for the design of footings are well established and are discussed in foundation design textbooks. The methods for determining allowable stress have been converted to nominal strength in NCHRP Report 343 (*Barker et al. 1991*) and these procedures are contained in the LRFD Bridge Code. Spread footing strength depends on the footing size and embedment depth. But footing size cannot be determined until nominal bearing pressure is known. Therefore, the design process is basically iterative.

Spread footing design is usually controlled by settlement limitations and many design organizations satisfy settlement limitation first and then check bearing capacity. In this case, settlement analysis is placed in Block SF4 and the bearing capacity check takes place in SF3. The procedures given in the LRFD Bridge Code will allow higher bearing pressures than have been used previously, so bearing strength will be the limiting factor even less frequently than is currently the case.

The problem with this aspect of the design process is that footing design has been traditionally performed by the structural engineer using presumptive bearing pressures or pressures determined by the geotechnical engineer using a geotechnical analysis. If the size of the footing must be known to determine the nominal bearing pressure, a very close cooperation between the structural and geotechnical

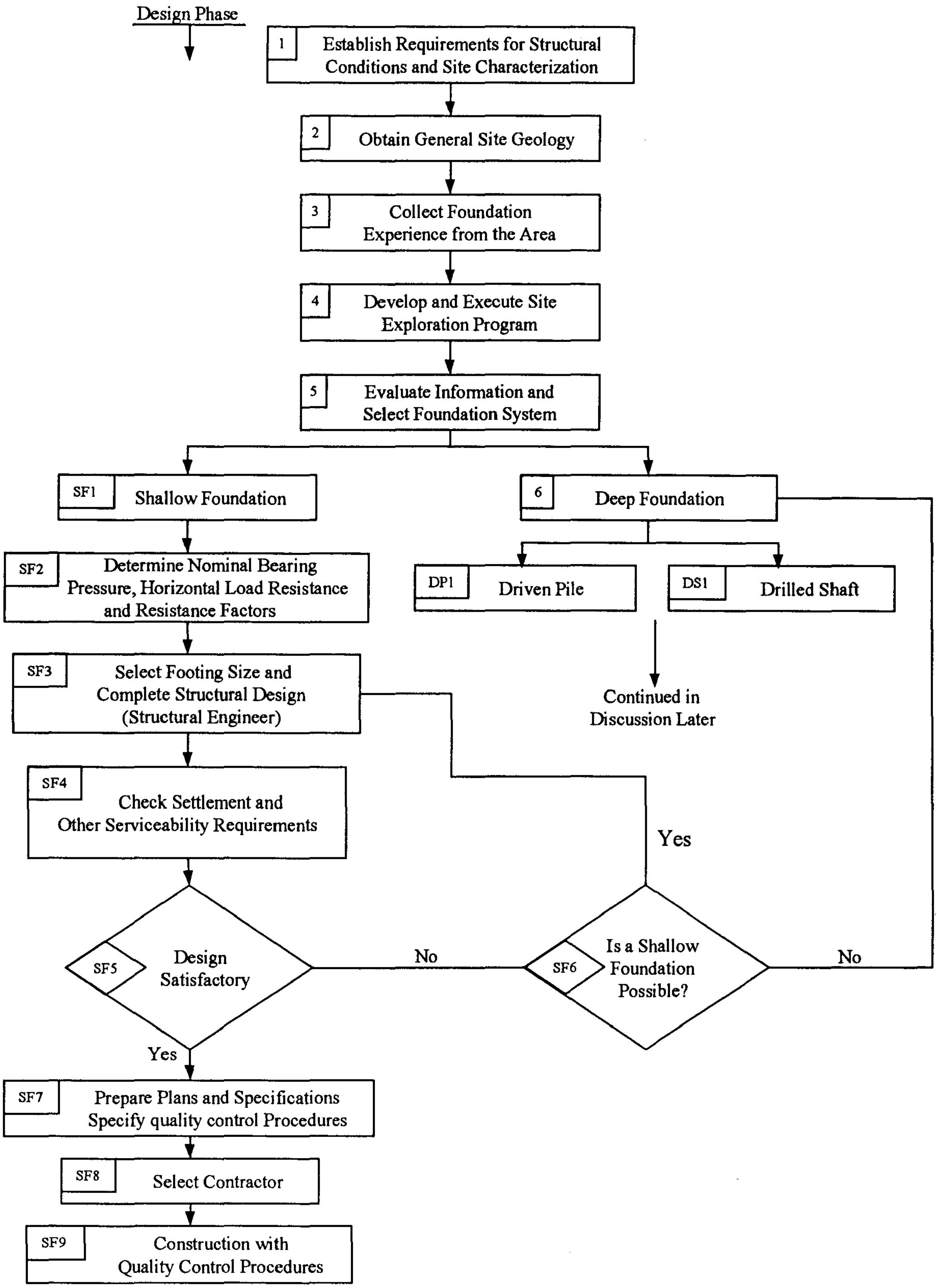


FIGURE 6 Flow chart describing foundation design process *(after Hannigan et al. 1996).*

engineer is implied. Some design organizations, including state DOTs, make it a practice to provide data on design bearing pressures as a function of the footing dimensions and depth. (In many cases, an appropriate depth will be defined by the soil profile but footing dimensions remain a variable.) Thus, a curve can be supplied to the structural footing designer expressing the nominal bearing pressure as a function of footing dimensions. Another approach is for the structural engineer to provide the geotechnical engineer the factored loads. The geotechnical engineer can then estimate the footing size and determine the appropriate required nominal bearing strength. Other organizations, where information is continually exchanged, depend on a close working relationship between the geotechnical and structural engineers.

After selection of the trial footing dimensions in Block SF3, horizontal loads and serviceability (settlement) requirements must then be checked by the geotechnical engineer in Block SF4. Like the strength condition discussed above, the footing response can only be determined after its size and elevation are known. Methods are well developed for determining settlement and horizontal resistance, but they must be applied after the footing size and depth are determined. Communication between the structural and geotechnical engineers is again necessary so that the appropriate checks can be performed, but the usual procedure is to develop the design based on strength and then adjust the design as necessary to satisfy the requirements arising from settlement, horizontal forces, and other requirements. If all requirements cannot be satisfied, it may be necessary to use a deep foundation.

Nominal Resistance Determination

The strength of the foundation soil supporting spread footings has been determined by three methods in general geotechnical practice. In most cases, presumptive bearing capacities have been defined based on the experience of the geotechnical engineer. The presumptive values can then be supplied to the structural engineer and the footing design can be completed. This approach to design has obvious, serious weaknesses but with extensive local experience it has proven to be a reliable and effective approach to design, although it is generally conservative. Furthermore, the real limitation for spread footings is frequently settlement and vertical load failure considerations do not necessarily govern.

The second approach that has been followed in the determination of nominal strength has been the use of in situ test results. These methods have been empirically calibrated to footing performance and they can be used easily with the available soil exploration data. This approach requires a knowledge of footing dimensions in order to determine an allowable or nominal bearing pressure so the problems of cooperation between the structural and geotechnical engineers discussed above apply to in situ methods.

The last general approach is the use of "rational" methods with soil properties based on subsurface exploration information and laboratory testing or perhaps soil properties obtained from in situ test data. Since many different conditions exist, such as load inclination, load eccentricity, soil layering, and a variety of soil types under the footing, the method becomes quite complex in its detail.

Substantial data base information from static load tests on spread footings taken to failure does not exist. Therefore, only limited data are available for use in substantiating the methods used for capacity determination. A simple, direct method of verification is not available, so indirect methods must be used. The computation method variability can be evaluated based on the variability and accuracy of the mechanical model and the variability and accuracy of the quantities used in the model. There may be an additional variable, in that the input information for the models may be obtained from curves or tables using subsurface exploration information, which also has some variability. Now the variability of both the subsurface information and the tabulated quantities in addition to the model becomes important.

The LRFD Bridge Code gives specific methods for nominal capacity determination including curves and graphs for the definition of required constants in the computational procedures to be used for vertical capacity determination based on the soil type and the soil conditions. Other conditions, such as eccentric loads, inclined loads, groundwater, and sloping ground surface, are specifically defined and numerical procedures are specified. The above methods include both the use of "rational" methods and methods based on in situ test data. There are no presumptive bearing pressure values for spread footings on soil in the LRFD Bridge Code.

The determination of soil properties from subsurface exploration information is the single most difficult and pervasive problem in geotechnical engineering. The sampling and testing of soils is complicated by the problems of dealing with a granular medium that is highly variable and usually below the water table. The LRFD Bridge Code discusses subsurface exploration procedures in Section 10.4, Determination of Soil Properties. Reference is made to procedures from AASHTO and ASTM standards. However, no guidance is given for the selection of soil properties given a variety of values from tests in a particular layer. Other standards, Eurocode for example, specify procedures for selecting values for design. A large change in the safety of the facility can result for designs based on conservatively selected material properties versus optimistic values. This topic will be discussed in more detail in the review comments on Eurocode.

The methods in the LRFD Bridge Code for determining nominal strength of spread footings are quite complete and thorough. However, in stating these methods specifically in the Code, the use of other computational procedures is effectively prohibited. Thus, what may have long been successful practice in many locations is no longer acceptable. Perhaps a more general statement of the procedures to be used for nominal strength determination should be considered. This would have the obvious difficulty that resistance factors are not available for these methods and would have to be generated by the organization using the method.

The serviceability limit state often governs the design of spread footings and this makes much of the discussion above of less concern. The importance of the strength limit state can best be examined by studying comparative designs for several realistic sites and loads.

Footings on rock are handled in a similar way except presumptive bearing pressure values are included in addition to empirical methods. It is noted that the strength design procedures for rock include specified bearing pressures at the "service limit state." Since values are also available for the "strength limit state" the appropriate use of these two values is unclear. The designer is directed to use two specific strength values for two strength limit states.

Resistance Factors for Spread Footings

Resistance factors for spread footings were studied by Barker et al. *(1991)* and the results of that study were used as a basis for the LRFD Bridge Code. The variability of the basic laboratory measured soil properties was determined from the literature and used as the variability of the footing capacity for the purpose of determining the resistance factors. This ignores the variability of the capacity calculation method and data on that variability is probably not available. Studies were done to examine the variation of the safety index, β, as a function of footing size and bridge span length. Based on this calibration effort, resistance factors were selected using the probabilistic analysis discussed in chapter 2. It is important to note that at the time this work was done the load factors available to the researchers were 1.3 for dead load and 2.2 for live load. It may be useful to examine the magnitude of the resistance factors for load factors of 1.25 and 1.75 for dead and live load, respectively. This would result in reductions in the resistance factors, implying that the current designs are not conservative. However, the changes would probably not be very great in view of the other assumptions that had to be made in generating the current resistance factors.

Data base information is now available *(Briaud and Gibbens, 1995)* on the strength of spread footings. While these data are limited they give response information on the results of full-scale tests. This makes it possible to check designs against test results. It is likely that additional data are available from centrifuge tests. The literature in this area was not checked but tests have certainly been made and the results may be useful. The use of centrifuge tests to generate additional data might be cost-effective.

Resistance factors were also determined for the sliding failure mode. Based on comparative designs, larger values of resistance factors are used to give reasonable values. In this case, the fact that a much lower factor of safety is commonly used in ASD is the basis. A good description of the LRFD design procedure for spread footings is given by Withiam et al. (1997).

Serviceability Design for Spread Footings

Serviceability design for spread footings is important because settlement behavior will often control the design. The prediction of spread footing settlement is one of the important problems that have challenged geotechnical researchers for many years. Procedures for analysis are well established and are discussed in foundation design textbooks. Load combinations and load factors to be used in settlement calculations are mentioned in the LRFD Bridge Code. Structural analysis methods to determine foundation loads are also discussed in that code and, while nonlinear analyses are permitted, it is likely that the analysis will be based on the assumption of linear elastic behavior.

Displacement limits are discussed but specific limits are not given in the code. FHWA sponsored *(Moulton et al. 1985)* one of the few studies in this area and it is referenced in the Commentary of the LRFD Bridge Code. Included in the Commentary are suggestions for limitations on angular distortions of 0.008 for simple span structures and 0.004 for continuous structures. This limit seems to be somewhat large. For example, a relative displacement of almost 6 inches would be permitted in a 60-foot simple span bridge. More severe limitations are suggested by the LRFD Bridge Code if determined to be necessary by studies based on cost, rideability, aesthetics, and safety. Probably more specific requirements are not possible at this time and any stated limit in this area can help establish a practice. Clearly, the structural engineer is responsible for establishing the tolerable settlement if it is different from that required by the code. Analysis methods with the capability of coupling structure and soil response to produce a more rational displacement are becoming available.

Other Codes and Standards

Ontario Bridge Code

The Ontario Bridge Code *(Ontario Ministry of Transportation and Communication 1992)* gives resistance factors for spread footing design and they are listed here in Table 2. Methods of capacity calculation are outlined quite specifically in the commentary to the code. The computations include all of the failure modes and are quite clear. This approach may resolve the geotechnical engineer's concern with the limitations imposed by requiring specific analysis methods. Of course, single values of resistance factors are currently specified independent of the analysis procedure. This section of the Ontario Code should be reviewed carefully and compared with current United States practice.

TABLE 2

RESISTANCE FACTORS (from Ontario Bridge Code)

Shallow Foundations	
Type	Factor
Bearing resistance	0.5
Horizontal shear resistance	
a) calculated based on tan ϕ' or tan ϕ_u	0.8
b) calculated based on c' or c_u	0.5
Horizontal passive resistance	0.5

Canadian Bridge Code

It is presumed that this code *(Canadian Standards Association 1988)* governs the design of bridges in Canadian provinces other than Ontario, which has its own document and British Columbia where the responses to the questionnaire circulated for this study indicated that they did not intend to adopt the LRFD method. The Canadian Bridge Code is more concise than the Ontario Code. Resistance factors are applied to the soil parameters, cohesion, and internal friction. A multiplier of 0.5 is applied to cohesion and 0.8 to friction. This is a strong departure from the procedure used in Ontario and a comparison between the two approaches would require comparative designs. It should be noted that the load factors are also different, as discussed in chapter 3.

Methods are given for nominal strength calculation for the ultimate limit state and tables and graphs are also specified. Presumptive bearing capacity values are given for foundations on rock. Settlement determination is discussed briefly, but methods of analysis are not mentioned.

Eurocode

The Eurocode 7 *(European Committee for Standardization 1994)* describes the methods that can be used in the design of spread footings. They include both an analytical method and a semi-empirical method that uses in situ test results. Specific formulas are not included but example problems are given in an appendix. The example problem describes a design process but other similar methods are permitted. Load and resistance factors are summarized in Table 3. It is interesting to note that three load and resistance factor combinations are given. The resistance factors are applied directly to the soil properties. Both load and resistance factors are "bracketed quantities" and these values can be set by the various governments in the immediate future. The resistance factors are applied to the resistances as divisors rather than multipliers, as is the case in North American codes. In Table 3 the resistance factors from the Eurocode 7 have been inverted to simplify comparison with North American resistance factors.

The Eurocode is primarily descriptive, with a very large number of conditions to be qualitatively satisfied by the designer. Many of these requirements are very general and of a type that would be discussed in a textbook. There are few specific, quantitative requirements.

The basic approach for resistance factors used in Eurocode 7 is fundamentally different than that used by North American Codes. In Eurocode 7 the resistance factor is applied to the soil properties while in the LRFD Bridge Code and the Ontario Bridge Code the resistance factor is applied to the nominal strength.

Danish Code of Practice for Foundation Engineering

The Danish Code of Practice *(Danish Geotechnical Institute 1985)* covers the design of spread footings. In an early

TABLE 3

PARTIAL FACTORS—ULTIMATE LIMIT STATES IN PERSISTENT AND TRANSIENT SITUATIONS (from Eurocode 7)

	Actions			Ground Properties			
	Permanent		Variable				
Case	Unfavourable	Favourable	Unfavourable	tan ϕ	c'	c_u	q_u[1)]
A	[1.00]	[0.95]	[1.50]	[0.91]	[0.77]	[0.83]	[0.83]
B	[1.35]	[1.00]	[1.50]	[1.0]	[1.0]	[1.0]	[1.0]
C	[1.00]	[1.00]	[1.30]	[0.80]	[0.63]	[0.71]	[0.71]

[1)] Compressive strength of soil or rock.

TABLE 4

PARTIAL COEFFICIENTS FOR SPREAD FOOTINGS, NORMAL FOUNDATION CLASS (from Danish Code)

	Safety Class	
Partial Coefficients	Normal	High
γ_ϕ tangent of angle of internal friction	0.83	0.77
γ_c^1 cohesion (load carrying capacity of foundations)	0.56	0.50
γ_c^2 cohesion (stability and earth pressures)	0.67	0.61

TABLE 5

MATERIAL RESISTANCE FACTORS FOR ULTIMATE LIMIT STATE OF SPREAD FOOTINGS (from AUSTROADS Bridge Design Code)

Material Factor for Ultimate Limit States	Values
Ultimate resistance of shallow footings on cohesionless soils where SPT (N) values are used	0.5
Ultimate resistance of shallow footings on cohesionless soils where CPT values are used	0.6

section, methods are specified for determining soil properties. Then the bearing capacity formula is given with the constants defined in a curve as a function of soil properties. Other more complex conditions of loading and soils are dealt with by a few very brief, specified procedures. Resistance factors from the Danish Code of Practice are given in Table 4. Information was not given on how the resistance factors were determined. Discussions with Dr. Ovesen *(Dr. Krebs Ovesen, Danish Geotechnical Institute, March 1998, personal communication)* indicate that they were determined by direct correlation with existing practice.

AUSTROADS Bridge Design Code

The AUSTROADS specification (*AUSTROADS 1992*) governs the design of bridges in Australia. Only the geotechnical section was available. The design of spread footings is dealt with generally in the specification. In the commentary, soil properties are given for both cohesive and cohesionless soils based on density. Also, presumptive bearing capacity values are given for both soils and rock. The specified resistance factors for spread footings are given in Table 5.

