Design, Analysis, and Testing of Laterally Loaded Deep Foundations that Support Transportation Facilities

FHWA GEC 009

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FOREWORD

The Geotechnical Engineering Circular (GEC) *Design, Analysis, and Testing of Laterally Loaded Deep Foundations that Support Transportation Facilities* provides recommended guidance for the design, analysis, and testing of laterally loaded deep foundations for transportation facilities in accordance with the Load and Resistance Factor Design (LRFD) platform. The intended audience for this document includes geotechnical, structural, and highway design and construction specialists involved with the selection, design, analysis and testing of laterally loaded deep foundations.

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16. Abstract

This Geotechnical Engineering Circular (GEC) is intended to provide recommended guidance for the LRFD design, analysis, and testing of laterally loaded deep foundations for transportation facilities. This document applies to deep foundation elements such as driven piles, drilled shafts, micropiles, and continuous flight auger (CFA) piles that are used to resist lateral loads, often in combination with axial loads, for new construction, rehabilitation, or reconstruction of transportation facilities. Applications include both single and groups of deep foundation elements for bridge foundations, excavation support, earth retention structures, noise walls, sign and signal foundations, landslide repairs, vessel or vehicle impact mitigation measures, and seismic event resistance. The objective of this document is to provide a single reference source for the state-of-the-practice with regard to recommended methodologies and guidance for the design, analysis, and testing of laterally loaded deep foundations for transportation facilities.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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1 INTRODUCTION AND OVERVIEW

1.1 PURPOSE

The purpose of this Geotechnical Engineering Circular (GEC) is to provide recommended guidance for the design, analysis, and testing of laterally loaded deep foundations for transportation facilities in accordance with the Load and Resistance Factor Design (LRFD) platform. The intended audience for this document includes geotechnical, structural, and highway design and construction specialists involved with the selection, design, analysis, and testing of laterally loaded deep foundations. The geotechnical and structural engineering communities have not had a comprehensive design and analysis document to address this geo-structural topic, which has resulted in designs that may be overly conservative or costly.

This document applies to deep foundation elements such as driven piles, drilled shafts, micropiles, and continuous flight auger (CFA) piles that are used to resist lateral loads, often in combination with axial loads, for new construction, rehabilitation, or reconstruction of transportation facilities. Applications include both single and groups of deep foundation elements for bridge foundations, excavation support, earth retaining structures, noise walls, sign and signal foundations, landslide repairs, vessel or vehicle impact mitigation measures, and seismic event resistance.

This GEC draws heavily from published work by the Federal Highway Administration (FHWA), the American Association of State Highway Transportation Officials (AASHTO), state and local departments of transportation (DOTs), and other reference publications that address laterally loaded deep foundations. As such, this document does not represent “new” research. The objective of this document is to provide a single reference source for the state-of-the-practice regarding recommended methodologies and guidance for the design, analysis, and testing of laterally loaded deep foundations for transportation facilities. Information presented in this document is not intended to be prescriptive, but rather the information that is described is to be considered current practice for transportation facilities and should be used with good engineering judgment. The recommended procedures presented herein are not intended to preclude deviations based on sound local engineering practices, demonstrated performance, or testing results.

This document focuses on laterally loaded deep foundations. Other considerations regarding deep foundations, such as the selection of foundation type, the design, and testing of deep foundations for axial loads, foundation construction, or other foundation considerations are beyond the scope of this document. Although this document is applicable to multiple types of deep foundations, throughout the document these may be referred to generically as piles/shafts unless a distinction is necessary. In the remainder of the text, the term “foundation” or “foundation element” is understood to refer to a deep foundation. Similarly, the use of the word “pile” is understood to refer to a deep foundation element, i.e., “lateral pile analysis” refers to analysis of deep foundations under lateral loads.

1.2 BACKGROUND AND HISTORY OF ANALYSIS OF LATERALLY LOADED DEEP FOUNDATIONS

Lateral loads in deep foundations occur under various conditions. In some applications, such as noise walls or landslide stabilization, lateral loads are the principal design load. For bridge foundation applications, lateral loads must be fully considered in the design. In many cases, lateral loads control the diameter or width of the foundations. Even relatively small lateral loads may influence the structural response of bridge foundations, depending on the load and structure characteristics. The magnitude, point (or area) of application, orientation, duration, and frequency of occurrence of lateral loads, and the response of the surrounding geomaterials will be different for each project application.

Lateral loads on deep foundations may result from either the structure to be supported or from the surrounding ground. A concentrated horizontal load and/or an overturning moment applied at the top of the foundation element are typically encountered in many transportation applications including bridge foundations, signal and sign structures, and noise walls. In these applications, loads applied by the superstructure are relatively independent of the subsurface conditions. In other applications, such as deep foundations used for slope stabilization or earth retaining structures, or piles subjected to lateral spread, loads acting on foundation elements are heavily dependent on soil conditions and the magnitude of soil movement, which in turn is affected by the selection of the deep foundation.

Numerous methods are available to analyze the response of deep foundations subjected to lateral loads. These methods have evolved from simple methods based on elasticity or plasticity theories to fully non-linear methods. Simple analytical methods use geotechnical parameters estimated from conventional geotechnical tests. More advanced analyses methods rely not only on geotechnical parameters but also on other types of data, including pile load test results. Significant improvements in the analysis, design, and load testing of laterally loaded piles have been made (Reese and Van Impe 2001) and powerful computer programs have been developed for handling complex calculations of pile resistance under lateral loads.

One of the first analysis methods available was based on the concept of the subgrade modulus (Terzaghi 1955). In this method, which considers an elastic beam resting on an elastic foundation (Hetenyi 1946), the lateral reaction of soil against the pile is assumed to be dependent on the soil deflection. Subsequently, other elastic-based methods were developed to provide preliminary estimates of the response of piles (e.g., Poulos and Davis 1980). Methods based on plasticity theory, including that proposed by Broms (1964a), were developed to estimate the ultimate lateral capacity of single piles and are based on the well-known concept of lateral passive earth pressure. The concept of an equivalent pile length was introduced in the late 1960s to facilitate structural calculations. In this concept, the effects of flexibility of the soil embedding the pile are replaced by a fixed pile having an equivalent stiffness.

The nonlinear response of deep foundations under lateral loads can be considered the “p-y” method (McClelland and Focht 1958). In this method, where “p” is the lateral soil reaction along the pile and “y” the lateral pile displacement. The soil is represented by a series of nonlinear springs, which are characterized according to the type of surrounding soil and other subsurface conditions (Reese et al., 1975). This methodology gained popularity in the 1960s and 1970s after several studies were conducted to support the development of offshore platforms for oil exploration (e.g., Matlock and Reese 1961; Matlock, 1970; and Reese et al. 1975).

Computer-based analysis techniques incorporated the p-y concept to efficiently analyze the response of deep foundations under combined lateral loads and moments. One of the first computer programs to incorporate the p-y methodology was the program COM624P (Wang and Reese 1993), which was developed under contract to the Federal Highway Administration (FWHA). This program analyzes the response of a single pile/shaft to the application of moments and lateral loads at the top of the pile. The p-y method is largely empirical.

The strain wedge method was derived in the 1990s to provide a more theoretical basis for correlating lateral soil resistance for laterally loaded deep foundations. In the strain wedge method, the soil resistance is correlated with mobilization of forces of a 3-D passive soil wedge from a limit equilibrium solution of passive earth pressure resistance. Estimated strains in the passive soil wedge associated with the mobilization of the passive resistance are correlated to lateral displacements. The strain wedge method is performed using software in a similar fashion to the p-y method.

Various commercial computer programs, which are based on either the finite element or finite difference methods, have been developed to handle more complex features including:

- three-dimensional (3-D) loading
- foundation geometry effects
- load nonlinearity
- simultaneous vertical and lateral loading
- nonlinear response of concrete pile sections
- pile groups

These computer programs are often used in the design and analysis of deep foundations under combined vertical and lateral loads.

The need for this document stems from the fact that, although various design procedures are currently used for the analysis of laterally loaded deep foundations, these procedures are not presented in a single reference. In addition, since the analysis requirements for laterally loaded deep foundations vary by locality and by published standards, there is a need for general guidelines to provide more consistent application of these procedures in the design of transportation facilities.

1.3 LITERATURE REVIEW

A literature review was performed as part of the development of this document. The intent of the literature review was to identify gaps in the literature and existing practice so that this document would enhance, rather than duplicate, the existing state-of-the-practice. The literature review included review of various publications and guidelines by state DOTs, publications by other U.S.-based sources including FHWA, AASHTO, and other organizations, as well as a review of published international sources that could be considered comparable to U.S. practice. Published case histories and lateral load test studies were also reviewed. The literature review provided information about how various state DOTs have implemented the LRFD design platform with regard to laterally loaded deep foundation applications, and how other organizations, both U.S.-based and internationally, address requirements for laterally loaded deep foundations. The literature review report is included in Appendix E.

1.4 ORGANIZATION OF MANUAL

The remainder of this document is organized as follows:

- *Chapter 2: Lateral Load Applications and Selection of Deep Foundation Type for Transportation Projects*. This chapter provides an overview of lateral load applications for transportation projects, and discusses the common types of deep foundations that are used in situations where lateral loads control the design. Chapter 2 also identifies foundation types that are excluded from consideration.
- *Chapter 3: Geotechnical Characterization for Design of Laterally Loaded Deep Foundations*. This chapter discusses subsurface investigation and testing methods, and identifies the relevant soil and rock engineering parameters needed for analyzing and designing laterally loaded deep foundations.
- *Chapter 4: LRFD Design Requirements and Limit States for Laterally Loaded Deep Foundations*. This chapter provides a discussion of the LRFD design platform including Limit States, load factors, and resistance factors as they relate to the design of laterally loaded deep foundations.
- *Chapter 5: Design Process and Design Team Roles for Analysis of Laterally Loaded Deep Foundations*. This chapter provides an overview of the design procedures typically used in practice, along with typical roles for geotechnical and structural engineers who collaborate in the design process.
- *Chapter 6: Analysis for Laterally Loaded Single Deep Foundation Elements*. This chapter presents methods for analyzing single laterally loaded deep foundation elements, including simplified methods and more complex methods requiring computer software. Discussion of the applicability and use of each design method and other design considerations are also presented.
- *Chapter 7: Lateral Analysis of Groups of Deep Foundations*. This chapter addresses lateral analysis of groups of deep foundation elements including considerations for groups versus single foundations, spacing of deep foundation elements, group effects, frame action, and other considerations.
- *Chapter 8: Design for Extreme Events.* This chapter presents recommended design methods for laterally loaded piles/shafts under extreme conditions such as seismic and vessel impact loading.
- *Chapter 9: Design for Earth Retention Structures.* This chapter addresses considerations for deep foundations for earth retention structures such as wall types, earth pressures, geotechnical strengths, and evaluation of deformations.
- *Chapter 10: Design for Slope Stabilization.* This chapter addresses analysis of deep foundations used for slope stabilization applications including determination of earth loads and soil-pile models.
- *Chapter 11: Structural Design and Performance*. This chapter presents considerations for structural design for shear and moments in deep foundations, foundation types and considerations, connections to superstructure, and foundations in rock which may develop high shear near the top of rock.
- *Chapter 12: Lateral Load Tests*. This chapter provides a description of the procedures used to conduct and interpret the results of lateral load tests on deep foundation elements, including considerations for use of such tests, limitations, alternatives to testing, types of tests, instrumentation and data analyses for the tests, and lateral load test reports.
- *Chapter 13: Construction Considerations*. This chapter discusses construction considerations that may impact lateral load capacity of deep foundations.
- *References*. A compiled list of references cited in the text of the report.

- *Appendix A:* This appendix presents example p-y curves and parameters for various conditions based on published sources.
- *Appendix B:* This appendix provides detailed design examples.
- *Appendix C:* This appendix provides example load test results and interpretation for a laterally loaded deep foundation.
- *Appendix D:* This appendix provides a guide specification for lateral load tests.
- *Appendix E:* This appendix consists of a report that documents the findings of a literature review regarding current state of practice for laterally loaded deep foundation design.

2 LATERAL LOAD APPLICATIONS AND SELECTION OF DEEP FOUNDATION TYPE FOR TRANSPORTATION PROJECTS

This chapter provides an overview of laterally loaded deep foundations commonly used in transportation projects.

2.1 LATERAL LOAD APPLICATIONS FOR TRANSPORTATION PROJECTS

2.1.1 Typical Lateral Load Applications for Vertical Deep Foundations

Lateral loads in transportation projects can originate from a variety of sources including: vehicle acceleration and braking on bridge decks and approaches; wind effects on traffic, on bridge decks, and/or structures; wave and current action in rivers and streams on bridge piers; forces caused by debris and ice floating in water courses; thermal effects (e.g., in integral abutments); vessel/vehicle impact on bridge piers and abutments; earth pressures acting behind abutment or retaining walls; slope movements; and seismic events. In some cases, the magnitude of lateral loads may be of comparable magnitude to that of axial loads. Examples of situations when deep foundations are used to resist lateral loads include:

- bridge abutments (Figure 2-1(a));
- bridge piers (Figure 2-1(b));
- temporary excavation support (Figure 2-2);
- permanent retaining walls (Figure 2-3);
- noise barrier walls (Figure 2-4);
- slope or landslide stabilization (Figure 2-5); and
- signs and traffic signals.

Lateral loads caused by traffic, braking, and wind forces along the longitudinal and transversal directions of a bridge can act on a bridge pier as shown in Figure 2-6 in addition to vertical loads. In the case of bridge abutments, loads due to lateral earth pressures must be added to the lateral loads from the bridge structure. Bridge pier foundations may be either a single deep foundation element, such as a large diameter drilled shaft, or may be a group of deep foundation elements such as a group of piles, drilled shafts, or micropiles.

Lateral earth pressures generated behind temporary and permanent retaining structures can be resisted using deep foundations. Conventional cantilever retaining walls may be supported by piles similar to a bridge abutment. Cantilever or non-gravity retaining walls can support lateral loads from retained earth using soldier piles, secant piles, or tangent piles. The lateral retained earth loads are supported by a combination of passive resistance of the portion of the piles that is embedded below grade and the structural capacity of the piles. If additional lateral capacity is needed, external braces or tieback anchors can be incorporated into on-gravity wall systems.

Figure 2-7 suggests that a vertical deep foundation used to stabilize slopes resists lateral loads above the potential slip surface and transfers this load to the ground below the potential slip surface. Lateral earth pressures above the slip surface are destabilizing and those below the slip surface are stabilizing. In the case of foundations for sound walls, traffic signals, and signs, lateral loads are most commonly caused by wind action and these are typically resisted by a single element, such as a drilled shaft (Figure 2-8) or occasionally a group of piles or small diameter drilled shafts.

(a)

(b)

Figure 2-1: Bridge abutment (a) and bridge pier (b).

(a)

(b)

Figure 2-2: Temporary excavation support using (a) external braces and (b) tieback anchors.

Figure 2-3: Permanent retaining walls: non-gravity cantilever wall.

Figure 2-4: Noise barrier wall.

Note: Soldier piles are embedded in drilled shafts extending below the slope failure surface

Figure 2-5: Slope stabilization with pile and lagging wall.

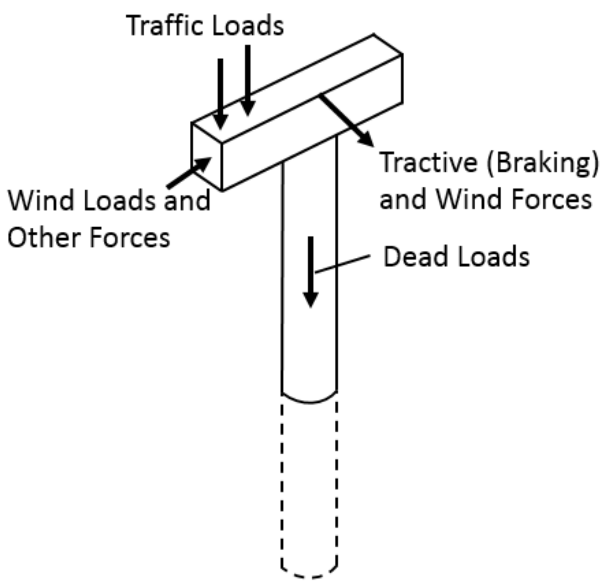


Figure 2-6: Schematic loading in a bridge pier (from O’Neill and Reese 1999).

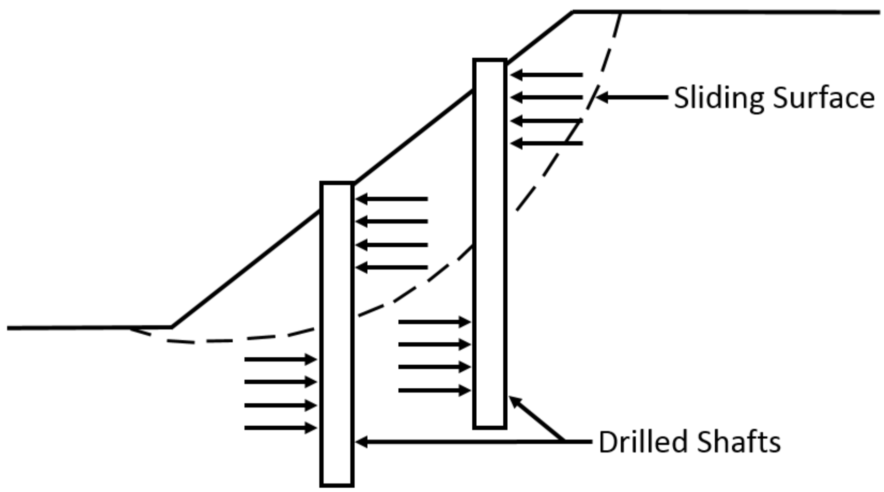


Figure 2-7: Schematic loading for slope stabilization (from O'Neill and Reese 1999).

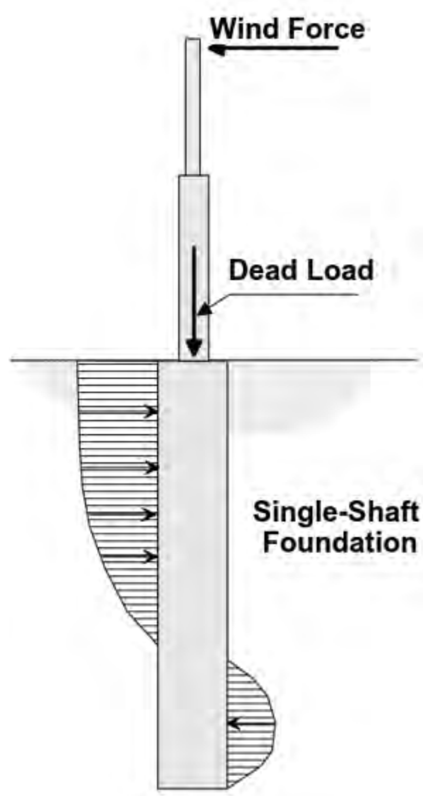


Figure 2-8: Schematic loading in a noise wall (from O'Neill and Reese 1999).

Lateral loads can also include those due to vessel impact on fender systems and other structures around bridge piers in rivers and channels (Figure 2-9), loads that result from scour undermining bridge piers (Figure 2-10), and loads that result from liquefaction or soil softening of soil layers around bridge piers and abutments. When a fender system (i.e., dolphin or mooring system) is an integral part of a bridge pier, lateral loads from vessel collision or mooring would be transmitted to the foundation. In rivers and streams with a high scour potential, deep foundations must extend sufficiently below the scour zone to maintain necessary lateral support. Scour does not apply a load directly to the foundation. However, scour of material from one side of a foundation or from around the entire foundation will result in unbalanced earth pressures and/or additional moments on the foundation due to the increased unsupported pile length. Lateral loads acting on deep foundations can be significantly larger during seismic events than for the static case because of the added inertia forces of the structure and the potential reduction of lateral resistance if liquefaction occurs. Liquefaction reduces the resistance of the soil to lateral load, thereby reducing the lateral resistance of the foundation, and may cause an increased load demand as a result of lateral spreading or lateral flow of the ground above the liquefied layer.

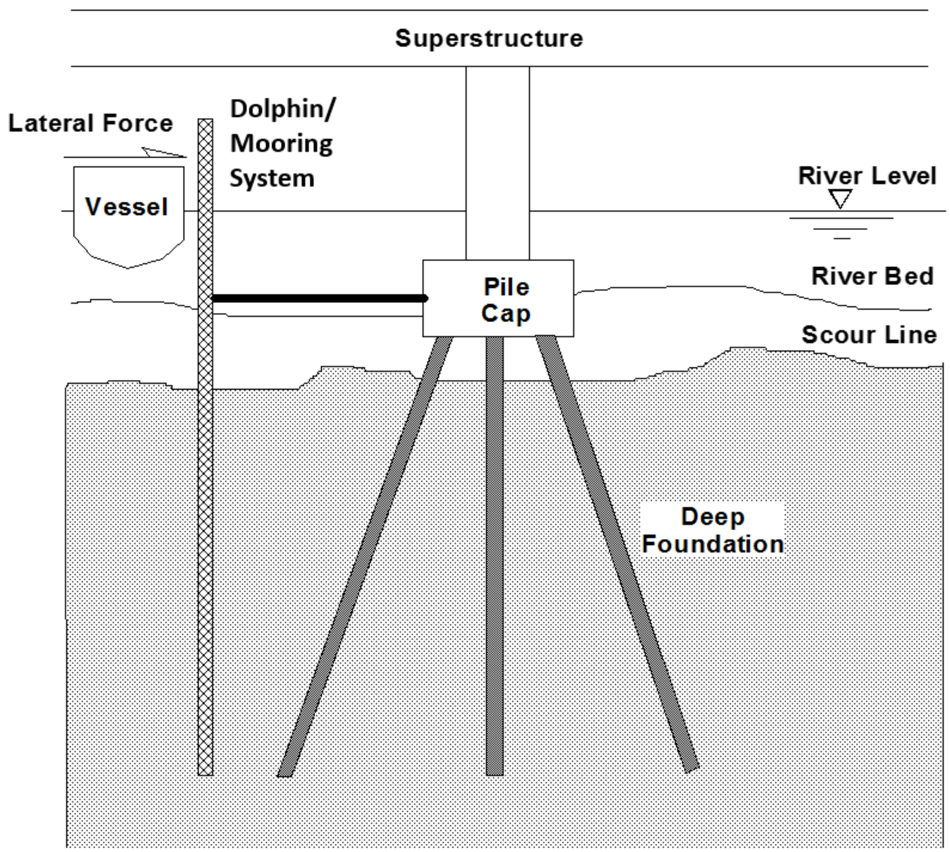


Figure 2-9: Vessel collision on fender piles (from O'Neill and Reese 1999).

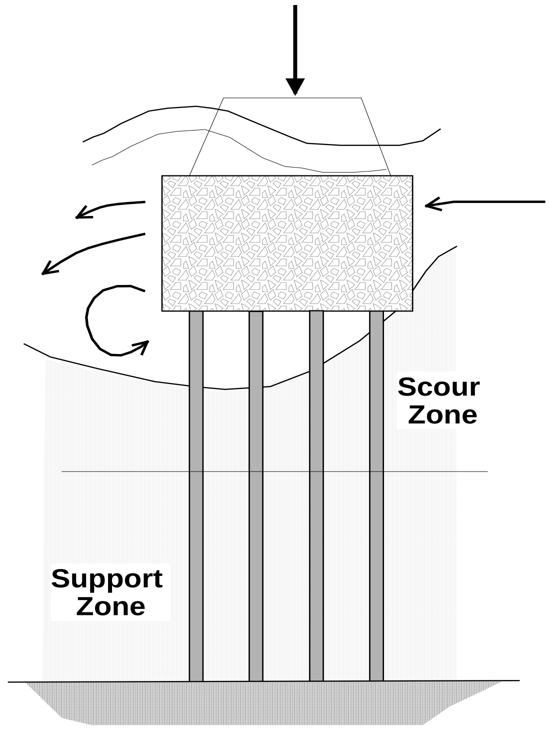


Figure 2-10: Scour effect on deep foundations (from O'Neill and Reese 1999).

2.1.2 Batter Piles for Lateral Load Applications

Most the applications discussed herein are for vertical, or plumb, deep foundation elements. However, batter piles can also be used to resist lateral loads. Batter piles are installed at an angle (batter) that typically varies from 1H:12V to 1H:3V. Batter piles resist lateral loads primarily through the horizontal component of the axial load in the pile as determined by the angle of the batter, whereas vertical piles resist lateral loads primarily through structural shear and bending resistance. Bending of batter piles can also contribute to resistance of applied lateral loads, particularly for cases with small batter angle. The axial resistance in batter piles can be either in compression or tension. Typical batter pile applications include two or more rows of driven piles for bridge foundations with batter piles in one or more rows of the group (see Figure 2-9). Micropiles can also be installed at a batter for similar applications, as well as for slope stabilization in which one or more rows of micropiles may be installed at different batter angles as shown in Figure 2-11. Micropiles in this type of configuration, often referred to as "A-frame," resist the sliding forces of the slope through axial compression, tension, and shear through the micropiles.

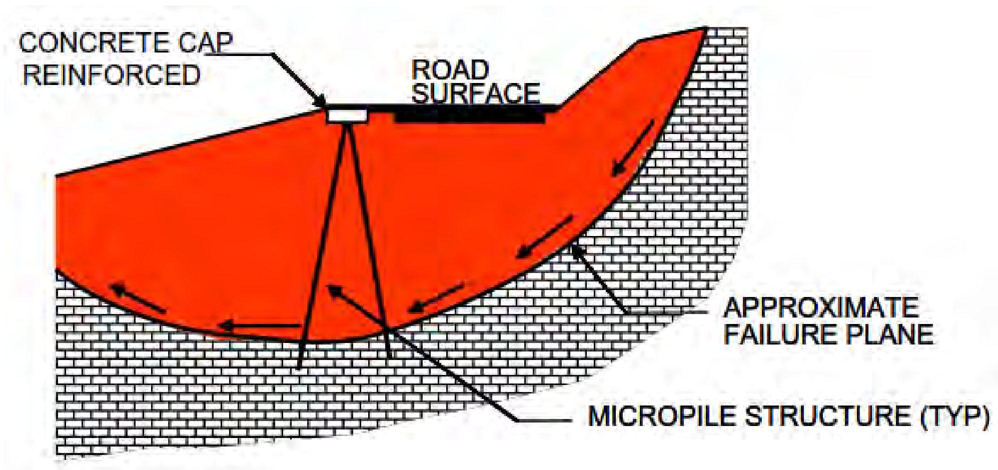


Figure 2-11: Illustration of battered micropiles for slope stabilization (from Sabatini et al. 2005).

Plumb, or vertical, piles are commonly used to resist vertical and lateral loads because they are easier and less costly to construct; however, batter piles typically provide the necessary resistance with less displacement and often with a smaller group of piles. In recent years, the use of batter piles has been discouraged in seismic areas or in projects where a ductile foundation response is necessary. It has been observed that batter piles exhibit relatively poor performance during earthquakes due to the lack of ductility in compression and tend to fail abruptly in shear (Ferritto et al. 1999). As a stiffer foundation, batter piles can also lead to greater lateral load demand during a seismic event. When batter piles are used in seismically active areas, the increased stiffness of the foundation system must be accounted for in the design. Batter piles are also typically discouraged in areas where overlying soil is anticipated to settle significantly relative to the foundation.