DEEP FOUNDATION DESIGN PROCEDURE

The development of a design code requires a firm basis in the design process that is used. If the code does not truly reflect the requirements of the process it will fail to be useful to the designer. The design process for deep foundations has changed dramatically over the past three decades. For driven piles, larger hammers have become common, wave equation driveability predictions have become routine, new and larger piles have appeared, and dynamic capacity prediction has reduced the cost of testing. Drilled shafts have increased in popularity, new and larger installation equipment is available, new methods of installation have been developed, integrity testing techniques are available, and less costly capacity testing methods have been introduced. A modern design specification must reflect these developments so the design process will be reviewed including the new developments.

The process used in the design of deep foundations is presented in Figures 6, 7, and 8. In Block 6 of Figure 6, the decision is made to use a deep foundation. In some cases it may be desirable to perform preliminary designs and cost evaluations. In Figure 7, Block DP1 a driven pile is selected and then in Block DP2 pile type, method to be used in determining capacity, and quality control procedure are selected. For a drilled shaft design, similar decisions are made in DS2 of Figure 8. With the capacity determination method and the quality control procedure established, the resistance factor can be established for either a driven pile (DP3) or a drilled shaft (DS3). The nominal strength for an individual pile or drilled shaft is selected and the estimated pile length calculated in Block DP4 or DS4. Serviceability evaluations must also be made at this stage. This can include lateral load behavior, settlement, and any other serviceability requirements.

The basic LRFD design condition contained in the LRFD Bridge Code was stated in chapter 1, as Eq. 1. It contains a multiplier, η, on the load side of the expression. This term is defined as a multiplier to be selected based on the ductility, redundancy, and importance of the element in question. Some catastrophic superstructure failures of nonredundant and nonductile bridges have made bridge engineers very sensitive to the importance of ductility and

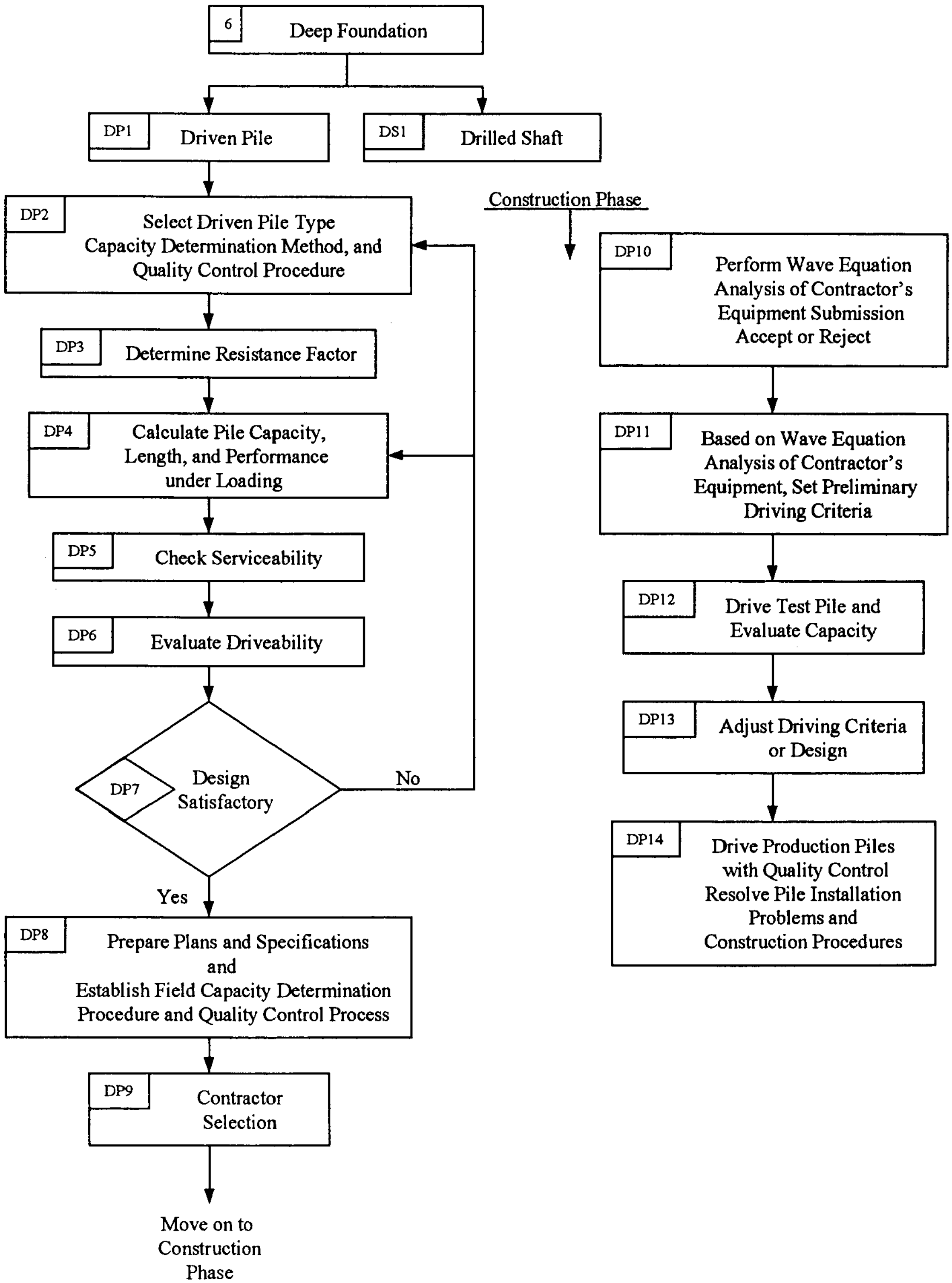


FIGURE 7 Design procedure for driven piles.

redundancy. Ductility will assure that, if the structure is loaded in excess of its nominal strength, it will give a warning of the problem by excessive deflections. Thus, catastrophic failure is avoided. Deep foundations will almost always be very ductile in axial behavior, as illustrated in Figure 9. There are cases of sensitive clays where the loss of strength with increasing deflection can occur, but this behavior is rare and can be predicted from subsurface information.

Deep foundations will usually have a sufficient number of elements to be redundant. As an example of the desirability

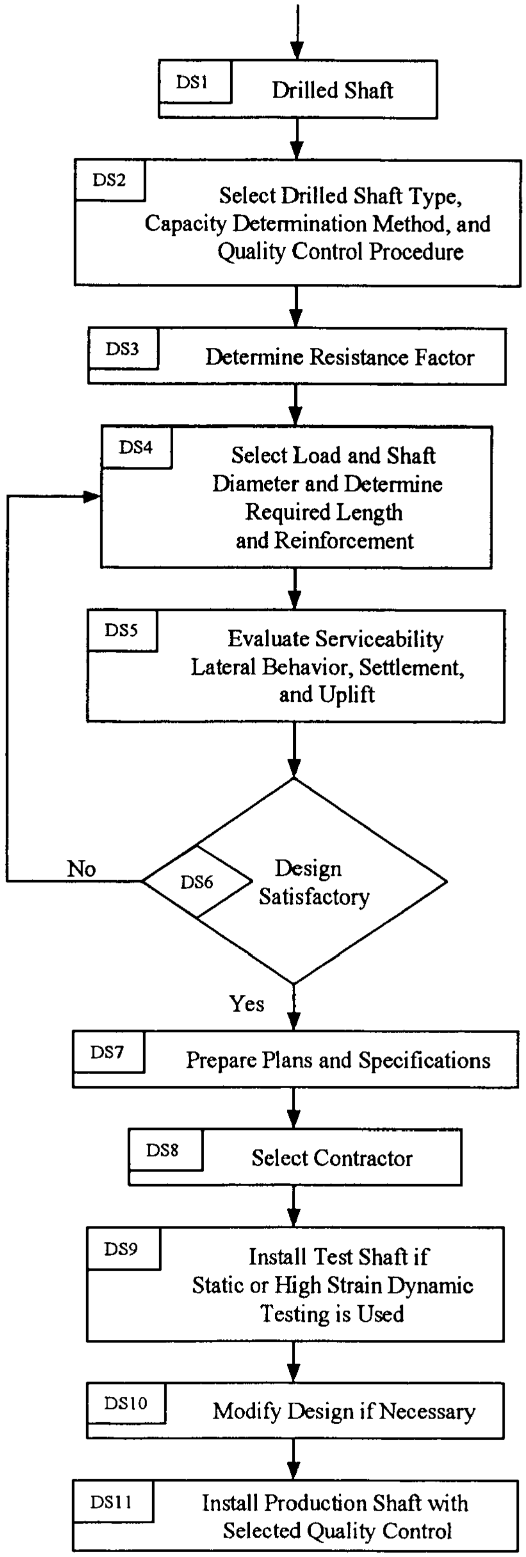


FIGURE 8 Design procedure for drilled shafts.

of redundancy consider, qualitatively, some specific examples. Assume that a 9-pile foundation constructed in a square pattern is subjected to vertical compression load only. If one pile has a capacity less than that required to carry its share of the load, the excess load will be transferred to the other piles and the safety is not impaired. Now consider the same pile group subjected to combined bending and vertical compression load. The maximum individual pile load will be applied to the three piles on the compression side of the footing. If one of these piles is weak, the excess load will be transferred to the neighboring two piles. It is unlikely that failure would result but the level of safety is not as great as for the case of vertical compression load only because the excess load must be shared between two piles instead of eight. Of course, not all pile foundations are strongly redundant. This topic merits additional study so that the designer can be provided with clear and specific recommendations.

It should be noted that there is a trend to the use of more nonredundant foundation systems. Bridge piers are more frequently designed with only two drilled shaft supporting elements and even single shaft foundations are becoming common. In this case, additional conservatism is appropriate. Chapter 3 of the LRFD Bridge Code suggests values of h for the case of nonredundant systems.

The meaning of redundancy should be more precisely defined and quantified for deep foundations in the LRFD Bridge Code. NCHRP Project 12-47 "Redundancy in Highway Bridge Substructures," is now studying this problem. The emphasis of this project is on the development of methods for evaluating substructure failure, including consideration of serviceability at several levels: 1) cases where the structure can continue to carry normal traffic loads; 2) cases where the structure can carry normal traffic and can be repaired; and 3) severely damaged structures that will allow existing traffic to safely leave the bridge but the bridge must then be closed. A report documenting the project and its findings is expected to be published as the final phase of the project.

Deep Foundation Nominal Strength Definition

The definition of nominal pile strength is not always obvious given a static load test result. An example of the results of two different pile load tests is shown in Figure 9. These examples are simply illustrative of the problem and are not taken from specific load tests. Case 1 represents an example of a pile test in cohesive soil. Notice that at one point the curve breaks quite sharply and then shows additional displacement at little change in load. This behavior is typical of piles with only a small portion of the load carried by the toe. The definition of the nominal capacity can be taken at the maximum load and that value is easily and clearly defined.

Case 2 in Figure 9 is typical of piles in noncohesive soil with substantial toe capacity. In this case there is no clear

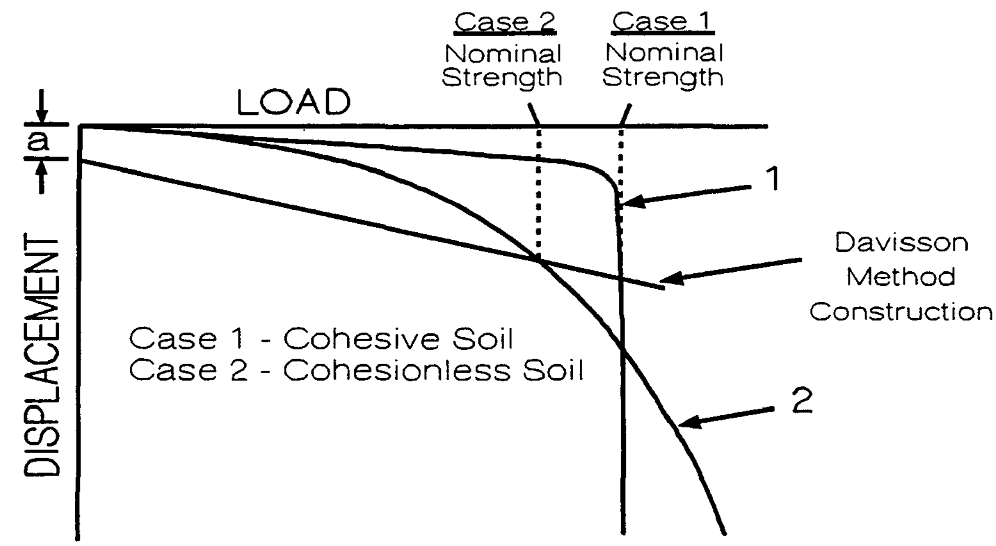


FIGURE 9 Idealized load test curves for different soil types.

break in the curve and the load continues to increase, sometimes indefinitely. For example, in the Michigan Pile Test Program *(Michigan State Highway Commission 1965)* one pile in sand was loaded to a displacement of 2 feet and the capacity was still increasing. This characteristic usually becomes more pronounced as the diameter increases. The problem of defining pile capacity has been studied by several authors. For instance, Fellenius *(Fellenius 1975)* has summarized several different methods of capacity definition.

Where piles have the type of load response illustrated in Case 2 of Figure 9, it becomes necessary to have a defined capacity. In the past two decades, the Davisson Method *(Davisson 1972)*, illustrated in Figure 9, has become increasingly popular in the United States. In the application of this method, a straight line representing the axial pile structural stiffness is plotted with a defined offset, a, on the displacement axis. The intersection of that line with the load test curve is defined as the failure load. The value, a, is often defined as 3.75 (0.15) + (pile diameter/120) mm (inches) but it has not been standardized. Although this method has become popular, neither it nor other procedures are specified universally in codes. The Davisson Method will usually provide a smaller nominal strength than almost all other methods.

O'Neill has suggested the importance of determining a "creep" load *(O'Neill and Hawkins 1983)*. This is the load, which, if exceeded, would cause the pile to penetrate slowly into the ground. Perhaps this value would make a more appropriate nominal strength.

The value of the nominal strength as measured by load test is important in defining the resistance factor. If a definition of failure produces a smaller value for a given load test curve, conservatism is increased and a larger resistance factor may be justified than if a larger capacity is obtained from the evaluation procedure. Additional research in this area is necessary.

Pile capacity is also affected by the loading procedure used in the static load test. Many years ago a "maintained" load test procedure was usually used. A large increment of load was applied and held until a prescribed time had elapsed after movement had stopped. As the load increased, the holding time was also increased until a specified failure criterion was reached. Thus, the loading rate decreases during the test as the incremental movement increases. Because the load is applied in large increments, failure will tend to occur at the end of an increment of load application influencing the definition of the failure load. Because the loading rate varies in this procedure the test interpretation becomes less reliable. However, since the load is maintained for longer periods of time some engineers find the procedure attractive because a slow failure might be identified at a lower load. The entire load test using the maintained load test procedure can take over 24 hours, it can become difficult to execute accurately, and it is more expensive than quick tests.

Quick load tests have become popular over the past decade. In this procedure, the load is applied in small increments, perhaps 1/20 of the design load, and each increment is held for a specified and short time period *(ASTM 1997)*. This test is easy to perform and reproducible results can usually be obtained.

It is interesting to compare pile load test evaluation with the beam tests that were used in developing the LRFD codes for structural design. Most of the early beam tests used in nominal strength definition were performed with little effort exerted to control loading rate and to define failure. When full plastic conditions were reached, the load required to achieve the plastic condition was called failure regardless of loading rate. When full plastic conditions were reached, the load could not be maintained on the beam. However, the ultimate capacity was used to define failure and develop analytical methods for predicting strength. Analytically calibrated structural codes have been based on a safety index of 3.5, a value that may seem large for geotechnical practice. A general examination for selecting an appropriate safety index for geotechnical application could be interesting and useful.

Driven Pile Design Procedure

The driven pile design process continues in Figure 7. After a pile type, capacity determination method, and quality control procedure are selected in DP2, the appropriate resistance factor is determined in DP3 based on the decisions made in DP2 and the judgment of the engineer. The strength aspects of the design are completed in DP4. For a selected pile capacity, the estimated pile length is determined by static analysis and other performance characteristics of interest are evaluated. After checking serviceability in DP5, the pile driveability is evaluated in DP6 by wave equation analysis. When a satisfactory and economical design has been selected, plans and specifications are prepared.

A contractor is selected in DP9 and submits his driving system for review in DP10. Based on a wave equation analysis the preliminary driving criteria are established in DP11. The pile capacity is determined in DP12 and if necessary the driving criteria are adjusted in DP13. *So the design is not complete until this stage when the final driving criteria are established.* The production piles are then driven using the selected quality control methods. The driven pile design procedure is discussed in more detail in Volumes I and II of *Design and Construction of Driven Pile Foundations (Hannigan et al. 1996).*

Nominal Strength Determination

Two strength limit states must be considered: structural pile capacity and pile-soil strength. In the first case, failure will occur with the fracture or collapse of the pile material such that after failure, the pile will not be satisfactory to carry the required load. In the case of the pile-soil strength limit state, the pile penetrates into the ground and, after removal of the excessive load, the pile is still able to carry loads without regard for the fact that it has "failed." In fact, in many cases pile capacity is increased by the loading process.

Strength Limit State—Structural

Considerable research has been performed in determining the axial strength of pile elements. An embedded pile will usually behave as a short compression member. If it extends above the ground surface in either water or air, buckling must be considered. All of these topics have been studied extensively by structural engineering research.

Davisson et al. *(1983)* studied and made recommendations for allowable stresses for driven piles. These values were very low, and in some cases, lower than the allowable stresses used at the time the report was written. There is no history of structural pile failure under service loads without other unexpected critical conditions (e.g. downdrag loads).

Timber—Extensive testing has been performed for timber piles on short axially loaded specimens *(Thompson nd; Peterson nd, and Goble et al. 1983)*. These test data are sufficient to provide good information on mean strength and coefficient of variation for both treated and untreated round timbers of Southern Pine and Douglas Fir. They can be used to arrive at appropriate nominal strength and resistance factor values. Currently used ASD values were developed primarily based on experience.

The structural strength for particular species of timber is specified directly in the LRFD Bridge Code. The methods for evaluating timber strength are straightforward.

Steel—The behavior of steel sections subjected to axial or combined bending and axial loads has been studied extensively in connection with column behavior analysis, and reduced to a form that can be used for routine capacity evaluation by designers *(Johnston, ed. 1976)*. Bjorhovde et al. *(1978)* used the available information to generate a practical column design framework that was used in the AISC Code *(AISC 1986)*. These methods were used to develop the nominal strength determination methods that are contained in the LRFD Bridge Code. The calibration work *(Nowak nd)* used this data to obtain resistance factors for column design and this work was applied directly to pile design in the LRFD Bridge Code.

For steel piles under axial load only, the nominal structural resistance is further reduced by 22 percent for H-piles and 13 percent for pipe piles in the LRFD Bridge Code. This reduction is due to unintended eccentricities, according to NCHRP Report 343 *(Barker et al. 1991)*. A similar reduction is not specified for concrete or timber piles.

Corrosion can reduce the area of steel sections. The *FHWA Driven Pile Design Manual (Hannigan et al. 1996)* gives methods for evaluating the magnitude of corrosion based on the environmental exposure. The reduced section is used in design for strength.

Concrete—Extensive testing has also been performed on concrete sections in compression, although less information is available on prestressed members *(Mirza et al.*

1979, 1979a-b). Computer programs have been developed to determine the strength of prestressed concrete sections subjected to combined bending and axial loads with given material properties, cross section characteristics, and effective prestress, and they have been used to generate interaction curves for a wide variety of prestressed concrete pile sections *(Anderson 1970)*. These curves are used routinely in the ASD design of piles that also serve as bridge bent columns *(PCI nd)*. Since the curves are used to determine ultimate strength, they can be applied directly in the LRFD approach.

Bridge Bent Piles—Piles that are not fully embedded must be checked for their buckling strength and that requires estimating the effective length that is used in the analysis *(Davisson et al. 1965)*. The primary problem is that the point of fixity is dependent on the soil properties. Combining the research that has been done on laterally loaded piles with modern analysis methods, it should be possible to do parameter studies to provide data that could be used to recommend better empirically defined effective lengths for timber, steel, and concrete piles that are not fully embedded.

Strength Limit State—Soil

In the design procedure described in Figure 7, nominal soil strength determination can be done using six different general methods: presumptive bearing capacity, static analysis, dynamic formula, wave equation analysis, dynamic monitoring, and static load test. These methods are commonly combined in various ways to establish the capacity and to verify it for quality control requirements. The resistance factors should be selected based on the reliability of the pile capacity determination method. Selection should also account for the variability of the method used to verify the capacity during installation, (e.g., the quality control procedures), and the percentage of piles tested during production.

Presumptive Bearing Capacity—Presumptive bearing capacities refer to the use of prescribed values, usually from building codes, that can provide a rough guide regarding the capacity that may be used for piles. Historically, these values have all been allowable values and they are used for applications where small loads are involved. For example, in New Orleans many dwellings are supported on driven timber piles. A specified length is driven but quality control such as blow counting is not normally used since large amounts of setup will occur and the end-of-driving blow count has little meaning. Load tests are rarely performed. Large factors of safety probably exist. Presumptive loads have not been used in recent AASHTO bridge specifications and they are not used by state DOTs for other than, possibly, some cases of temporary structures. There will be no further discussion of presumptive driven pile capacities, as this practice is not recommended.

Static Analysis—Static analysis refers to the use of subsurface exploration information together with empirical or semi-empirical methods to determine pile capacity. All of these methods define pile capacity as the sum of the shaft resistance and the toe capacity in the form

$$Q_{ult} = Q_s + Q_p - W$$

where Q_{ult} is the nominal strength of the pile under axial load, Q_s is the ultimate shaft capacity, Q_p is the ultimate toe capacity and W is the weight of the pile. Normally the pile weight can be ignored.

A large number of methods have been presented for determining Q_s and Q_p ; they have been reviewed elsewhere *(Barker et al. 1991, Vesic 1977)* and are not discussed extensively here. The *FHWA Driven Pile Design Manual (Hannigan et al. 1996)* recommends some specific procedures for determining the capacity of driven piles. These methods are the Meyerhof and Nordlund method for piles in sand using SPT data and the Laboratoire Centrale des Ponts et Chaussees (LCPC) and Schmertmann Method if CPT data are used. The same document recommends the use of the a-method for piles in clay. Foundation engineers have used a variety of static analysis methods often modified with local experience and static load test results. If specific static methods are required by code, this advantage of local experience is lost.

The prediction of geotechnical pile capacity from subsurface exploration data is a subject that has interested researchers for decades. A large number of analysis methods have been proposed and they produce capacity predictions that are quite unreliable. This is particularly true for piles in cohesionless soils *(Dennis et al. 1983, pp. 389–402)*, while the results for cohesive soils are more predictable but still poor *(Dennis et al. 1983, pp. 370–388). For design, the use of static analysis only implies that a pile length is selected based on the results of the analysis and then the pile is driven to that predetermined length without other methods used for capacity determination or quality control.* This method is not used in practice and is not recommended.

The driving process generates changes in pore pressures in most soils and associated changes in effective stresses and shear strength. The strength change can be either a strength loss or a strength gain. If a strength loss occurs during driving, the strength will be regained with time. In noncohesive soils, this strength gain is rapid while in cohesive soils days, weeks, or months may be required. In some granular soils, continued strength gain occurs over a long period of time. This effect, called aging by Schmertmann *(1991)* is not well understood.

Strength may also increase due to driving and, in these relatively infrequent cases, the strength will decrease with time. This phenomenon is known as relaxation. Observations made during the driving process will not necessarily reflect the final pile resistance without some form of correction. Relaxation can occur in some dense fine sands or dense silty sands. It is probably caused by the generation of negative pore pressures and increased shear strength due to dilation of the soils near the pile tip. Strength reduction occurs fairly rapidly in this case, since pore pressures will increase rapidly in these granular soils. This behavior may be quite common but its effect on total pile capacity is often canceled by setup of other soil types along the pile shaft. There are some shales where strength loss has been observed, although it may require several days to develop. After the pile strength change has occurred, re-strike testing can be performed to observe blow count or make dynamic measurements, or static load tests can be performed. These tests must be performed some time after the end of driving.

It should not be surprising that static analysis has proven to be very unreliable for driven piles in granular soils. The driving process surely changes the soil density so the subsurface investigation prior to driving will not reflect conditions after driving is complete. This has been shown by Antorena *(1996)* at a site where concrete piles were being driven in sands. He made dynamic measurements during driving and on piles that were re-struck after pile driving of the group was complete. Signal matching analyses of the dynamic data showed that the shaft resistance changed (increased) with driving of additional adjacent piles. In addition, CPT tests were run at the site before driving and near some of the piles after driving; it was clear that the driving process increased the soil strength. This data indicates that it will probably not be possible to ever predict pile capacity from subsurface exploration data unless it becomes possible to predict soil strength changes caused by driving.

Driven pile capacities in granular soils are not determined by the static analysis but rather by observations made during driving. The purpose of the static analysis is to determine an estimated length for pile bidding, specifying, and ordering.

Dynamic Formula—The oldest approach for the prediction of pile capacity, even preceding static analysis, is the use of a dynamic formula with field observations of the blow count and a knowledge of the rated energy of the hammer. Extensive studies of the accuracy of the various formulas have been made over the past half-century *(Olsen 1967, Rausche et al. 1997, Briaud et al. 1988, Fragasny et al. 1988. and many others)*. It has been shown that, in spite of very poor accuracy, the dynamic formulas are more accurate than static analyses. The advantage of the dynamic formula is that the observations reflect the conditions during the driving operation. If soil changes have occurred during driving, or if the subsurface investigation was inadequate or erroneous, it is reflected in the driving resistance. One of the primary disadvantages of the driving formulas is that they depend on the rated hammer energy and the performance of the driving system under the specific conditions of hammer maintenance, driving system configuration, and site characteristics.

In comparisons between the various formulas, the Gates Formula is usually found to be the most accurate *(Fragasny et al. 1988)* and the *FHWA Driven Pile Design Manual (Hannigan et al. 1996)* recommends its use if a formula is used. One of the problems that arise in converting to LRFD is that all of the formulas have a factor of safety hidden in the constants contained in them. Care must be taken to remove the factor of safety correctly. This is further complicated by the adjustment of the constants in the formula based on experience.

Wave Equation Analysis—The use of a discrete dynamic analysis of the pile and pile driving system was first suggested by Smith *(1960)* and since that time extensive development has occurred *(Hirsch et al. 1976, GRL 1996)*. The FHWA has made a considerable effort to encourage the implementation of this tool over the past two decades and today its usage in North America is widespread and increasing steadily. The approach is commonly used for both driveability analysis and capacity determination. For capacity determination, a bearing graph is usually generated and then the pile is driven to a blow count corresponding to the required capacity. Of course, capacity can only be determined by observing the driving resistance and thus the method only gives capacity information when piles are driven. Like all other capacity determination methods that are based on observations made during driving, wave equation analysis is useful for dealing with cases where the strength changes with time. The pile can be driven to a resistance less than that required by the bearing graph and then re-struck at a later date after strength change has occurred to verify that the capacity is indeed achieved. In this way, a reduced driving resistance can be proven or, in those cases where relaxation occurs, the magnitude of strength loss can be quantified and an appropriate driving resistance determined.

Dynamic Monitoring—The use of dynamic measurements of force and acceleration at the pile top to predict pile capacity and also to study pile driving problems *(Goble et al. 1970, Rausche et al. 1985)* is now well established in practice in the United States and most of the developed countries of the world. A majority of the state DOTs use the equipment with their own forces or by contracting for testing services. Standards have been adopted for the performance of the test *(ASTM 1997, AASHTO 1998)*. Capacity predictions are obtained for every hammer blow in real time in the field by solving a closed form expression based on one-dimensional wave mechanics *(Rausche et al. 1985)*. Large volumes of data have been collected to correlate with static load test information *(Rausche et al. 1997)*. The accuracy of the capacity predictions can be improved by use of signal matching analysis techniques to further evaluate the measurements *(Rausche et al. 1972)*. The availability of fast personal computers has made it possible to apply signal matching easily and quickly. A large amount of correlation data is available to evaluate the accuracy of the method.

Static Load Test—The most accurate and reliable method for determining pile capacity is the static load test. Of course, it is expensive and time consuming, but it is the standard by which other methods are evaluated. The additional problem with the static load test is that it must be carefully performed and if that is not done the results can be substantially affected. Procedures for performing static load tests have been standardized *(ASTM 1997)*. It should be noted that a poorly performed static load test may give erroneous results. The evaluation of the accuracy of all methods of capacity prediction is based on static load test results. It should be noted that all methods of capacity determination discussed here are for single piles and they do not deal with group capacity.

Requirements of the LRFD Bridge Code—The largest number of responses to the Questionnaire concerned the driven pile code requirements. This is not surprising because driven piles are a widely used bridge foundation type. Most of these comments were quite general, expressing difficulty using the code with no specific criticism. The most specific comments came from the Florida DOT which expressed concern specifically for the resistance factors. This issue will be discussed in more detail in the next section.

As shown in Figure 7, the function of static analysis is to obtain an estimate of pile length for bidding purposes. Also, it is important that a static analysis be performed in order that the foundation engineer gain a clear understanding of pile-soil load transfer and possible construction problems. It is unnecessary that specific methods of analysis be required because the designer may have local experience with other methods that are not in the Code. It may be desirable to recommend specific analysis methods in the Commentary, but it should be possible to use other methods with their own resistance factors.

The FHWA does not recommend the use of dynamic formulas and they are not mentioned in the LRFD Bridge Code. Current practice in a substantial number of state DOTs make use of a dynamic formula for at least some classes of projects. The *FHWA Driven Pile Design Manual* recommends the use of the Gates formula for projects controlled by driving formulas. It is true that modern practice is rapidly reducing the use of dynamic formulas, but they are still being used widely in practice.

Wave equation analysis is mentioned in the Code but reference is made only to driveability analysis. Wave equation analysis is frequently used in modern practice to establish pile capacity. The maximum allowable driving loads (stresses) are specified using the values that have been established by practice over the past decade or more, multiplied by the associated resistance factors. The reduction of allowed driving stresses below current practice should be examined.

In the code, dynamic monitoring is mentioned as a tool for dealing with driving problems or for cases where static load tests are not justified. Dynamic monitoring is now used by several DOTs for establishing the nominal strength, for use together with static load tests for increasing the number of piles tested at only modest cost, as well as an overall quality assurance tool.

Resistance Factors

Some work has been performed in determining resistance factors for single driven piles. Goble et al. in 1980 reported a set of resistance factors consistent with the requirements of pile design. These factors were obtained by calibration with existing factors of safety. Goble and Berger *(1994)* and Berger *(1989)* reported on resistance factors obtained using available data bases from Dennis and Olson *(1983)*, Pennsylvania DOT *(nd)*, and Rausche et al. *(1997)* together with a probabilistic analysis. These calibrations were obtained using the software developed during the National Bureau of Standards study *(Ellingwood et al. 1980)* and, therefore, they are related to the NBS load factors, which were developed for building loads. Some of the results obtained by Berger are summarized in Table 6. A large volume of pile capacity data has been compiled since publication of Berger's work.

TABLE 6

RESISTANCE FACTORS FOR A β-VALUE OF 3.0 (from *Goble and Berger 1994*)

Method	Resistance Factor
Static analysis	0.42
ENR	0.42
Wave equation	0.50
CAPWAP	0.73

TABLE 7

RESISTANCE FACTORS FROM THE LRFD BRIDGE CODE

Method/Soil/Condition		Resistance Factor
Ultimate Bearing Resistance of Single Piles	Skin Friction: Clay	
	α-method (Tomlinson, 1987)	0.70 λ_v
	β-method (Esrig & Kirby, 1979 and Nordlund method applied to cohesive soils)	0.50 λ_v
	λ-method (Vijayvergiya & Focht, 1972)	0.55 λ_v
	End Bearing: Clay and Rock	
	Clay (Skempton, 1951)	0.70 λ_v
	Rock (Canadian Geotech. Society, 1985)	0.50 λ_v
	Skin Friction and End Bearing: Sand	
	SPT-method	0.45 λ_v
	CPT-method	0.55 λ_v
	Wave equation analysis with assumed driving resistance	0.65 λ_v
	Load Test	0.80 λ_v
Block Failure	Clay	0.65
Uplift Resistance of Single Piles	α-method	0.60
	β-method	0.40
	λ-method	0.45
	SPT-method	0.35
	CPT-method	0.45
	Load Test	0.80
Group Uplift Resistance	Sand	0.55
	Clay	0.55

Method of controlling installation of piles and verifying their capacity during or after driving to be specified in the contract documents	Value of λ_v
Pile Driving Formulas, e.g., ENR, equation without stress wave measurements during driving.	0.80
Bearing graph from wave equation analysis without stress wave measurements during driving	0.85
Stress wave measurements on 2 to 5 percent of piles, capacity verified by simplified methods, e.g., the pile driving analyzer	0.90
Stress wave measurements on 2 to 5 percent of piles, capacity verified by simplified methods, e.g., the pile driving analyzer, and static load test to verify capacity	1.00
Stress wave measurements on 2 to 5 percent of piles, capacity verified by simplified methods, e.g., the pile driving analyzer, and CAPWAP analyses to verify capacity	1.00
Stress wave measurements on 10 to 70 percent of piles, capacity verified by simplified methods, e.g., the pile driving analyzer	1.00

Barker et al. *(1991)* generated the resistance factors for the static analysis methods contained in the LRFD Bridge Code using the rational probabilistic approach on available estimates of basic soil property variability. This approach does not include the model variability, site variability, or the fact that other means are used to determine capacity. They do not report any calibration analyses of the other nominal strength determination methods nor do they mention the use of available pile capacity data bases.

Recently the Florida DOT adopted a set of resistance factors (*1997*) to deal with problems they had encountered in implementing the LRFD Bridge Code in their local practice. Several of the questionnaires indicated that additional research is underway.

Requirements of the LRFD Bridge Code—Resistance factors for the soil strength limit state for driven piles are tabulated in the LRFD Bridge Code and that information is included here as Table 7. The use of the resistance factors presented has proven to be unclear to foundation designers from some of the DOTs that responded to the Questionnaire. Consider some examples: Suppose that the capacity is determined by the SPT-method at a site with piles having end bearing and friction in sand. The specified resistance factor is 0.45 λ_v. If a bearing graph is used for verifying capacity during driving, λ_v is 0.85, giving a resistance factor of 0.38, a value associated with a factor of safety of about 3.7, a considerably larger value than would be used in current practice. As another example, if the ultimate bearing resistance is determined by the SPT method, the resistance factor is specified as 0.45 λ_v. If quality control is by stress wave measurements, λ_v is 0.90. This gives a resistance factor of 0.40 and an equivalent ASD safety factor of about 3.25, a value that is much greater than current practice.

Several methods are given in Table 7 for determining the ultimate bearing resistance. For example, if the SPT method were used together with wave equation analysis and static load test, then what should be used for the multiplier on λ_v? No guidance is offered in the text of the LRFD Bridge Code. Which resistance factor should be selected?

The concept of having the resistance factor related to the static analysis method is inconsistent with current practice. The resistance factors contained in the LRFD Bridge Code 1997 Interim were new. They represented a major change from the approach contained in the AASHTO LRFD Bridge Design Specification of 1994. In the version of 1994, the variable λ_v giving the multiplicative value for the resistance factor, was not present. Probably the preferred solution would be to eliminate the λ_v multiplier and simply specify particular capacity determination methods and quality control procedures.

In general, the use of the currently specified resistance factors (1997 Interim) will give equivalent factors of safety that are usually larger than current practice. The driven pile is the standard foundation for most state DOTs and the use of this Code could substantially increase pile foundation costs.