2.2 TYPES OF DEEP FOUNDATIONS ADDRESSED IN THIS MANUAL

Lateral load applications typically involve the application of significant lateral forces and/or moments on a deep foundation, resulting in significant internal bending moments and shear forces. Adequate design for such bending moments and shear forces requires deep foundations with adequate structural section and bending moment resistance. For this reason, most lateral load applications for deep foundations involve the use of piles or drilled shafts. However, micropiles and CFA piles, especially with permanent steel casings (for micropiles) and internal reinforcing, can also be used to resist lateral loads, especially if the lateral loads are not significant. Micropiles can also be used in battered configurations for resistance of lateral loads as discussed in Section 2.1.2. These foundation systems are described below; however, the majority of this document focuses on piles and drilled shafts as these are the most common types of deep foundations. Deep foundations systems not addressed in this manual include those with limited tension or moment capacity, such as vibro-compacted concrete columns and unreinforced auger-cast piles; and those not typically used in transportation facility applications, such as helical piles, and timber piles.

In summary, deep foundation types addressed in this manual include:

- Driven Piles
- Drilled Shafts
- Micropiles
- Continuous Flight Auger (CFA) Piles

2.3 SELECTION OF DEEP FOUNDATION TYPES FOR RESISTANCE OF LATERAL LOADS

The selection of the optimum foundation type for a project involves considering first whether shallow foundations are better alternatives than deep foundations. In general, shallow foundations are typically more economical, less complex, and easier to construct than deep foundations. However, deep foundations are preferred or required when a competent stratum is not present within an economical depth, when the footing dimensions required for stability cannot be accommodated, when settlement is excessive, when significant uplift or lateral loads are present, and when the bearing support may be subject to scour or liquefaction. The evaluation of foundation types must be based on vertical loads and settlement, and lateral resistance and displacement. Vertical loads, axial foundation capacity, and settlement are beyond the scope of this document. The discussion herein focuses on considerations for selection of deep foundation type when lateral loads are a significant or controlling aspect of the design.

With regard to lateral resistance, shallow foundations generally only offer limited capacity in the form of passive resistance on the side of the footing and/or shear resistance along the base of the footing. Deep foundations may be more advantageous when lateral loads are large because of the ability to develop high lateral resistance over the depth of deep foundation elements. Deep foundations are generally more economical when overturning loads are dominant (e.g., foundations for noise barrier walls and sign posts). Deep foundations can also be used for slope stabilization applications to intercept and develop sufficient lateral resistance below a failure plane.

The factors to be considered when selecting deep foundations to resist lateral loads include:

- subsurface conditions and geomaterial properties
- design loads
- potential for scour or liquefaction
- structural properties of the deep foundation elements
- constructability
- cost-effectiveness
- structural redundancy
- acceptable magnitude of lateral displacement

In addition, availability of local experience and construction practices may be a factor when considering one type of deep foundation over another. The factors above are discussed in the following subsections.

2.3.1 Subsurface Conditions and Geomaterial Properties

Subsurface conditions specifically affecting the selection of deep foundations for lateral load applications include: occurrence and distribution of hard or soft strata (for establishing range in tip elevations for pile/shafts), the presence of soft soils and fill at shallow depth that will have a major influence on available lateral soil resistance, depth to rock, and groundwater levels. As the resistance to lateral foundation loads depends on the engineering properties of geomaterials, these properties must be adequately established from field and laboratory testing. Guidelines for performing subsurface investigations are provided in the AASHTO LRFD design specifications, Mayne et al. (2002), Loehr et al. (2016), and local standards, if applicable. Chapter 3 presents specific information on the engineering properties necessary to analyze deep foundations under lateral loads.

2.3.2 Properties of Deep Foundation Elements

Piles or shafts with a relatively large bending stiffness will be more suitable to resist horizontal loads because structural resistance is mobilized with smaller deformation. If horizontal loads occur primarily in one direction, foundation elements with a greater bending stiffness in one direction (i.e., such as steel H-section piles) must be oriented to resist the primary loading. Alternatively, batter foundation elements may be advantageous because they may add a significant lateral resistance without increasing the number of resisting elements. If large dynamic or cyclic loads are anticipated, the use of a ductile material such as steel H-piles or pipe piles may be preferred.

Structural resistances (i.e., compression, tension, shear, bending, buckling) are a function of the element materials (timber, steel, or concrete) and cross-sectional properties. Where applied lateral loads are significant, shear and bending moment typically govern the size of both piles and drilled shafts, except when batter piles are used.

2.3.3 Constructability Considerations

Constructability of deep foundations must be addressed during design. If the design is not constructible as designed (i.e., foundation cannot be installed to the required depth or size), then it may not be possible to achieve the required lateral load resistance. Therefore, constructability, and the associated cost to overcome potential construction challenges, must be considered when selecting the type of deep foundation. Some aspects influencing the constructability of deep foundations are briefly discussed below.

<u>General Constructability</u>

- Site access. Is the site accessible for large cranes that would be necessary for pile driving or large drilled shaft construction? If site accessibility is an issue for large foundations, then smaller elements may be more appropriate. If the site is relatively small or if headroom is limited, micropiles may be the only feasible foundation type.
- Available space and accessibility for the foundation units. Sufficient space must be available for the construction of a pile or shaft group and the cap; otherwise a single large diameter drilled shaft may be more appropriate.
- Local practice or precedent. Some DOTs prefer a certain type of foundation element for certain applications.
- Experience of local contractors. Some foundation types, notably micropiles and large diameter drilled shafts, require contractors experienced with these types of foundations.

<u>Subsurface Conditions</u>

- Obstructions. The presence of hard layers or obstructions may make the use of driven piles impractical. Foundation that can penetrate obstructions, such as drilled shafts or micropiles, may be more appropriate.
- Depth to rock. Laterally loaded deep foundations require a minimum depth of embedment that may require penetration of rock if rock is present and relatively shallow. Driven piles and CFA piles may not be able to reach the minimum lengths in such conditions.

- Groundwater conditions. High groundwater, artesian conditions, or flowing or running sands can be an issue for drilled shaft construction. Other foundation types, such as driven piles, may be more appropriate under these conditions.
- Contaminated soils. If contaminated soils are present, driven piles may be preferable because they do not result in spoils. Drilled shafts, micropiles, and CFA piles would all require disposal of contaminated spoils.

Installation

- Pile driving considerations.
  - The ability to drive a pile (drivability) to the required tip elevations must be evaluated during design. Piles must have sufficient capacity to overcome soil resistance during driving and reach specified minimum pile penetration.
  - A pile must have sufficient structural stiffness and strength to withstand driving forces without damage. A drivability assessment is recommended in the current AASHTO specifications (2014).
- Equipment tooling and changes. Drilled shaft construction may require changing equipment or drilling tools to penetrate hard layers, clear obstructions, or penetrate rock. The need for casing and the need for additional equipment, such as casing oscillators, should be assessed as well as the need for slurry and a slurry plant if the wet method of excavation is used.
- Time for installation. Installation times vary by foundation type and may impact the selection of the foundation type. For example, groups of CFA piles may be preferred to groups of driven piles or small diameter drilled shafts owing to faster production for installation, assuming the axial load capacity is not an issue.
- Construction testing. The need for construction testing should be considered. Drilled shafts may require multiple types of testing, such as integrity testing (e.g., CSL testing), axial load tests, and lateral load tests. Lateral load testing may be easier on smaller elements, such as an individual pile for a design based on pile groups, compared to a design requiring large diameter drilled shafts. The required equipment, space, set-up, and performance of the testing should be considered.
- Construction inspection. Inspection requirements vary by foundation type and some require specialized inspection techniques or procedures. Pile driving inspection is relatively simple. Inspection of drilled shafts constructed with slurry may require more specialized techniques to verify the alignment, proper bottom clean out and concrete placement.

Effects on Nearby Structures or Public Perception

- Noise and vibration. Noise or vibrations due to pile driving can be an important factor that may limit the use of driven piles. In urban areas, downhole hammers or drop chisels for excavating rock sockets for drilled shafts may produce relatively high noise and vibration from a public perception standpoint in an urban setting. Noise or vibrations may also have negative impacts on marine environments.
- Disturbance of the adjacent ground.
  - Driving piles can densify, displace, or heave the adjacent ground and nearby structures. Low-displacement piles (e.g., steel H-section piles) or pre-drilling can be used to reduce disturbance around the pile.
  - Post-driving consolidation of cohesive soils may occur as excess pore pressures built-up during pile driving dissipate.
  - Installation of casing for drilled shafts using vibratory hammers may also cause densification and settlement of the adjacent ground that may potentially damage nearby structures.

Additional details and considerations regarding constructability can be found in Hannigan et al (2016) for driven piles, Brown et al. (2010) for drilled shafts, Brown et al. (2007) for CFA piles, and Sabatini, et al. (2005) for micropiles.

2.3.4 Cost Effectiveness

A cost analysis is recommended when selecting among different types of deep foundations for lateral load resistance, particularly when the performance of alternative foundation types are comparable. A cost analysis should consider all aspects of the potential foundation design. Factors such as constructability, seismic design considerations, structural connections, temporary construction works and requirements, etc., can significantly affect the cost of a particular deep foundation system. Failure to understand and consider these factors may increase final costs or may lead to the selection of a foundation type that is not the most economical. Overly conservative designs and inappropriate construction practices may result in significantly larger foundation costs as well.

A cost analysis should include mobilization, materials, installation, locally available practices, local restrictions such as permit requirements or time-of-year restrictions, potential time delays, cost of load testing program(s), total number of piles/shafts required, cost of a pile/shaft cap, and other factors that depend on different types of deep foundations and construction operations required for such foundation systems. Cost-efficiency can be analyzed as total cost for a foundation unit or unit cost per linear foot for walls and slope stabilization (linear foot along the length of wall or slope). In general, individual driven piles are less costly than individual drilled shafts. However, drilled shafts often have higher individual axial and lateral load resistance such that fewer drilled shafts can be used in place of a larger quantity of piles.

The relative cost is also affected by the project size. For small, single-span bridges in which some lateral resistance is required, the use driven piles are often more economical than installing drilled shafts. For piers of large bridges, more piles and a larger cap would be needed for a foundation unit compared to a group of drilled shafts. A single large diameter drilled shaft can eliminate the need for a group of piles and a pile cap. Batter piles may have some advantages with regard to cost and lateral load resistance compared to drilled shafts.

Although this discussion focuses on considerations for lateral load applications, a cost analysis is a broad topic that should include all aspects of the foundation design, including factors for the axial design, structural design, etc. For example, costs for axial load tests may impact the overall cost and selection of the foundation type. In addition, the duration of foundation construction may impact the overall project schedule, with related impact to construction cost.

2.3.5 Redundancy of the Foundation System

Like other structural elements, structural redundancy may provide adequate support in case individual elements fail, or the lack of redundancy should be considered in the design of the structure. Pile/shaft groups may be considered redundant depending on the pile/shaft layout and if there is a sufficient number of piles/shafts in the group. In such a group, if an element (e.g., pile/shaft) fails, other piles/shafts in the group may overcome the deficiency. However, the ability to transfer the load originally resisted by the failing element to the other elements is related to group effects.

Deep foundations consisting of a single element are non-redundant. Therefore, when a single drilled shaft is designed to replace a number of driven piles or smaller drilled shafts, the redundancy present in the group of driven piles is lost. Redundancy is typically a more significant consideration for axial capacity than for lateral capacity. Redundancy is typically addressed by resistance factors in the LRFD design platform, which is discussed in Chapter 4.

2.4 EXCLUSIONS

There are several types of deep foundations that this document does not address or is not applicable to. Deep foundation elements that have little lateral capacity, such as unreinforced CFAs or vibro concrete columns, are not addressed in this document. Also, speciality foundations or deep foundations that are not common for transportation projects in the U.S. are not addressed. Examples include helical piles, screw piles, and similar types of foundations. This document is not applicable to continuous cantilevered retaining walls, such as sheetpile walls, secant pile walls, or tangent piles walls, because the methods used for analyzing continuous walls differ from those for analyzing individual or groups of laterally loaded deep foundations. More complex deep foundations systems that use external supports for lateral load resistance, such as anchored piles or braced piles, are also not addressed in this document.

3 GEOTECHNICAL SITE CHARACTERIZATION FOR DESIGN OF LATERALLY LOADED DEEP FOUNDATIONS

3.1 INTRODUCTION

This chapter presents information on the characterization of subsurface conditions for design and constructability assessments of laterally loaded deep foundations. The intent of this chapter is not to repeat detailed information presented in other publications, but rather to focus specifically on considerations for applications where lateral loads on deep foundations may be a significant aspect of the foundation design, as well as the development of parameters that may be specifically needed for laterally loaded deep foundation analyses.

Subsurface investigations and evaluation of soil and rock parameters in general are addressed in *Subsurface Investigations* (Mayne et al. 2002) and Geotechnical Site Characterization (Loehr et al. 2016). Specific requirements for LRFD design considerations are addressed in Article 10.4 of the *AASHTO LRFD Bridge Design Specifications* (2014). Subsurface characterization for design and construction of deep foundations for transportation projects is addressed in other publications for drilled shafts (Brown et al., 2010), CFA piles (Brown et al. 2007), and driven piles (Hannigan et al. 2016). Those publications include details regarding planning of subsurface investigations, testing programs, and development of geotechnical parameters for analysis of those types of deep foundations.

Regardless of the type of deep foundation, the subsurface investigation and testing program must be adequate for the design of the deep foundation system, including design for lateral loading. The selection of resistance factors for design in the LRFD framework, discussed in Chapter 4, is based in part on the adequacy of the investigation and testing program.

For the analysis and design of piles/shafts under lateral loads, all geomaterials are categorized as either granular soils, cohesive soils, rock, or cohesive intermediate geomaterial (IGM).

3.2 GEOTECHNICAL DESIGN PARAMETERS

Subsurface investigations should consider geotechnical design parameters needed for analysis of deep foundations under lateral loads. Parameters needed for both soil and rock typically used in p-y analyses, described in Chapter 6, are identified in this section. Procedures for obtaining geotechnical parameters are outlined in GEC 5, Geotechnical Site Characterization (Loehr et al. 2016).

3.2.1 Soil Geotechnical Design Parameters

The analysis of deep foundations under lateral loads requires various geotechnical parameters including basic strength, stiffness, and deformation parameters. These are summarized in Table 3-1 for both cohesive and cohesionless soils.

Table 3-1: Geotechnical parameters for analysis of laterally loaded deep foundations in soils.

Material	Parameter: Basic	Parameter: Strength	Parameter: Deformation	Parameter: Stiffness
Cohesive Soils(1)	Unit Weight, γ Poisson's ratio, v	Undrained Shear, S_u	Strain, ε_{50}	Subgrade Modulus, k
Cohesionless Soils	Unit Weight, γ Poisson's ratio, v	Friction Angle, ϕ	N/A	Subgrade Modulus, k Elastic Modulus, E

Notes: (1) Including cohesive IGMs treated as clay-type materials for analyses. Refer to Table 3-2 for parameters if IGM is treated as weak rock.

Additional parameters may be needed for vertical loading considerations, constructability and other aspects of deep foundation design not addressed in this manual.

3.2.2 Rock Geotechnical Design Parameters

It is important to distinguish properties of intact rock and properties of the rock mass when assessing geotechnical parameters for design of laterally loaded deep foundations. Intact rock refers to the consolidated and cemented particles that form the rock material and is characterized by index and strength properties of rock samples recovered from rock coring operations. Rock mass refers to the mass of intact rock and discontinuities including joints and fractures that break up the intact rock. Rock mass is characterized by the properties of the intact rock materials and the characteristics of the discontinuities (Brown et al. 2010).

The information presented in this section applies to materials defined either as rock or as cohesive intermediate geomaterials (cohesive IGM). Cohesive IGM is defined as material that exhibits unconfined compressive strengths in the range of 10 ksf to 100 ksf. In some cases, these materials may also be referred to as soft rock or very weak rock. Specific materials identified by O'Neil el at. (1996) as being cohesive IGMs include (1) argillaceous geomaterials such as heavily overconsolidated clays, clay shales, saprolites, and mudstones that are prone to smearing when drilled, and (2) calcareous rocks such as limestone and limerock and argillaceous geomaterials that are not prone to smearing when drilled.

Rock may be characterized as soft rock or hard rock. For driven piles, soft rock generally refers to rock that can be penetrated by pile driving operations. Hard rock in pile driving operations generally refers to rock that cannot be penetrated by pile driving. For drilled shaft operations, hard rock may refer to rock that requires special tooling for shaft excavation such as rock augers, core barrels, downhole hammers, or chisels.

The analysis of deep foundations under lateral loads, specifically p-y analysis, requires various geotechnical parameters for rock, such as basic strength, stiffness, and deformation parameters. These are summarized in Table 3-2. Not all parameters presented in Table 3-2 are used in every type of lateral pile analysis.

Table 3-2: Geotechnical parameters for analysis of laterally loaded deep foundations in rock.

Material	Parameter: Basic	Parameter: Strength	Parameter: Deformation
Rock(1)	Unit Weight, γ	Unconfined Compressive Strength of intact rock, q_u;	Elastic Modulus of intact rock, E_R; Elastic Modulus of rock mass, E_m; Poisson's ratio, v

Notes: (1) May also include cohesive IGM treated as weak rock

3.3 SUBSURFACE EXPLORATION

Characterization of subsurface conditions and parameters for design and construction of deep foundations requires an adequate subsurface investigation and testing program. Such a program consists of field investigations and testing, including in-situ testing, and laboratory testing of recovered soil and rock samples. These are described briefly herein with emphasis on considerations for laterally loaded deep foundation design.

3.3.1 Subsurface Exploration Program Requirements

Data for the foundation design should be obtained from a subsurface exploration, consisting of field investigation and testing techniques. The subsurface exploration will depend on the type of structure, or application, and the site variability. Table 3-3 provides recommendations regarding the minimum number of exploration points for the design of deep foundations for lateral loading applications. The requirements presented in Table 3-3 are consistent with general recommendations for deep foundation design. Exploration points typically consist of soil and rock borings, but may also include other types of in-situ exploration techniques.

Table 3-3: Minimum number of exploration points and depth requirements for deep foundations subject to lateral loads (modified from AASHTO 2014 and Loehr et al. 2016).

Application	Minimum Number of Exploration Points and Location of Exploration Points	Minimum Depth of Exploration
Retaining walls on deep foundations (soldier pile & lagging walls or cast-in-place walls supported on deep foundations)	▪ Minimum one exploration point per wall. ▪ For walls > 100 feet in length, exploration points every 100 to 200 feet. ▪ For walls with anchors, additional exploration points in the anchorage zone every 100 to 200 feet.	Exploration depth should be 1 to 2 times the height of the wall below the base of the wall, and should be deep enough to penetrate through compressible soils (peat, organic soils, soft silt & clay) and a sufficient depth into competent bearing layers such as hard or dense soils or rock. For slope stability applications, explorations should penetrate a sufficient depth below potential or pre-existing failure surfaces.
Bridge foundations	▪ For piers or abutments < 100 feet wide, one exploration point per substructure. ▪ For > 100 feet wide, a minimum of two per substructure.	In soil, depth should be at least 20 feet below the estimated foundation tip elevation or a minimum of two times the minimum pile/shaft group dimension, whichever is deeper. For foundations bearing on or in rock, a minimum length of 10 feet of rock should be cored (to verify it is not a boulder). For drilled shafts socketed in rock, minimum rock core length should be two times the minimum shaft group dimension or three times the diameter of isolated shafts.

The recommendations in Table 3-2 may be adjusted based on local conditions, knowledge of geology, local practice, and precedent. For example, in highly variable conditions, additional borings may be warranted. Along retaining walls, borings should be spaced in front of and behind the walls to the extent practical to define conditions for the earth pressures on the rear of the wall and conditions for the lateral resistance available in front of the walls.

In rock, geologic knowledge based on local experience should be considered and may take precedence over the recommendations for exploration depths given above. In rock mass that is known to be uniform, free of cavities, voids, weathered zones, etc., it may not be necessary to extend explorations more than a few feet into rock. By contrast, rock masses that are known to include cavities, weak or weathered zones, or other highly variable and potentially adverse conditions, additional explorations and/or additional depth may be warranted.

Other types of structures not addressed in Table 3-2 should be addressed based on local site conditions, availability of subsurface information, and engineering judgment. For example, borings along noise walls are typically more widely spaced than retaining walls. However, noise walls tend to be built along roadways which may have additional boring data for cuts, fills, culverts, or retaining walls that can provide supplemental subsurface information for use in the noise wall foundation design. Similar considerations apply for foundations for sign structures, light posts, etc., except that large cantilever signs and sign bridges may warrant site-specific borings.

3.3.2 Subsurface Exploration Techniques

Several in-situ techniques, including the standard penetration test (SPT) and the cone penetration test (CPT) are routinely used in field investigations to establish the stratigraphy at a site and obtain geotechnical parameters. Other techniques are also used including the pressuremeter test (PMT), the vane shear test (VST), and the dilatometer Marchetti test (DMT). These various types of in-situ tests are illustrated in Figure 3-1. Details regarding the specifics of the investigation types are presented in Mayne et al., 2002.

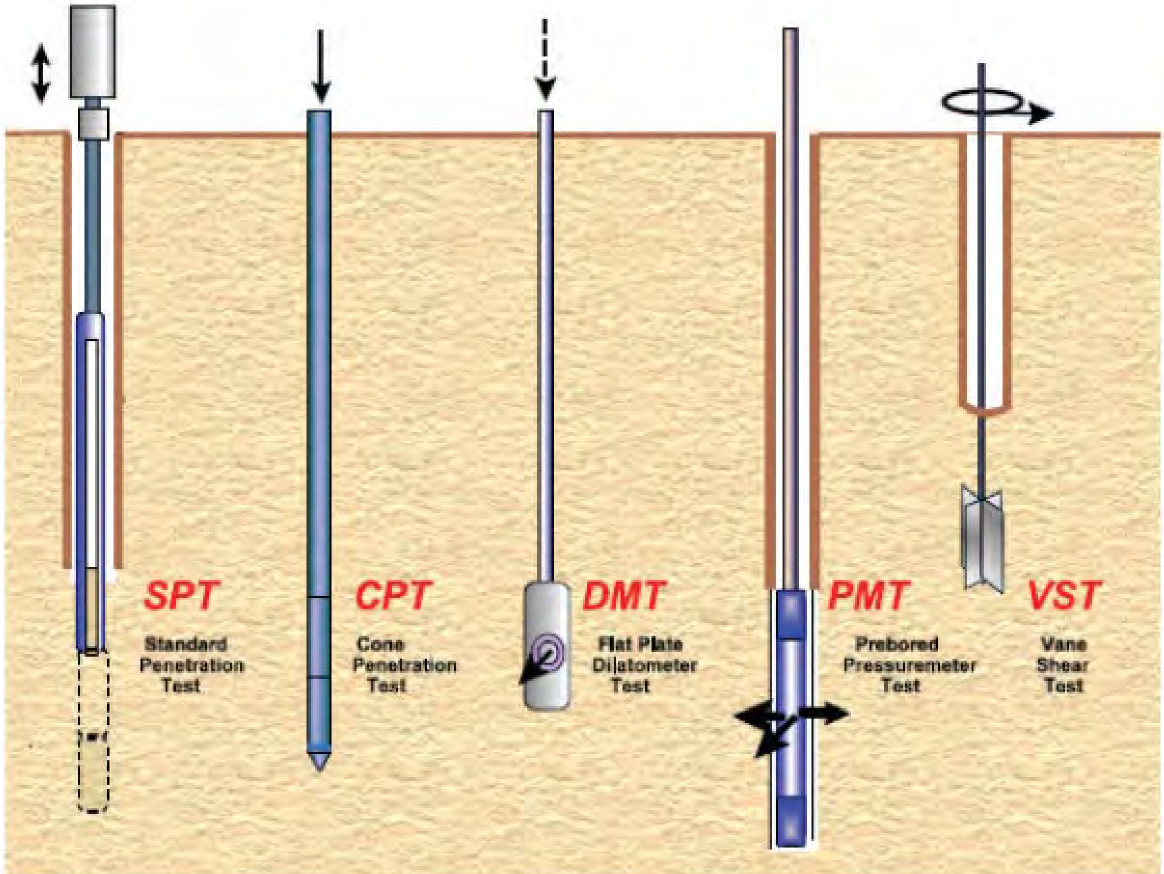


Figure 3-1: Schematic of various in-situ tests (Mayne et al. 2002).

The use of geophysics methods is becoming more common in site investigations. However, due to the need for samples for soil and rock classification and laboratory testing, it is recommended that techniques that provide soil or rock specimens always be used whenever a geophysical investigation is performed. Geophysics can be used in preliminary or planning investigations, or for investigating specific issues for foundation design such as karst features or areas with highly variable rock surface. These methods may be beneficial for general site characterization and planning subsurface investigations, but are generally of limited value for development of design parameters for this application.

For soils, soil borings and SPT tests, or similar drive tests appropriate for site soils, should always be conducted on projects involving deep foundations in order to retrieve samples for proper subsurface characterization and laboratory testing. Similarly, where deep foundations are expected to bear on or in rock, borings with rock coring should always be performed. Additional explorations methods, such as CPT and DMT, may be useful and may be more economical by allowing a reduction in the number of exploratory borings, and may provide data that SPT and rock coring do not. However, the CPT and DMT do not recover soil samples (and do not penetrate rock), but could be used in conjunction with SPT borings in order to correlate the in-situ testing results with actual soil classification and laboratory test data, as well as with SPT N-values. Some types of in-situ tests, such as the PMT and VST, can be performed within an SPT boring if the casing size is large enough to accommodate the testing device. This may offer opportunities for combined types of data acquisition (recovered samples and in-situ test measurements) at a single exploration point.

3.3.3 Considerations for Subsurface Explorations when Lateral Loads are Significant

When lateral loading on deep foundations is expected to be a significant aspect of the structure design, consideration should be given to performing certain in-situ testing to obtain data that can specifically be used for lateral pile analyses. The additional cost of these methods may be offset by a more economical design for lateral loading.

The PMT can be used to estimate the stiffness (E_m) and the mass strength of weak rock and fractured rock for development of p-y curves for lateral pile/shaft analysis (p-y curves will be discussed in more detail in Chapter 6). This technique has been increasingly used in the U.S. and elsewhere to estimate the elastic modulus of a soft rock mass.

The PMT involves inflating a cylindrical probe against the sidewalls of a boring drilled in soil or rock. The most common technique is to insert the inflatable probe in a pre-bored hole before expansion takes place, although self-boring PMTs are also used. Pressuremeter testing for rock uses a device similar to that used in soil but with a stiffer membrane and higher pressure range. The term rock dilatometer is sometimes used to describe a pressuremeter used for testing rock; this is not the same device as a flat-plate dilatometer for testing soil. According to ASTM D 4719, one of the most important aspects of the PMT is assuring that the sidewalls of the borehole are smooth, consistent, and of uniform diameter.

In general, the PMT offers advantages in providing estimates of rock modulus over laboratory methods, particularly in weak rocks, because PMT provides a direct estimation of the modulus of the rock mass with little disturbance, does not require sampling, and it automatically considers the softening effects of fractures, joints, weathering on the lateral deformability of rock. The method also allows the indirect estimation of the rock mass strength. The PMT (and the SPT) can produce estimates of strength and modulus estimates in very weak and weak rocks (Abu-Hejleh et al. 2005). These categories can be defined as rock having unconfined compressive strengths between 20 ksf and 100 ksf, and between 100 ksf and 500 ksf, respectively; however, the definition of weak or very weak rock may vary by local practice area or formation. In general, sampling of soil-like rock for subsequent laboratory testing can be particularly challenging.

Some local agencies have developed region-specific guidelines or correlations for investigations and estimating parameters for weak and very weak rock. For example, the Colorado DOT has produced correlations between PMT and SPT and unconfined compression test results for weak rock (Abu-Hejleh et al., 2005).