A resistance factor of 0.9 specified for the structural limit state for timber piles will produce an equivalent factor of safety on strength in the range of 1.6, which equates to an allowable stress of about 12.5 MPa (1800 psi), a much larger value than used in current practice. A 12.5 MPa allowable stress sounds reasonable to a structural engineer accustomed to dealing with timber in the superstructure where the moisture content is low. For piles, the moisture content is higher and therefore the strength is much lower.

A separate resistance factor of 1.15 is specified for driving stresses in timber piles. Current practice is well established at 20 MPa (3000 psi), considerably less than that allowed by the 1.15 resistance factor.

Ontario Bridge Code—The Ontario Highway Bridge Code defines the limit states for strength and specifies resistance factors for deep foundations subjected to axial compression load, as given in Table 8. The resistance factors seem to be quite small when compared with the LRFD Bridge Code. There is no discussion of the methods used to obtain these values.

TABLE 8

RESISTANCE FACTORS FOR DEEP FOUNDATIONS (from Ontario Bridge Code)

Axial Load	Factor
Static analysis, compression	0.4
Static analysis, tension	0.3
Static test, compression	0.6
Static test, tension	0.4
Dynamic analysis, compression	0.4
Dynamic test, compression field measurements and analysis	0.5

Canadian Bridge Code—The design requirements given in the Canadian Bridge Code are quite brief. The soil limit state is dealt with in three separate categories. For load tests, the resistance factor is specified as 0.5 for routine testing and 0.6 for high-level testing and the two levels of testing are defined. However, the definitions are quite qualitative. If dynamic testing is used, the resistance factor is specified as 0.4 for routine analysis and 0.5 for analysis based on parameters obtained from dynamic field measurements.

The third category is geotechnical formula (static analysis) and the particular formula that is used is not given but must be approved. The soil properties used in the formula are specified to be factored by 0.5 for cohesion and 0.8 for friction angle. The capacity resulting from the use of the factored soil properties is then factored by the resistance factors given in Table 9. The resulting capacity shall not be greater than 2.5 EAp/C or the structural nominal resistance. The quantity E is defined as the modulus of elasticity of the pile material, C is the velocity of stress wave propagation, and the other two terms are not defined. The source of this approach is not given.

TABLE 9

RESISTANCE FACTORS (from Canadian Bridge Code)

Type of Unit	Resistance Factor
Precast reinforced concrete	0.4
Cast-in-place concrete	0.4
Expanded-base concrete	0.4
Prestressed concrete	0.4
Steel H-section	0.5
Unfilled steel pile	0.5
Concrete-filled steel pipe	0.4
Wood	0.4

Florida DOT—Table 10 presents the resistance factors recently developed in a calibration study by the Florida DOT (*1997*) whose practice is among the most modern in the United States.

The resistance factor for wave equation analysis seems to be rather small when compared with the value for SPT 97. However, the Florida DOT follows the design process outlined in Figures 6 and 7, so the static analysis is only used to estimate pile length for bidding. It is interesting that a resistance factor is used in this application. This is the first presentation of such an approach and it merits further study. If data is collected on the error in the length

TABLE 10

FLORIDA DOT RESISTANCE FACTORS

Foundation Type	Design Consideration	Design Methodology	Resistance Factor, ϕ
Piles	Compression	SPT97	0.65
		PDA (EOD)	0.65
		Wave equation analysis	0.35
		Static load testing	0.75
	Uplift	SPT 97	0.55
		Static load testing	0.65
	Lateral Load	Structure stability consideration	1.00

estimate on each job, it would be possible to adjust the resistance factor to arrive at the best possible length prediction or the analysis method could be modified.

This calibration effort was undertaken when the Florida DOT had difficulty in using the LRFD Bridge Code in their practice.

Eurocode—This document is complex with a large number of descriptive, limiting conditions that are usually not stated quantitatively. The requirements for driven piles will be summarized briefly and should be read with the understanding that only the more quantitative portions of the document are discussed. In the Eurocode, both driven piles and drilled shafts are handled in a single section and they are discussed here in the same way.

A nominal strength as used in the United States codes is not defined. Rather, the "characteristic bearing capacity" is defined to be

$$R_{ck} = \xi\, R_{cm}$$

where R_{cm} is the "measured capacity," and the values of ξ are given in Table 11. The quantity ξ is determined by the number of load tests used to determine the capacity, with values given for up to three load tests. The values for ξ are specified without regard to the total number of piles on the job. Thus, a job with 2,000 piles and one with 100 piles would have the same ξ-factor for three load tests even though for the large job three tests would be a much smaller sample. The characteristic bearing capacity is further reduced in a second step by "component factors" to obtain the design bearing resistance according to the relationship

$$R_{cd} = \gamma_b\, R_{bk} + \gamma_s\, R_{sk} = \gamma_t\, R_{tk}$$

where R_{bk}, and R_{sk} are the toe and shaft characteristics bearing capacities, respectively, and γ_b and γ_s are the toe and shaft component factors, respectively. The value R_{tk} is the total characteristic bearing capacity where it cannot be divided between toe and shaft and γ_t is the associated component factor on the total capacity then divided into R_{cm}. They have been inverted here to more easily compare with the United States practice.

TABLE 11

ξ FACTORS[1] (from Eurocode 7)

Number of Load Tests	1	2	3
Factor on mean R_{cm}	0.67	0.74	0.77
Factor on lowest R_{cm}	0.67	0.80	0.91

[1]The values of ξ contained in the Eurocode are actually the inverse of those presented here. They were inverted to more easily compare with United States practice.

The values of γ_b, γ_s, and γ_t are given in Table 12. The problem with multiplicative resistance factors is illustrated with this code. If the highest factors and the lowest factors are combined this yields a range of resistance factors of 0.70 to 0.52. The resistance factors contained in Eurocode 7 were developed by calibration to current practice. No probabilistic calibration was performed. The limit state for pile structural failure is not mentioned in the geotechnical code.

Danish Code of Practice for Foundation Engineering—The Danish Code discusses the use of static analysis and makes recommendations for establishing the soil properties from subsurface investigation information. It also recognizes the use of static load tests and the Danish Formula for dynamic analysis of pile capacity based on driving resistance. As discussed above, it divides piles into two safety classes with different resistance factors.

The following is quoted from the Danish Code of Practice:

> The partial coefficients γ_{b1} given for piles and ground anchors and used for static design (performed with characteristic strength parameters) as well as for assessment of the driving resistance, only apply to the load bearing capacity determined from shear strength tests on soil samples. In cases where the load-bearing capacity is established by test loading, γ_{b3} is applied to the piles (anchors) actually subjected to test loading, while γ_{b2} is applied to the other (non-tested) piles or anchors where test loading according to information relating to soil conditions and pile driving etc. may be considered representative.

The resistance factors contained in the Danish Code, and presented in Table 13, were selected to match current practice. No probabilistic calibration was done.

1992 AUSTROADS—This design specification is quite modern, permitting all of the usual methods of capacity

TABLE 12

VALUES OF γ_b, γ_s, γ_t[1] (from Eurocode 7)

Component Factors	γ_b	γ_s	γ_t
Driven piles	0.77	0.77	0.77
Bored piles	0.63	0.77	0.67
CFA piles	0.70	0.77	0.71

[1]The γ-values are inverted here to more easily compare with United States practice.

TABLE 13

PILES AND ANCHORS[1] (from the Danish Code)

	Safety Class	
Partial Coefficient	Normal	High
γ_{b1} without test loading	0.50	0.45
γ_{b2} with test loading	0.63	0.57
γ_{b3} for piles and anchor actually subjected to test loading	0.71	0.65

[1]The γ-values are inverted here to more easily compare with United States practice.

determination. The range of resistance factors covering the various methods is quite large, as shown in Table 14. Compared with other codes, the value for static load test seems to be quite high. The methods that were used to obtain the resistance factors are not described. Probably they were calibrated by comparison with existing ASD practice.

Australian Standard—The Australian Standard for the design and installation of piles is concise and specific. All aspects of design are covered in addition to the strength requirements. The resistance factors, given here in Tables 15 and 16, are complete and reasonable. No information is available as to the methods used to generate the factors. It is interesting that resistance factors are specified to be within a range, with suggestions contained in Table 16 for selecting the appropriate value within the range. Structural design specifications have traditionally given nominal strength values that were not-to-exceed quantities and lower values than those specified were used, as appropriate, in the judgment of the engineer. This approach would imply that the lower number in the given range is unnecessary. However, the availability of the guidance given in Table 16 makes the range of values useful to the designer. Some of the factors of Table 16 will not be relevant to all of the capacity verification contained in Table 15. For example, the method used in selecting geotechnical properties is irrelevant to the reliability of the capacity obtained by signal matching.

No recommendations are given regarding the use of measurements made at the end of driving versus beginning of re-strike. The specification does not deal extensively with changes in the resistance factor with increased numbers of tests. However, there is a qualitative recognition of the value of quality control testing in some of the recommendations of Table 16.

The specification also includes some resistance factors for the structural limit state for concrete and timber piles. This is the most complete pile design specification of all those reviewed.

American Petroleum Institute—The code of the American Petroleum Institute, "Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms—Load and Resistance Factor Design" deals quite specifically with the design of driven piles in the soil types usually encountered in the offshore environment. The specification emphasizes static analysis and gives specific methods for common soil types. Dynamic methods of capacity determination, either wave equation or dynamic measurements, are not mentioned, probably because most offshore piles are designed with a heavier steel section at the mud line to carry lateral loads and the pile must be driven to depth. In practice, the capacity is verified by wave equation analysis and the hammer performance is checked with dynamic monitoring. If the capacity is inadequate, a serious problem results. Due to the heavier pile section at the mud line, used to resist lateral loads, the pile cannot be simply driven deeper or it will lose lateral

TABLE 14

MATERIAL RESISTANCE FACTORS FOR PILES (from AUSTROADS Foundation Code)

(a)	Routine proof load tested	0.8
(b)	Load tested to failure	0.9
(c)	Piles analyzed by dynamic formulae or wave equation methods based on assumed driving system energy and soil parameters	0.4 to 0.5*
(d)	Piles subjected to closed-form dynamic solutions, e.g., Case method	0.5
(e)	Piles subjected to closed-form dynamic solutions correlated against static load tests or dynamic load tests using measured field parameters in a wave equation analysis (e.g., CAPWAP)	0.6
(f)	Piles subjected to dynamic load tests using measured field parameters in a wave equation (e.g., CAPWAP)	0.8

*Note: A value of 0.4 should be used for cohesive soils and structures where permanent loads dominate. In noncohesive soils and for structures where transient loads dominate, values up to 0.5 may be used.

TABLE 15

RANGE OF VALUES FOR RESISTANCE FACTORS FOR PILES (from the Australian Standard)

Method of Assessment of Ultimate Geotechnical Strength	Range of Values of ϕ_g
Static load testing to failure	0.70–0.90
Static proof (not to failure) load testing[1]	0.7–0.90
Dynamic load testing to failure supported by signal matching[2]	0.65–0.85
Dynamic load testing to failure not supported by signal matching	0.50–0.70
Dynamic proof (not to failure) load testing supported by signal matching[1,2]	0.65–0.85
Dynamic proof(not to failure) load testing not supported by signal matching[1]	0.50–0.70
Static analysis using CPT data	0.45–0.65
Static analysis using SPT data in cohesionless soils	0.40–0.55
Static analysis using laboratory data for cohesive soils	0.45–0.55
Dynamic analysis using wave equation method	0.45–0.55
Dynamic analysis using driving formulae for piles in rock	0.50–0.65
Dynamic analysis using driving formulae for piles in sand	0.45–0.55
Dynamic analysis using driving formulae for piles in clay	Note 2
Measurement during installation of proprietary displacement piles, using well established in-house formulae	0.50–0.65

Notes:
[1] ϕ_g should be applied to the maximum load applied.
[2] Signal matching of the recorded data obtained from dynamic load testing should be undertaken on representative test piles using a full wave signal matching process.
[3] Caution should be exercised in the sole use of dynamic formulae (e.g., Hiley) for the determination of the ultimate geotechnical strength of piles in clays. In particular, the dynamic measurements will not measure the 'set up' which occurs after completion of driving. It is preferable that assessment be first made by other methods, with correlation then made with dynamic methods on a site-specific basis if these latter are to be used for site driving control.

For cases not covered in Table 15, values of ϕ_g should be chosen using the stated values as a guide.

TABLE 16

GUIDE FOR ASSESSMENT OF RESISTANCE FACTORS FOR PILES (from the Australian Standard)

Circumstances in Which Lower End of Range May be Appropriate	Circumstances in Which Upper End of Range May be Appropriate
Limited site investigation	Comprehensive site investigation
Simple method of calculation	More sophisticated design method
Average geotechnical properties used	Geotechnical properties chosen conservatively
Use of published correlations for design parameters	Use of site-specific correlations for design parameters
Limited construction control	Careful construction control
Less than 3 percent piles dynamically tested	15 percent or more piles dynamically tested
Less than 1 percent piles statically tested	3 percent or more piles statically tested

strength. A variety of methods are used, including driving a smaller pile inside of the existing pile to a greater depth and grouting the inner and outer pile together. A resistance factor of 0.7 is specified for operating environmental conditions and 0.8 for extreme environmental conditions. In the case of extreme loads, a larger resistance factor is justified because a lower safety margin is acceptable.

DRILLED SHAFT DESIGN PROCEDURE

It is useful to review the design and construction procedures as illustrated in the flow chart of Figure 8. After selecting a drilled shaft as the deep foundation type in DS1, the capacity determination method and the quality control procedures are selected in DS2. For the factored cap loads supplied by the structural engineer, the foundation designer must select the factored loads per shaft to be used as a basis for design. The geotechnical engineer will have a general idea of the most economical capacity for a single shaft. Then he or she can select the number of shafts and from that the value of the factored load per shaft for design. With the capacity determination method and quality control procedures known, the resistance factor can be selected in DS3. Final issues regarding the required nominal strength for design are established. Then in DS4 the shaft diameter, length, and reinforcement are determined so that the strength requirements are met. The diameter and length are chosen to carry the required load based on a static analysis. The reinforcement is selected to carry the factored load but may be designed primarily to provide structural resistance to uplift and lateral loads.

The shaft may be installed with no further methods used for determination of nominal strength beyond the use of a static analysis. In some cases, static load tests are used to verify the pile capacity and in this case it is clear that the reliability of the shaft has been improved. Therefore, a larger resistance factor can be justified.

The Osterberg test is being used with increasing frequency and it offers the opportunity for increased reliability of the foundation. Dynamic tests are not frequently used for capacity determination in the United States, but this method of determining capacity is very common in Europe, China, and Southeast Asia. Use of these dynamic methods results in increased foundation capacity reliability.

With the selection of the strength determination method and quality control procedures, a resistance factor is known and the strength portion of the design is complete. Serviceability is then evaluated in DS5 and, if necessary, the design is modified. The plans and specifications are prepared and a contractor is selected. Then a test shaft is installed and tested, if testing is required (DS9), and if the capacity is unsatisfactory the design is modified. The production shafts are installed using the specified quality control procedures.

Nominal Strength Determination

As with driven piles, there are two strength limit states that must be considered in the design of drilled shafts: the structural strength of the shaft and the strength of the soil surrounding the shaft or the soil-shaft interface.

Strength Limit State—Structural

If a drilled shaft fails structurally, then at some modest level of displacement the shaft will begin to lose strength and that strength will continuously decrease with further displacement. The structural element will not be able to carry the load and is rendered unsatisfactory. Depending on the amount of reinforcement, the failure can range from ductile to quite brittle, but for typical shafts there will not be sufficient spiral reinforcement to assure ductile failure. If lateral or eccentric loads are present, combined bending and axial loads can cause failure and this failure mode must be considered. In all of these cases, failure implies the possibility of strength reduction with increased displacement.

The drilled shaft is cast in place so quality control must be performed to assure that the required concrete strength is achieved and that no discontinuities are created during the construction process. Cylinders are cast from concrete samples during the concreting process to verify the concrete strength. These procedures are well established in the construction of all cast-in-place concrete structures and they apply here as well.

Verifying the structural uniformity and continuity of the shaft has proven to be difficult. A variety of nondestructive testing techniques (NDT) have been developed (*Stain 1982, Davis et al. 1974, Hussein et al. 1993*) and their use has become widespread.

Most of the testing in the United States has been done using two basic methods: low strain impact and cross-hole sonic logging. In the low strain impact test an accelerometer (or geophone) is attached to the top of the shaft, which is struck with a small hand-held hammer, generating an axial stress wave in the shaft. Stress wave reflections are sensed by the accelerometer (or geophone), processed, and displayed. The operator examines the measurements and judges from the signal if cross-section reductions exist in the shaft. The same measurement can be examined in the frequency domain to assist in evaluating the measurement or the velocity (obtained from the geophone or by integrating the measured acceleration) can be integrated to obtain a scaled representation of the shaft cross-sectional area.

Cross-hole sonic logging is performed by installing small-diameter holes the length of the shaft. Usually they are installed with small pipes of plastic or steel during shaft construction. They are filled with water and a piezoelectric impact device is lowered and raised in one pipe together with a sensor in one of the other pipes. The stress wave travels between the two pipes, is sensed and recorded. Voids and weak concrete can be detected.

Some gamma logging is also used. This method is primarily used in California. It has not been popular in other states because of the problems with storing and handling a radioactive source.

Often the stresses on the shaft are quite low and the structural loads can be carried even by a shaft that has discontinuities. The dimensions of the shaft may be set by requirements for load transfer to the soil not for the structural requirements of the shaft. If rather low design stresses result, the shaft could carry the load even if discontinuities are present. Hence, identification of a discontinuity may not be an indication that a drilled shaft structural capacity is unsatisfactory.

However, the shaft design is frequently controlled by lateral loads rather than axial loads, particularly in high seismic areas. In this case, it is important that the shaft have adequate bending strength near the top and discontinuities in this region could be very undesirable for the bending strength of the shaft. If the discontinuities are in the lower part of the shaft they may be less important. Some state DOTs have found the use of NDT to be an effective means of quality control.

Strength Limit State—Soil

The nominal strength determination for the soil strength limit state is simpler for drilled shafts than for driven piles because the drilled shaft installation procedure does not produce quantitative information that can be used in the determination of nominal strength.

For many cases, nominal strength is determined by static analysis only. The length of shaft of the selected diameter required to carry the load is determined by static analysis and the shaft is constructed to that length and diameter. Static capacity prediction has been studied extensively over the past two decades and many methods have been proposed. Some large data bases have been assembled but they were not available at the time that the AASHTO Code calibration was done (*Barker et al. 1971*). The data generated since the completion of the work reported in NCHRP Report 343 could be used to generate statistical information on the accuracy and reliability of the various static analysis methods. Drilled shaft performance is strongly dependent on the construction procedures and good quality inspection is particularly important.

It is fairly common to perform static load tests on one or a small number of drilled shafts at a site. Usually the tested pile will be the first one installed and, due to the cost of the test, it may be the only one tested. The Osterberg test (*Osterberg 1984*) was developed to reduce the cost of static testing. In this test a specially constructed hydraulic cell (Osterberg Cell) is installed at the base of the shaft or at some point along the lower part of the length of the shaft. During the testing process, the cell is expanded with hydraulic fluid, the upward displacement of the top and the downward displacement at the bottom of the shaft are measured. Failure can occur due to upward displacement of the shaft or downward displacement of the toe. The failure load of the shaft is assumed to be at least twice the measured load. The Osterberg test does not determine the ultimate capacity of a drilled shaft because neither toe nor shaft has reached its maximum load. However, it does provide a lower bound and is, therefore, a very useful test.

Dynamic monitoring is now being used with increasing frequency for the capacity testing of drilled shafts. In Europe, China, and Southeast Asia this procedure is generally accepted and widely used. Recently, a number of these tests have been used in the United States (*Townsend et al. 1994*). The dynamic testing of drilled shafts uses the same procedures that would be used for the re-strike of a driven pile except the pile driving hammer is not available. Usually a drop hammer of appropriate size is used. It may be necessary to fabricate a special hammer for the particular requirements of the job.

Requirements of the LRFD Bridge Code—The LRFD Bridge Code specifies methods for determining nominal strength including static analysis, and static load test. By reference, it also includes dynamic monitoring and signal matching capacity predictions. The principal emphasis is placed on static analysis methods (*Reese 1984*) and the methods used are specifically stated. This approach is open to debate because other, and possibly, obscure methods may be known to be superior for specific locations based on the local experience of the designer.

Although use of the Osterberg Test is not mentioned in the Code, its use is expanding. It is less expensive to perform than a standard static test in cases of high capacity, and it has the advantage of providing results similar to a conventional static test. When drilled shaft capacities become very large, say greater than about 1,000 tons, static load testing becomes very difficult and expensive. The Osterberg Test becomes the only practical way to perform a static test.

Dynamic monitoring is a method that is well proven with extensive correlation data available. A major advantage of the dynamic test is that it is much less expensive than a conventional test, so many more tests can be run for the same cost. In cases of large diameter shafts with the toe in soil, the displacement necessary to mobilize the capacity is difficult to achieve with a dynamic test. It will produce a lower bound on capacity.

Resistance Factors

The resistance factors contained in the LRFD Bridge Code are given in Table 17. A large number of different design methods are given with associated resistance factors. Also, static testing is included with the same resistance factor used for driven piles. No resistance factors are given for Osterberg testing or dynamic testing. The basis for the generation of the resistance factors was the estimated variability of the capacity predictions as described in NCHRP Report 343 (*Barker et al. 1991*) and in volumes I and II of *Design and Construction of Driven Pile Foundations* (*Hannigan et al. 1996*). Several significant data bases are now available and could be used to improve the quality of the resistance factors.

The resistance factor for static load test is given as 0.80, the same value used for driven piles. Driven piles will vary in length when driven to a blow count criteria across a specific site. Thus, site variability is accounted for in the installation process for driven piles. This is not the case for drilled shafts since they are usually drilled to a constant depth. A recognition of the effect of site variability on drilled shafts as compared to driven piles could be made by adjusting the relative size of the resistance factors.

TABLE 17

RESISTANCE FACTORS FOR GEOTECHNICAL STRENGTH LIMIT STATE IN AXIALLY LOADED DRILLED SHAFTS (from LRFD Bridge Code)

Method	Soil	Condition	Resistance Factor
Ultimate Bearing Resistance of Single Drilled Shafts	Side Resistance in Clay	α-method (Reese & O'Neill, 1988)	0.65
	Base Resistance in Clay	Total Stress (Reese & O'Neill, 1988)	0.55
	Side Resistance in Sand	Tourma & Reese (1974) Meyerhof (1976) Quiros & Reese (1977) Reese & Wright (1977) (Reese & O'Neill, 1988)	See Discussion in Article 10.8.3.4
	Side Resistance in Rock	Carter & Kulhawy (1988) Horvath and Kenney (1979)	0.55 0.65
	Base Resistance in Rock	Canadian Geotechnical Society (1985) Pressure Method (Canadian Geotechnical Society, 1985)	0.50 0.50
	Side Resistance and End Bearing	Load Test	0.80
Block Failure	Clay		0.65
Uplift Resistance of Single Drilled Shafts	Clay	α-method (Reese & O'Neill, 1988) Belled Shafts Reese & O'Neill (1988)	0.55 0.50
	Sand	Touma & Reese (1974) Meyerhof (1976) Quiros & Reese (1977) Reese & Wright (1977) (Reese & O'Neill, 1988)	See Discussion in Article 10.8.3.7
	Rock	Carter & Kulhawy (1988) Horvath & Kenney (1979) Load Test	0.45 0.55 0.80
Group Uplift Resistance		Sand Clay	0.55 0.55

It is noted that resistance factors are not given for the shaft resistance of drilled shafts in sands, although several methods are suggested. Also, there is no discussion of resistance factors for drilled shafts subjected to lateral loads. In this case, the lateral load behavior is challenging to deal with since the soil behavior is probably limited by displacement and is a serviceability condition. On the other hand, the shaft structural strength due to bending induced by lateral loads is a strength problem that may require different resistance factors.

The effect of additional testing could be used to adjust the values of the resistance factors. Data is available to measure the accuracy and reliability of dynamic testing of drilled shafts and it can be used to generate the required resistance factors.

In all of the other codes reviewed, drilled shafts were included together with driven piles and, in those cases, the tabular data on resistance factors is contained in the section on driven piles. It is interesting that, in most cases, no design specification was given specifically for drilled shafts.

SERVICEABILITY DESIGN FOR DEEP FOUNDATIONS

Foundation settlement is the most common consideration that is evaluated in examining serviceability for foundations. The procedures used for settlement evaluation of deep foundations are empirical and have not always been examined in foundation design in the past. This is one of the design considerations that have been added and emphasized in the FHWA educational programs that have been presented in the past few years. In the case of abutments, lateral displacement must also be a consideration.

The LRFD Bridge Code specifies methods for determining settlement. The same question arises as mentioned above, "Should specific methods be required, thus excluding others that may be well established in some locations."

The other important serviceability requirement is lateral load behavior. This problem is treated by the LRFD Bridge Code as a serviceability consideration. It may also be necessary to consider lateral behavior from a strength basis, particularly in dealing with the structural failure mode as discussed above. In the case of extreme events such as earthquake or vessel impact, strength may be the limiting condition. The limit state behavior is strongly nonlinear and in this regard a strength based design requirement would be appropriate. However, deflection is an important limit state and because the performance is non-linear a serviceability limitation is proper. A method of analysis developed by Reese dominates North American practice and it has been added to the AASHTO LRFD Code in its most recent interim *(AASHTO 1994)*. Load factors and load combinations should be examined to verify that they are appropriate for each limit state.

CHAPTER FIVE

EARTH RETAINING SYSTEMS

INTRODUCTION

The fundamental problem in the area of earth retaining systems design is the fact that both the load and the resistance sides of Equation (1) contain soil parameters. Thus, care must be used in applying load and resistance factors to assure that appropriate margins of safety are used. All or part of the load may come from earth loads. This changes the problem substantially from what is the case for foundations. For example, a bridge abutment may receive part of its applied load from earth pressure and part from bridge dead and live load. Therefore, the load factors used for the earth pressures must be appropriately scaled to match the load factors applied to the structural loads. There must be a correlation between the load factors for the two load types in the design of abutments. In traditional designs involving the stability of earth slopes, the factors of safety have seemed to be rather small when compared with factors of safety used in foundation design. In traditional procedures used for slope stability, loads and strengths were evaluated at their ultimate state and then a global factor of safety was used. In the LRFD approach for stability considerations, the previously used global factor of safety is separated into at least two parts.

The probabilistic approach described in chapter 2 implies that rational factors would be developed based on a probability of occurrence and magnitude of both the superimposed loads and the earth loads. It is unlikely that such an analysis will be possible in the near future due to the lack of statistical data on wall failures. However, the problem is still more complex. Separate factors must be determined for both the load and the strength sides of the limiting condition of Equation (1). For structures and foundations, this is a realistic task because it is possible to examine the variability of the loads separately. Large volumes of traffic load data are available, in addition to even larger amounts of bridge strain history data. Extensive testing of structural elements and deep foundations has been completed and strength models have been generated and checked against the test data. So, two completely separate problems that can be evaluated separately are given—load and strength variability.

The problem of generating the LRFD method for earth retaining systems is much more difficult than for structures and foundations. Large volumes of load and strength data are not available for earth retaining structures and it will be difficult to generate such information. For example, an earth retention system could be loaded to soil failure with some system of mechanical loads, which would give an insight into its resistance characteristics. The earth loads in an actual retaining wall, however are affected by the deformations associated with failure. The clear separation of soil loads from resistances in such tests will still be difficult. The work of NCHRP Project 20-7/88 will produce an improved calibration of resistance factors for earth retention systems, and resulting specifications will be published in the AASHTO Specifications.

The problem of earth retaining systems design by LRFD has been studied and discussed extensively in Europe during the development of the Eurocode. The basic issues of this discussion were presented to the recent FHWA Scanning Team by Dr. Krebs Ovesen at the Danish Geotechnical Institute and were clearly illustrated by an example, the basic elements of which are presented here. Consider the problem shown in Figure 10 without regard to specific codes. Assume that the load P_1 is a tank filled with water. This applies what Eurocode calls a variable load (live load) with a load factor from typical codes ranging from 1.5 to 1.75, depending on the specific code involved. Load P_2 is the weight of the soil inside the failure surface that acts together with the water tank loading to cause failure. It is unfavorable, so the load factor would be 1.2 to 1.35 depending on the specific code being used. Load P_3 is favorable, that is it stabilizes the failure surface, so it would have a load factor of 0.9.

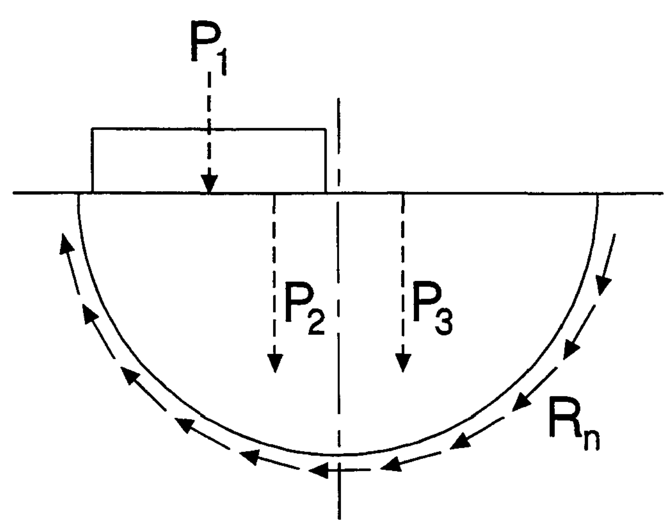


FIGURE 10 Stability example.

Several questions can then be asked. For the load coming from the full water tank, is it possible that the water force would have substantial variability? Is it reasonable that the soil load P_2 could have an error of 20 to 35 percent or even 10 percent in the case of load P_3? Now if the nominal strength, R_n, is calculated using very conservative soil properties as discussed in chapter 3 and then reduced by a resistance factor, very conservative designs could result. It may be possible that in some cases the analysis would indicate instability even without the external load coming from the water tank! This example illustrates some of the problems associated with LRFD applied to soil stability problems.

DESIGN PROCEDURE FOR EARTH RETAINING SYSTEMS

The limit states that must be considered in the design of earth retaining systems are, generally, the same for all systems although in some cases the nature of the design is such that particular limit states may not be present. The strength limit states are as follows:

1. Bearing resistance
2. Lateral sliding
3. Excessive loss of base contact
4. Overall instability
5. Pull out of anchors or soil reinforcements
6. Structural failure.

The design procedure for earth retaining systems has been discussed by Withiam et al. (1997). The design process is presented in Figure 11 with some amplification on the procedure presented by Withiam et al. and it will be reviewed here. Block 1 defines the geometry of the design. This will be set by the requirements placed on the earth retaining system. What height of earth must be retained? Are external loads applied? What are the soil conditions for the wall foundation and for the backfill material (or the retained earth)? What type of wall is most appropriate for the requirements?

At the end of this investigation a decision must be made in Block 2 regarding the wall type to be designed. Four different categories are considered: cantilever retaining walls and abutments, anchored walls, mechanically stabilized earth walls, and prefabricated modular walls. Other wall types could have been considered but these were selected because they are covered specifically in the LRFD Bridge Code. The design process for each of these wall types differs to a small degree, but is summarized in the single flow chart in Figure 11. A schematic of each of the four wall types considered here is shown in Figure 12.

After selection of a wall type, the configuration of the selected wall is established in Block 3. The wall geometry is established, reinforcement or anchors are selected if necessary, and wall details are established so that a preliminary design is set. In Block 4, the soil properties are determined from the soil characteristics established in

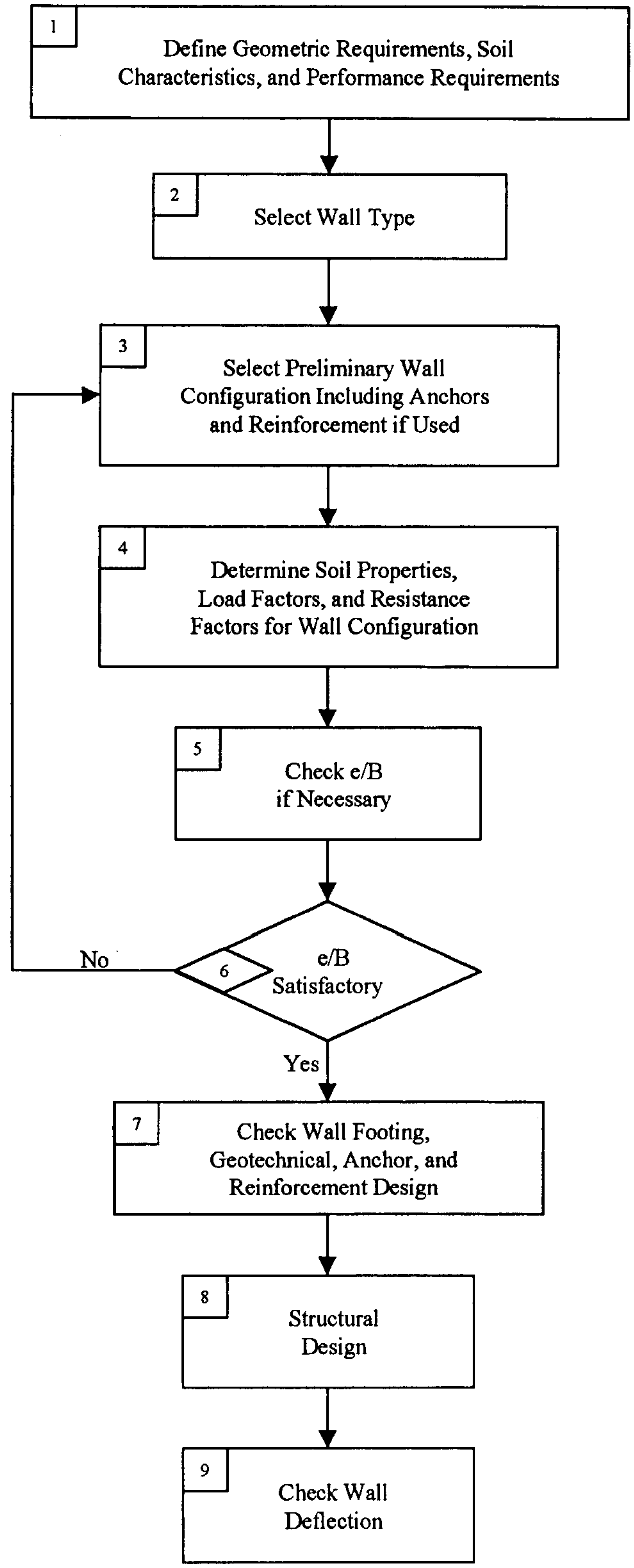
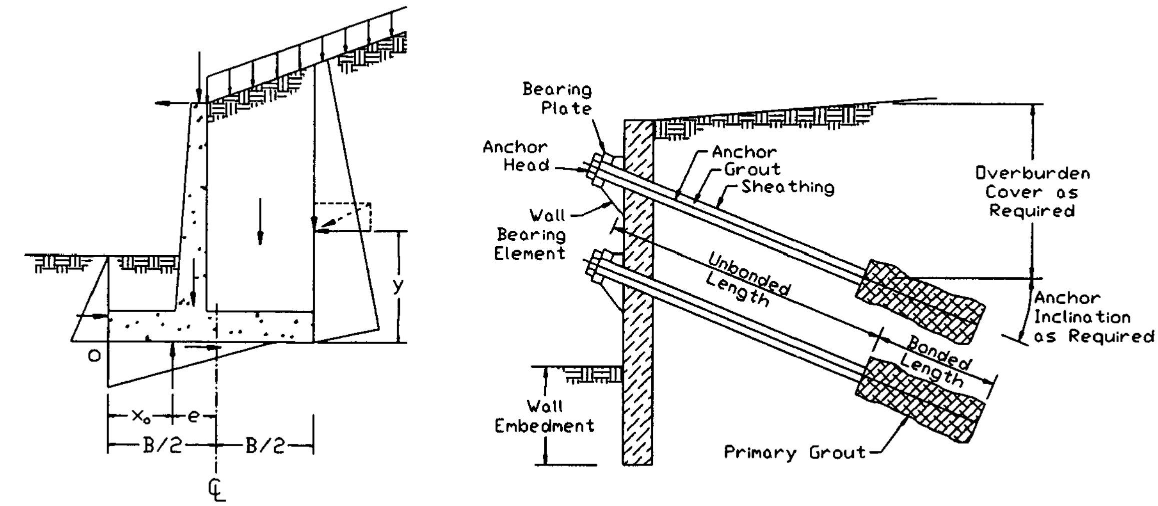
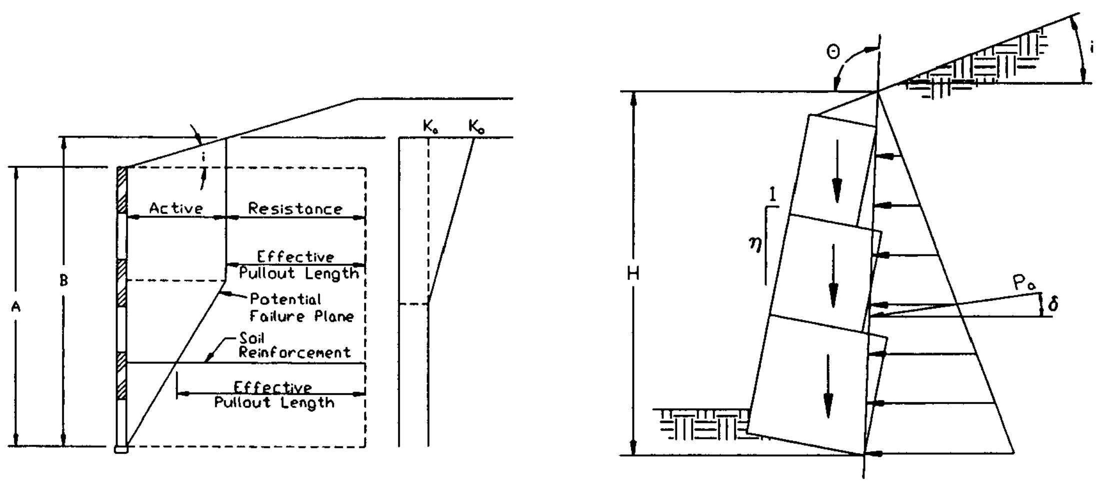