The PMT can be used with unusual soil types that may not correlate well with other tests, such as dense gravelly soils, for developing engineering parameters for laterally loaded pile/shaft analysis. PMT tests in soil formations can be used to develop load-deflection response curves that can be used in computerized lateral pile analyses, without being associated with a particular soil classification or without having to develop estimates of other engineering parameters.

In addition to the above procedures, the Goodman Jack or borehole jack test is used occasionally (Goodman et al. 1972) in rock formations to estimate their strength and deformation properties. A borehole jack device consists of exerting a unidirectional pressure on the walls of a borehole using two opposed curved steel plates. This apparatus is designed for use in 3-inch diameter holes. The advantage of this system is that it allows higher pressures than the PMT; therefore, the response of stronger rock can be investigated. The borehole jack results must be corrected to account for the stiffness of the steel plates.

The rock pressuremeter and borehole jack devices require proper calibration and operation, and these may vary depending on the type or manufacturer of the particular device. The system should be calibrated prior to and after testing. Tests should include multiple loading and unloading cycles with pressure readings taken during both loading and unloading cycles (Brown et al. 2010).

The DMT may also be used for developing correlations for laterally loaded pile/shaft design. The DMT is primarily intended for soils with particle sizes smaller than fine gravel and is not well-suited for soils with large gravel, cobbles, boulders, concretions, cementations, large shells, or rock layers. The DMT can be used with correlations to estimate the soil type, at-rest earth pressure, overconsolidation ratio, effective friction angle of sands, undrained shear strength of cohesive soils, and the dilatometer modulus.

The use of PMT, DMT, and borehole jack tests and the calibration of the results of such tests is part of an evolving state-of-the-practice with regard to laterally loaded deep foundations, especially as additional research and testing is performed. For example, a study on rock socketed drilled shafts in Ohio included in-situ pressuremeter and dilatometer testing and full scale lateral load testing. The results of pressuremeter and dilatometer tests were used to develop load-displacement curves (p-y curves) for analysis of the laterally loaded shafts. The p-y curves were compared with and adjusted to match the full-scale load test results. One of the findings of the study was that the dilatometer test results could provide reasonable predictions of p-y curves for drilled shafts in rock by using a method developed by Briaud et al. (1983) with the modification of reducing the p-values by 50 percent (Nusairat et al. 2006).

The in-situ tests in rock have several limitations. Most notably is that they test only a relatively small area of the rock mass. As a result, depending on the joint spacing, test results may or may not be representative of the overall rock mass behavior. A similar consideration would apply with interbedded rock types which may have different properties, such as sandstone and shale. The testing should be performed at sufficient intervals within each rock type to prevent bias in the results for evaluating the overall rock mass. The rock pressuremeter is generally limited to soft to weak rocks.

3.3.4 Laboratory Testing

Laboratory tests are used to estimate geotechnical parameters of soils and rock for use in laterally loaded deep foundation analyses. Index testing consisting of sieve tests (ASTM D6913) and Atterberg limit determination (ASTM D4318) should be performed on recovered samples to verify field classifications and aid in characterization of subsurface conditions. Data regarding the gradation of coarse grained soils (fines content, percentage of fine or coarse grains, etc.) should be considered when correlating engineering parameters for such soils. This is discussed in more detail in subsequent sections. Tests for undrained shear strength of cohesive soils and unconfined compressive strength of rock should also be performed.

The undrained shear strength, S_u, is a key design parameter used in the design of deep foundations in cohesive soils. Several in-situ tests can be used to estimate this parameter; however, laboratory tests are more commonly performed and are often more economical. Laboratory tests for estimating the undrained shear strength of cohesive soils must be performed only on undisturbed samples. The most common laboratory tests to estimate the undrained shear strength of soils are the unconfined compression (UC) tests (ASTM D2166), the consolidated-undrained (CU) triaxial test (ASTM D4767), and the unconsolidated-undrained (UU) triaxial test (ASTM D2850). The CU and UU tests are preferred to the UC tests because the use of confining pressures more closely models the in-situ condition. Descriptions of these tests and the interpretation of UU and CU test results can be found in detail in Chapter 5 of GEC 5 (Loehr et al. 2016).

The compressive strength of intact rock (q_u or σ_{ci}) can be evaluated using the unconfined compressive strength test that is conducted on intact rock core specimens (ASTM D7012). In this test, rock specimens of regular geometry, generally rock cores, are used. After the rock core specimen is cut to a length-to-diameter ratio between 2.5 and 3.0 and the specimen ends are machined-flat, the specimen is placed in a loading frame and loaded. Unconfined compression tests can be used to determine the deformation properties of the intact rock, but correlations with rock mass classifications are needed to estimate the deformation properties of the fractured and weathered rock mass.

Discussion on the use of these and other tests in relation to the development of geotechnical parameters for the analysis of laterally loaded piles/shafts is included in subsequent sections.

3.3.5 Groundwater Conditions

Groundwater conditions must be considered in the design of laterally loaded deep foundations as part of the determination of stratigraphy. Groundwater observations should be made during drilling, upon completion of drilling, and where possible, after stabilization of groundwater levels at least 24 hours or more following completion of drilling. Long-term readings are especially important when water has been used in the drilling process, either for stabilization of the borehole in soils, as part of rock coring operations, or as part of other in-situ testing or drilling operations. A detailed discussion on the measurement of groundwater levels and water pressures, as well as piezometers and groundwater observation wells, is provided in Mayne et al. (2002) and GEC 5, Geotechnical Site Characterization (Loehr et al. 2016).

Groundwater conditions affect the design of deep foundations for vertical and lateral loads because they affect the effective stresses in the soil. In the specific case of analysis of lateral loads on deep foundations, some of the parameters used in lateral pile/shaft analyses have been explicitly developed for above- or below-water conditions. Therefore, the groundwater level is a critical parameter for these types of analyses. Where no groundwater data are available, an assessment of groundwater level must be made based on soil moisture descriptions, local knowledge of the site, regional geology, or other methods. In addition, assessment of the groundwater level should include consideration of flood levels, tidal and seasonal variations, and potential variations due to other sources, such as industrial operations, changes in hydrogeological conditions, etc. Where groundwater data are not available or groundwater levels may fluctuate significantly, a conservatively high groundwater level should be used to ensure that the most adverse groundwater condition has been considered in the lateral pile/shaft analysis. If necessary, a parametric study or sensitivity analysis can be performed by considering the static groundwater level and potential variations in groundwater conditions to assess the economics of designing for extreme or uncertain variations.

4 LRFD DESIGN REQUIREMENTS AND LIMIT STATES FOR LATERALLY LOADED DEEP FOUNDATIONS

4.1 INTRODUCTION

The design of laterally loaded deep foundations must address all applicable Limit States using appropriate load combinations, load factors, and resistance factors. The design of laterally loaded deep foundations requires interdisciplinary coordination between the structural engineer and the geotechnical engineer, especially for determining LRFD design requirements for laterally loaded deep foundations.

4.2 LOAD COMBINATIONS AND LOAD FACTORS

The Limit State design approach in LRFD requires an identification of all potential failure modes, or Limit States. A Limit State is defined as a condition for which some component of the structure does not fulfill its design function. Four Limit States are identified in AASHTO 2014: Strength, Service, Extreme Events, and Fatigue. Limit States that typically govern deep foundation design include Strength I, Strength IV, Extreme Events I and II, and Service I. Service Limit States II, III, and IV and Fatigue Limit States I and II are typically associated with superstructure behavior and not generally not applicable to foundation design.

The total factored load for a given Limit State is calculated as the sum of the individual load effects and corresponding load factors and load modifiers that apply to the given Limit State. For the applicable load, load combinations and load factors associated with each Limit State, refer to AASHTO 2014.

Figures 4-1 through 4-3 present simplified illustrations of typical loads on deep foundations for transportation structures. Figure 4-1 illustrates typical loads on a bridge abutment in the longitudinal direction (loads arising in the transverse direction are not included in this illustration). Typical loads for piers supported on deep foundations are shown in Figure 4-2. Design loads transmitted from the superstructure include dead and live loads, wind on structure, wind on live load, and temperature forces. Piers also resist loads from self-weight, wind on substructure, stream flow, buoyancy, ice flows, creep, shrinkage, and other loads. When piers are in a river, stream, or other navigable waterway, the deep foundations must be designed against factors such as scour, stream flow effects, temperature effects associated with the water stream, and potentially vessel collision. Horizontal forces and bending moments caused by lateral loads acting on noise walls and similar structures are illustrated in Figure 4-3.

When deep foundations are used to stabilize landslides or slopes, the soil behind the foundation elements can generate very high lateral forces. The loads that need to be considered for such an application are illustrated in Figure 4-4. In general, the computation of lateral loads in slope stabilization cases is more complex than the load computation for retaining structures. Deep foundations for slope stabilization are discussed in Chapter 10.

As illustrated in Figure 4-5, the reactions at the column-foundation joint, or pile cap, computed by the structural analysis are taken as the force effects transmitted to the foundations. For deep foundations, the reactions are resolved into vertical, horizontal, and moment components, and these are taken as the factored values of axial, lateral, and moment force effects, respectively, at the top of the foundation or pile cap. Multiple iterations are typically performed to obtain agreement between deformations and forces at the structure/foundation interface as calculated by both the structural and geotechnical analysis. The resulting factored force effects are substituted into Equation 4-1. Although this is a somewhat oversimplified description of the actual process, it is the general procedure by which factored foundation force effects are determined for each applicable Limit State.

Also, note that load factors for permanent loads are specified at maximum and minimum values. For foundation design, modeling of the structure while varying the load factors is necessary to determine the combination resulting in maximum force effect acting on the foundation, which are then used in Limit State checks.

The loads in Figures 4-1 through 4-5 include the permanent and transient loads that should be considered:

- Permanent Loads
  - *CR* = Force effects due to creep
  - *DD* = Downdrag force
  - *DC* = Dead load of structural components and nonstructural attachments
  - *DW* = Dead load of wearing surfaces and utilities
  - *EH* = Horizontal earth pressure load
  - *EL* = Miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
  - *ES* = Earth surcharge load
  - *EV* = Vertical pressure from dead load of earth fill
  - *PS* = Secondary forces from post-tensioning for Strength Limit States; total prestress forces for Service Limit States
  - *SH* = Force effects due to shrinkage
- Transient Loads
  - *BL* = Blast loading
  - *BR* = Vehicular braking force
  - *CE* = Vehicular centrifugal force
  - *CT* = Vehicular collision force
  - *CV* = Vessel collision force
  - *EQ* = Earthquake load
  - *FR* = Friction load
  - *IC* = Ice load
  - *IM* = Vehicular dynamic load allowance
  - *LL* = Vehicular live load
  - *LS* = Live load surcharge
  - *PL* = Pedestrian live load
  - *SE* = Force effect due to settlement
  - *TG* = Force effect due to temperature gradient
  - *TU* = Force effect due to uniform temperature
  - *WA* = Water load and stream pressure
  - *WL* = Wind on live load
  - *WS* = Wind load on structure

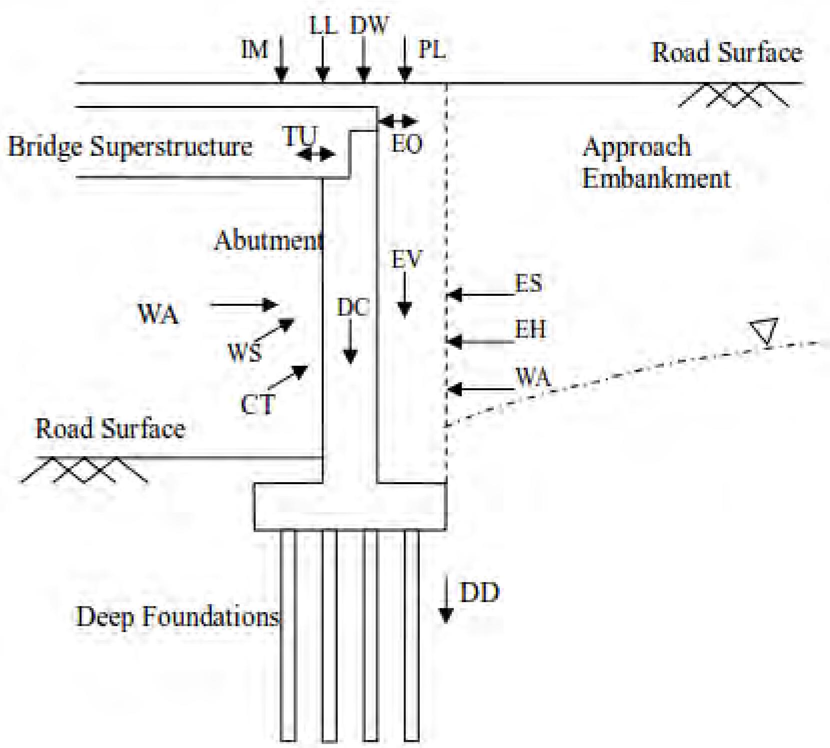


Figure 4-1: Typical loads in bridge abutments supported on deep foundations.

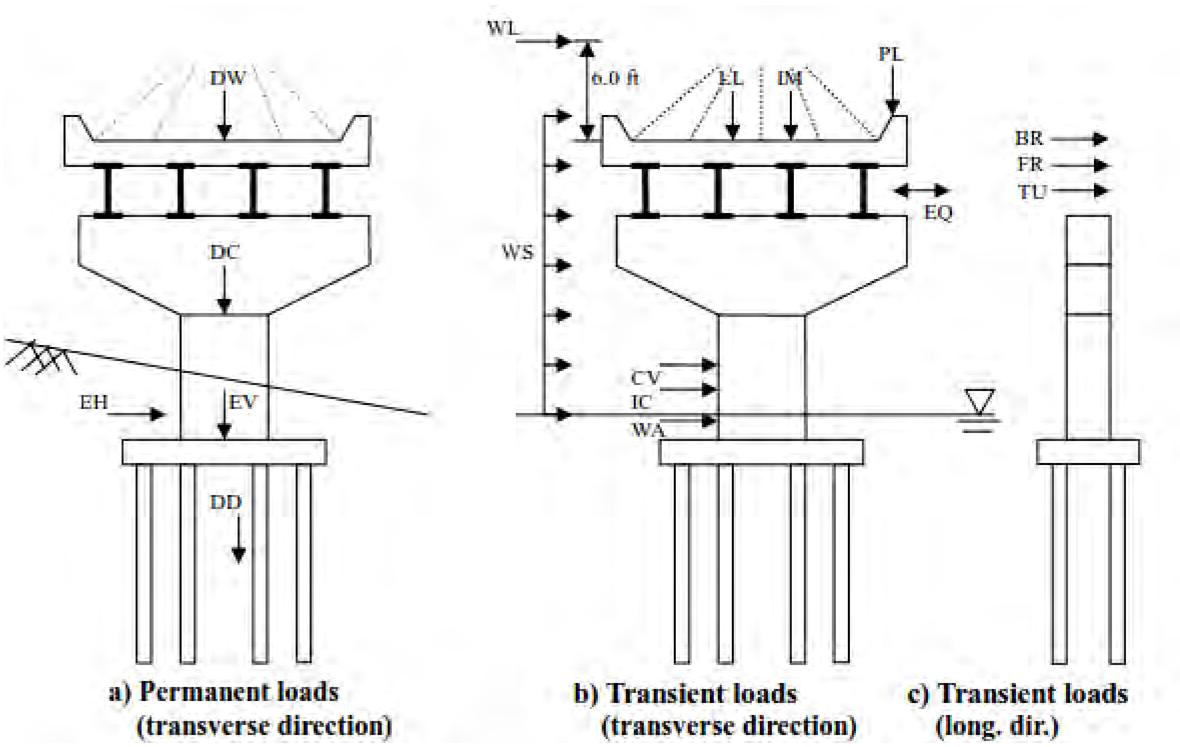


Figure 4-2: Typical loads in bridge piers supported on deep foundations.

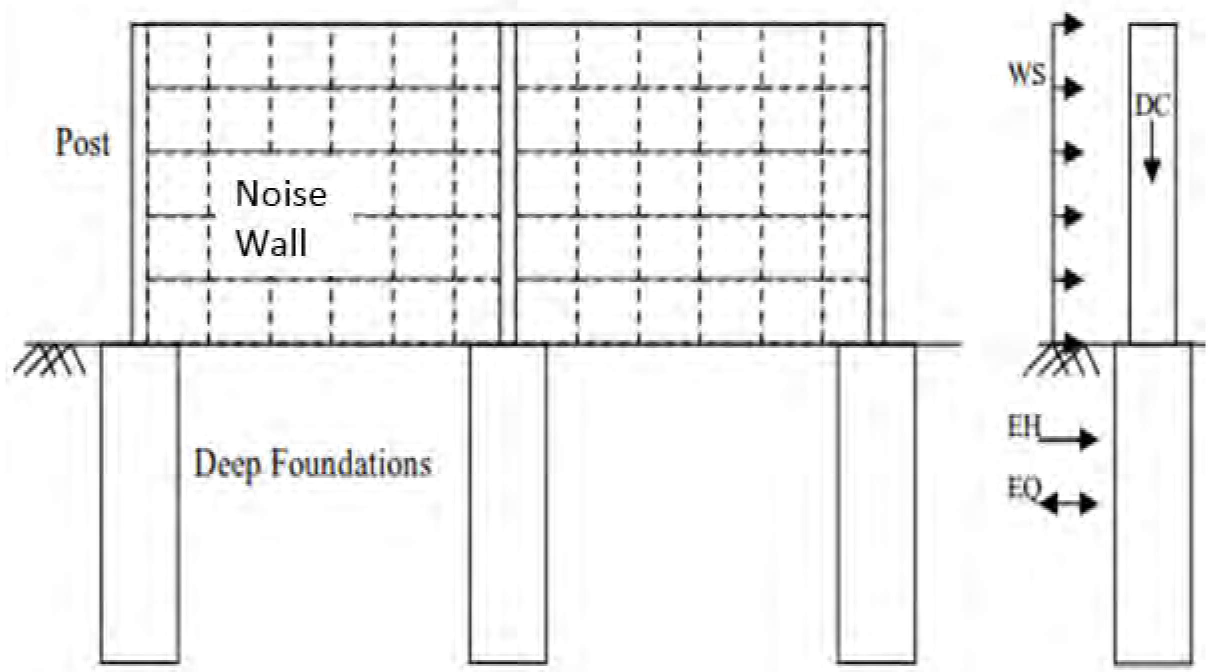


Figure 4-3: Typical loads on noise walls supported on deep foundations.

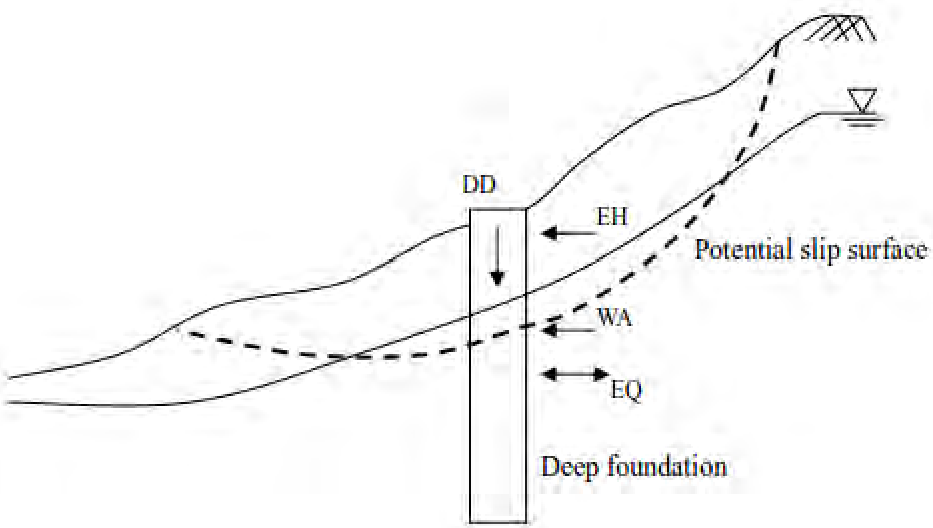


Figure 4-4: Typical loads in deep foundations for slope stabilization.

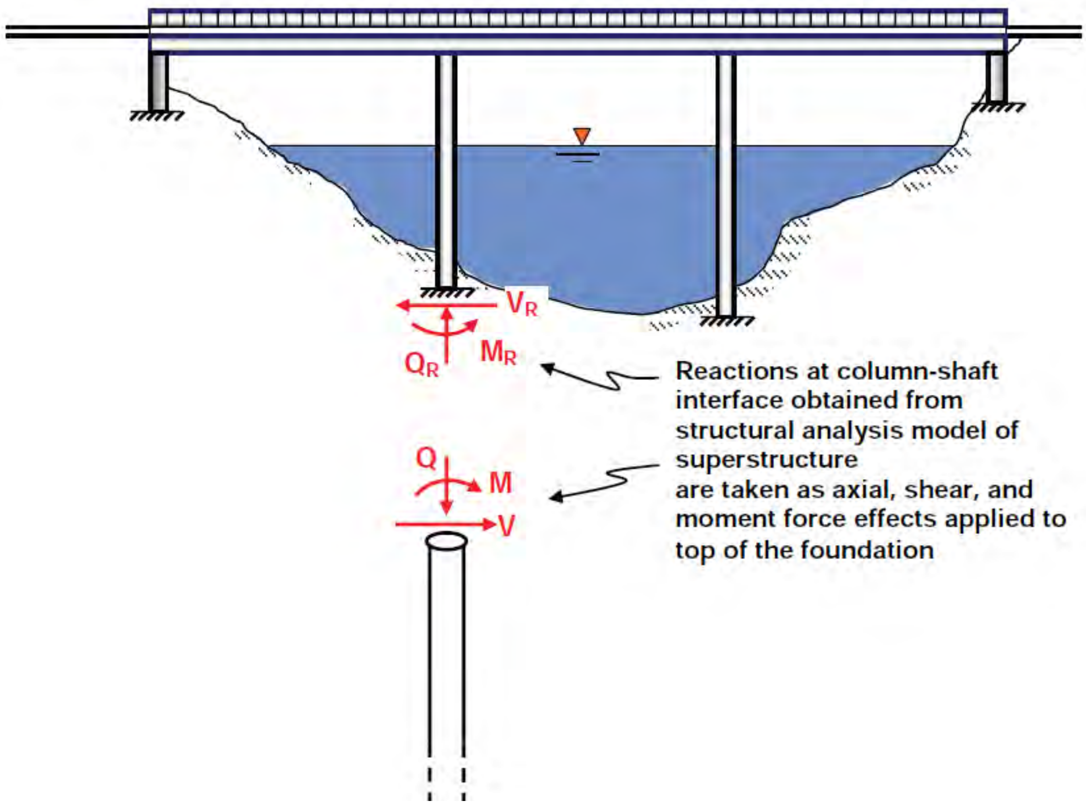


Figure 4-5: Illustration of force effects on bridge foundation (after Abu-Hejleh et al. 2011).

4.3 STRENGTH LIMIT STATE FOR LATERALLY LOADED FOUNDATIONS

Strength Limit State failure modes are related to the strength and stability of the foundation under loads applied during the design life of a bridge or structure. This Limit State includes an evaluation of typical geotechnical and structural resistances to resist the loads applied to them with an adequate margin of safety against damage or collapse. For laterally loaded deep foundations, Strength Limit States include geotechnical strength (failure of the soil) and structural strength (failure of the foundation element) Limit States. Structural Strength Limit State is discussed further in chapter 11.

Resistance factors have not been calibrated for lateral analysis of deep foundations; however, Table 4-1 provides recommended values for geotechnical strength limit state design. The values recommended in the table are suggested based on engineering judgement (Brown et al., 2010) in recognition that this recommended approach provides a check for ductility and geotechnical stability that exceeds the level of reliability provided by the AASHTO (2014) code provisions. The recommended resistance factors apply to lateral analysis of driven piles or drilled shafts for the Strength Limit State using load-displacement (p-y) analyses. These resistance factors provide a check for ductility and geotechnical stability that exceeds the level of reliability provided by the AASHTO (2014) requirements.

Table 4-1: Resistance factors for lateral Geotechnical and Strength Limit State (from Brown et al. 2010)

Application	Resistance Factor, ϕ_r
Pushover of individual deep foundation element; head free to rotate	0.67
Pushover of single row of deep foundation elements, retaining wall or abutment; head free to rotate	0.67
Pushover of deep foundation element within multiple-row group, with moment connection to cap	0.80

The state of the practice for analysis of laterally loaded deep foundations is evolving as additional research and lateral load testing results are obtained. For example, local DOTs have developed locally calibrated resistance factors based on substantial successful experience and load tests (Abu-Hejleh et al. 2011). Additional references discussing local resistance factors include the Idaho Transportation Department *Bridge Design LRFD Manual* (2008), a study by Robinson et al. (2006) for NCDOT, and a study by MoDOT (Boeckmann et al. 2014).

4.4 SERVICE LIMIT STATE FOR LATERAL DISPLACEMENTS

Service Limit State failure modes are related to performance problems (e.g., deformation) for a structure under regular operating conditions. For laterally loaded deep foundations, Service Limit States relate to lateral displacement of foundation elements considering interaction with the structure. Laterally loaded deep foundations must have adequate structural and geotechnical resistance to keep bridge or structure displacements within established tolerable levels. Tolerable lateral settlement is project specific and should be determined by the project structural engineer.

A resistance factor of 1.0 is applicable to Service Limit States. Soil-structure interaction analyses methods are to be used for Service Limit States and may include Broms method, strain wedge method, and p-y analyses depending on the type of structure and foundation (AASHTO 2014). Note that p-y analyses are the recommended method.

Lateral displacements of foundations elements are analyzed at Service Limit State load conditions to ensure that the lateral foundation displacements are below tolerable lateral structure displacements. The tolerable lateral displacements of foundation elements are often limited based on lateral displacements that will adversely impact the structure, such as closure of joints on bridge structures, excessive structure lean or rotation, displacement of one structural element into another, aesthetics, etc. The magnitude of tolerable lateral displacements should be determined by the structural engineer and may be based on considerations other than the geotechnical resistance or the structural capacity of the foundation (these displacements are often less than the amount of displacement that the foundation elements themselves can withstand, especially for ductile elements such as steel piles). The limiting deflection of the structure, and therefore the limiting deflection of the foundation, are assessed using Service Limit State load combinations and load and resistance factors equal to 1.0.

Analyses of foundations for deformation are generally performed using p-y methods, although alternative methods such as the strain wedge method or FEM can be used; analysis methods are discussed in more detail in Chapter 6. Lateral displacements can also be determined from lateral load tests.

Each individual or group of foundation elements should be designed so that deformations do not exceed criteria established for the bridge or other structure. The deformations to be checked relative to lateral loads on deep foundations include:

- Horizontal movement at the top of the foundation
- Rotation at the top of the foundation
- Horizontal movement under scour at the design flood
- Horizontal displacement at the superstructure level

Lateral displacement may also be part of the criteria used to define Geotechnical Strength Limit States, as discussed in Chapter 5. Lateral displacements may also be a consideration for Extreme Event loading, to avoid adjacent structures from impacting each other or to avoid excessive damage to the structure. Earthquake loads can result in liquefaction in subsurface soil layers or lateral displacement of a slope (in combination with excess pore pressures and gravity loads). Lateral slope movement or movement of surficial soil blocks overlying a liquefied layer may result in lateral displacements to deep foundations, which can result in additional loads on the foundation elements and reduce the available deformation capacity for the service limit resistance.

Much work has been done in the field testing of drilled shafts and piles to measure lateral displacements. Examples of these are included in Brown et al. (2010) and shown in Figure 4-6. Measurements of lateral deformations and displacements should be part of any lateral load testing program. Measurements of lateral displacements can also be included as part of a monitoring program to verify performance of a foundation system. For example, lateral deformation measurements, such as inclinometers, may be included within drilled shafts used for slope stabilization. Deformations of such shafts can be monitored following installation to verify that the slope has been stabilized or to establish the need for the installation of additional elements if on-going or future movements are observed. A more detailed discussion of field testing of deep foundations and associated instrumentation is included in Chapter 12.

Figure 4-6: Measurement of lateral deformation in a test shaft using an inclinometer (from Brown et al. 2010).

4.5 EXTREME EVENT LIMIT STATE

Extreme Event Limit State failure modes are related to the strength and stability of the foundation under unique loading events that have a return period greater than the bridge design life. These events include check flood (500-year event for scour), earthquakes, and/or major vessel or vehicle collision. The design concern is survival of the bridge and protection of life safety with the expectation that some damage to the structure will occur without resulting in collapse.

The recommended load and resistance factor for extreme events is 0.80 for geotechnical lateral resistance (Brown et al. 2010). See Chapter 8 for a detailed discussion of the Extreme Event Limit State.

4.6 CONSIDERATIONS FOR LIMIT EQUILIBRIUM APPLICATIONS

For evaluation of global stability of retaining structures and slope stabilization, limit equilibrium applications are needed. Resistance factors and additional load factors in current standards are typically not applied to loads and resistances in limit equilibrium analyses. Using software to assess global stability, the analyses are performed at the Strength I Load combinations and targeting a load factor of 1.0 and a resistance factor of 0.75. If supporting a structure foundation, the resistance factor of 0.75 is used in combination with applying Strength Limit load factors to foundation loads to be consistent with current Strength Limit load groups and load factors. A resistance factor of 0.65 should be used when the geotechnical parameters and subsurface stratigraphy are highly variable, or are based on limited information. The analysis of deep foundation elements for slope stabilization is further discussed in Chapter 10.

5 DESIGN PROCESS AND TEAM ROLES FOR ANALYSIS OF LATERALLY LOADED DEEP FOUNDATIONS

This chapter provides an overview of the design procedure for laterally loaded deep foundations and team roles for geotechnical and structural engineers during the design process. The design procedures are intended to focus on procedures and considerations specific to laterally loaded deep foundations. This is not intended to be a comprehensive description of design procedures for all aspects of deep foundation design; rather this section focuses on design for lateral loading only. Refer to other references, such as Hannigan et al. (2016), Brown et al. (2010), Brown et al. (2007), and Sabatini et al. (2005) for further details on the design of driven piles, drilled shafts, CFA piles, and micropiles, respectively.

Although the design procedure is presented as a step-by-step process, in practice it often does not follow a linear path. The size and scope of the project, contracting approach, and complexity of the foundation loadings (lateral load demand as well as axial loads) affect the actual progression of tasks. The design process often includes multiple iterations in order to refine the design, incorporate additional subsurface data or testing information, accommodate design changes, or address constructability considerations. The design procedure presented herein is intended to provide a logical general design procedure to enable the designer to consider the necessary steps for adequate lateral pile design and analysis; the actual progression of the steps may vary. The design procedure is illustrated in Figure 5-1.

Similarly, the roles of design engineers presented herein are general roles that are common in practice, but may differ for individual projects based on the type of project, complexity of the work, local practice, contract approach and responsibilities, and the experience of the individual designer.

5.1 DESIGN PROCESS

Block 1 – Establish Project Type, Performance Requirements, and Constraints

The first step in the process is to define the type of project and project needs, including the need for laterally loaded deep foundations.

1. Establish the general structure requirements. Is it a new bridge, replacement bridge, retaining wall, noise wall, slope stabilization, sign or light post, etc.?
2. Identify and define project or site conditions that may impact the selection of deep foundation type or construction, especially regarding selection of deep foundation type and lateral loading considerations. Examples include limited right-of-way, constrained site areas or access, overhead constraints, potential for scour, potential for construction over bodies of water (construction considerations for foundation type selection, potential for lateral loads due to waves or vessel impacts, etc.), wetlands or other areas with environmental restrictions, existing or adjacent structures, restrictions regarding vibrations, etc.
3. Identify the general structure layout and site grades, surficial site characteristics, and general geology.
4. Identify any special design events or considerations, such as seismic, scour, downdrag, vessel impacts, etc.
5. Determine preliminary load types and estimates (even order of magnitude) to aid in determining whether deep foundations may be needed and whether lateral loads will be a significant design consideration.

6. Determine load factors for the applicable load types and resistance factors for Strength, Service, and Extreme Event Limit States. As indicated in Chapter 4, Strength Limit States will be defined based on the factored load combinations for design. Multiple load combinations may need to be assessed unless a single, most critical load combination can be identified. Service Limit States include tolerable deflections and/or global stability using service limit loads. Global stability requirements for retaining walls or slope stabilizations should consider consequences of failure in the selection of the resistance factors as discussed in Section 4.6.
7. Define lateral foundation design performance criteria. This may include limiting deflection values, a maximum factored resistance, and/or achievement of a maximum resistance against global stability (in the case of slope stabilization or retaining walls).

In some cases, it will be known at the early stages of a project that deep foundations will be needed and will be subject to lateral loads that are significant enough to control the overall foundation design. Examples of this type of project include noise walls, continuous retaining walls for excavation support, or retaining walls or deep foundations for slope stabilization. In other cases, it may not be evident at this stage that laterally loaded deep foundations will be needed. For example, a new bridge or bridge replacement in an area where rock or dense soils are relatively shallow, especially if an existing bridge is on shallow foundations. It may not be evident that the new bridge will need deep foundations until other aspects of the project are better defined, such as the structure loads, strength of surface materials, scour depth, seismic loads, etc. The steps that follow assume that deep foundations with lateral loads are expected.

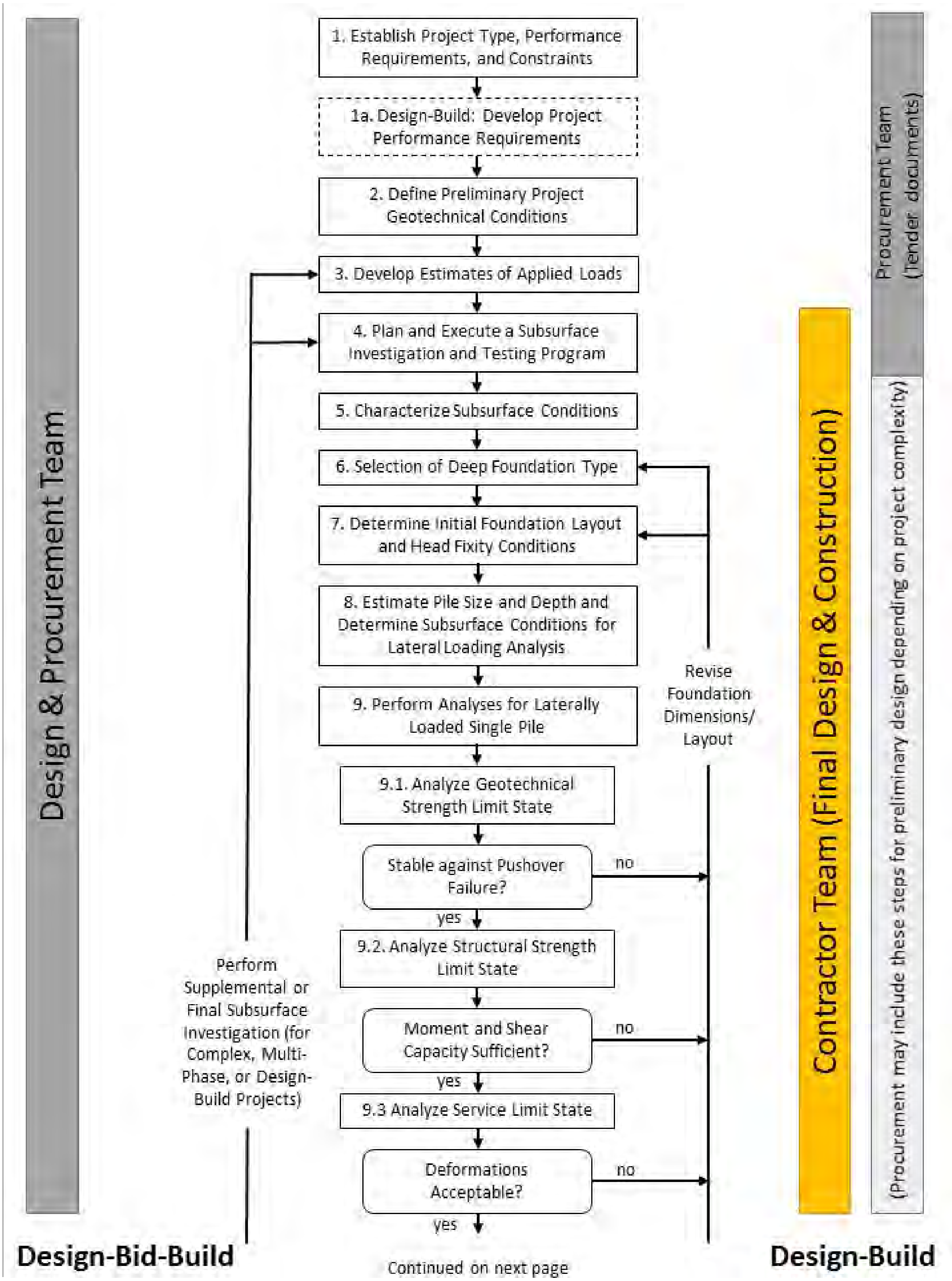


Figure 5-1: Design procedure for laterally loaded deep foundations (continued next page)

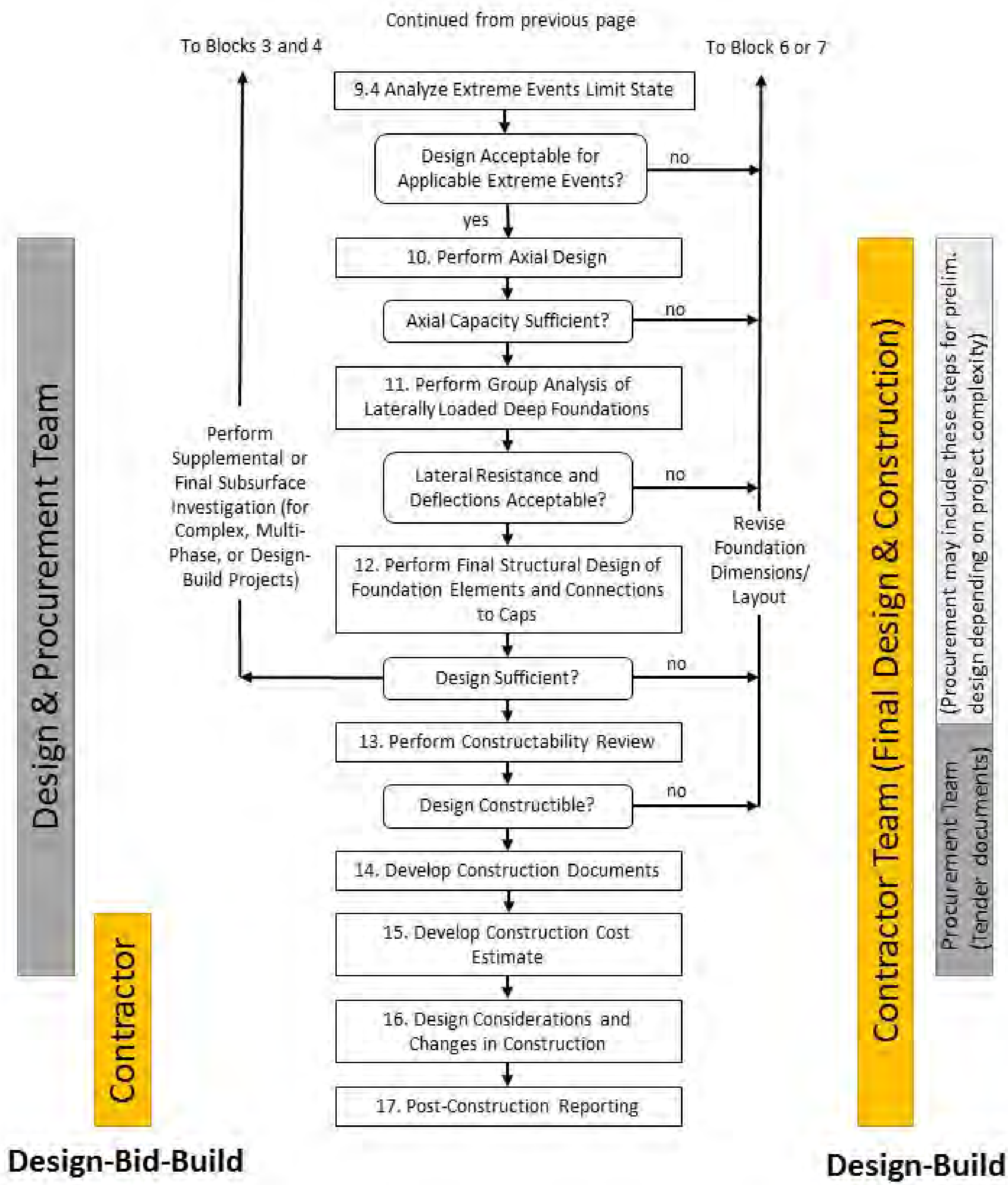


Figure 5-1: Design procedure for laterally loaded deep foundations (continued from previous page)

Block 1a – Design-Build: Develop Project Performance Requirements

In design-build, the project owner will develop project performance requirements before the tender design stage. These requirements will include the design specifications that are to be followed, including any local guidelines or practices. These may include defining the required subsurface investigation requirements, procedures to be used for lateral foundation design, the maximum allowable resistance factors for design of laterally loaded deep foundations, minimum lateral load testing requirements, and construction inspection and testing requirements. Many aspects of these are covered in the AASHTO specifications. However, this step allows the project owner the opportunity to specify deviations from or supplemental information to the AASHTO specifications. These may include site- or location-specific geologic models for development of p-y curves, deflection criteria for Service or Geotechnical Strength Limit States, and/or minimum requirements for use of certain resistance factors (local practice, etc.). In general, the design-builder will use the maximum resistance factor allowed (in order to be competitive) unless the values of resistance factor are tied to other criteria, such as local practice or load testing, in the project performance requirements, and subject to the judgment of the design-builder's designer.

The project owner may want to specify or preclude the use of a certain foundation type, or include minimum requirements regarding analyses and testing for each particular feasible foundation type. Additional project performance requirements should be developed for other aspects of the design, such as axial capacity, allowable settlement and differential settlement, structural performance, etc.

Block 2 – Define Preliminary Project Geotechnical Conditions

A preliminary characterization of the project geotechnical conditions should be developed for use in planning the subsurface investigation and preliminary foundation selection. A desktop study of available records should be performed.

The desktop study should help to identify geotechnical or geologic conditions that may be relevant to the design of laterally loaded deep foundations, such as the presence of poor soils, IGMs, type and depth of rock, etc. Also, key for this step is to identify any precedent that may exist for laterally loaded deep foundation design or previous work that can be used for such design, such as records of load tests, foundation performance, previous investigation data relative to laterally loaded deep foundations (PMT or DMT results), local practice with regard to testing and development of parameters for lateral pile analysis, etc. This will help to inform the development of the subsurface investigation, including the planning of sampling and testing types and depths. In some cases, particularly in design-build procurement, a sufficient subsurface investigation program to establish geotechnical baselines for procurement may be completed at this stage.

Block 3 - Develop Estimates of Applied Loads

The proposed structure design should be advanced far enough to develop an estimate of the loading conditions that will be applied to the deep foundations. A preliminary foundation layout should be used to develop an estimate of the load distribution among the foundation elements. For the purposes of this manual, and this design procedure in particular, it is assumed that the estimates of applied loads will establish that deep foundations are needed.

Development of loads includes both axial and lateral loads because the performance of deep foundations subjected to lateral loads is also influenced by the applied axial load. Load combinations and appropriate load factors must be developed and included in the development of the loads as discussed in Chapter 4. The most critical load combination(s) can then be determined and used for developing the foundation design.

Block 4 - Plan and Execute a Subsurface Investigation and Testing Program
Based on the results of Blocks 1, 2, and 3, a subsurface investigation program can be planned and performed to obtain the information necessary to characterize the geotechnical conditions and develop design parameters.

The performance of the subsurface investigation program may be done in multiple stages, such as a preliminary investigation followed by more detailed investigation, or a supplemental investigation may be performed later in design to verify initial findings or obtain more detailed data for finalizing the design. This is especially true for design-build contracts where an initial investigation is performed for the development of the preliminary design for the procurement documents, followed by a final investigation for final design by the design-builder. In design-build projects, and often for design-bid-build projects, the locations of structure foundations are not known or finalized at the time that the initial subsurface investigation is performed.

Block 5 - Characterize Subsurface Conditions
The results of the subsurface investigation and testing program are used to characterize the subsurface conditions for analysis of laterally loaded deep foundations. A subsurface stratigraphy and profile should be developed for the site for use in design, including soil layers, rock layers, and groundwater levels. Subsurface parameters should also be developed along with inputs for individual soil and rock units for lateral pile analysis.

Note that stratigraphy and parameters developed for lateral pile analysis may differ from that used for axial foundation analysis.

Block 6 - Selection of Deep Foundation Type
Understanding the project needs (Block 1), loads (Block 3), and subsurface conditions (Block 5), select a deep foundation type for preliminary selection. In some cases, the foundation type may be decided based on local practice or standards regarding similar types of structures or by structural design and connection considerations. In other cases, there may be several viable alternatives that may need to be evaluated in more detail.

In selecting the type of deep foundation, consideration should be given to:

- Lateral load capacity
- Lateral soil displacements
- Subsurface conditions and their impact on constructability
- Proximity of adjacent structures
- Low headroom
- Noise and vibration restrictions
- Availability of local contractors
- Project size/number of foundation elements
- Corrosivity of site soils
- Whether the work is over water
- Others

Determination of the foundation type often requires a significant amount of engineering judgment and should be performed by engineering professionals with sufficient experience in this field. In some cases, more than one foundation type may be feasible and additional considerations, such as economics and relative risks, will need to be considered. It may be possible that two or more foundation types are considered for preliminary analyses or evaluations. Axial load capacity may also influence or control the selection of the foundation type. Considerations of axial loading will be needed if axial loads are a significant aspect of the design. For complex projects or design-build projects, the process of selection of the deep foundation type and size may require iterations as the subsurface conditions are more thoroughly characterized during later stages of design or as the loading conditions are more fully developed.

Block 7 - Determine Initial Foundation Layout and Head Fixity Conditions

Based on the selection of the foundation type(s) and the characterization of the subsurface conditions, the structure type and geometry are advanced far enough to determine an initial foundation layout, including fixity condition. For example, if a retaining wall or abutment is a cantilever wall supported on piles, the size of the footing may be established along with estimated pile size, spacing, and number of piles within the group.

The spacing between foundation elements may impact the available soil resistance for analysis through shadowing or overlapping areas of influence under lateral loads. This is discussed more in Chapter 7 regarding analyses of groups of deep foundation elements. Often the design will be started assuming no reduction in capacity is needed based on spacing and may be refined later as the foundation layout is finalized.

At this stage of design, it may also be determined whether the pile head will be sufficiently fixed within a cast-in-place footing (pile cap), or whether it will be a free head condition that is allowed to rotate. This will be needed for some computer analyses. A free head condition is appropriate for single or isolated pile/shaft foundations and a fixed head for a foundation with multiple rows of piles/shafts. This assumption can be revised if necessary during the design process. This approach will likely reflect the final design conditions.

Block 8 – Estimate Pile Size and Depth and Determine Subsurface Conditions for Lateral Loading Analysis

Based on the selected foundation type and layout and the characterization of subsurface conditions, develop an estimate of the pile size and depth and determine the subsurface profile for lateral load analysis. The pile size includes an estimate of the minimum pile width or diameter. A minimum depth for analysis should be determined based on considerations regarding the subsurface profile for each load case. In many cases, the estimate of pile size and depth will be an initial trail size that is analyzed and refined, either increased or decreased, until a suitable design is determined that satisfies the project performance criteria.

A specific subsurface profile for analysis should be developed for each lateral load case to be analyzed. This should include considerations for the variability of subsurface stratigraphy and parameters at different foundation locations. Other design considerations that may impact the subsurface profile and parameters should also be considered. These include scour depth, liquefied soil layers during seismic events, additional soil layers (fill) due to proposed construction, compression of soft layers due to proposed fill, etc. The design for laterally loaded deep foundations will often include consideration of scour conditions, which must be assessed for the design flood and the check flood. The design flood is typically a 100-year flood and is considered under the strength and service limit conditions. The check flood is typically a larger interval, up to a 500-year interval, and is considered under the Extreme Event Limit State. Consideration of scour events may alter the geotechnical subsurface profile used in analyses rather than the actual loads, resistances, or applied load or resistance factors.

Block 9 – Perform Analyses for Laterally Loaded Single Pile

The next step in the design process involves analyzing the preliminary foundation type, size and length selected previously for Geotechnical Strength, Structural Strength, Service, and Extreme Event Limit States. To perform this step, the designer computes deflection and rotation at the head of a deep foundation element and the maximum bending moment and shear force within the deep foundation element. The designer also determines the nominal bending moment resistance of the deep foundation element (this is the moment at which a plastic hinge will develop in the foundation). These analyses are usually performed with the use of software programs. The details of various methods for performing these analyses are discussed in Chapter 6 for individual lateral pile analyses, Chapter 7 for pile group analyses, and Chapter 11 for structural analyses.

This step must include consideration of the axial loads. The axial loads will influence the maximum bending moment capacity as well as the amount of deflection that occurs under lateral loading.

Typically, the designer selects the pile/shaft length and performs analysis on a trial-and-error basis. Some computer programs can perform this step internally and produce a summary of pile lengths versus deflection (or other criteria) from which a minimum embedment length can be determined. This iterative procedure is repeated for different size piles or shafts (diameter, width, or pile section) to develop the most appropriate design. However, the designer will need to determine if a greater pile/shaft penetration is needed to satisfy axial load demand and foundation settlement criteria.

For drilled shafts or CFAs, a trial longitudinal reinforcement must be selected for analysis. A typical initial value is a longitudinal reinforcement of about 1 percent of the overall shaft cross sectional area (Brown et al. 2010; Brown et al. 2007). Engineering judgment and experience may be helpful in assessing reasonable reinforcement amounts, especially for applications with relatively high lateral loads or bending moments.

Block 9.1 – Analyze Geotechnical Strength Limit State

The Geotechnical Strength Limit State is analyzed for an individual deep foundation element using factored load combinations and applicable resistance factors for Limit States. One load combination may stand out as being the most critical, although it may be necessary to evaluate multiple cases, especially if there is no clearly distinct critical case by inspection or if different loading conditions correspond to different subsurface profiles, such as scour or extreme event conditions. Loads used in analyses must include both the factored lateral and axial loads. Several methods of analyses are described in detail in Chapter 6.

Block 9.2 – Analyze Structural Strength Limit State

The complete structural design of a deep foundation must consider combined axial load, shear, and bending. The nominal axial, shear, and flexural resistance of a foundation element cross section must exceed the factored axial load, shear, and bending moments. Based on the results of the Geotechnical Strength Limit State analysis, the trial foundation size and depth should be analyzed to verify that the structural section is adequate for the factored load cases. The structural design includes the following considerations (Brown et al. 2010):

1. Factored loads are used to determine axial load, shear, and bending moment.
2. The nominal structural strength of the foundation element is determined and must exceed the combined factored axial forces and bending moments for each load combination or for the critical load combination.
3. The nominal shear resistance of the foundation element is determined and compared to the factored shear forces.

If the factored axial load, factored maximum bending moments, and/or factored shear forces exceed the nominal structural resistance of the foundation elements, then the foundation design must be modified. This may include adding additional reinforcement in the case of concrete elements, using a heavier steel section in the case of steel piles, and/or increasing the size (diameter or exterior dimensions) of the foundation elements. If the size of the foundation is increased, the Geotechnical Strength Limit State should be reviewed and re-analyzed, if necessary, to see if the length can be reduced based on the increased size. This may therefore be an iterative process.

The structural design of foundations for lateral loading is described in detail in Chapter 11.

Block 9.3 – Analyze Service Limit State

Service load combinations, with applicable load and resistance factors as discussed in Chapter 4, are evaluated to check that deflections are within tolerable limits. Although for preliminary assessments, a rule of thumb deflection may be acceptable, the final design should include evaluation of the actual tolerable deformation for the serviceability of the structure. Service Limit State deflections should include potential for amplification of or additional deflection above the top of the foundation element, such as for pier columns or at the top of a cantilevered retaining wall. In some cases, the Service Limit State deflections may be set based more on aesthetics than on actual serviceability of the structure, such as noise walls, sign posts, or cantilevered retaining walls.

The Service Limit State deflections will be a function of both the ground response as well as the structural stiffness of the foundation element. Therefore, an analysis method that accounts for soil-structure interaction, such as the p-y method, must be used in this step. This analysis can often be performed using the same software and the same models that were used for the Strength Limit State analyses, however, the inputs must be adjusted to use the applicable load and resistance factors for the Service Limit States, and the criteria for evaluating the results must be based on Service Limit States. If the same models are used for both Limit States, the analyses results should clearly distinguish the service limit analyses from the strength limit analyses to avoid confusion in the project reporting.

Block 9.4 – Analyze Extreme Event Limit State

Extreme Event Limit State loading, discussed in Chapter 8, should also be analyzed as applicable. A resistance factor of 0.80, as discussed in Chapter 4, will apply.

Extreme Event Limit States involve events with a low probability of occurrence. Such events are considered to be unique and their return period may be significantly greater than the design life of the structure (AASHTO 2014). The Extreme Event Limit State is intended to ensure the survivability of the structure during such an event. This Limit State is to protect against collapse of the structure and loss of life; some damage or loss of functionality may be acceptable in such a case.

Extreme Event Limit States for design of deep foundations include the following:

1. The check flood for scour
2. Earthquakes/seismic events
3. Loading from ice
4. Vessel collision
5. Vehicle collision

Block 10 – Perform Axial Design

The minimum design length and size for axial capacity must be determined. The minimum pile/shaft penetration will be the greater of:

1. The penetration required for axial design,
2. The penetration required for lateral loads, and
3. The penetration required to meet settlement criteria.

Typically, approximate analysis is initially done to estimate the pile/shaft size for lateral loading. This element size is then used for axial load analysis, with depth increases as needed for axial resistance. In many cases, an iterative process may be needed to arrive at a final design that satisfies both lateral and axial loading criteria.

Axial foundation design is not addressed in this manual. For axial capacity considerations and design guidelines refer to Hannigan et al. (2016), Brown et al. (2010), Brown et al., (2007), and Sabatini et al. (2005) for driven piles, drilled shafts, CFA piles, and micropiles, respectively, as well as the current version of the AASHTO LRFD Bridge design specifications.

Block 11 – Perform Group Analysis of Laterally Loaded Deep Foundations

For designs that include groups of piles/shafts, the analysis of the group must be taken into account because of overlapping zones of influence of the foundation elements and the influence of frame action on the load distribution within the group. In many cases, the interaction of foundation elements can be accounted for by including appropriate reduction factors in the analysis of a single pile (p-multipliers in p-y analyses, as discussed in Chapter 7). However, for complex foundations or for complex soil–structure interaction, an analysis of the pile/shaft group lateral resistance and deflections should be performed. This is most often done using computer software as discussed in Chapter 7. Similar to the steps outlined above, this should be checked for Geotechnical and Structural Strength, Service, and Extreme Limit States. Results of the group analysis may result in revisions to the pile size, depth, or layout, and therefore may require iteration with the previous steps.