FIGURE 11 Earth retaining system.



a) Cantilever Wall or Abutment b) Anchored Wall



c) Mechanically Stabilized Earth Wall d) Prefabricated Modular Wall

FIGURE 12 Wall types (from the AASHTO LRFD Bridge Design Specification, 1st Edition, 1997 Interim.)

Block 1. Loads, load factors, and resistance factors are also determined. At the end of this activity, it is possible to check the first five limit states of the six listed above.

The abutment design is of interest because it may carry substantial structural loads, which are factored loads with well-established load factors. In Figure 12a the structural loads are shown acting at the top of the wall. For bridge abutments, the geometry of the wall may be quite different from that shown here, but the design approach will still apply. The determination of earth pressures is discussed in chapter 3.

The LRFD Bridge Code specifies that the eccentricity of the base reaction be kept within the middle one-half of the footing width as indicated in Block 5 of Figure 11. As shown in Figure 11, e is the eccentricity of the applied load on the foundation and B is the width of the footing. Limiting values of e/B were developed by correlation with ASD practice (*Barker et al. 1991*). This correlation recognized the influence of the load factors in moving the location of the resultant force and assumed that ASD was giving satisfactory results. This study was done with the LFD load factors *(AASHTO 1977),* which are different from the LRFD values, so the comparable limiting e/B values will change. Withiam et al. *(1997)* state that the differences will be small but probably that conclusion should be given at least a limited check.

When e/B has been satisfied, the footing design is checked in Block 7 according to the requirements on the design of spread footings discussed in chapter 4. Separate requirements are in place for footings on soil and rock. When these conditions are satisfied, the structural design is completed in Block 8, possibly by the structural engineer and the wall deflection is determined in Block 9. At any stage in the process it may be necessary or desirable to go back to Block 3 and select a new wall configuration or possibly even to Block 2.

A typical anchored wall cross section is shown in Figure 12b. The construction of this type of system is normally performed from the top down. First, the wall is installed from the original ground surface. It can consist of driven soldier piles, sheet piles, or various types of reinforced cast-in-place walls usually constructed under slurry. After these elements are installed the excavation proceeds to somewhat below the level of the top anchors. Those anchors are then installed and after the grout has hardened they are tensioned and proof tested. The anchors can also be tested to determine their pull-out strength. Excavation proceeds to the next level of anchors and they are installed, continuing in this way until the wall is completed. If soldier piles are used, lagging is installed between the piles as the excavation and anchor installation proceeds. The design process for this system is quite direct. The design geometry and the requirements for earth retention will be defined by the site. So, a wall configuration is selected together with the embedment depth.

A typical mechanically stabilized earth wall (MSE) cross section is shown in Figure 12c. The wall is constructed by "assembling" the earth-structure system from the bottom up. Reinforcing material is placed and attached to the wall facing elements as the fill is placed, gradually constructing a stabilized earth mass. Except for the evaluation of pull-out and rupture of the reinforcement, the limit states for MSE walls are identical to those for cantilever retaining walls.

A typical prefabricated modular wall section is shown in Figure 12d. The wall is constructed by assembling from the bottom up. The bottom element is installed after excavating the necessary material. These systems may or may not have a structural footing. It is then backfilled, possibly with select material and the next element is installed and backfilled. The process is continued until the wall is complete.

The design of prefabricated modular walls is similar to other gravity and semi-gravity wall types, except that only 80 percent of the weight of soil backfill in the modules is considered effective in resisting overturning.

Several different configurations are possible, depending on the characteristics of the soil and the geometry of the material to be retained. For example, the front edge of the individual wall elements can be offset and the wall can be battered. The design process of prefabricated modular walls requires that the limit states be checked for each prefabricated module.

NOMINAL STRENGTH DETERMINATION FOR EARTH RETAINING SYSTEMS

The soil strengths that must be determined in the process of designing an earth retaining system are as follows:

1. Soil/rock bearing resistance (all walls)
2. Soil/rock sliding resistance (all walls except anchored walls)
3. Passive embedment (usually only anchored walls)
4. Anchor or soil reinforcement pull-out strength anchored and MSE walls only)
5. Overall stability (all walls).

Soil or rock bearing resistance is discussed in chapter 4 in the section on spread footings. Methods are discussed for determining strength using presumptive values, in situ tests, or rational methods. Linked to this strength problem is the limitation of e/B ratios. There are two possible reasons for limiting e/B ratios. It may be considered undesirable to have the footing "lift-off" and overturning of the earth retention system is also an obvious failure mode. If the bearing pressures do not exceed the nominal strength then overturning will not occur. Is it physically undesirable to have bearing pressures at zero? Long tradition in the United States has treated this condition as undesirable, but perhaps it was undesirable only because it was associated with excessive compression stresses. It seems that perhaps the requirement of a limiting e/B ratio is unnecessary. It is interesting to note that neither the Eurocode *(European Committee for Standardization 1995)* nor the Danish Code of Practice *(Danish Geotechnical Institute 1985)* contains this specific limitation although high compressive stresses will be associated with large e/B ratios. These basic questions should be examined to arrive at the correct solution and in this way the best code limitations are established.

The strength of anchored walls for bearing pressure is evaluated in a fashion similar to other wall types. Nominal foundation bearing pressures must be determined.

Soil or rock sliding resistance is also defined in chapter 4, together with procedures for establishing resistance. Sliding of the base of a wall may be controlled by embedding the wall in the base soils. Then the passive resistance of the base soil acts to stabilize the wall. All codes that deal with this topic spend some effort in the discussion of passive soil resistance because of concern that the passive strength could be disturbed, leading to failure of the wall.

The overall stability must be checked by a limiting equilibrium analysis. Here the issues discussed above regarding loads and load factors are also important. No further guidance is given on this topic in the LRFD Bridge Code. Is the overall stability to be evaluated using the loads and load factors specified for other aspects of the design? How shall the failure surface be modeled? This aspect of the design could control so the methods to be used must be defined.

The structural design of the cantilever wall can use the nominal strength definition contained in the reinforced concrete section of the LRFD Bridge Code with the appropriate limit states. The earth pressure loads should be supplied by the geotechnical engineer and structural loads for abutments provided by the structural engineer. (Presumably the structural portion of the retaining wall is designed by the structural designer, although this aspect of the design may as well be done by the geotechnical engineer.)

In the design procedure for anchored walls as described in the flow chart of Figure 11, the general wall configuration is selected, the nominal soil pressures are found, and then the factored loads are determined. Anchor loads can then be determined. The LRFD Bridge Code implies that the anchor loads are found by tributary area but this procedure is not specifically stated. Anchors can be designed using presumptive anchor resistances given in the LRFD Bridge Code in Table 11.8.4.2-1 for soil and Table 11.8.4.2-2 for rock. The load-deformation behavior and a fractional portion of the ultimate capacity are determined by conducting proof or performance tests of each constructed anchor.

The wall is subjected to horizontal loads from the soil pressures, hence the dominant force that it must carry is bending moment. Procedures for determining factored wall moments are given in the Commentary part of the LRFD Bridge Code in Section 11.8.5.2 in the form of moment coefficients. The procedures are similar to those used for slab design in the ACI Building Code (*ACI 1995*) where comparable coefficients are given in a simplified procedure for determining design moments. Axial forces are generated in the wall due to the inclination of the anchor. These forces must be supported in soil or rock at the tip of the embedded wall.

If the structural aspects of the design are performed by the structural engineer, then a close cooperation between the structural engineer and the geotechnical engineer is necessary. Stability is checked by the use of a limiting equilibrium analysis.

RESISTANCE FACTORS FOR EARTH RETAINING STRUCTURES

No resistance factors are contained in the LRFD Bridge Code specifically for cantilever retaining walls and abutments. The necessary resistance factors all come from the provisions for foundation design, depending on the type of foundation used to support the wall. The resistance factors for the reinforced concrete structural elements are contained in the appropriate superstructure sections of the code.

The resistance factors specified by the LRFD Bridge Code for anchored walls are tabulated in Table 18. Almost all of these resistance factors are concerned with the anchor strength and pull-out resistance. The only resistance factors associated with the soil are those governing passive resistance. The resistance factors for the other geotechnical limit states are contained in the relevant foundation sections of the Code and are discussed in chapter 3 of this document. Resistance factors are given for the flexural structural resistance of the "vertical elements." Specifically, the vertical elements could include soldier piles or slurry walls. In the structural portion of the LRFD Bridge Code the same structural behavior is specified. The same resistance factors would not be used in the superstructure. Also it may be appropriate to check the shear failure mode around the anchor for slurry walls and resistance factors would then be required.

The resistance factors for MSE walls specified by the LRFD Bridge Code are tabulated in Table 19. These resistance factors are concerned only with the strength of the soil reinforcement, except for one pull-out resistance factor. The resistance factors for the relevant geotechnical limit states are contained in the spread footing sections of the Code and are discussed in chapter 3.

No resistance factors are contained in the LRFD Bridge Code specifically for prefabricated modular walls. The necessary resistance factors would all come from the sections on spread footing design. The resistance factors for the structural elements are contained in the appropriate superstructure sections of the code.

OTHER CODES AND STANDARDS

Canadian Bridge Code

The Canadian Bridge Code discusses the design and construction of piers, bin-type walls, and sheet walls. All of

TABLE 18

RESISTANCE FACTORS FOR ANCHORED WALLS (from the LRFD Bridge Code)

Wall Type and Condition		Resistance Factor
Anchored Walls		
Overturning	Passive resistance of vertical elements	
	• In soil	0.60
	• In rock	0.60
	Anchor pullout resistance	
	• Sand	
	Correlation with SPT resistance-corrected for overburdened pressure	0.65
	Pullout load tests	0.70
	• Clay	
	Correlation with unconfined compressive strength	0.65
	Using shear strength from lab tests	0.65
	Using shear strength from field tests	0.65
	Pullout load tests	0.70
	• Rock	
	Correlation with rock type only	0.55
	Using minimum shear resistance measured in lab tests—soft rock only	0.60
	Laboratory rock-grout bond tests pullout	0.75
	Load tests	0.80
Tensile resistance of anchor	Permanent	
	• Yielding of the gross section	0.90
	• Fracture of the net section	0.85
Flexural capacity of vertical elements	Permanent	0.90
	Temporary	1.00

TABLE 19

RESISTANCE FACTORS FOR MECHANICALLY STABILIZED EARTH WALLS (from the LRFD Bridge Code)

Wall Type and Condition		Resistance Factor	
Tensile resistance of metallic reinforcement			
	Strip reinforcements		
	• Yielding of gross section less sacrificial area	0.85	
	• Fracture of net section less sacrificial area	0.70	
	Grid reinforcements		
	• Yield of gross section less sacrificial area	0.75	
	• Fracture of net section less sacrificial area	0.60	
	Connectors		
	• Yielding of gross section less sacrificial area	0.75	
	• Fracture of net section less sacrificial area	0.60	
Strength limit state tensile resistance of polymeric reinforcement	From laboratory creep tests of 10,000 hours, minimum duration	0.27	
	From wide-width tensile test—ASTM D4595	Ultimate at 5% Strain*	
	• Polyethylene	0.05	0.08
	• Polypropylene	0.05	0.08
	• Polyester	0.11	0.16
	• Polyamine	0.09	0.14
	• High Density Polyethylene	0.09	014
Service limit state tensile resistance of polymeric reinforcement	From laboratory creep tests of 10,000 hours minimum duration	0.41	
	From limit state tensile strength of "4b"	0.66	
Ultimate soil pullout resistance		0.90	

*The two different values for strength limit state resistance factors are based on different evaluations of the wide-width tensile test. The smaller values are for the ultimate strength and larger values are for the 5% strain strength. These values are small because they include construction damage, creep, long-term degradation and biological and chemical deterioration.

TABLE 20

MATERIAL RESISTANCE FACTORS FOR ULTIMATE LIMIT STATE
(from AUSTROADS Bridge Design Code)

Material Factor for Ultimate Limit States	Value
Soil Properties	
Unit weight of soil	1.0
Tangent of angle of internal friction (tan ϕ)	0.8
Tangent of friction angle for soil/structure interface (tan δ)	0.8
(a) Cohesion (stability and earth pressures)	0.7
(b) Cohesion (ultimate resistance of foundations)	0.5
(c) Cohesion obtained by correlation with CPT data	0.5
(d) Cohesion obtained by correlation with pressuremeter tests	0.5

the requirements are general and do not contain any of the detailed requirements given in the LRFD Bridge Code. The procedures used for spread footings are used for walls.

Danish Code of Practice

The Danish Code of Practice discusses the design of earth retaining systems only briefly. They do not mention mechanically stabilized earth walls and in a recent visit to Denmark none of these systems were seen in uses characteristic of U.S. practice. Methods are discussed briefly for calculating earth pressures and resistance factors are given for anchors (Table 13). The procedures used for spread footings are applied to earth retaining systems.

1992 AUSTROADS Bridge Design Code

The AUSTROADS Bridge Design Code has a section that deals with design of earth retaining systems. Material (resistance) factors to be applied to soil properties are provided in the Code and shown in Table 20. These quantities seem to be applied in addition to resistance factors applied to spread footing nominal strengths given in Table 5. This is not discussed in the code but if the double usage is required, it would produce very conservative designs. Resistance factors are not given specifically for earth retaining structures.

There is an extensive section for the calculation of anchor strength in the soil pull-out limit state. Specific assumptions to be used in determining anchor capacity are stated for several different conditions. However, mechanically stabilized earth wall reinforcement is not discussed. Resistance factors for anchor pull-out or anchor structural strength are not given and neither are resistance factors for earth reinforcement specified.

Australian Standard

The available Australian Standard did not deal with earth retention systems.

CHAPTER SIX

LRFD FOR CULVERTS

INTRODUCTION

Culverts are constructed in a wide variety of geometries and sizes from several different materials. The engineering behavior of culverts is primarily controlled by the stiffness of the structure cross section and the characteristics of the surrounding soil backfill. For the purposes of design, culverts can be divided into two classes, flexible and rigid, to differentiate their response to loading. Flexible culverts (typically corrugated metal and thermoplastic systems) depend on a large deformation capacity and interaction with the surrounding soil to maintain their shape. If the backfill envelope is not constructed to develop adequate passive resistance and stiffness, flexible culverts will deflect beyond their tolerable limit and collapse. Because of their limited deformation capacity, rigid culverts (reinforced concrete systems) develop significant ring stiffness and strength to support the vertical pressures imposed on them.

In this presentation the subject is divided into the two categories, flexible and rigid. The design procedures are quite different for these two cases. A number of special flexible systems are used to construct very large culverts. The largest of these special systems is of a size approaching a small bridge; that design is specialized and beyond the scope of this study.

FLEXIBLE CULVERTS

Flexible Culvert Design Procedure

It was noted above that a review of the design of long span metal culverts is not included here. Thus, this section is concerned with the design of flexible pipes. The design of culverts is interdisciplinary involving hydrologic and hydraulic considerations, roadway, geotechnical, and structural design considerations. The design procedure is illustrated in the flow chart of Figure 13. First, the design geometry must be determined including factors such as culvert location, elevation, and roadway grade. Flow requirements for the culvert must be established based on the hydrologic requirements at the site. Then the hydraulic design can be performed and the culvert size (pipe diameter) selected. After this has been accomplished in Blocks 1-3, the design loads and load factors are determined in Block 4. With the loads known the structure can be selected based on the simple strength requirements in Block 5.

For flexible culverts, ring compression, buckling, and seam strength must be evaluated for the strength limit states (Block 6) and settlement and handling must be evaluated for serviceability requirements (Block 7).