Block 12 – Perform Final Structural Design of Foundation Elements and Connections to Caps

The structural design is finalized once the lateral and axial geotechnical designs have been fully developed. The final structural design is addressed in Chapter 11 and includes:

- Determination of the force effects on the foundations using the appropriate foundation model, structural model, load combinations, and load and resistance factors.
- Determination of dimensions of the foundation elements to resist the force effects and resulting behaviors (such as deflections). The dimensions determined from the Geotechnical Limit State analyses can be used as a starting point and increased if needed.
- Check of the structural design for axial loading, shear, and moment resistance, including evaluation of the material strengths (steel, concrete, grout, etc.), structural section capacity, amount of reinforcement, etc.
- For concrete foundation elements, performance of design and distribution of reinforcing as applicable, including both longitudinal and transverse reinforcing. This should include a check on spacing of reinforcement for constructability considerations such as flow of concrete through openings in the reinforcing cage.
- Design of connections to caps or structures, as applicable. This will include design of the embedment and/or connections, and will include evaluation of the performance of the overall foundation system to ensure that the connections are adequate and consistent with head fixity used in design analyses.
- Addition of iterations, as needed, among the steps above (and other steps such as geotechnical analyses or constructability review) to finalize the structural design of the individual element.

Block 13 – Perform Constructability Review

The proposed design should be reviewed with considerations for possible means and methods of construction to verify that the design is constructible. Elements of construction that may pose risks should be identified and mitigated if possible. Example mitigation measures may include a required or anticipated sequence of construction, precluding particular means and methods of construction, requiring minimum qualifications or pre-qualifying contractors (if allowed by procurement procedures), providing minimum opening between drilled shaft reinforcement for concrete flow, or requiring certain inspection and testing procedures. Chapter 13 further discusses construction considerations.

Block 14 - Develop Construction Documents

As the design is finalized, construction plans and specifications are prepared. In many cases, progress plans will be developed at earlier stages of the design process and periodically updated as the design progresses. The plans and specifications must clearly communicate the design to the bidders, including requirements for minimum tip elevations or penetration depths for the foundation elements, installation procedures, testing requirements, and quality control procedures.

Requirements for testing should also be clearly communicated in the bid documents. If lateral load testing is required, then requirements should include a load test location, minimum design requirements for lateral load test setup, procedures for testing, definition of acceptance criteria, and reporting requirements. Lateral load tests are discussed in more detail in Chapter 12.

Block 15 – Develop Construction Cost Estimate

A construction cost estimate should be developed for the design that includes all relevant considerations such as likely construction issues or difficulties, specialty construction requirements, risks, testing requirements, etc. Cost estimates may be developed at multiple stages of a project, such as preliminary design, design development, and final design, similar to the development of in-progress and final plans. The cost estimate may include estimates of more than one alternative design.

Block 16 – Design Considerations and Changes in Construction

Construction includes award of the contract, submittal of required documents, installation or construction of the foundations, construction inspection and testing, and construction reporting. Potential impacts, changes or considerations regarding the design for lateral loads are discussed in Chapter 13.

Design changes during construction could have a major impact to project cost and schedule, and should be avoided by giving greater attention to site variability, design risks and constructability issues during the design process.

Block 17 – Post-Construction Reporting

Final reporting for construction of deep foundations often includes compiling the installation records, testing results, and documentation of any construction changes for the project files. Input for as-built records often includes locations of final pile/shaft location, cut-off and tip elevations, and changes in foundation element type or size recorded on the record plans. This documentation is typical for all deep foundation projects, and are not requirements just for laterally loaded deep foundation projects.

Full documentation and publication of data at load test sites are recommended. Such data are needed for use in future similar projects and in the reliability calibration to develop quality resistance factors. See Chapter 12 for further discussion.

Additional Considerations regarding Design-Build Projects

The process above includes discussion in various steps with regard to how the process may differ for design-build projects. In a design-build project, the overall process is generally the same; however, the roles may differ. For example, the agency or a consultant working for the owner agency may perform site exploration, site characterization, conceptual selection of foundation type and preliminary design, establish minimum performance criteria and minimum foundation testing requirements, and develop requirements for completing the design and construction. The design-build contractor's design consultant, as the final designer or engineer of record, will develop the final design, including performing additional investigations, design analyses, and testing.

The actual tasks performed by each party will vary by project. Some design-build projects will be procured after performing the tasks in Blocks 1 through 4 (possibly with only a limited subsurface investigation), with the design-build contractor responsible for determining the foundation type and developing the full foundation design. Less frequently, other projects will include a relatively detailed preliminary design that requires progressing through to Block 14, with overlapping procedures by the design-build contractor who would perform the tasks in Blocks 4 through 14 again as part of finalizing the design. The roles for the procurement team and the contractor are shown conceptually in Figure 5-1, with the potential overlap for design-build project roles conceptually indicated.

5.2 DESIGN TEAM ROLES

The design team for deep foundations consists primarily of the geotechnical and structural engineers. Additional team members include hydraulic and civil engineers, architects, or other disciplines who may provide secondary input with regard to the design process for laterally loaded deep foundations.

Because the analysis of laterally loaded deep foundations involves soil-structure interaction, it requires coordination of the combined expertise of the geotechnical and structural engineers. It is not appropriate for the geotechnical engineer to analyze laterally loaded deep foundations without a complete understanding of the structural response of the foundation or the connection of the foundation to the structure. Similarly, it is not appropriate for the structural engineer to analyze laterally loaded deep foundations without a complete understanding of how the foundation section, depth, or spacing may impact the geotechnical response under the applied loads, or how external system loads, such as seismic loads, may affect the geotechnical parameters or the surrounding soils. Therefore, a proper design of laterally loaded deep foundations must involve adequate coordination between the geotechnical and structural engineers. Often this may require an iterative process between the two disciplines, rather than simply one discipline providing input, parameters, or a check of the results of the other discipline.

Typical roles for the design professionals are discussed below. These roles are based on general industry practice, including practice by some agencies. However, the roles on a specific project may vary due to local practice, contractual arrangements, or other considerations.

5.2.1 Geotechnical Responsibilities in Lateral Loading Analysis

The geotechnical engineer has primary responsibility for tasks relating to the subsurface conditions. The geotechnical engineer has secondary responsibility for tasks relating to the structural design. Primary responsibility for some tasks may be allocated to either the geotechnical engineer or the structural engineer, depending on the project type, team roles, experience or qualifications of individual team members, contractual arrangements, or other considerations.

The tasks relating to the subsurface conditions include defining the preliminary project geotechnical conditions, planning and executing the subsurface investigation and testing program, characterizing subsurface conditions, and development of resistance factors (which are based in part on considerations regarding the characterization of subsurface conditions). The geotechnical engineer is also primarily responsible for determining external geotechnical loads such as loads due to external soil loads (retained earth pressure), slope movement, and possible change to soil conditions (liquefaction, lateral flow or lateral spreading). In some cases, the geotechnical engineer may simply provide the required input parameters (unit weight, earth pressure coefficient, etc.) for the structural engineer to develop the loads. Development of the loads also includes axial loads related to geotechnical conditions, such as downdrag, as well as assessment of load effects, such as ground settlement at the deep foundations (individual or group). Development of geotechnical design for axial loads, assessment of constructability issues, and development of construction inspection and testing requirements will also be the primary responsibility of the geotechnical engineer.

Responsibilities for other tasks may vary depending on project type. For projects that are geotechnical in nature or where the loads are primarily geotechnical, such as slope stabilization or cantilevered retaining walls, the geotechnical engineer may be primarily responsible for additional analysis and design tasks. Similarly, if projects include unusual or challenging geotechnical conditions or require specialty geotechnical construction, the geotechnical engineer may have primary responsibility definition of the project type and needs, selection of the deep foundation type and preliminary size, determination of initial foundation layout and head fixity conditions, and determination of performance criteria.

Analysis of a single pile/shaft may be the primary responsibility of the geotechnical engineer or the structural engineer. The geotechnical engineer should have primary responsibility if the analysis involves Broms method or strain wedge theory. The soil-structure interaction analysis (p-y method) involves both geotechnical and structural inputs and considerations. In some cases, these analyses are performed by the geotechnical engineer; the structural engineer then reviews the results for reasonableness, and/or may perform the Structural Strength Limit State analysis. In other cases, the geotechnical engineer develops the geotechnical inputs, such as the subsurface profile for analysis and geotechnical parameters for soil and rock layers, including p-y curves or input parameters for development of p-y curves; the structural engineer then performs the analyses for a single pile, including the Geotechnical Strength and Service Limit State analyses. Similarly, analysis of group behavior involves specialty computer software that includes both geotechnical and structural inputs and considerations, and may be performed by either the geotechnical or structural engineer depending on the project roles.

The geotechnical engineer will likely also have a lead role in the constructability review and development of construction documents. The geotechnical engineer should lead the development of specifications for construction or installation of foundation elements and construction inspection and testing related to foundation elements. The geotechnicap-yl engineer may also lead the development of plans or details for geotechnical construction such as slope stabilization of specialty foundation details such as micropiles or CFA piles. For other plans, details, or specifications that are primarily structural, such as pile layout for bridge foundations or structural details for drilled shafts, the geotechnical engineer should have secondary responsibility (review of work developed by others disciplines).

The geotechnical engineer should be responsible for aspects of construction related to geotechnical conditions or construction, such as review of submittals by specialty geotechnical contractors (installation plans, equipment, etc.), oversight or review of foundation inspection and testing, and review and approval of foundation installation records. For projects that are geotechnical in nature, such as slope stabilization, the geotechnical engineer should lead the preparation of as-built records. For most other projects, the geotechnical engineer will generally have a secondary responsibility for development of construction records or as-built drawings, such as providing input or review.

5.2.2 Structural Responsibilities in Lateral Loading Analysis

Most transportation projects, such as bridges, retaining walls, noise walls, signs, and light poles, are primarily structural design projects and are therefore led by the structural engineer. The structural engineer has primary responsibility for tasks relating to the structural design. These generally include definition of project type and needs, determination of the foundation layout and head fixity conditions, determination of performance criteria (allowable deflection at the pile/shaft head and shear and moment capacity), development of applied loads (structural loads and reactions at the top of the foundation elements), analysis of the Structural Strength Limit State, preliminary and final structural design of foundation elements and connections to caps, development of construction documents, and development of as-built records. As discussed above, the structural engineer may also have primary responsibility of the design of an individual foundation element and group analysis, including Geotechnical Strength and Service Limit States, using the geotechnical inputs provided by the geotechnical engineer. The structural engineer should also participate in the constructability review.

As discussed in Section 5.3.1, for projects that are primarily geotechnical in nature or that involve specialty geotechnical construction or unusual geotechnical conditions, the geotechnical engineer may take the lead role for some of these aspects of the design process with the structural engineer providing a secondary or support role. For these types of projects, the structural engineer may review the development of the loads, structural design of the foundation elements and connections, and review foundation layouts and foundation specifications, as well as perform review of construction submittals relating to structural design (foundation test apparatus set-up, design of shoring systems, etc.).

5.2.3 *Other Team Members*

Other team members include hydraulic engineers and civil engineers, architects, construction engineers, or other specialty disciplines. Hydraulic engineers will determine loads due to hydraulic forces, such as currents and wave impacts, as well as flood elevations and depths of scour for design. Civil engineers will determine site grading which may impact loading conditions, such as cuts or fill that may result in unbalanced earth pressures (resulting in additional lateral loads), or fill that may result in settlement and downdrag loads. Construction engineers will be the project representative during the actual construction. Construction engineers will be responsible for ensuring that the foundations are installed in accordance with the plans and specifications, performing coordination with the contractor regarding construction operations, and will alert the designer if there is a difference in site conditions or the as-built foundation that potentially impacts the design. Other design considerations that may be primarily determined by other design professionals, such as architects or other engineering disciplines, may include cost estimating, constructability reviews, determination of noise or vibration thresholds limits, or determination of space restrictions for structure or foundation layout. Other professionals may have the lead responsibility for determining the availability of local contractors or materials (which may impact the selection of type of deep foundations); whether there is a preferred method of procurement (design-build vs design-bid-build and whether it may impact the type of foundation used); whether the project is an emergency response (which may impact the procurement or the type of deep foundation); or other considerations that may impact the design tasks outlined in this chapter.

5.3 NEED FOR COMMUNICATION AND COORDINATION

As discussed in Sections 5.3.1, 5.3.2, and 5.3.3, there is a need for communication and coordination between team members for the design of deep foundations. Design requirements, such as tolerable deflections, loading conditions, subsurface conditions, or other inputs, must be determined and communicated between team members to avoid using assumed or overly conservative values or to avoid double application of load and resistance factors. Clear communication should include an indication of whether loads and resistances are factored or unfactored/nominal, as well as what load or resistance factors should be applied in analysis, as well as other factors such as group factors. Individual pile/shaft analyses and group analyses performed by one discipline (geotechnical or structural) should be reviewed by the other to confirm that the design is adequate and that there are not missed opportunities for improvement. Coordination is also needed when developing construction plans, specifications, testing requirements, and performing constructability reviews, especially for more complex projects.

It is often the case that there is conservatism included in the design parameters or inputs developed at early or preliminary stages. This could be conservatism in the allowable deflection limit, the geotechnical input parameters, the characterization of subsurface conditions, the size and/or type of foundation selected for design, or the magnitude of the design loads. The intent of such conservatism is often to ensure that additional refinements do not result in increases in the size, depth, or number of foundation elements, and therefore result in increases in the overall project cost; i.e., the intent is that as the design is refined, the preliminary design will be verified and the potential to optimize the design and reduce costs may arise.

Too often, the respective disciplines do not coordinate, choosing instead to provide criteria or parameters that are conservative but reliable from the standpoint of the individual designer. When the design is complete, the initial criteria are often not re-assessed to see if a more efficient and economical design can be realized.

Without communication and coordination between the disciplines, in particular the structural and geotechnical disciplines, there may be lost opportunity to improve the economics of the design. For example, IGM materials may be characterized conservatively as stiff or dense soils, which may be acceptable for some foundations but may result in excessive depths for other applications. Without the opportunity to review the analysis results, the geotechnical engineer may not be able to assess how his initial characterization impacted the overall design.

Similarly, an initial limiting deflection may be provided by the structural engineer; during analysis, the geotechnical engineer may find that a heavy pile section is needed, although a lighter and more economical section could be used with a slightly greater deflection. The geotechnical engineer should communicate this to the structural engineer to see if the initial deflection criteria can be relaxed to allow the more economical design.

Similar communication with other disciplines and stakeholders may help avoid other design issues or produce more economical designs. The use of specialty foundations, such as micropiles, may be more appropriate or economical in some cases, such as avoiding costs associated with mobilizing and creating access for large cranes for piles driving in sites with limited access. As noted in Section 5.2, geotechnical and structural engineers should coordinate in the development of drawings and specifications, and should be included in the constructability review. A well-coordinated project with clear communication between respective disciplines will ensure that an economical design is developed that meets all performance requirements.

6 ANALYSIS FOR LATERALLY LOADED SINGLE DEEP FOUNDATION ELEMENTS

This chapter presents methods for analyzing single laterally loaded deep foundation elements, including simplified methods and more complex methods requiring computer software.

6.1 INTRODUCTION

The interaction between a vertical deep foundation element and surrounding soil when the foundation element is subjected to a lateral load, P_t, applied at the top is schematically illustrated in Figure 6-1. Horizontal pressures are uniform around the foundation element at a depth, Z_1, before lateral loads are applied. After the load is applied, the foundation element deflects away from the load with a magnitude, y_1, at depth Z_1, and the existing soil pressure distribution is modified. In the upper part of the foundation element, pressures in front of the foundation element increase as shown in Figure 6-1c. If the applied load is large enough, the pressures may approach the passive Limit State. Conversely, pressures on the back of the foundation element decrease and may approach the active Limit State, or may even approach zero if the element moves away from the soil. The stress distribution may reverse its sign in the lower portion of the foundation element. The foundation element will tend to rotate if it is relatively short and stiff, resulting in a condition close to passive wedge failure, whereas if the foundation element is relatively long, it will deflect and bend but will not deflect enough to reach the passive state (see Error! Reference source not found.). Shear stresses, not shown in Figure 6-1, are also mobilized on the sides of the deep foundation element. The net effect is a soil resistance, p_1, corresponding to a deflection, y_1, at depth Z_1 as shown in Figure 6-1c. The relationship between soil resistance, p, and displacement, y, is referred to as the "p-y" relationship. The p-y relationship changes with depth, soil type and other factors (McClelland and Focht 1958; Reese and Matlock 1956). The p-y concept captures most aspects of the soil-foundation interaction under lateral loading.

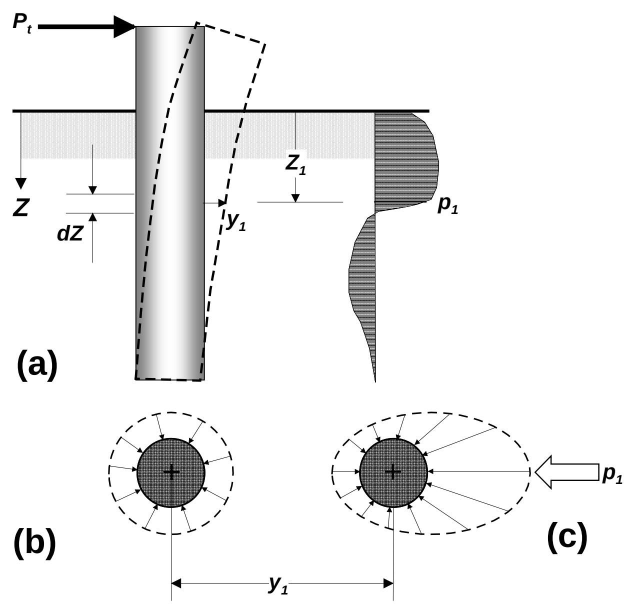


Figure 6-1: Vertical deep foundation element subjected to lateral loading: (a) elevation view; (b) plan view of earth pressure distribution at depth Z_1 prior to lateral load application; and (c) plan view of earth pressure distribution at depth Z_1 under applied lateral load.

The problem of loaded elastic beams resting on an elastic medium (Hetenyi 1946; Vesic 1961) was the original concept used in the solution of laterally loaded deep foundation elements. The classical problem of the elastic beam can be "rotated" to consider the case of a deep foundation. In this case, the pile/shaft reaction *p* is considered as the distributed load of the beam. Applying the concept of subgrade reaction, $p = k\,y$, in which *k* is a subgrade reaction modulus and *y* is the deflection, solutions can be found for various boundary and load conditions for the elastic problem (Terzaghi 1955). Solutions of the elastic problem are overly simplified and unreliable to be used in practice, but the concept provides the framework on which many analytical methods are based.

There are a number of available methods for analyzing laterally loaded deep foundation elements. The most widely accepted methods are the p-y method, which is a computer based solution based on p-y behavior, and the strain wedge model (SWM), which is another computer based solution based on a limit-equilibrium approach and the Broms method for short piles. As discussed in this chapter, the p-y method is recommended for analysis.

All loads used in analysis should be factored loads, with applicable load factors for the Strength, Service, and Extreme Limit States (refer to Chapter 4).

6.2 GEOTECHNICAL STRENGTH LIMIT STATE OF LATERALLY LOADED PILES

Typically, laterally loaded deep foundation elements are deep enough such that they will bend and deflect rather than rotate. If the foundation element is not sufficiently deep, then geotechnical failure of the soil material can occur (geotechnical strength is exceeded) and the foundation element behaves as a rigid element that rotates about a point at or near the bottom of the element; this is referred to as short pile behavior or "push-over" or "fence-posting" and is illustrated in Figure 6-2a. If the foundation element is deep enough to avoid push-over failure, then its tip will remain essentially fixed and the element will bend and deflect in a ductile manner under applied lateral load, referred to as long pile behavior and illustrated in Figure 6-2b. In this case, the geotechnical strength is not exceeded. This is the general behavior for the vast majority of applications and is controlled by limiting deflections and/or structural resistance of the foundation element. The intent of a Geotechnical Strength Limit State analysis is to verify that the strength limit state of the foundation material exceeds the structural resistance of the pile in flexure, and therefore the foundation will behave in a ductile manner and push-over failure will not occur. For these types of deep foundations, the Geotechnical Strength Limit State can be performed by using a p-y analyses and performing a push-over type of analysis (the p-y method of analysis is described in the subsequent section).

For a push-over analysis, the factored axial and lateral loads are used in computer analyses. Loads are factored according to the Strength Limit States as discussed in Chapter 4. P-y curves developed for the soils or rock at the site are used to model the geotechnical resistance and soil-structure interaction. For piles in groups, p-multipliers, which account for overlapping zones of influence of the piles, must be included (analyses of groups of deep foundations is discussed in detail in Chapter 7). Note that p-multipliers, which are less than 1.0, are used to address overlapping zones of geotechnical resistance and should not be confused with or take the place of resistance factors. Appropriate resistance factors are incorporated into the analyses based on the project design criteria and considerations, as discussed in Chapter 4.

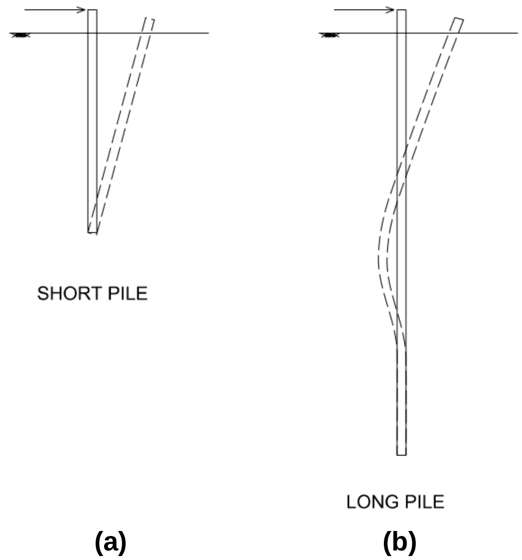


Figure 6-2: Illustration of short pile behavior and long pile behavior under lateral loads.

For a push-over type analysis, the following analysis procedure can be followed (Brown et al. 2010):

1. The deep foundation is modeled as a simple linear elastic beam with the elastic modulus equal to that of the foundation material (concrete or steel). The moment of inertia is based on an equivalent uncracked section in the case of concrete.
2. The soil profile is developed. Separate profiles are used as appropriate depending on site variability and design conditions (scour, liquefaction, etc.).
3. The load at the head of the foundation element is applied in multiples up to and exceeding the factored design load to compute deflections. An unstable condition will result in the computer program being unable to converge or converging at extremely large deflection values.
   a. The computed deflection should be a reasonable value at and slightly above the factored design loads to ensure that geotechnical strength requirements are satisfied. Some judgment is needed to assess the reasonableness of the computed deflection. For drilled shafts, a value of 10 percent of the diameter is suggested by Brown et al. (2010).
   b. Local practice or agency requirements may also define the limiting deflection. For example, as indicated in Chapter 4, the Idaho Transportation Department (2008) defines the Geotechnical Strength Limit State for seat type abutments as a deflection of 2 inches in combination with a resistance factor of 0.9.
4. The geotechnical strength, as evidenced by a reasonable deflection at or exceeding the factored design load, must be adequate to ensure that a ductile lateral foundation response exists; i.e., that the foundation element does not fail through push-over.
   a. The geotechnical strength can be increased, and the deflection decreased, by increasing the length of the foundation element and/or its size (diameter or width) to engage more geotechnical resistance. Once the initial model parameters are established (loads, soil profile, etc.), repeated trials of different lengths and shaft diameters can be performed quickly to determine a minimum length and diameter that satisfies the criteria. For most laterally loaded deep foundations, there is a point at which an additional increase in depth does not result in an additional decrease in lateral head deflection; this is often referred to as the critical depth or critical penetration concept and is

illustrated in Figure 6-3 (Reese 1986). If the foundation element being analyzed has reached the critical depth under the applied loads and has not satisfied the deflection criteria, then the diameter or width of the element must be increased, or additional foundation elements can be provided to reduce the load demand, or a different type of element must be considered.

b. Site variability, both in terms of the profile used and the design parameters, should be considered in assessing the geotechnical strength. For relatively variable conditions, adequate or reserve geotechnical resistance may be desirable when assessing the Geotechnical Strength Limit State.

c. Note that in assessing the Geotechnical Strength Limit State, “failure” is not necessarily geotechnical failure of the soil, rather it is defined by a deflection limit for consistency and applicability of the p-y approach. The analysis must converge at a reasonable deflection limit to represent a stable condition.

5. The factored loads may exceed the strength of the trial foundation element. This is not an issue though as the intent of this Geotechnical Strength Limit State analysis is to verify that the embedment is adequate for the pile to behave as a ductile element rather than to fail through rotation (geotechnical failure of the soil). It is for this reason that the Geotechnical Strength Limit State is modeled with the foundation element as a linearly elastic beam. The Structural Strength Limit State is addressed as a separate section.

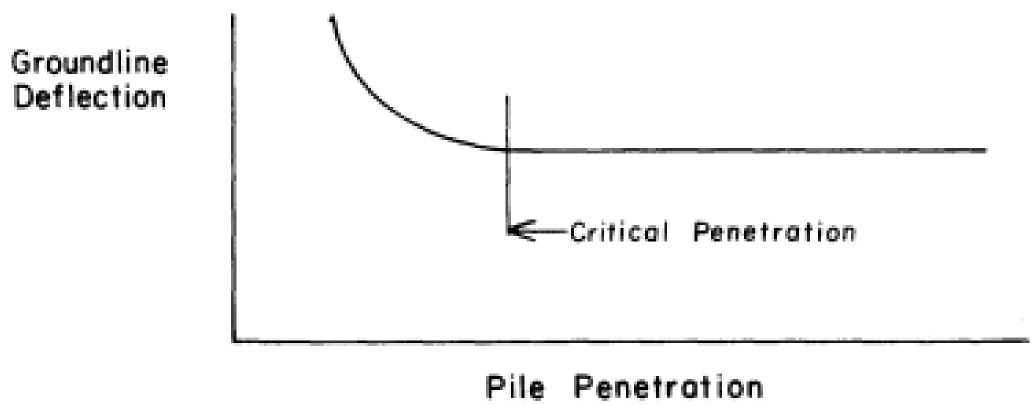


Figure 6-3: Illustration of the critical depth concept for deep foundations (from Reese 1986).

Local practice may vary from the procedure described above or local agencies may require a different approach. Designers should ensure that their analyses procedures comply with applicable design standards including local agency guidelines or requirements, especially as the state of the practice may continue to evolve based on research or local experience.

For larger diameter, relatively short foundation elements such as short drilled shafts or relatively short and stiff piles, the foundation element may rotate rather than bend, i.e., develop a “fence-posting” or “push-over” failure. For such cases, the p-y analysis will not converge at a reasonable depth and error messages may result; this is a result of the pile not being long enough to behave as a long pile, and indicates that the pile is more likely to fail through rotation rather than bending. For these types of foundations, the Geotechnical Strength Limit State can be analyzed using theoretical analyses such as Strain Wedge Model method (SWM) to verify that the foundation materials have adequate strength to support the factored loads. For multiple deep foundations with potentially overlapping zones of influence, the effect of the overlap must be accounted for in the analysis; p-multipliers for p-y analyses are specific to that method and are not applicable to Broms method or SWM. The Broms method can be used as well for short pile behavior analysis of the Strength Limit State, but the Broms method is typically only used for simple structures (such as sign post or light pole foundations) or for preliminary analyses.

For non-gravity cantilevered walls, analyses should be performed as described in Chapter 9.

6.3 P-Y METHOD

The analysis of a laterally loaded deep foundation element is a complex problem that involves nonlinear reactions of the foundation element and nonlinear reactions of the surrounding geomaterial. The most common nonlinear analysis method used in practice is the p-y method. This method captures the essential mechanisms of the problem, has a wide industry acceptance, a history of use in the transportation industry, and can be performed with readily available commercial software. The p-y method is the recommended method for design of all major deep foundation projects.

The p-y method can accommodate variable subsurface layers, axial loads, lateral deflections as inputs, distributed loads, sloping ground conditions, fixed or free head conditions, nonlinear bending characteristics (such as cracked sections in drilled shafts), and non-linear soil response. The outputs of the p-y method include distributions versus depth of lateral displacement, shear forces, bending moments, soil resistance, and soil and pile moduli. These distributions can be tabular or graphical. The output information also allows for an analysis of the foundation’s structural resistance. An illustration of conceptual p-y analysis graphical results is provided in Figure 6-4.

The p-y method was originally developed under an FHWA research grant and was developed into the DOS-based computer program COM624. COM624 has been discontinued, but has been succeeded by other commercial software programs for p-y analyses. Any commercially available software program that properly performs the p-y method should be adequate for analysis.

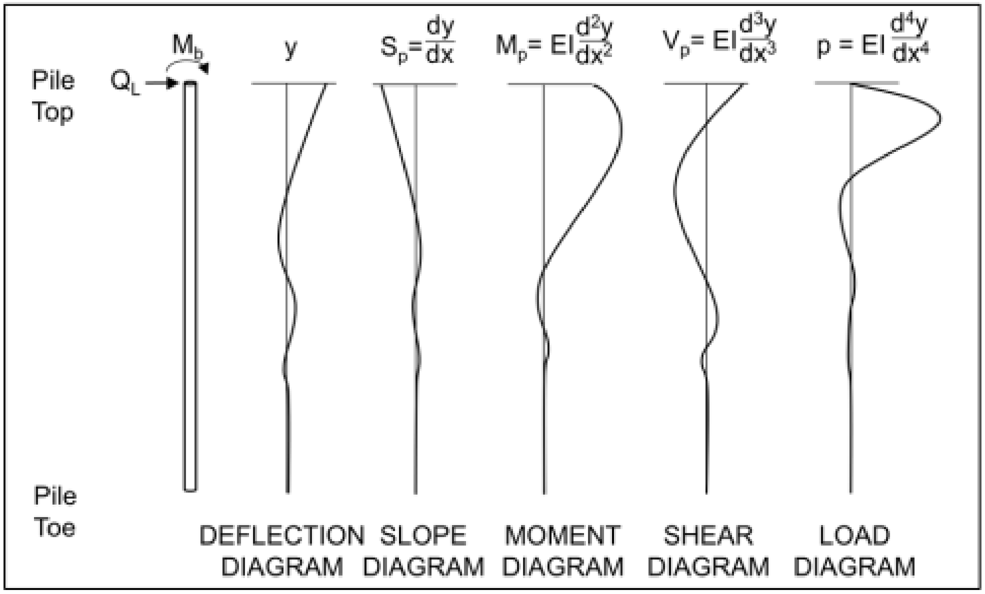


Figure 6-4: Illustration of graphical p-y analysis results (from Hannigan et al. 2016, modified from Reese 1986).

The p-y method is a method that has been developed based on the extension of the elastic solution and the subgrade reaction concept. The deep foundation element is treated as a beam-column with lateral support, following the approach described previously of an elastic beam rotated on an elastic foundation. The general behavior of the foundation element under lateral and axial loading can be obtained by solving the fourth-order differential equation (Hetenyi 1946):

$$E_p I_p \frac{d^4 y}{dx^4} + P_x \frac{d^2 y}{dx^2} - p - w = 0$$

(Equation 6-1)

Where:

- P_x = Axial load in the pile/shaft.
- y = Lateral deflection of the pile/shaft.
- x = Depth along the pile/shaft.
- E_p = Elastic modulus of the pile/shaft.
- I_p = Moment of inertia of the pile/shaft.
- p = Lateral soil reaction per unit length of the pile/shaft.
- w = Distributed load along the length of the shaft, if applicable.

It can be seen from Equation 6-1 that the axial load influences the bending moments and lateral deflections of a laterally loaded deep foundation. Other beam formulae in the analyses include:

$$E_p I_p \frac{d^3 y}{dx^3} = V$$

(Equation 6-2)

$$E_p I_p \frac{d^2 y}{dx^2} = M$$

(Equation 6-3)

$$\frac{dy}{dx} = S$$ (Equation 6-4)

Where:

- V = Transverse shear in the pile/shaft.
- M = Bending moment in the pile/shaft.
- S = Slope of the deflection diagram.

The soil reaction, *p*, is a function of the deflection, *y*. The relationship between *p* and *y* is nonlinear, and is referred to as the *p-y* curve. The soil-pile interaction is modeled as a nonlinear elastic beam and the soil resistance replaced by a series of discrete, non-linear “springs”, in which:

$$p = E_{py}\, y$$ (Equation 6-5)

Where E_{py} is the soil modulus or reaction modulus, also referred to as the “spring constant” for the soil spring model. The soil modulus is a function of deflection, *y*, and depth, *x*.

The physical model of the p-y method is presented in Figure 6-5. This figure illustrates how the soil around the deep foundation is represented by non-linear springs, and how the p-y relationship varies with displacement, *y*, and depth, *x*.

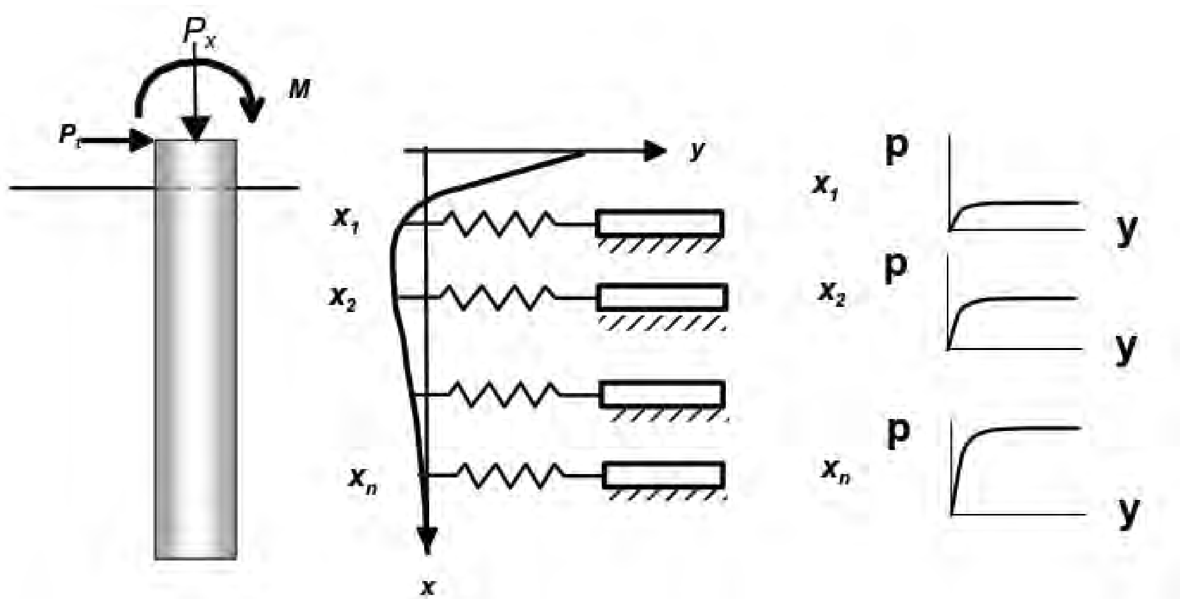


Figure 6-5: Model of a deep foundation element under lateral loading conditions showing concept of nonlinear soil springs and p-y curves.

The software analysis involves the simultaneous solution of a series of equations numerous times. The general process within the software analysis consists of assuming a deflected shape of the deep foundation element (initial deflection), obtaining *k* values from the p-y curves based on the initial deflections, solving the equations to determine a new set of computed deflections, and repeating the process with multiple iterations until the initial deflections and computed deflections are within a specified tolerance. Once the p-y analysis has finished, the bending moment, shear, and slope of the deflection can be calculated from the results using Equations 6-2 through 6-4. Graphical and tabular outputs of the displacement, shear, moment, slope, and soil resistance can be generated from the results of the analysis.

The ability of the analysis above to accurately predict the behavior of a deep foundation element under lateral loading is dependent on the ability to represent the soil response by an appropriate set of p-y curves. The p-y curves available in commercial software programs have been largely empirically developed based on the results of full-scale lateral load tests and experiments. The p-y method is therefore predominantly an empirically based design method rather than a fundamentally theoretical method.

The software analysis for the p-y method is generally easy to use and produces results quickly. This provides the user the opportunity to investigate a large number of variables and their potential impact on the design with relative ease. This can include investigating variations on loading conditions (magnitude, type, and location of loads), subsurface conditions (soil or rock layer depth, thickness, density, strength; and groundwater depth), and geometry (ground slope, foundation embedment, and exposed height of foundation element), among other factors. The analysis of deep foundations using the p-y method should include parametric evaluation of key input parameters in order to assess the potential sensitivity of the results to such inputs. This will help identify which parameters may be most critical and to what degree, which may influence the decision to require additional field investigation and testing, selection of foundation type, construction testing, or other aspects of the design.

Most commercial software programs include a wide variety of p-y curves for various geomaterials, allowing the practitioner to reasonably model real-world conditions. However, there are limitations with applying the available models to conditions that do not match those upon which the models were originally based; the subsurface conditions (soil, rock, and groundwater conditions), foundation characteristics (foundation type, material, shape, and size), and loading conditions (static, cyclical, loading rate) used in the experiments versus the problem condition should be understood to appreciate how applicable the p-y curves are and what inherent risks in the analysis and design may be present as a result. The p-y method can be used to develop site-specific or project-specific p-y curves based on lateral load tests, as discussed in Chapter 12 and Appendix C. New or updated p-y curves are frequently being developed and published, or included in updated software versions, as a result of new research and developments in the industry.

6.3.1 Characteristics of P-y curves

This section presents general descriptions and overview of p-y curves for use in laterally loaded deep foundation analysis. The purpose of this discussion is to provide an understanding of the characteristics of p-y curves and provide some guidance on their appropriate selection and use in practice. More detailed discussion on development of specific curves or on application in design can be found in the references cited and in technical manuals that accompany software programs.

P-y curves are largely empirical, having been primarily developed based on the performance of instrumented lateral load tests on deep foundations in different soil and rock conditions. The p-y curves therefore are based on specific loading conditions, subsurface conditions, and pile type and geometry. These p-y curves are often used for project conditions that differ from the original experiment conditions. It is therefore important that the designer understand the experimental basis for the p-y curves that are used in analysis as well as the limitations of the p-y curves so that inherent risks are appreciated.

P-y curves for a single deep foundation element typically consist of three portions, as shown in Figure 6-6: (i) an initial linear portion (origin to point “a”), representing the almost linear elastic response of soil for small displacements; (ii) a nonlinear transition portion (*ab*), representing soil nonlinear stress-strain behavior; and (iii) a horizontal portion (*bc*), representing the soil ultimate resistance, p_{ult}.

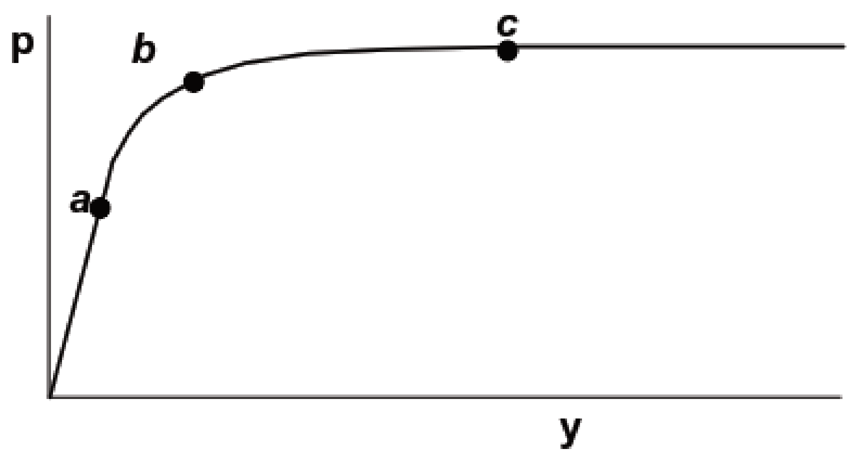


Figure 6-6: Typical p-y curve for a single deep foundation element.

As indicated in Figure 6-5, the soil response to an applied horizontal load, P_t, or moment, M_t, can be represented by a series of discrete nonlinear soil springs located at various distances below the head of the foundation element. The soil modulus, E_{py}, (or spring constant) in Equation 6-5 is the secant modulus of the p-y curve, which represents the reaction modulus of soil. The maximum value of the soil modulus, $E_{py\text{-}max}$, occurs for y = 0 and is proportional to the soil elastic modulus, E_s (Reese and Van Impe 2001). As displacement of the foundation element increases, the secant modulus of the p-y curve decreases nonlinearly. In general, each soil-spring has a particular p-y curve representing the soil-pile interaction at that location. If the soil conditions are relatively uniform, then the shape of the p-y curve is similar for all soil springs along the depth of the foundation element. However, the ultimate resistance of the p-y curve, p_{ult} (i.e., segment bc and beyond in Figure 6-6) tends to increase with depth, as suggested in the lower part of Figure 6-5.

Several factors affect p-y curves including soil properties, foundation material and geometric properties, spring location, and loading characteristics. These factors affect the initial, ultimate, and transition portions of the p-y curves.

The initial portion of p-y curves represents the elastic response of soil to small pile/shaft deflections and is typically characterized by K, which coincides with the maximum reaction modulus $E_{py\ max}$. The parameter K has units of force per square length (F/L^2) and is not a soil property (Terzaghi 1955). In general, K depends on:

i. geomaterial properties, such as strength, stiffness parameters, modulus of subgrade reaction, etc.
ii. depth of the equivalent “spring”
iii. deep foundation element section properties such as diameter, moment of inertia, etc.
iv. subsurface conditions, such as soil above or under the groundwater surface, etc.

Based on work by Skempton (1951), Terzaghi (1955), and McClelland and Focht (1958), Reese et al. (1974) developed the following empirical equation for estimating *K* in sands:

$$K = kz \qquad \text{(Equation 6-6)}$$

where *k* is a proportionality coefficient with units of F/L^3 and *z* is the depth below ground. The coefficient *k* and the conventional subgrade reaction modulus can be related although these parameters are numerically different. While the subgrade reaction modulus is related to the loading of a rectangular plate resting on an elastic horizontal surface, *k* is related to a long “beam” (i.e., pile/shaft) loaded with a horizontal load and exhibiting a different failure mode. Correlations are available for estimating k, and many modern computer programs can estimate k based on soil parameters inputs.

Reese (1997) proposed the following empirical equation for estimating *K* in a rock mass:

$$K = k_i E_m \qquad \text{(Equation 6-7)}$$

Where: E_m = modulus of the rock mass and k_i = dimensionless constant that depends on the drilled shaft diameter and the depth below ground (Turner, 2006).

6.3.2 Factors Affecting P-y Curves

P-y curves are described as a set of expressions to capture the three portions of the curve described above. Most p-y curves were derived from horizontal load tests conducted on full-scale deep foundation elements installed in more or less uniform soil conditions and under a particular loading condition. Some p-y curves are based on in-situ test results. In addition, p-y curves can be derived using the strain wedge model (SWM), which is described in a later section.

P-y curves are largely empirical and are affected by a number of factors, including but not limited to:

- Geomaterial properties
  - Undrained and drained strength
    i. Undrained Shear Strength (S_u)
    ii. Friction Angle (ϕ)
  - Stress-strain behavior
    iii. Poisson’s ratio (v)
    iv. Strain ($\varepsilon 50$)
    v. Subgrade Modulus (k)
    vi. Elastic Modulus (E)
  - Unit weights for soil and rock
  - Rock mass properties
  - Intact rock properties

- Subsurface conditions
  - Depth to groundwater
  - Homogeneous or layered profile
  - Depth to rock
- Size and stiffness of deep foundation elements
- Type of loading (static, dynamic, or cyclical)
- Liquefaction of soil
- Head fixity
- Sloping ground surface

In some cases, several factors will act in combination to affect the p-y curve, such as cyclical loading of cohesive soils. A more detailed discussion and specific examples of p-y curves are presented in Appendix A.

6.3.3 Limitations

A limitation of the p-y method is that it is a one-dimensional solution; it does not directly account for 3-D effects of the loading of the soil mass, although the empirically derived curves from load tests are based on actual three-dimensional field tests. Another significant limitation of the p-y approach is that it is mostly empirical and difficult to adapt to new conditions if empirical data are not available. Soil parameters used in this method are not fundamental parameters; therefore, parameters obtained with conventional laboratory tests, such as triaxial tests, cannot always be linked in a straightforward manner to those that are used in development of p-y curves. In addition, p-y methods are generally more applicable to deep foundations that are relatively long and slender, and as a result, can bend and deflect, i.e., structural failure of the deep foundation element in bending usually controls. The p-y methodology is not fully applicable to short piles/shafts that tend to rotate, where soil failure near the ground surface controls. Also, care must be used for large diameter elements, such as large diameter drilled shafts, because the majority of published curves were developed based on smaller diameter elements.

6.3.4 Recommendations Regarding P-y Method

The p-y method is recommended for use in laterally loaded deep foundation analyses. Although this is an empirical method and there are theoretical shortcomings, the method has been widely and successfully used in a variety of subsurface conditions. There is a high degree of confidence in the results for most typical applications involving the use of deep foundations that are long enough to be governed by deflection rather than rotation. A number of lateral load tests and research projects have been performed to investigate, confirm, and refine p-y curves for various soil and rock conditions (refer to Appendix E). Most agencies and practitioners are therefore familiar with its principles and use, and have confidence in the results obtained using this method.

Summary points regarding the use of the p-y method include:

- Commercial software can be used to perform p-y analyses, as it is generally easy to use and produces results quickly.
- The inputs of the p-y method typically include loading conditions (magnitude, type, and location of loads), subsurface conditions (soil or rock layer depth, thickness, density, strength; and groundwater depth), and geometry (ground slope, foundation embedment, and exposed height of foundation element).

- The outputs of the p-y method include distributions versus depth of lateral displacement, shear forces, bending moments, soil resistance, and soil and pile moduli.
- The analysis of deep foundations using the p-y method should include parametric evaluation of key input parameters to assess the potential sensitivity of the results to such inputs.
- Most commercial software programs include a wide variety of p-y curves for various geomaterials, allowing the practitioner to reasonably model real-world conditions.
- Site-specific or project-specific p-y curves can be developed based on lateral load tests

6.4 STRAIN WEDGE MODEL

The strain wedge model (SWM) (Ashour et al. 1988) was developed to provide a theoretical basis for correlating lateral soil resistance and pile/shaft deflections compared to the predominantly empirical p-y method. The SWM method is an approximate methodology that incorporates the stress and strain responses as well as strength (drained or undrained) of soils. This soil behavior occurs in a hypothetical 3-D passive wedge of soil mobilizing strength behind a single pile/shaft to derive resistance, as illustrated in Figure 6-7. The soil resistance is based on a limit equilibrium solution of passive earth pressure within the 3-D wedge.

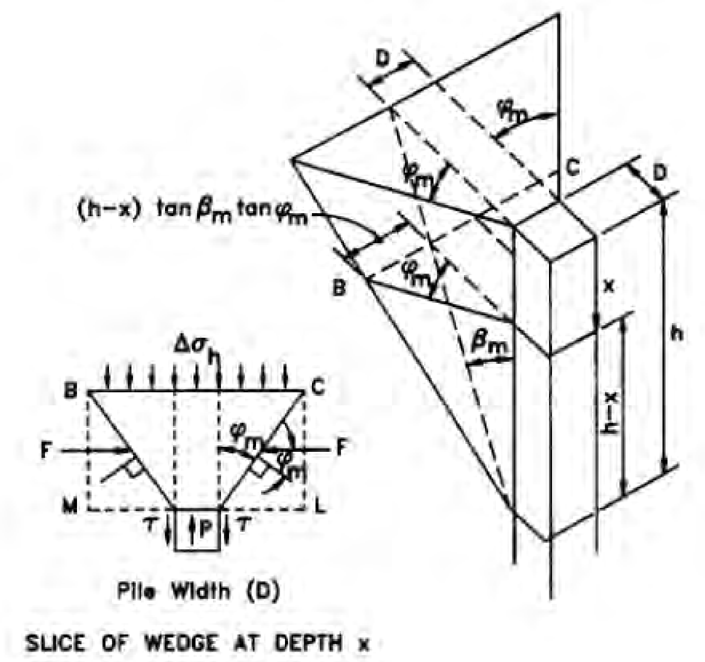


Figure 6-7: Basic wedge in uniform soil for SWM (from Ashour et al. 1988; refer to reference for details on nomenclature and derivation).

Similar to the p-y method, this method must be analyzed using computer software. Multiple programs capable of performing SWM are available commercially. The software inputs and design process for the SWM are similar to those for the p-y method, although the way in which the soil resistance is computed is different from the p-y method. The SWM method is briefly described herein. Additional information regarding the use of SWM can be found in Norris and Abdollaholiaee (1985), Gowda (1991), Ashour et al. (1996), Ashour et al. (1998), Pilling et al. (2001), Ashour (2002), Ashour et al. (2002a), Ashour et al. (2002b), Ashour and Norris (2003), Ashour et al. (2004), Ashour and Norris (2005), Shamsabadi et al. (2005), and Ashour and Norris (2006).

The method relates soil response and parameters participating in the 3-D response of the wedge to the response and parameters of the one-dimensional case of the “beam on elastic foundation” pile/shaft system. The soil stress-strain and strength properties, which can be determined from laboratory triaxial testing, are used to relate the horizontal strain in the passive wedge in front of the pile/shaft to the deflection (y) versus depth. The horizontal stress change is related to the non-linear modulus of subgrade reaction, which is the slope of the p-y curve. The SWM can therefore be used to develop p-y curves for soil (Brown et al. 2010).

In this method, the passive wedge is divided into various horizontal slices or sublayers of constant thickness. In each slice, the soil is considered to behave uniformly, to be under plane stress conditions, and to have the same properties, which are a function of the slice location. The deflection of the pile/shaft in front of the slice is controlled by equilibrium conditions according to the soil-pile/shaft interaction established. The wedge shape is affected by soil type and properties. The mobilized depth of the passive wedge at any time is a function of the various soil parameters, stress levels, pile/shaft bending stiffness, and pile/shaft head fixity conditions. In general, the geometry of the passive wedge changes as the load increases while satisfying compatibility between pile/shaft deflections and soil modulus profiles. The strains are assumed to vary linearly over the depth of the passive wedge, as shown in Figure 6-8.

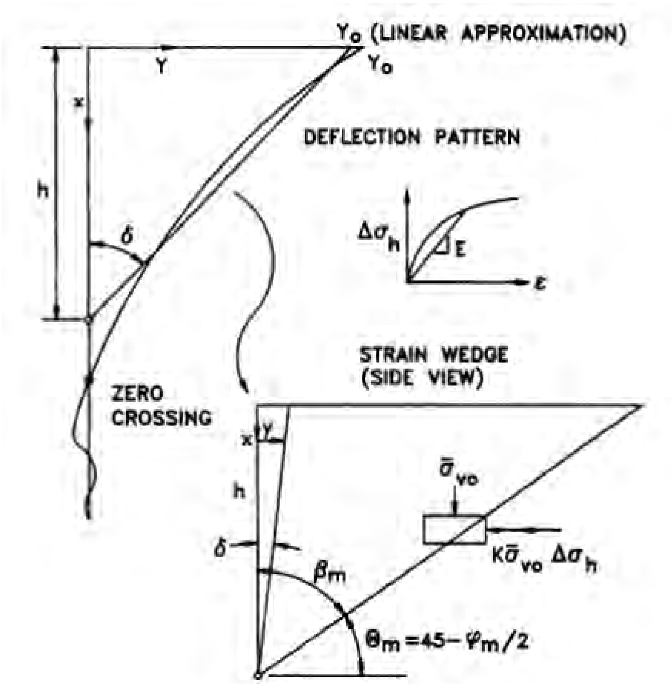


Figure 6-8: Deflection pattern of laterally loaded long pile/shaft and associated strain wedge (from Ashour et al. 1988).

The SWM can directly accommodate the effect of stratigraphic variations on soil properties and behavior at a specific elevation, and thereby address a theoretical limitation of the p-y approach which uses independent non-liner springs to model the soil resistance (Brown et al. 2010). Variable subsurface conditions and soil properties can be addressed through the inclusion of multiple soil material layers. The shape of the wedge and strain patterns within each separate material wedge are modified to accommodate a theoretical composite wedge as shown in Figure 6-9.

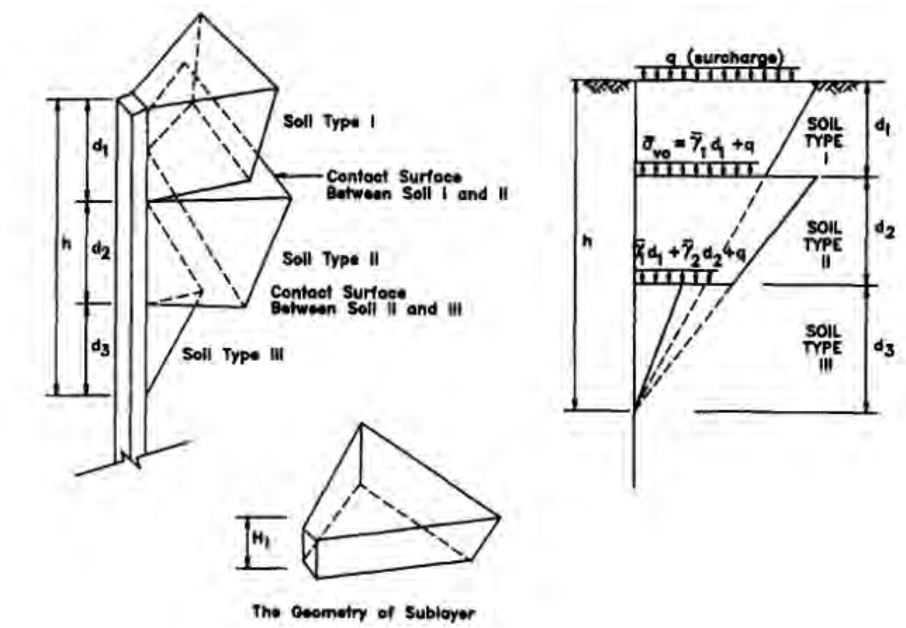


Figure 6-9: Geometry of a compound passive wedge (Ashour et al. 1988).

Compared to the p-y methodology, the SWM has advantages for analysis of laterally loaded deep foundations in the following areas:

- Soil response curves are developed not only as a function of soil properties and pile/shaft width but also to account for pile/shaft-head fixity, pile/shaft cross-section shape, bending stiffness, and soil property distribution with depth. This advantage is significant for the analysis of large diameter shafts because not all of these aspects can be accounted for using the conventional p-y method.
- For pile/shaft group analyses:
  - This method allows evaluating group response by overlapping the effects of passive wedges on the relative stiffness of the soil-pile system (p-multipliers, as discussed in Chapter 7 for the p-y method, are not required).
  - The presence of a pile/shaft cap can be accounted for by also evaluating the development of a passive wedge over the depth of the pile/shaft cap (Refer to Chapter 7 for discussion regarding the use pile cap resistance).
  - Individual members in a group are analyzed based on their location in the group, longitudinal and transverse spacing, and the soil type that surrounds them. These aspects are not currently addressed in the current p-y practice.
- The analysis of individual piles/shafts and/or groups in liquefied soil may be more realistic as the development of excess pore pressure can be accounted for.