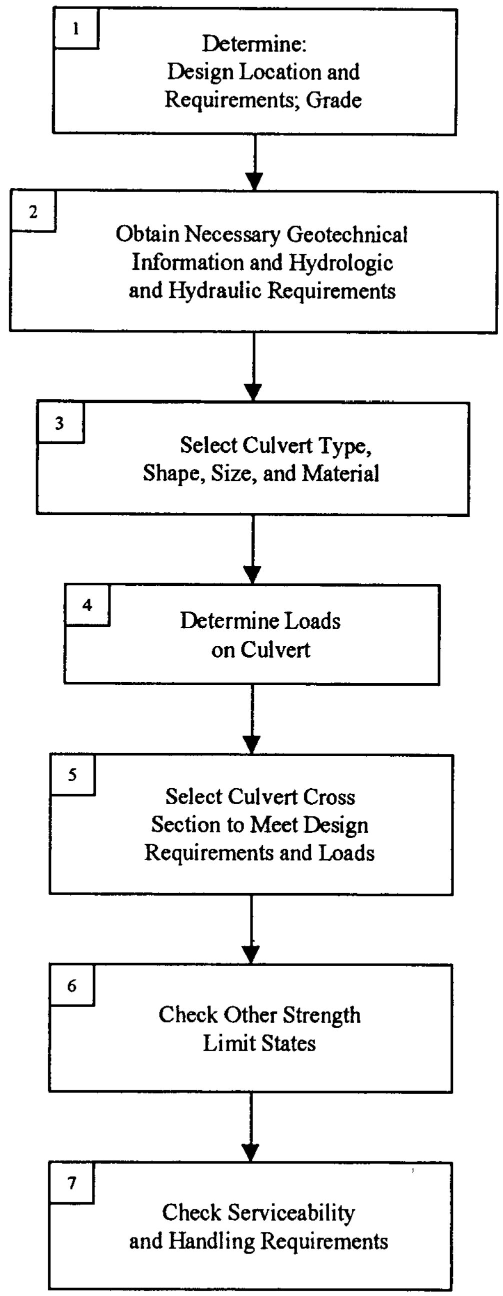


FIGURE 13 Design process for culverts.

The loads and load factors for LRFD according to the LRFD Bridge Code are defined in chapter 3 of that document. The definition of the soil pressure due to the overburden and to the traffic loads is empirical but well established by large-scale tests and finite element studies (*Spangler 1941*; *Watkins and Moser 1969; Duncan and Drawsky 1983*). The use of the AASHTO specified loads is discussed extensively by Withiam et al. (*1997*), including design examples. The load factors are also given in Section 3 of the LRFD Bridge code.

Nominal Strength of Flexible Culverts

The nominal strength of flexible culverts is defined for the various limit states in Section 12 of the LRFD Bridge Code. There are three strength limit states for a flexible pipe culvert and an additional limit state for noncircular culverts. For the flexible pipe the strength limit states are thrust (compression yield) failure, buckling, and seam failure. In practice these three limit states can be reduced to two since, if there is a bolted seam, it will be critical compared with the thrust failure. For the thrust case, the plate can be loaded to the yield strength of the material reduced by the resistance factor. The seam failure condition is limited by the strength of the bolted seam and for convenience in design, the strength can be easily tabulated.

The buckling strength limitation contains two buckling conditions, one elastic and one plastic. As for columns, increasing slenderness produces reduced buckling strength. The problem is complicated by the restraint provided by the soil envelope. While the form of the two strength equations has a rational appearance, they contain a parameter for soil stiffness that has been established empirically.

In addition to the above conditions, there is a limit state for flexibility required by construction conditions. This limit is given in the LRFD Bridge Code.

Resistance Factors for Flexible Culverts

The resistance factors from the LRFD Bridge Code for flexible culverts are given in Table 21. These factors were obtained by comparison with the ASD factors of safety and they, in turn, were based on extensive tests. Withiam et al. (*1997*) refer to this source of the factors but no reference is given. The load factors, generated by the AASHTO Code Calibration Committee chaired by Nowak, are given in Section 3 and are quite detailed but a reference to their source could not be found. It is interesting to examine these factors. In every case for flexible culverts, the resistance factors are the same for all of the limit states in each category of culvert type. This is not surprising when one considers that the nominal strength determination procedures and the loads are obtained by methods that are highly empirical.

Probabilistic analysis was performed in the determination of either the load factors or the resistance factors. This is realistic in view of the available load and resistance information. The pipe design information was obtained to a considerable extent from tests conducted in large test chambers. In these cases, it is difficult to measure both the load and the resistance separately in such a way that a rational analysis can be verified.

Serviceability Evaluation of Flexible Culverts

The serviceability considerations for flexible culverts include settlement and handling requirements for factory-made and field-assembled pipe. If differential settlement occurs along a culvert, the movement must be limited to maintain structural integrity and the hydraulic effectiveness of the culvert. The other settlement considerations are actually concerned with strength. If differential settlements occur, it is critical that they be greater in the soil under the culvert invert than in the soil surrounding the sides of the pipe. The additional settlement under the culvert invert will help reduce the load transferred from the soil to the pipe. If the reverse situation occurs, the culvert receives additional load from the soil adjacent to the culvert and this can be sufficient to cause failure of the culvert structure. This problem is covered in a clear and concise manner in the LRFD Bridge Code.

RIGID CULVERTS

Rigid Culvert Design Procedure

Rigid culvert design is concerned with the design of precast, reinforced concrete pipe of circular section, reinforced concrete cast-in-place box culverts, and reinforced concrete cast-in-place arches. The design of rigid culverts is interdisciplinary involving hydrologic and hydraulic, roadway, geotechnical, and structural considerations. The design procedure is illustrated in the flow chart of Figure 13. In Blocks 1 and 2 the design grades, location, and other site-specific requirements are determined, the geotechnical conditions are established, and the hydrologic and hydraulic requirements determined. The culvert type, material, shape, and size can then be established in Block 3. The loads on the culvert are determined in Block 4. The LRFD Bridge Code, Section 3 specifies the procedures to be used in determining the soil pressure due to overburden and also to vehicle loads.

TABLE 21

RESISTANCE FACTORS FOR BURIED STRUCTURES (from the LRFD Bridge Code)

Structure Type	Resistance Factor
Metal Pipe, Arch and Pipe Arch Structures	
Helical pipe with lock seam or fully-welded seam:	
Minimum wall area and buckling	1.00
Annular pipe with spot-welded, riveted or bolted seam:	
• minimum wall area and buckling	0.67
• minimum seam strength	0.67
Structural plate pipe:	
• minimum wall area and buckling	0.67
• minimum seam strength	0.67
Long-Span Structural Plate and Tunnel Linear Plate Structures	
• minimum wall area	0.67
• minimum seal strength	0.67
Structural Plate Box Structures	
• plastic moment strength	1.0
Reinforced Concrete Pipe	
Direct design method:	
Type 1 Installation:	
• flexure	0.90
• shear	0.82
• radial tension	0.82
Other type installations:	
• flexure	1.00
• shear	0.90
• radial tension	0.90
Reinforced Concrete Cast-in-Place Box Structures	
• flexure	0.90
• shear	0.85
Reinforced Concrete Precast Box Structures	
• flexure	1.00
• shear	0.90
Reinforced Concrete Precast Three-Sided Structures	
• flexure	0.95
• shear	0.90
Thermoplastic Pipe	
PE and PVC pipe:	
• minimum wall area and buckling	1.00

Once the design soil pressures are determined, the task for rigid pipes is to determine the axial forces, moments, and shears in the pipe. Two general procedures are permitted by the LRFD Bridge Code. The more advanced of the two methods is taken from an ASCE report (*ASCE 1993*) that was prepared to assimilate the results of extensive research (*Heger 1963, 1982*). The Code does not specify the analysis method that is to be used, but with the loads known, analysis capabilities are readily available. The result of the analysis is a force distribution in the pipe.

The strength limit states (Block 6) that must be addressed are all structural conditions. The pipe must carry the thrust, tension, bending, and shear forces. The Code gives several specific requirements that must be satisfied in the reinforcement area to carry the various loads. This is an interesting approach in that it makes the design process direct, i.e. a process of calculating the necessary reinforcement. While this procedure appears to be quite complex it is, in fact, presented in a specific and clear manner. Crack width control is also presented in this section of the code as a serviceability requirement.

The same design procedures can be used for box culverts and other similar reinforced concrete systems. In these cases, when the design pressures are available, an analysis can be performed and a design of the reinforced concrete structure completed using the usual concrete design requirements.

Resistance Factors for Rigid Culverts

The resistance factors from the LRFD Bridge Code for rigid culverts are given in Table 21. The source of these factors is not given. The resistance factors are the same for the two methods of design that are contained in the code. For reinforced concrete pipe the resistance factors depend on the method used in the installation. Both sets of resistance factors are larger than is typical for reinforced concrete design and are given in the chapter in the LRFD Bridge Code. In the case of favorable installation procedures the resistance factors are considerably larger than would be used in superstructures.

Serviceability Evaluation of Rigid Culverts

The serviceability considerations for rigid culverts include settlement and crack width control. The LRFD Bridge Code gives the serviceability requirements for all culvert types, both flexible and rigid. Settlement must be checked to verify that hydraulic functionality is maintained. In the case of the rigid culvert, the possibility of large differential settlements must be considered to deal with the possibility of disjointing.

Crack width is checked during the analysis for strength and specific limits are given in the Code section on serviceability.

OTHER CODES AND STANDARDS

Of all the LRFD codes reviewed, only the Ontario Bridge Code and the 1992 AUSTROADS Design Code discussed culvert design. The Ontario Bridge code contains an extensive and very specific set of requirements for the design of flexible steel pipes and arches. The requirements are written in an LRFD format and contain clearly specified methods for analyzing the soil-structure system. The AUSTROADS Code contains some general requirements but does not include detailed information of the type contained in the LRFD Bridge Code. The specific requirements given in the LRFD Bridge Code make the design of culverts possible despite the complex soil-structure interaction behavior of these systems.

CHAPTER SEVEN

CONCLUSIONS

In 1994, AASHTO adopted an LRFD-based code for bridge design, including the design of the geotechnical facilities. Questions have been raised regarding some aspects of the geotechnical portion of the code. The goal of this synthesis study was to report on the entire area of LRFD development and application in geotechnical design.

A questionnaire was sent to departments of transportation in the all of the states and Canadian provinces. The results of the questionnaire showed that about half of the state DOTs had plans for implementing the geotechnical aspects of the LRFD Bridge code and those that had implemented or experimented with implementation had encountered some problems. Nearly all the Canadian provinces have implemented LRFD for the geotechnical aspects of bridge design and all indicated that they had encountered some problems but have overcome them. The Ontario Ministry of Transportation has used the LRFD approach in all aspects of bridge design, including foundations, for over 10 years, more experience in LRFD bridge design than any other agency worldwide, and their design code is still evolving. In the United States, LRFD has been used for the structural design of concrete buildings according to the Building Code of the American Concrete Institute for more than 30 years. The implementation of that code was accomplished in about 2 years. Bridge design in the United States has used some aspects of LRFD for several years but this has not included the design of geotechnical facilities.

The earliest application of LRFD in the design of geotechnical facilities was made in Denmark about 30 years ago. In the past few years, there has been some further effort in other countries to develop and implement LRFD for both structural and geotechnical facility design. Codes have been developed in Australia, Canada, and the European Community. The effort made in Europe was a major one involving all countries of the European Community and, because of the size of the effort, review of the resulting document is useful.

After the implementation of the LRFD Code by the American Concrete Institute, a firm theoretical basis was created from probability theory and load and resistance factors were generated rationally and analytically. The ASD safety factor was divided between the load and the resistance side of the fundamental design requirement. The load and resistance factors can be generated from statistical data on load occurrences and from data on the strength of the element. Where this information is available, determining resistance factors is a direct process. Load factors have been established analytically during the development of the LRFD Bridge Code based on load data and on the behavior of structural elements.

In this study, the available LRFD design codes for geotechnical systems were reviewed and compared with the LRFD Bridge Code and all were reviewed relative to each other. Load and resistance factors spanning a wide range were found to be in use.

In the review of all of the available LRFD codes, including the LRFD Bridge Code, some conclusions were drawn:

- The load factors for the various available LRFD Codes were assembled and tabulated. For the most common gravity load condition, the load factors on dead load ranged from 1.0 to 1.40 and on live load from 1.3 to 1.75. The load factors used for the Strength I case in the LRFD Bridge Code have a rational, analytical basis. The methods used to generate the load factors for the LRFD Bridge Code, the AISC Code, the Ontario Bridge Code, and the Canadian Bridge Code have been presented, but the same information was not found for some of the other codes. Some of the load factors used in the other load conditions are the product of further analysis but include engineering judgment and comparison with the Strength I case.
- The Eurocode gives guidance in the selection of soil properties from data taken from a particular soil stratum. In the Eurocode, it is stated that the values used for design should be such that there is a 5 percent probability that a worse value will exist for the soil. While this requirement seems conservative, it may be desirable to examine the possibility of making some recommendations in the LRFD Bridge Code for selecting soil property values from test data. However, the use of very conservative soil properties, as in the Eurocode, combined with a resistance factor applied to the nominal strength could produce excessively conservative designs.
- Geotechnical resistance factors for equivalent applications given in the various LRFD codes have an even greater range than the load factors. In almost all cases, resistance factors were determined by correlation with ASD or by engineering judgment. Only in a very limited number of cases was an analysis of statistical data used.
- The definition of methods for determining nominal strength and resistance factors requires that established methods of design be followed. For driven pile foundations,

the design procedures recommended by the Federal Highway Administration were not followed in any of the codes. Static load tests or dynamic methods such as dynamic testing, wave equation analysis, or dynamic formulas are almost always used to determine driven pile capacity. Static analysis together with driving to a specific depth is very rarely used and never used in granular soils. The Australian Code and the procedures used by the Florida DOT were the most advanced and complete. The resistance factors in the codes that were reviewed vary significantly. There are no published reports of deep foundation strength data being used analytically to determine resistance factors contained in the code.

- Resistance factors are not available for some of the methods used to determine pile capacity.
- Quality control procedures should also affect resistance factors. Larger resistance factors are justified for more thorough quality control procedures.
- For drilled shafts, resistance factors must be available for all commonly occurring soil conditions.
- Extensive data bases from tests on driven piles and drilled shafts are available. They should be used to determine resistance factors based on probabilistic analyses.
- The procedures to be used in dealing with the structural limit states for deep foundations must be clear, the resistance factors consistent and specified.
- The use of LRFD in the design of earth retaining systems is fundamentally more difficult than for foundations because the soil properties are involved in determining both the load and the resistance. Even with test data to failure, it will be difficult to separate the soil contribution to the load from the definition of the strength. The determination of resistance factors by analytical means based on a probabilistic method will be difficult.
- The problem of earth retaining systems designed by LRFD has been studied extensively in developing the Eurocode, and the methods used warrant study.

Based on the studies reported in this synthesis, the following research efforts to improve the LRFD Bridge Code were identified:

- Make design procedures for deep foundations contained in the LRFD Bridge Code consistent with established practice. Generate resistance factors analytically using existing deep foundations data bases.
- Review load combinations and load factors for geotechnical applications to assure that they are consistent with current practice.
- Establish the proper load and resistance factors for applying LRFD to the entire area of designing earth retaining systems, which will probably require extensive research.
- Revisit the large amount of test data on culverts to investigate the establishment of rationally determined resistance factors.
- Undertake a strong program of performing designs with the new LRFD Code and the previous ASD to assure that substantial, unexpected differences are not produced. Undertake an effort to produce a clear, user-friendly design code. It would be desirable to apply decision tables to test the finished product.

The preparation of a design code requires that there be a clear understanding of the design process. This is particularly important when the design process spans two different disciplines, as in foundation design. There may be limit states that span the two disciplines. Without a careful coverage of all limit states and all steps in the design process, failure probabilities cannot be properly assessed.

Excessive emphasis should not be placed on the use of reliability-based methods in determining resistance factors. The exercise is certainly important and it can help to discover variabilities in safety factors in existing practice. If increased safety is found to be necessary in the reliability analysis, it should not be applied without an indication of some history of unsatisfactory performance. However, in deep foundations, for which test data is available, it should certainly be used and it may be possible to justify a reduction in safety factors.

The preparation of codes must have input from people familiar with the design process. When preparing a code, structural engineers have made it a practice to have the proposed code examined by a group of people representing a wide range of practice. This should also be the case in the geotechnical area.

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APPENDIX A

Questionnaire

National Cooperative Highway Research Program

NCHRP Synthesis Topic 28-02

Geotechnical Related Development and Implementation of Load and Resistance Factor Design (LRFD) Methods

QUESTIONNAIRE

Name of Respondent: ____________________

State DOT: ____________________

Title: ____________________

Phone and FAX Numbers: ____________________

1. Has your state completed any highway projects including foundation elements, retaining walls, MSE walls, slope stability and/or culverts using LRFD or are any underway?

   ❑ YES ❑ NO

2. If YES, How many? ____________

2a. If YES, please give a brief description of the size and scope of the project. What types of geotechnical features did it contain?

2b. If YES, what process was used to put your design practice into an LRFD format?

3. If NO, when do you plan to do your first design? ____________

4. When do you plan to implement LRFD on a routine basis? ____________

5. Based on the limited experience you have had with LRFD, or if you have not completed any design based on your review of the current (1997) AASHTO Standard Specifications for Highway Bridges, what problems do you anticipate in executing the geotechnical aspects of design. Please comment on the following topics:

   Spread Footings

   ____
   ____
   ____

   Driven Piles

   ____
   ____
   ____

Drilled Shafts

Earth Retaining Structures

Culverts

Other

6. Has your state supported or otherwise encouraged research on the Geotechnical aspects of LRFD?

❑ YES ❑ NO

7. Please supply references and sources of any reports that were issued as a result of the research.

8. Does your state have any research studies related to the subject now underway?

❑ YES ❑ NO

9. Please supply the name and address of the research organization and/or the principal investigator.

10. What research, development, and training needs must be addressed in order to implement LRFD?

Please send your response to:

George C. Goble
Goble Rausche Likins and Associates, Inc.
5398 Manhattan Circle, Suite 100
Boulder, Colorado 80303

If you have any questions, please call George Goble on 303-494-0702 or contact him by e-mail at bdigobleaol.com. If you wish to submit your questionnaire by FAX, please do so on 303-494-5027.