- The effect of lateral soil spreading on individual piles/shafts and/or groups can be assessed with a rational method.
- The effect of the vertical side shear on the lateral response of large diameter shafts can be explicitly considered.

The SWM has inherent limitations due to simplifications of the problem and inherent limitations of predicting in-situ stresses and stress-strain behavior of geomaterials; such limitations are common to all complex 3-D modeling of non-linear soil-structure interaction problems. The SWM is not as widely used or accepted in practice for transportation structures, and therefore there is not as much experience or correlation of results with testing databases for this method compared to the more widely used p-y method. It is therefore recommended that for long piles, the SWM method be used to supplement a p-y analyses to provide additional understanding and evaluation of the problem conditions, rather than replace the p-y analysis. The use of SWM in additional to p-y analyses may be useful for large or complex problems. Also, full scale load testing can be used to provide field verification testing of any designs that are based on SWM.

The SWM may be more applicable than the Broms method for shorter, large diameter foundations elements that tend to rotate rather than bend (AASHTO 2014).

6.4.1 Recommendations Regarding Strain Wedge Model Method

The SWM method is accepted by a handful of state agencies. The SWM method does not have the same history of use and familiarity within the transportation industry as the p-y method (refer to Appendix E for additional discussion). For long foundation elements that bend rather than rotate, this method can be used to supplement p-y analyses, especially for large or complex projects. If used as the primary method for design, lateral load testing for verification is recommended.

6.5 BROMS METHOD

Broms (1964a, 1964b, 1965) developed a method for estimating the nominal lateral capacity and corresponding moments and shear forces along a vertical foundation element subjected to lateral loads. The method, which is based on limit equilibrium principles, relies on the assumption that soil stresses around the deep foundation element have mobilized their nominal values (i.e., passive and/or active states). This method is therefore applicable to the Strength Limit States.

The method is applicable to relatively short, stiff foundation elements, such as drilled shafts, subject to lateral shear and overturning moments applied at the top of the foundation element which is free to rotate. In this method, only uniform pile/shaft dimensions and homogeneous soil profiles and properties can be considered. The Broms method is widely accepted for preliminary analyses or initial estimates of foundation lengths, or for design of simple, non-critical structures, such as sign post or noise wall foundations, in relatively uniform soil profiles. However, the p-y method enables a more complete modeling of problem conditions compared to the simplified Broms method.

Broms (1964a, 1964b, 1965) also developed expressions to estimate lateral deformations for long piles that bend rather than rotate. However, these methods are based on a simplified subgrade reaction model for an elastic pile. This method is not considered reliable and is not widely used in practice. Broms also proposed a method for analysis of piles with a moment connection to a fixed cap that prevents rotation of the head of the deep foundation element. However, this method also has significant limitations, is not widely accepted in practice, and is not recommended for use. Therefore, the Broms method referred to herein is only the method developed by Broms for short, stiff foundation elements with a free head that rotates rather than bends. Other methods of analyses developed by Broms for long piles or piles with fixed heads are not recommended for use in practice.

Foundations that are more significant than the simple cases indicated above for the Broms method, or foundations that require estimation of lateral deformations, or have more complex loading, head fixity, or variable subsurface conditions should be analyzed using the more sophisticated and widely accepted p-y method (discussed in Section 6.3).

The Broms method for short piles has not been calibrated to the LRFD design framework.

The Broms method is based on simple passive soil pressure diagrams and a limit equilibrium solution obtained through equations of static equilibrium of shear and moment in the shaft. Although the method is based on simplifications and has limited capabilities, it is still useful for simple structures, widely accepted by many agencies for such simple applications, and is useful for understanding the loads and resistances of laterally loaded foundations.

6.5.1 Broms Method for Cohesive Soils

The soil reaction distribution (earth pressures), shear, and moment diagrams for the Broms method in cohesive soils for short piles is shown in Figure 6-10. The pile is subjected to a lateral load, P_t, and an overturning moment, M_t. The embedment length of pile is L, which is defined as L = 1.5B_b + f + g, where B_b is the pile diameter or width and f and g are depths indicated in Figure 6-10. The maximum unfactored soil resistance per unit length of the pile is nine times the undrained shear strength, S_u, times the pile width or diameter (B_b), with the top 1.5 times B_b excluded as indicated in the figure. The earth pressure in the upper portion of the pile opposes the shear force as shown, and the earth pressure in the lower portion of the pile acts to restrain the pile toe. The resulting shear and moment diagrams are also shown in Figure 6-10.

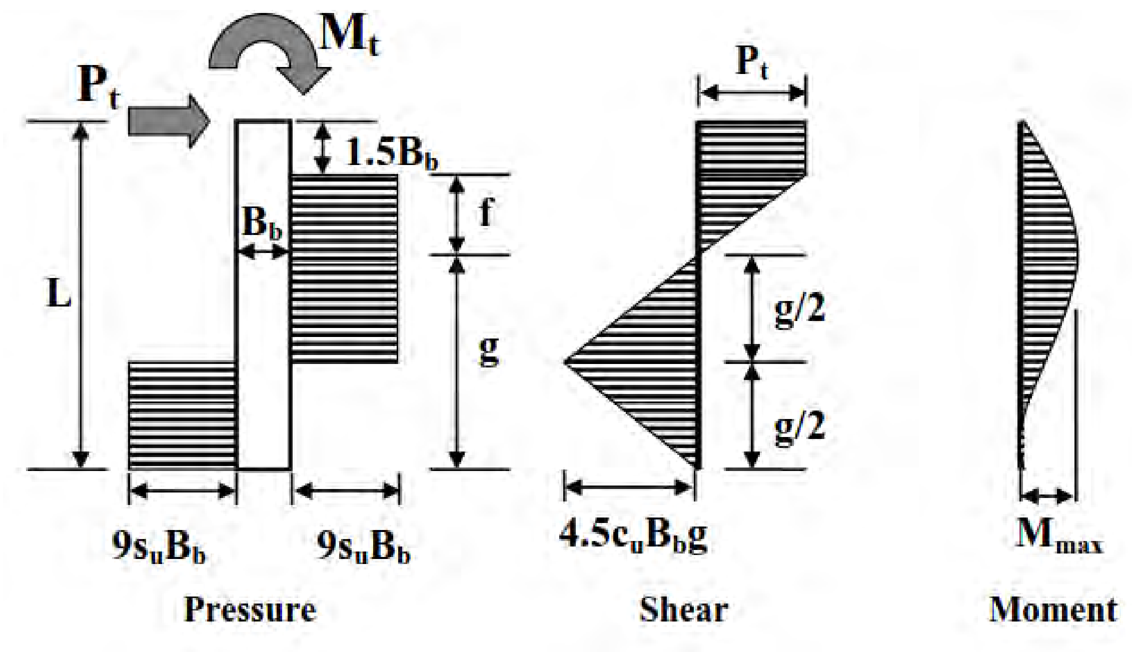


Figure 6-10: Earth pressure, shear, and moment diagrams for Broms method in cohesive soils (from Brown et al. 2010).

The point of zero shear, and therefore the maximum moment, occurs at the depth, *f*. To satisfy horizontal force equilibrium, the resultant of the earth pressures below that point, over the depth, *g*, must be zero, so the earth pressures are equally divided over the depth, *g*. The resulting moment due to the earth pressures acting over the depth, *g*, must be equal to the maximum moment, which is the moment due to the applied shear, moment, and earth pressures above the point of zero shear. Note that only horizontal forces and moments are considered, and that no vertical loads are considered. Also, the earth pressures are considered as fully mobilized on opposite sides of the pile, regardless of the magnitude of the actual deflections along the pile length.

Based on the diagrams, the location of maximum moment and zero shear is defined by the distance, *f*, given by:

$$f = \frac{P_t}{9S_u B_b} \qquad \text{(Equation 6-8)}$$

The maximum moment is given by:

$$M_{max} = M_t + P_t(1.5B_b + 0.5f) \qquad \text{(Equation 6-9)}$$

The maximum moment can also be calculated by applying moment equilibrium at the point of zero shear, or:

$$M_{max} = 2.25B_b g^2 S_u \qquad \text{(Equation 6-10)}$$

The depth, *g*, can be determined from:

$$g = \left(\frac{M_{max}}{2.25 S_u B_b}\right)^{1/2}$$ (Equation 6-11)

And the minimum pile length can be determined as:

$$L \geq 1.5B_b + f + g$$ (Equation 6-12)

6.5.2 Broms Method for Cohesionless Soils

For a short pile/shaft in cohesionless soil, the maximum soil resistance per unit length of the pile/shaft by the Broms method is the passive earth pressures applied for three times the width of the pile. Passive earth pressure is assessed using Rankine theory. The earth pressure, shear, and moment diagrams for a pile/shaft in cohesionless soil according to the Broms method are shown in Figure 6-11.

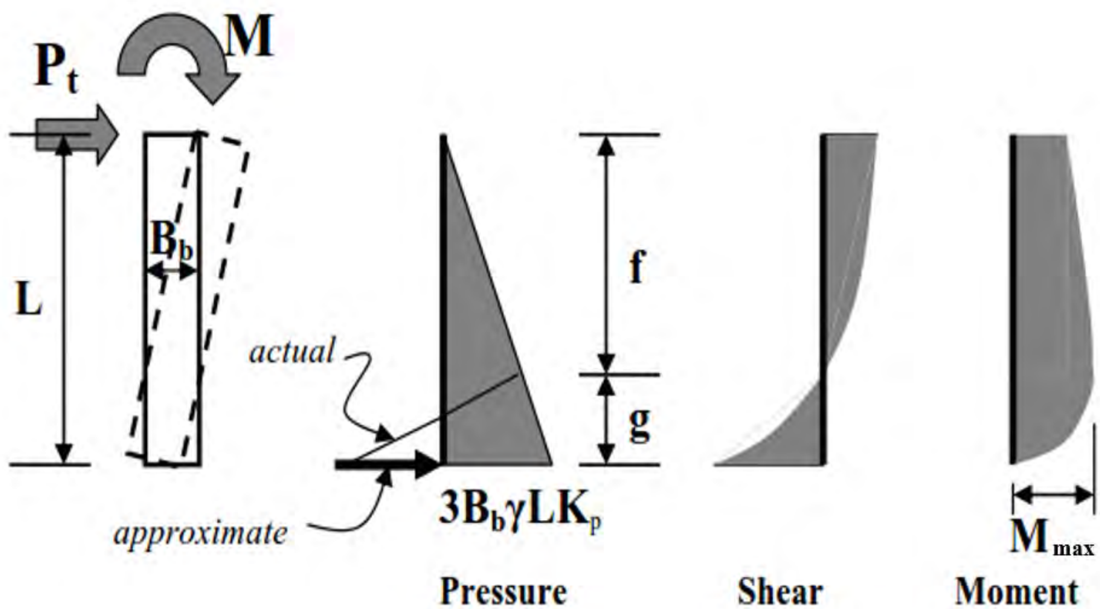


Figure 6-11: Earth pressure, shear, and moment diagrams for Broms method in cohesionless soils (from Brown et al. 2010).

Therefore, at a depth, *z*, below the ground surface the soil resistance per unit length of shaft, p_z, is:

$$p_z = 3B_b \gamma z K_p$$ (Equation 6-13)

Where B_b is the width of the pile/shaft, $\gamma' z$ is the vertical effective stress (effective unit weight multiplied by depth), and K_p is the Rankine passive earth pressure coefficient, given as:

$$K_p = \tan^2\left(45 + \frac{\phi'}{2}\right)$$ (Equation 6-14)

In which ϕ' is the effective friction angle of the cohesionless soil.

The pressure distribution in Figure 6-11 represents a simplification with a concentrated force at the bottom of the pile on the left-hand side. In actuality, the passive earth pressure will cross the vertical axis at the point of rotation. For a pile of minimum length, L_{min}, the depth z is replaced by L_{min}. The requirements of overall moment equilibrium at the base of the shaft are applied (using the simplified approximate pressure distribution) to determine the maximum force at the pile head for L_{min}:

$$P_t = \frac{\gamma' B_b L_{min}^2 K_p}{2L_{min}} - \frac{M_t}{L_{min}} \qquad \text{(Equation 6-15)}$$

Equation 6-15 can also be solved to determine L_{min} for an applied force, P_t.

The point of zero shear and the point of maximum moment occurs at depth, *f*, which can be determined by:

$$f = \sqrt{\frac{P_t}{1.5 B_b \gamma' K_p}} \qquad \text{(Equation 6-16)}$$

The maximum moment, M_{max}, is determined from the sum of the moments about depth f:

$$M_{max} = M_t + P_t(f) - \left(\frac{B_b \gamma' f^3 K_p}{2}\right) \qquad \text{(Equation 6-17)}$$

An example of the Broms method for cohesionless soils is included in Appendix B.

6.5.3 Recommendations Regarding Broms Method

The Broms method is acknowledged by many state agencies as being acceptable for preliminary analysis of laterally loaded deep foundations or for simple projects of low complexity. This method is most applicable to short, stiff foundations that tend to rotate rather than bend, such as sign post or noise wall foundations.

6.6 OTHER ANALYSIS METHODS FOR LATERALLY LOADED DEEP FOUNDATIONS

Some state agencies also allow the use of empirical charts or tables, but these must be used on a case by case bases as intended by the agency that has published them. These are often for preliminary analyses or simple structures and are based on local practice or precedence. These should not be extrapolated beyond their intended use.

Other analysis methods are not as widely used and should not be solely relied upon for analysis. Many of these methods have fundamental shortcomings or lack of precedence, use, and/or familiarity in the transportation industry. For large or complex projects, additional soil-structure interaction modeling, such as finite element modeling (FEM), may be appropriate. Such advanced modeling should be done only by practitioners that are familiar with the nuances of FEM modeling to avoid creating erroneous or misleading results. Consideration should be given to performing lateral load tests, either in design or for construction verification, for large or complex projects, or projects that analyses with different methods indicate uncertainty in the results.

Other methods include the Characteristic Load method, developed by Duncan et al. (1994) and Evans and Duncan (1982) to analyze laterally loaded piles/shafts; the Brinch Hansen method (l961), developed as a general procedure to estimate the lateral resistance of laterally loaded piles with a general distribution of soil resistance; and elastic solutions based on the boundary element method by Poulos and Davis (1980) and Poulos and Hull (1989).

6.7 OTHER DESIGN CONSIDERATIONS

This section presents additional design considerations for analysis of laterally loaded single deep foundation elements. The topics presented in this section generally apply to more complex loading conditions, problem geometry, or subsurface conditions, and therefore are generally not applicable to the Broms method for simple analyses as previously discussed. Therefore, the considerations discussed herein are presented assuming the applicability of the p-y method.

6.7.1 Selection of Deep Foundation Type and Size

As discussed in Chapter 5, one of the key steps in the design of laterally loaded deep foundations is the selection of the deep foundation type. In some projects, the selection of a deep foundation may be dictated by project conditions, project type, or local practice. For example, an area with limited footing space may be designed with a single drilled shaft foundation because of that restriction, or a pile and lagging wall may be the only viable wall type for a particular site. However, in many projects, the design of a deep foundation to resist lateral loads will have multiple types of foundations and sizes that will produce a satisfactory technical result.

The solution for a deep foundation that has adequate resistance and tolerable deflections to applied lateral loads is a combination of the type of foundation (steel pile, concrete pile, drilled shaft, micropile, etc.), the size (width or diameter), the depth of embedment, and the number of elements used to resist the load (whether the problem is adequately addressed as an individual element or if a group of elements is considered). As previously noted, p-y curves are a function of the pile type, size, and soil conditions, and the soil response varies with depth. Therefore, it is generally found that multiple solutions regarding foundation type and size can be developed for a given project condition. Design of individual deep foundations for lateral load applications often involves an iterative process of evaluating different types, sizes, and depths of individual foundation elements to determine which combinations will produce satisfactory results. The computer software programs for p-y analyses make evaluating multiple foundation types and sizes relatively easy and straightforward.

6.7.2 Point of Fixity or Equivalent Depth of Fixity

The analysis of laterally loaded deep foundations has often included use of the term “fixity” in practice, sometimes also referred to as point of fixity or depth of fixity. Fixity is defined as the depth of a deep foundation element at which both lateral deflection and the slope of the deflected element are zero. For design, a practical depth to fixity can be initially estimated using procedures in AASHTO (2014), and verified using p-y analyses in final design. The depth to fixity is used for structural analysis of a foundation element. Equivalent depth to fixity can also be used to perform a preliminary analysis for buckling of unsupported pile lengths. However, for both applications, p-y analyses should be performed to verify the results for final debsign. Note that the final depth of embedment is typically below the depth to fixity, to achieve fixity for the foundation element.

Fixity herein refers to considerations relative to the embedded section of the pile. The fixity of the pile head; i.e., the connection of the pile head into a cap, is a separate topic that is discussed in a subsequent section.

6.7.2.1 Fixity for Structural Analysis

AASHTO (2014) Section 6.15.2 indicates that for structural analysis of piles subject to axial loads, the selection of resistance factors for structural analysis at the Strength Limit State should consider the depth to fixity as indicated in Figure 6-12. Above this depth, the pile is subject to compression and bending, and it will therefore need to be structurally analyzed for axial load, shear, and moment. Below this depth, the pile only needs to be analyzed for axial loads. Because of different load combinations, different values of resistance factors are applicable above and below the depth of fixity. The approach presented in AASHTO for selection of the resistance factor for axial compression, ϕ_c, and the resistance factor for flexure, ϕ_f, is presented in Figure 6-12.

The depth of fixity is used for structural analysis at the strength limit state in order to determine over what depth the shear and moment will apply and therefore which resistance factors to use.

For preliminary purposes, the depth to fixity below the ground, d_f, can be calculated from procedures in AASHTO (2014) Section 10.7.3.13.4 as:

For clays:

$$d_f = 1.4\sqrt[4]{\frac{E_p I_w}{E_s}} \qquad \text{(Equation 6-18)}$$

For sands:

$$d_f = 1.8\sqrt[5]{\frac{E_p I_w}{n_h}} \qquad \text{(Equation 6-19)}$$

Where:

E_p = Elastic modulus of the pile/shaft (ksi).

I_w = Weak axis moment of inertia of the pile/shaft (ft^4).

E_s = Soil modulus for clays = 0.465 S_u (ksi).

n_h = Rate of increase of soil modulus with depth for sands – refer to AASHTO (2014) Section 10.4.6.3.

For foundation elements embedded in rock, a depth of fixity of half the foundation element diameter below the top of rock can be assumed.

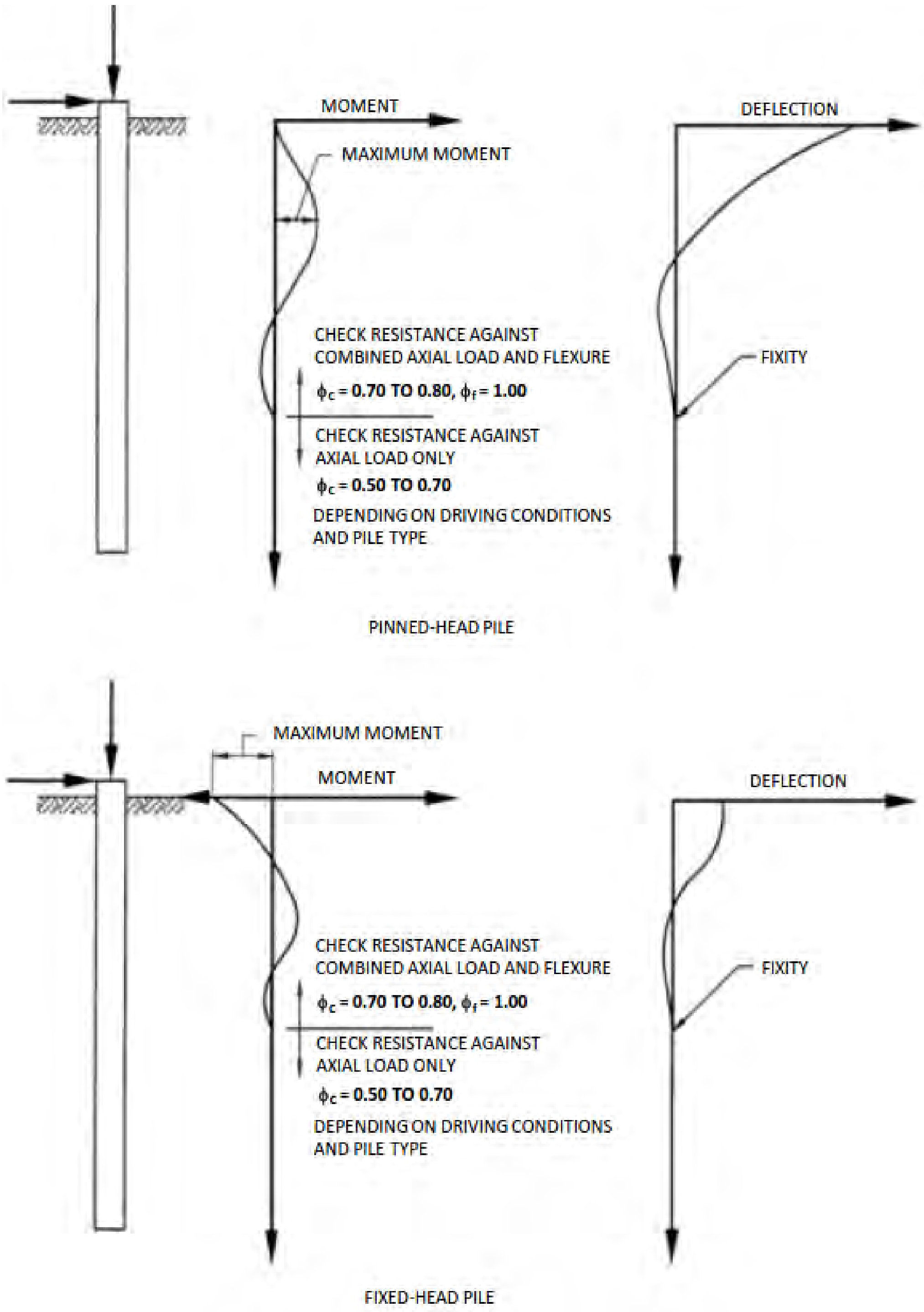
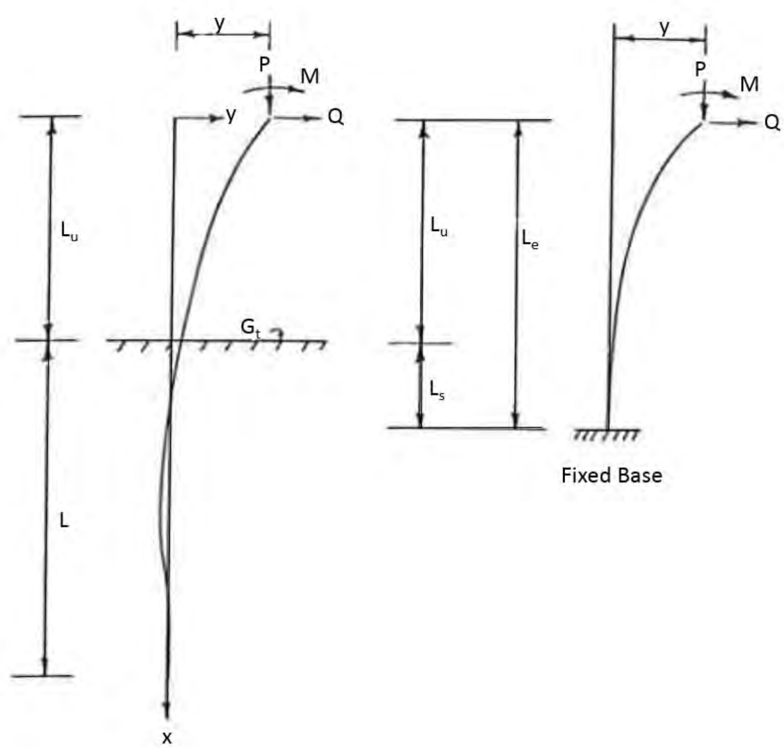


Figure 6-12: Illustration of the selection of resistance factors for structural Strength Limit State of piles (from AASHTO 2014).

6.7.2.2 Fixity for Equivalent Cantilever Length for Buckling

Some structures supported on deep foundations and subjected to axial and lateral loads will include free-standing foundation lengths; i.e., partially embedded and partially unsupported piles or drilled shaft lengths. An example of such a case is a bridge pier over open water, with an unsupported pile/shaft length from the mudline to the base of the pile cap or column, subjected to lateral loads from water, wind, ice, vessel impacts, etc. A pile in such a condition can be analyzed as an equivalent cantilever attached to a fixed base at a particular depth (Davisson and Robinson 1965; Greimann and Wolde-Tinsea 1988; Abendroth and Greimann 1989; and, Abendroth et al. 1989). This type of analysis is appropriate for a preliminary analysis for buckling of unsupported lengths of the pile. However, as discussed in Chapter 11, p-y analyses should be used in final design.

Figure 6-13(a) shows such a case, in which L_u is the length of unsupported pile and L is the length of the embedded section of the pile. For such a condition, a flexural analysis may control the design of the foundation element. Such a design is indeterminate and often difficult to solve without simplifying assumptions, especially if the pile head is attached to a girder or cap. From a structural analysis standpoint, it is convenient to simplify the analysis by assuming the pile is fixed at some depth below the ground surface (Davisson and Robinson 1965).



(a) **Partially embedded pile** **(b) Equivalent pile attached to a fixed base**

Figure 6-13: Illustration of a partially embedded pile and equivalent cantilever pile (from Davisson and Robinson 1965).

Figure 6-13(b) shows the equivalent cantilever model of overall length, L_e, with a length below the ground to an equivalent fixed base, L_s. The depth, L_s, is considered the depth to fixity for the equivalent cantilever model. This model is assumed to behave the same from a structural standpoint as the original model (Davisson and Robinson 1965).

The length of the equivalent cantilever, L_e, is given by (Abendroth and Greimann 1989):

$$L_e = L_u + L_s \qquad \text{(Equation 6-20)}$$

Procedures in Davisson and Robinson (1965) only apply to long foundation elements, meeting the criteria:

For Clays:

$$\frac{L_e}{\sqrt[4]{\frac{E_p I_w}{E_s}}} > 4 \qquad \text{(Equation 6-21)}$$

For Sands:

$$\frac{L_e}{\sqrt[5]{\frac{E_p I_w}{n_h}}} > 4 \qquad \text{(Equation 6-22)}$$

Where:

L_e = Length of the equivalent cantilever (ft).

E_p = Elastic modulus of the pile/shaft (ksi).

I_w = Weak axis moment of inertia of the pile/shaft (ft^4).

E_s = Soil modulus for clays = 0.465 S_u (ksi).

n_h = Rate of increase of soil modulus with depth for sands.

For piles that do not meet the criteria in Equations 6-21 and 6-22, piles will behave as a rigid member rather than a long flexible member, and the equivalent fixed pile conditions will not apply (Davisson and Robinson 1965).

As long as criteria in Equations 6-21 and 6-22 is met, the depth to fixity below the ground, L_s, can be estimated as d_f for preliminary purposes (Equations 6-18 and 6-19).

Equations 6-18 and 6-19 are for an assumed loading condition of axial load only, with the shaft/pile assumed to be fixed at both ends. It is noted that these equations give depth to fixity from the ground line; the unbraced length, L_u, must be determined by the designer considering the boundary conditions at the top of the pile. Other pile tip and loading conditions are addressed in Davisson and Robinson (1965).

The pile spacing in this analysis has an effect on the soil modulus. For pile spacing of three times the pile width, the effective soil modulus should be reduced to 25 percent of the value for a single pile. For a pile spacing of 8 times the pile width, no reduction is necessary. For spacing between 3 and 8 times the pile width, interpolation between these limits indicated above can be used (AASHTO 2014).