TABLE A-1

QUESTIONNAIRE RESPONSES/UNITED STATES

Name of Respondent	Question 1 Yes	Question 1 No	Question 2	Question 3	Question 4	Question 6 Yes	Question 6 No	Question 8 Yes	Question 8 No
Alabama		X		?	?		X		X
Arkansas	X		4		1999		X		X
California		X		None	None		X		X
Colorado	X		~50	N/A	N/A		X		X
Connecticut		X		None	After the software packages		X		X
Delaware		X		1997	1999		X		X
Florida		X		1998	1998	X		X	
Georgia		X		?	?		X		X
Hawaii		X		When adequate training is available	When adequate training is available		X		X
Idaho		X		1999	2000		X		X
Illinois		X		Not yet	Not yet		X		X
Iowa		X		None	None		X		X
Kansas		X		1999	1999		X		X
Louisiana		X		2000	2000		X		X
Maryland		X		?	?		X		X Responses/United States
Massachusetts	X		3		2000		X		X
Michigan	X		1	None	Will probably take Federal Mandate		X		X
Minnesota		X		2000+	1999		X		X
Mississippi		X		None	We will follow Bridge Division		X		X
Missouri		X		1998	Uncertain - Depends on software availability		X		X
Nebraska		X		?	?		X		X
Nevada		X		When trained	After we are trained and we can evaluate the method		X		X
New Hampshire		X		When method is approved by AASHTO	When LRFD is implemented by Bridge Design Bureau		X		X
New Jersey		X		1998	1999		X		X
New York		X		1998	2000-1		X		X

TABLE A-1 *(Continued)*

Name of Respondent	Question 1 Yes	Question 1 No	Question 2	Question 3	Question 4	Question 6 Yes	Question 6 No	Question 8 Yes	Question 8 No
North Carolina		X		?	?		X		X
North Dakota		X		2003	2003		X		
Oklahoma	X		>10		Immediately		X		X Responses/United States X
Oregon		X		1997	1998?	X			X
Pennsylvania		X		We have now completed computer program and we will shortly initiate design	We have for all new projects	X			X
South Carolina		X		2000	2000		X		X
Tennessee		X		?	?		X		X
Texas		X		None	None		X		X
Utah		X		None	None		X		X
Vermont		X			After we've hosted the LRFD Workshop and our structures section has adopted the code	X			X
Washington	X		1		1998	X			X
West Virginia		X		1997	1998		X		X
Wyoming		X		1999	2000		X		X

TABLE A-2

QUESTIONNAIRE RESPONSES/CANADA & PUERTO RICO

Name of Respondent	Question 1 Yes	Question 1 No	Question 2	Question 3	Question 4	Question 6 Yes	Question 6 No	Question 8 Yes	Question 8 No
British Columbia		X		Unknown, perhaps when code is better developed and factors better calibrated	same as 3		X		X
Manitoba	X		3		No set date		X		X
New Brunswick	X		>100	N/A	N/A		X		X
Newfoundland	X		~20		Presently implemented		X		X
Nova Scotia	X		150+		N/A		X		X
Ontario	X		500+		1981+/-	X		X	
Puerto Rico		X	N/A	None	No policy related to this		X		X

Summarized and Paraphrased Comments Received from State DOTs:

1. The load combination appropriate for particular designs is not clear.

2. The tolerable displacement of the structure must be defined.

3. Resistance factors obtained by direct calibration with ASD factors of safety should produce LRFD designs that are similar to ASD-based designs so foundation cost savings cannot be expected.

4. The calibration of the resistance factors has been questioned.

5. If the methods used for nominal strength determination of spread footings or any other element do not have resistance factors specified by the LRFD Bridge Code, the designer must develop the factors.

6. Some DOTs found that spread footing designs by the LRFD Bridge Code were overconservative.

7. Soil information needs to be defined in terms of ultimate conditions for spread footing design.

8. The failure criteria need to be defined for deep foundation load test results.

9. The current version of the Code for piles cannot be directly implemented.

10. Concern was expressed regarding the appropriate resistance factors when dynamic formulas or dynamic testing is used to determine pile capacity.

11. The selection of the appropriate resistance factor for drilled shafts in interbedded soils is not defined.

12. Only one state commented on the earth retaining and culvert sections of the Code. That comment indicated that the Code is too conservative.

13. The current geotechnical section of the LRFD Bridge Code is silent with regard to seismic design.

14. Better communication between the foundation and bridge engineer is necessary.

15. There is an urgent need for standardization of notation in the LRFD Bridge Code.

16. There is a need for detailed design examples.

Summarized and Paraphrased Comments from Canadian Provinces:

1. Spread footing sizes seem to have become larger.

2. The use of load factors causes problems with inclined loads. Sometimes the statics does not "work" out.

3. The implementation of serviceability limitations for piles has become a problem. With ASD, serviceability was probably not checked for piles.

4. There has been a tendency for double counting of factors on both the load and resistance side.

5. The code originally used partial resistance factors but is now tending more to single factors.

APPENDIX B

Existing Code Load Factors and Combinations

AASHTO LRFD BRIDGE CODE, 1997 INTERIM

Definitions

Permanent Loads

DD = downdrag

DC = dead load of structural components and nonstructural attachments

DW = dead load of wearing surfaces and utilities

EL = accumulated locked-in force effects resulting from the construction process

EH = horizontal earth pressure load

ES = earth surcharge load

EV = vertical pressure from dead load of earth fill

Transient Loads

BR = vehicular braking force

CE = vehicular centrifugal force

CR = creep

CT = vehicular collision force

CV = vessel collision force

EQ = earthquake

FR = friction

IC = ice load

IM = vehicular dynamic load allowance

LL = vehicular live load

LS = live load surcharge

PL = pedestrian live load

SE = settlement

SH = shrinkage

TG = temperature gradient

TU = uniform temperature

WA = water load and steam pressure

WL = wind on live load

WS = wind load on structure

Notations

Y_p = load factor for permanent loading

Y_{SE} = load factor for settlement

Y_{TG} = load factor for temperature gradient

TABLE 1-B

AASHTO LOAD COMBINATIONS AND LOAD FACTORS (from AASHTO LRFD Bridge Code, 1997 Interim)

Load Combination Limit State	DC DD DW EH EV ES	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH EL	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 1.5	–	1.00	–	–	1.00	0.50/1.20	–	–	–	–	–	–
STRENGTH-V	γ_p	1.35	1.00	0.40	0.40	1.00	0.50/1.20	γ_{TG}	γ_{SE}	1.00	–	–	–
EXTREME EVENT-1	γ_p	γ_{EQ}	1.00	–	–	1.00	–	–	–	–	1.00	1.00	1.00
EXTREME EVENT-II	γ_p	0.50	1.00	–	–	1.00	–	–	–	–	–	–	–
SERVICE-1	1.00	1.00	1.00	0.30	0.30	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	–	–	–	–	–	–	–		–	–	–

TABLE 2-B

AASHTO LOAD FACTORS FOR PERMANENT LOADS, γ_P (from AASHTO LRFD Bridge Code, 1997 Interim)

Type of Load	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
EH: Horizontal Earth Pressure		
C Active	1.50	0.90
C At-Rest	1.35	0.90
EV: Vertical Earth Pressure		
C Overall Stability	1.35	N/A
C Retaining Structure	1.35	1.00
C Rigid Buried Structure	1.30	0.90
C Rigid Frames	1.35	0.90
C Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
C Flexible Metal Box Culverts	1.50	0.90
ES: Earth Surcharge	1.50	0.75

AC 318-95

$$1.4D + 1.7L$$
$$0.75\,(1.4D + 1.7L + 1.7W)$$
$$0.9D + 1.3W$$
$$1.4D + 1.7L + 1.7H$$
$$0.75\,(1.4D + 1.4T + 1.7L)$$
$$1.4\,(D + T)$$

where

D = dead loads
H = loads due to weight and pressure of soil
L = live loads
T = effects of temperature, shrinkage, and creep, and
W = wind load

AISC DESIGN CODE AND ACI 318-95 APPENDIX C

$$1.4D$$
$$1.2D + 1.6L + 0.5\,(L_r \text{ or } S \text{ or } R)$$
$$1.2D + 1.6\,(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W)$$
$$1.2D + 1.3W + 0.5L + 0.5\,(L_r \text{ or } S \text{ or } R)$$
$$1.2D + 1.5E + (0.5L \text{ or } 0.2S)$$
$$0.9D - (1.3W \text{ or } 1.5E)$$

ONTARIO BRIDGE CODE

TABLE 3-B

LOAD COMBINATIONS AND LOAD FACTORS (from Ontario Bridge Code)

	Permanent Loads (See Note 1, below)			Transitory Loads					Exceptional Loads (Use one only)			
Loads	D	E	P	L	K	W	V	S	Q	F	A	H
Fatigure Limit States FLS Combination 1	1.00	1.00	1.00	0.80	0	1.00	0	0	0	0	0	0
Serviceability Limit States SLS Combination 1	1.00	1.00	1.00	0.75	0.80	0.70	0	1.00	0	0	0	0
Ultimate Limit States ULS Combination 1	a_D	a_E	a_P	1.40	0	0	0	0	0	0	0	0
ULS Combination 2	a_D	a_E	a_P	1.25	1.15	0	0	0	0	0	0	0
ULS Combination 3	a_D	a_E	a_P	1.15	1.00	0.40	0.40	0	0	0	0	0
ULS Combination 4	a_D	a_E	a_P	0	1.25	1.30	0	0	0	0	0	0
ULS Combination 5	a_D	a_E	a_P	0	0	0.70	0	0	1.30	1.30	1.30	1.40

Note 1: For U.L. States, use maximum or minimum value of a

Notation: a_x Load Factor for load x, where xis a letter identifying a load

Loads:

- A ice accretion load
- D dead load
- E Loads due to earth pressure and hydrostatic pressure other than dead load. Surcharges shall be considered as earth pressure even when caused by other loads.
- F loads due to stream flow and ice pressure
- H collision load
- K all strains, deformations, displacements and their effects, including the effects of their restraint and those of friction or stiffness in bearings. Strains and deformation include those due to temperature change and temperature differential, concrete shrinkage, differential shrinkage and creep;but not elastic strains.
- L live load
- P secondary prestress effects
- Q earthquake load
- S load due to foundation deformation
- V wind load on L.L.
- W wind load on structure

TABLE 4-B

MAXIMUM AND MINIMUM VALUES OF LOAD FACTORS (from Ontario Bridge Code)

Dead Load	Maximum α_D	Minimum α_D
Factory-produced components excluding wood	1.10	0.95
Cast-in-place concrete, wood and all non-structural components	1.20	0.90
Wearing surfaces, based on nominal or specified thickness	1.50	0.65
Earth fill, negative skin friction on piles	1.25	0.80
Water	1.10	0.90
Earth Pressure & Hydrostatic Pressure	**Maximum α_E**	**Minimum α_E**
Passive earth pressure	1.25	0.50
At-rest earth pressure	1.25	0.80
Active earth pressure	1.25	0.80
Backfill pressure	1.25	0.80
Hydrostatic pressure	1.10	0.90
Prestress	**Maximum α_P**	**Minimum α_P**
Secondary prestress effects	1.05	0.95

CANADIAN BRIDGE CODE

TABLE 5-B

LOAD FACTORS AND LOAD COMBINATIONS (from Canadian Code)

	Permanent Loads					Variable Loads				Exceptional Loads (use only one)				
Load Case	D	D_s	E	B	S	L	W	V	T	F	C	Q	A	G
Serviceability Limit States														
1. Type I	1.0	1.0	1.0	1.0	1.0	0.9	0	0	0	0	0	0	0	0
2. Type II	1.0	1.0	1.0	1.0	1.0		0	0	0	0.8	0	0	0	0
Ultimate Limit States														
Combination 1						1.6	0	0	0	0	0	0	0	0
Combination 2						0	1.3	0	0	0	0	0	0	0
Combination 3	0.9	0	0.8	0	0.8	1.0	0.5	0.5	0	0	0	0	0	0
Combination 4	or	or	or	or	or	1.3	0	0	1.0	1.0	0	0	0	0
Combination 5	1.2	1.6	1.3	1.2	1.2	0	0	0	0	1.3	1.3	1.3	1.3	0
Combination 6						0	0	0	1.0	0	0	0	0	0
Combination 7						1.0	0	0	0	0	0	0	0	1.0

EUROCODE

TABLE 6-B

DESIGN VALUES OF ACTIONS FOR USE IN THE COMBINATION OF ACTIONS
(taken from Eurocode 1 (32a)

Design Situation	Permanent Actions G_d	Single Variable Actions: Dominant	Single Variable Actions: Others	Accidental or Seismic Actions A_d
Persistent and Transient	$\gamma_G G_k$ ($\gamma_p P_k$)	$\gamma_{Q1}\, Q_{k1}$	$\gamma_{Q1}\, \psi_{01}\, Q_{ki}$	
Accidental	$\gamma_{GA} G_k$ ($\gamma_{PA} P_k$)	$\psi_{11}\, Q_{k1}$	$\psi_{21}\, Q_{ki}$	$\gamma_A A_k$ or A_d
Seismic	G_k		$\psi_{21}\, Q_{ki}$	$\gamma_I A_{Ed}$

Symbolically the combinations may be represented as follows

a) persistent and transient design situations for ultimate limit states verification other than those relating to fatigue

$$\frac{\Sigma}{j \geq 1} \gamma_{Gj} G_{kj} \text{"+"} \gamma_p P_k \text{"+"} \gamma_{Q1} Q_{k1} \text{"+"} \frac{\Sigma}{i \geq 1} \gamma_{Qi} \Psi_{0i} Q_{ki}$$

b) combinations for accidental design situations

$$\frac{\Sigma}{j \geq 1} \gamma_{GAj} G_{kj} \text{"+"} \gamma_{PA} P_k \text{"+"} A_d \text{"+"} \psi_{11} Q_{kl} \text{"+"} \frac{\Sigma}{i \geq 1} \psi_{2i} Q_{ki}$$

c) combination for the seismic design situation

$$\frac{\Sigma}{j \geq 1} G_{kj} \text{"+"} P_k \text{"+"} \gamma_I A_{Ed} \text{"+"} \frac{\Sigma}{i \geq 1} \psi_{2i} Q_{kj}$$

TABLE 7-B

PARTIAL FACTORS: ULTIMATE LIMIT STATES FOR BUILDINGS *(from Eurocode 1(32a))*

Case[1]	Action	Symbol	Situations P/T	Situations A
Case A Loss of static equilibrium; strength of structural material or ground insignificant	Permanent actions: self weight of structural and non-structural components, permanent actions caused by ground, ground-water and free water			
	-unfavourable	γ_{Gsup}[4]	[1,10][2]	[1,00]
	-favourable	γ_{Ginf}[4]	[0,90][2]	[1,00]
	Variable actions -unfavourable	γ_Q	[1,50]	[1,00]
	Accidental actions	γ_A		[1,00]
Case B[5] Failure of structure or structural elements, including those of the footing, piles, basement walls etc., governed by strength of structural material	Permanent actions[6] (see above)			
	-unfavourable	γ_{Gsup}[4]	[1,35][3]	[1,00]
	-favourable	γ_{Ginf}[4]	[1,00][3]	[1,00]
	Variable actions -unfavourable	γ_Q	[1,50]	[1,00]
	Accidental actions	γ_A		[1,00]
Case C[5] Failure in the ground	Permanent actions (see above)			
	-unfavourable	γ_{Gsup}[4]	[1,00]	[1,00]
	-favourable	γ_{Ginf}[4]	[1,00]	[1,00]
	Variable actions ufavourable	γ_Q	[1,30]	[1,00]
	Accidental actions	γ_A		[1.00]

P: Persistent situation T: Transient situation A: Accidental situation

1) The design should be verified for each case A, B and C separately as relevant.

2) In this verification the characteristic value of the unfavourable part of the permanent action is multiplied by the factor [1,1] and the favourable part by the factor [0.9]. More refined rules are given in ENV 1993 and ENV 1994.

3) In this verification the characteristic values of all permanent actions from one source are multiplied by [1,35] if the total resulting action effect is unfavourable and by [1,0] if the total resulting action effect is favourable.

4) In cases when the limit state is very sensitive to variations of permanent actions, the upper and lower characteristic values of these actions should be taken according to 4.2 (3).

5) For cases B and C the design ground properties may be different, see ENV 1997-1-1.

6) Instead of using γ_G (1.35) and γ_Q (1,50) for lateral earth pressure actions the design ground properties may be introduced in accordance with ENV 1997 and a model factor γ_{Sd} is applied.

TABLE 8-B

Ψ FACTORS FOR BUILDINGS *(from Eurocode 1)*

Action	Ψ_0	Ψ_1	Ψ_2
Imposed loads in buildings[1]			
Category A: domestic, residential	[0,7]	[0,5]	[0,3]
Category B: offices	[0,7]	[0,5]	[0,3]
Category C: congregation areas	[0,7]	[0,7]	[0,6]
Category D: shopping	[0,7]	[0,7]	[0,6]
Category E: storage	[1,0]	[0,9]	[0,8]
Traffic loads in buildings			
Category F: vehicle weight ≤ 30kN	[0,7]	[0,7]	[0,6]
Category G: 30kN < vehicle weights ≤ 160kN	[0,7]	[0,5]	[0,3]
Category H: roofs	[0]	[0]	[0]
Snow loads on buildings	[0,6][2]	[0,2][2]	[0][2]
Wind loads on buildings	[0,6][2]	[0,5][2]	[0][2]
Temperature (non-fire) in buildings [3]	[0,6][2]	[0,5][2]	[0][2]

[1] For combination of imposed loads in multistorey buildings, see ENV 1991-2-1.
[2] Modification for different geographical regions may be required.
[3] See ENV 1991-2-5.

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