Once the actual pile conditions (Figure 6-13(a)) have been converted to an equivalent fixed pile condition (Figure 6-13(b)), then the structural design is relatively straightforward. The lower boundary, at L_s, is fixed in the structural analysis. If a frame analysis is performed, the deflection of the pile head, moment distribution, shear distribution, and axial loads at the pile head will be close to those for the original design condition. However, the moment at the depth of the fixed based, L_s, will be greater for this equivalent model compared to the actual condition. To analyze the embedded section of the pile, the loads at the pile head should be determined from the frame analysis and converted to loads at the ground surface through structural analysis. The loads at the ground surface can then be used to design the embedded portion of the pile using the methods previously described for analysis of a single deep foundation element (Davisson and Robinson 1965).

Some practitioners may elect to use the moment at the depth of the fixed base from this approach as the design moment for the internal structural analysis of the deep foundation for simplicity (it avoids performing another analysis) and conservatism (because it is greater than the actual condition). However, since this approach introduces unnecessary and unquantified conservatism, which may lead to an uneconomical design, it is not recommended practice.

The equivalent cantilever approach should only be used for preliminary estimates of required embedment lengths; the equivalent cantilever method is only applicable for a specific condition (laterally loaded deep foundations with unsupported lengths above the ground surface). Note that, with this method, the equivalent cantilever length is a function of pile and soil stiffness, and is not dependent on the applied loads. This allows an initial estimate of pile length for lateral loading requirements to be developed if structural loads are not yet available. However, once structural loads are available, the recommended approach is to use the p-y method to determine the required pile embedment length and the internal shear and moments.

6.7.2.3 Recommendations Regarding Fixity

In summary, the following guidelines are recommended for the use of depth to fixity in the design of deep foundation elements:

- Fixity is defined as the depth of a deep foundation element at which both lateral deflection and the slope of the deflected element are zero.
- Depth to fixity is applicable to structural analyses. Procedures are presented in AASHTO (2014) Section 6.15.2 to determine resistance factors above and below the depth to fixity.
- Depth to fixity can be initially estimated using Equations 6-18 and 6-19, but should be verified in final design by p-y analyses. The p-y analyses also provide a more reliable estimate of lateral displacement at the top of the pile/shaft.
- For foundation elements embedded in rock, a depth of fixity of half the foundation element diameter below the top of rock can be assumed.
- The equivalent cantilever approach, applicable for long foundation elements with unsupported lengths above the ground surface, should only be used for preliminary design.
- P-y analyses should be used in final design to verify foundation element length, buckling, and lateral stability.
- The total embedment of the foundation element is typically greater than the depth to fixity.

6.7.3 Free Head vs. Fixed Head (Effect of Pile Cap)

The connection of the deep foundation element to the cap can be fixed such that the rotation of the pile head relative to the cap is not permitted. Fixing a steel pile head to the pile cap requires that the pile be imbedded about two to three times the pile diameter or pile width into the cap. If the pile is embedded only a nominal amount into the cap, it behaves as a pinned connection and is free to rotate relative to the cap. Most installations are somewhere between these two extremes. Drilled shafts and concrete piles generally are considered fixed to the substructure unit if the reinforcing from the shaft or pile is fully developed into the substructure unit.

However, a pure fixed-head condition is seldom achievable in the field, even when a pile group is constrained by a stiff concrete pile cap, because the cap itself rotates. Pile group deflections increase as the degree of fixity at the pile head decreases; thus, assuming complete fixity (zero rotation at the pile head) can result in underestimated values of pile head deflection, and incorrect magnitudes and locations of maximum pile bending moments. On the other hand, assuming a free-head condition will most likely result in an over-conservative design. The degree of rotational restraint at the top of a pile group usually falls somewhere between the limiting boundary conditions represented by fixed-head and free-head cases.

At the Service Limit State, the pinned condition will generally result in a larger horizontal deflection. At the Strength Limit State, the fixed condition will generally result in a larger bending moment at the head of the pile but may result in a larger or smaller moment at depths below the head of the pile.

If there is uncertainty whether a connection should be treated as pinned or fixed, it is recommended to perform the single pile analysis with both types of connections to evaluate how sensitive the results are to the head condition. If the results are sensitive, a more detailed analysis, such as a pile group analysis, should be performed. Some computer programs for lateral analysis of pile groups include the effects of pile cap rotation in the analysis. Group pile analysis is discussed further in Chapter 7.

6.7.4 When to Consider Anchors or Braces for Lateral Support of Deep Foundations

Except for retaining wall and slope stabilization designs that are based on failure of the soil, the design of laterally loaded deep foundations is based on empirical soil-structure interaction that relies on limiting displacement as a design criterion for both Strength and Service Limit States. For conventional bridge or wall foundations, the size of the foundation element (diameter or width) can be increased, the length increased, and/or the number of foundation elements within a foundation increased until the limiting deflection criterion is met. However, there are some applications, in particular non-gravity cantilevered walls (soldier pile and lagging walls), where these alternatives may not be able to reduce the deflections sufficiently or may not be the most cost effective approach to meeting the design criterion.

Cantilevered soldier pile and lagging walls typically consist of a single row of foundation elements. Adding a second row of foundations and a cap to connect the two rows significantly impacts the design, constructability and cost of the wall and effectively changes the wall type to a conventional retaining wall on piles; typically, this type of construction is not feasible where a non-gravity cantilevered wall is required. Reducing the spacing between the soldier piles has a limited ability to increase the capacity of the wall because the zones of influence will overlap more, thus reducing the amount of passive soil resistance available to each pile (although closer pile spacing will increase the structural stiffness of the wall system). Increasing the size of the soldier pile, either the pile itself or the diameter of a pre-augered hole that it is set in, also reaches a point of diminishing return because of the increasing cost of each element (and therefore the cost per unit length of the wall) begins to increase significantly.

In these situations, anchors or external braces may offer a feasible and cost effective alternative to limit the deflections of the foundation. Such anchors or braces provide a second point of support for the walls, in addition to the passive resistance on the foundation elements, and therefore reduce the amount of load that is to be resisted along the embedded portion of the soldier piles. Typically, external anchors or braces are needed for cantilevered walls that are greater than 8 to 15 feet in height, depending on the external loads due to soil conditions and surcharges, the soldier pile type and size, the available passive resistance, and the magnitude of the limiting deflections. The design of anchored, tied-back, or braced walls is beyond the scope of this manual. Refer to the LRFD Bridge Design Specifications by AASHTO (2014) for detailed design guidance of the design of anchored or braced walls in the LRFD design platform.

6.7.5 Scour

Scour is a primary design consideration for bridges over waterways; other transportation structures may also have foundations in or near waterways that may need to be assessed for scour conditions as well. The evaluation of scour should be a multi-disciplinary task that includes hydraulic engineers, structural engineers, geotechnical engineers, and construction specialists. Evaluation of scour should be performed in accordance with Publication No. FHWA-HIF-12-003, *Evaluating Scour at Bridges*, by Arneson et al. (2012), also referred to as HEC-18.

Bridge foundations are to be designed for the worst conditions that may result from scour for the 100-year flood event, or from a lesser event with deeper scour potential. This is considered the *design* flood and is not an extreme event condition. Bridge foundations are also to be checked to ensure they will not fail for an extreme flood event, such as the 500-year flood. This is considered the *check* flood and is considered an extreme event.

Scour includes general scour from the flood (degradation of the river bed), as well as contraction scour due to the presence of the structure in the waterway, and local scour around the foundation elements themselves. Scour components for bridge foundations are illustrated in Figure 6-14.

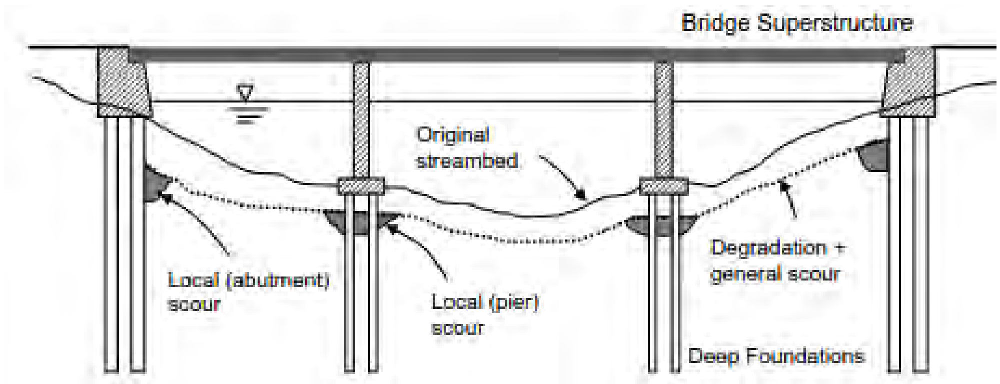


Figure 6-14: Illustration of scour conditions including degradation and general scour and local scour (after Brown et al. 2010).

The effects of scour will lower the surface elevation in the vicinity of the deep foundation elements. In some cases, entire soil layers may be removed due to scour. The lateral analysis for the scour condition must account for these changes in the subsurface profile (i.e., Block 5 in Chapter 5). Consideration should be given as to whether additional lateral loads are applicable to the scour condition, such as vessel impact loads, wave loads, loads due to water pressure, debris loads, larger moments or axial forces due to unsupported foundation lengths, etc. Multi-disciplinary coordination is required to ensure that all proper loads have been considered and accounted for.

The result of scour is that the lateral resistance of the deep foundation will be decreased (due to removal of materials) and the loads on the foundation element may be increased. Although actual scour conditions may be variable, for analysis of laterally loaded deep foundations, the entire soil depth within the scour zone should be considered to be removed by scour.

Additional considerations for design of drilled shafts and driven piles for scour conditions, including considerations for design of axial capacity and considerations for construction and installation, can be found in Brown et al. (2010) and Hannigan et al. (2016).

6.7.6 Sloping Ground Surface

Sloping ground is characterized by the angle, θ, which is the deviation of the ground surface from horizontal as indicated in Figure 6-15. For a ground surface that slopes down in the direction of the applied load, the resistance to the applied load will be decreased and the p-y curves must be modified. Modifications to the p-y curves for sloping ground conditions are generally only applied to the equations for the wedge-type failures near the ground surface, under the assumption that the flow-around failure that occurs at depth is not influenced by the sloping ground condition (Isenhower and Wang 2015).

It is noted that the ground slope must be assumed to be uniform. If the slope is not uniform or continuous, an approximation of a uniform slope must be made for analysis. For cases with variable slope angles or heights, it is recommended that multiple uniform slope approximations be made to assess the sensitivity of the results to the approximated input geometry.

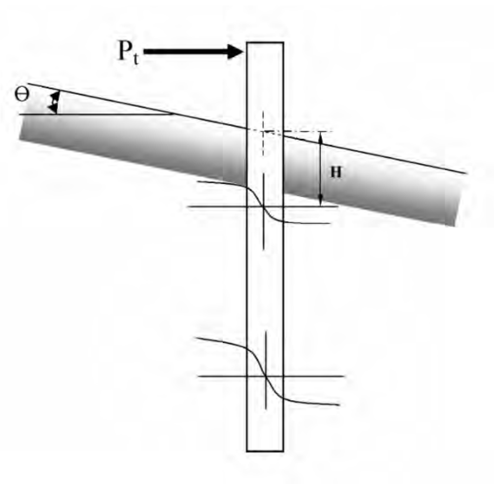


Figure 6-15: Effects of sloping ground.

6.7.7 Deep Foundations Socketed in Rock

Limited p-y curves are available for rock within the available computer software programs. Available p-y curves are based on a limited number of experiments and based on correlations. There have been a number of recent full scale lateral load test research programs performed with the intent of developing additional p-y curves in rock or weak or decomposed rock (examples include Robinson et al. 2005 and Boeckmann et al. 2014). Some of these test programs are applicable to a particular rock type or geology.

There are generally two types of rock addressed in available software programs, weak rock and strong rock. P-y curves for these materials are discussed in Appendix A. In general, the shape of the weak rock p-y curve is similar to that for cohesive soils, although different input parameters are used.

The p-y curve for strong rock is a bi-linear envelope based on tests in vuggy limestone of south Florida. This curve is recommended for rock with intact strengths greater than 1000 psi (Isenhower and Wang 2015). The tests that this p-y curve was developed from were run to only limited displacements. It was therefore assumed that brittle fracture may occur at higher displacements because the tested rock formation was known to be brittle in shear. As a result, the resistance of rock in the model drops to zero within the area of assumed brittle shear. Although conservative, this is a condition that is unlikely to occur in actual rock formations. This assumed brittle failure can result in misleading results, including the possibility of the strong rock analysis giving a weaker response than a weak rock analysis for the same input parameters (Brown et al. 2010).

The weak rock and strong rock models both include inherent assumptions and conditions based on limited experimental results in particular formations. Care must be exercised when using these models for other applications. For example, the strong rock model assumes that cyclic loading results in a loss of resistance, and the assumptions for the development of the weak rock curve appear to be valid only for static loading (Ensoft 2016).

Isenhower and Wang (2015) recommends performing lateral load proof tests if the deflection of a rock and foundation using the strong rock model are greater than 0.004b, where b is the width of the foundation element. Brown et al. (2010) recommend that the weak rock models be used for cases where shear failure of the rock mass is considered possible.

Another consideration regarding the design of rock socketed deep foundations are the shear and moments that occur internal to the foundation element at the top of rock. For strong, high quality rock, the top of rock boundary will result in high shear forces in the foundation element. For weak rock, especially if overlain by relatively stiff or dense soil, this transition will not be as abrupt and therefore the shear forces in the foundation may not be as high. The designer should consider this when evaluating which rock model is appropriate for use. Depending on the rock type and degree of fracturing, it may be appropriate to evaluate the embedment depth or deflections with one p-y model (strong or weak rock) and evaluate the shear forces for the structural design of the foundation using the other p-y model. The large shear force computed at the top of rock can also be addressed by reducing the design shear force to the average value along a length equivalent to one shaft diameter below the top of rock, as proposed by Brown et al. (2010).

It is clear based on the considerations above that the designer must use caution and judgment when analyzing deep foundations socketed in rock. Local practice or research program results, minimum socket depths required by AASHTO, and lateral proof load tests should be considered in the development of rock socketed deep foundation designs.

6.7.8 Loading Considerations

6.7.8.1 Axial Loads

The presence of axial loads can significantly influence the response of a deep foundation element to lateral loads and/or moments, and therefore axial loads must be included in the lateral load analysis when present. In general, for fully embedded piles (piles completely below the ground surface) with other considerations being equal, axial loads and the confining pressures of the surrounding earth create an increase in stiffness of the foundation element, and therefore reduce the amount of deflection that occurs under the same lateral load. For partially embedded piles with an unsupported length above the ground surface, axial loads will increase the amount of lateral deflection, assuming all other considerations being equal (Davisson and Robinson 1985).

If possible, lateral load proof tests should include conditions comparable to the actual design condition. This should include axial loads comparable to the design loads and/or unsupported pile lengths similar to the design condition. However, in many cases it is not practical to include axial loads within a lateral load testing program. Similarly, it is often not possible to include an unsupported pile length in a lateral load test program. An analysis should be performed to design the lateral load test so that the test load conditions produce a result that is as equivalent as possible to the design condition, or a condition that is more conservative compared to the design condition. For example, lateral proof load tests for fully embedded piles will tend to be conservative from the standpoint of deflection because the actual condition with axial loads will produce a stiffer response (i.e., less lateral deformation).

6.7.8.2 Cyclical Loads

The influence of cyclic loading can be significant. This effect has been assessed for deep foundations in clays and sands. Most p-y curves in the literature were developed for sustained loading. However, it was found that cyclic loading (caused by wind and ocean waves, among other factors), can cause a substantial loss in the lateral resistance of soils, especially at sites with stiff clay as a result of the progressive loss of contact between the soil and the foundation element, as shown in Figure 6-16. Figure 6-16(a) shows the general shape of a p-y curve for static loads versus a p-y curve for cyclic loads. The figure shows the general degradation of resistance over applied cycles of load to a lower final resistance compared to the static p-y curve. This strength degradation was observed by Matlock (1970) and Reese et al. (1975) in full-scale experiments. These researchers observed that after a number of loading cycles a loss of soil-pile contact was produced and some of the soil around the pile was suctioned into suspension in the water around the piles (Figure 6-16(b)), as evidenced by clouds of suspension around the front and back faces of the pile.

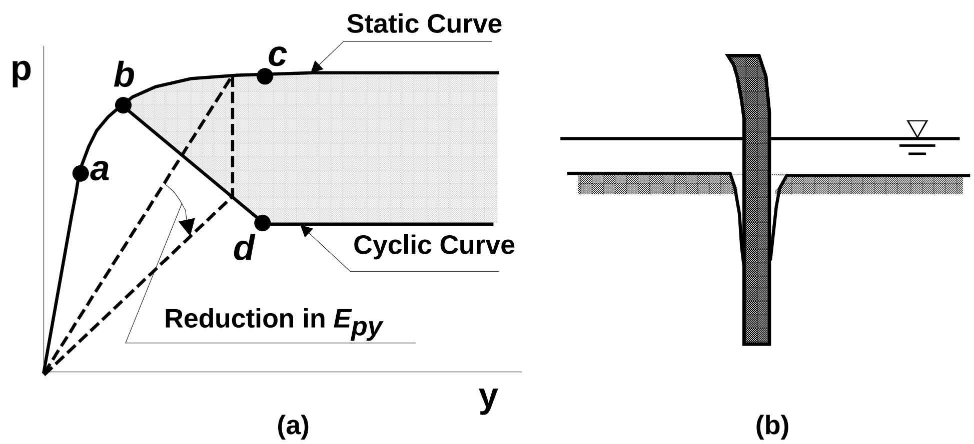


Figure 6-16: Effects of cyclic loading (a) schematic static and cyclic p-y curves and (b) loss of soil contact around deep foundation element (after Reese et al. 1975).

The influence of cyclic loading on p-y curves has been also studied by Wang (1982) and Long (1984), who observed that cyclic loading results in larger deflection and bending moments than with short-term static loading. For load levels mobilizing less than 50 percent of the static lateral resistance, deflections increase with cyclic load levels. For cyclic load levels, greater than 50 percent of the static resistance, deflections tend to increase only with the cycle numbers. This tendency was attributed to repeated cycles of relative large strains in the clay followed by scour of the soil around the pile if water is above the ground surface. Confirming other researchers' observations, Welch and Reese (1972) noticed that a gap between the soil and the pile may develop under cyclic loading. This mechanism was confirmed by Reese et al. (1975) and O'Neill and Dunnavant (1984). With the soil being washed out, the gap tends to increase and cause additional water to penetrate the gap.

6.7.8.3 Considerations for Transient Loads, Temporary Loads, and Permanent Loads

Transient loads are of short duration such as wind, wave, impact, or seismic loads. These loads are applied at a higher rate of loading than is typically accounted for in analysis or testing. Most load tests are performed under static loading conditions, and the analysis methods have been developed for static loads and therefore do not consider higher rates of loading. In general, rapid rates of loading would be expected to produce a higher lateral load resistance from the soil and a stiffer overall response. The use of methods based on static loads is therefore anticipated to be somewhat conservative for more rapid lateral load applications.

Loads can be also be of short, temporary duration or permanent. In this case, temporary loads refer to short term loads often associated with construction rather than rapid transient loads.

Short term temporary loads are analyzed in the same manner as permanent design loads. Where there are different considerations for temporary vs. permanent loads, these considerations are usually addressed through the level of reliability considered (different load or resistance factors) or through different performance requirements. For example, a lower load factor or higher resistance factor may be used for design or a temporary structure, or higher deflections may be allowed for a temporary structure. In some cases, such as performance of a pile and lagging support of excavation system adjacent to a critical structure, the reliability may be the same as a permanent structure and the performance requirements may be more stringent than may be used for a permanent structure.

For permanent loads, long term sustained lateral loads may result in soil creep or reduced lateral resistance over time. This is especially true for clay soils, where permanent loads may result in soil creep that may not need to be considered for temporary or transient loads. The resulting soil stiffness as a result of soil creep may be less than that predicted from p-y curves based on static loading conditions and undrained strengths.

6.7.9 Frost/Desiccation Depth, Loss of Contact, Etc.

Another consideration for analysis is disturbance to surficial soils that may impact the resistance to lateral loads of near surface soils. In many project settings, there is potential for natural disturbance of the near surface soils due to frost heave, desiccation or shrinkage, surficial erosion due to run-off, or other potential mechanisms that may result in loss of contact between the foundation element and the adjacent soil. These mechanisms may vary with time or year or precipitation patterns, or other outside influence. In such cases, it may be appropriate to neglect or reduce the soil resistance within the anticipated depths of which disturbance or loss of contact may occur. Construction disturbance, such as trenching for installation of utilities, may be another reason to neglect a nominal depth of resistance to lateral loads.

6.7.10 Other Design Considerations

6.7.10.1 Variations in Subsurface Conditions

Most project sites include some degree of variability with regard to subsurface conditions. The use of the p-y method allows parametric studies to be performed relatively easily. It is recommended that parametric software runs be performed when there are potential variations in subsurface layer thicknesses and depths, subsurface material strengths and stiffness, and significant variations in the water table depth. Selection of the p-y curve for modeling specific materials can also be varied to assess potential impacts of the specific p-y curve on the results, especially for conditions that the available p-y curves may not fit well with the actual subsurface conditions. Brown et al. (2010) present an example of drilled shaft through soft clay socketed into rock that demonstrates that some of the variations in subsurface stratigraphy may produce counterintuitive results.

If the site is highly variable, consideration should be given to subdividing the project site into smaller sites with more consistent conditions for analysis. If the results of parametric studies indicate the potential for significant impacts on the overall design, then consideration should be given to performing additional exploration and testing to better define the subsurface conditions and/or construction verification testing to verify the performance of the deep foundation design.

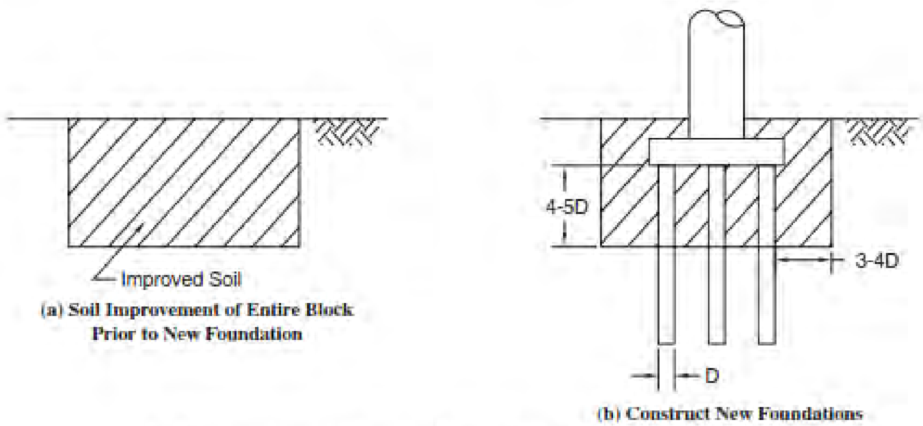
6.7.10.2 Anchors and Bracing against Deep Foundations

For deep foundation elements that are restrained by anchor devices, such as a tieback or strut, the tieback or strut can be simulated by using a very stiff p-y curve at the location of the support. The p-y curve should be consistent with the stiffness of the restraining element. This curve can be input in the same manner that other p-y curves can be specified for soil layers.

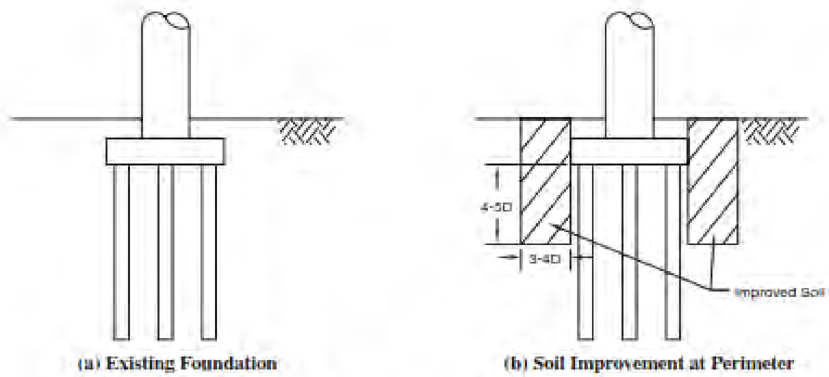
6.7.10.3 Increasing Lateral Resistance around Deep Foundations

In some problem cases the near surface soils are particularly weak or soft and may not provide adequate resistance to lateral loading of deep foundations. A typical approach for such a condition would be to increase the size of the deep foundations (larger diameters or width) and/or add more elements, resulting in a larger pile/shaft group. Another approach would be to use ground improvement to increase the resistance of the soils to lateral loads. Ground improvement methods for improving lateral soil resistance were evaluated and published by Rollins and Brown (2011) in the NCHRP report 698, *Design Guidelines for Increasing the Lateral Resistance of Highway Bridge Pile Foundations by Improving Weak Soils*.

Rollins and Brown (2011) indicate that significant increases in lateral resistance of soft clays and loose sands can be achieved through ground improvement and replacement techniques. Examples of treatment areas are shown in Figure 6-17. Ground improvement methods evaluated in the study included jet grouting, soil mixing, placement of flowable fill or compacted fill, and rammed aggregate piers. Cost comparisons were performed to evaluate potential cost savings of implementing ground improvement versus the additional of more piles and enlargement of the cap. Refer to the NCHRP study for additional details.



Soil improvement around new foundation.



Soil improvement around existing footing.

Figure 6-17: Ground improvement treatment concepts for increasing lateral resistance of pile groups in weak soils (modified from Rollins and Brown 2011).

Analysis of ground improvement for improving lateral load resistance requires application of judgment. Two key aspects to be assessed are the coverage of ground improvement and the development of parameters for input in the lateral pile analysis. The coverage of ground improvement must be sufficient to provide a large enough volume of improved soil to provide the required resistance. For example, Figure 6-17 indicates a zone of three to four times the pile width as a minimum. The development of the parameters for input in lateral analysis is also critical to developing an effective design. This include developing appropriate soil parameters and/or p-y curves for the modified ground, as well as considerations for shear or adhesion on the sides of the deep foundation elements, shear, adhesion, and/or passive resistance for the pile cap, etc. Rollins and Brown (2011) present details regarding the results of trial sections and recommendations for design. The study focuses on groups of piles, but the concepts of ground improvement for increasing lateral resistance are applicable to individual deep foundations as well.

7 LATERAL ANALYSIS OF GROUPS OF DEEP FOUNDATIONS

7.1 INTRODUCTION

The concepts related to the response of a single laterally loaded deep foundation element presented previously can be extended to a group of deep foundation elements. Groups can be analyzed using the p-y method or the strain wedge method discussed in Chapter 6. However, the lateral load resistance of a deep foundation element in a group is less than if the same foundation were an isolated element in the same subsurface conditions. Group efficiency and interaction must be accounted for in a group analysis. Most deep foundation groups involve foundations that are relatively closely spaced and therefore need to account for the interaction and reduction in efficiency within the group.

7.2 GROUP EFFECTS IN LATERAL LOADING

The behavior of a group of deep foundation elements is more complex than that of a single deep foundation because of two additional aspects that affect the response of the group (Reese and Van Impe 2001):

1. Reduced efficiency due of lateral load resistance among closely-spaced deep foundation elements as a result of the overlapping the shear zones mobilized to resist the lateral load (i.e., the “shadowing” or “shading” effect); and
2. Distribution of loads and moments to individual deep foundation elements within a group from the cap.

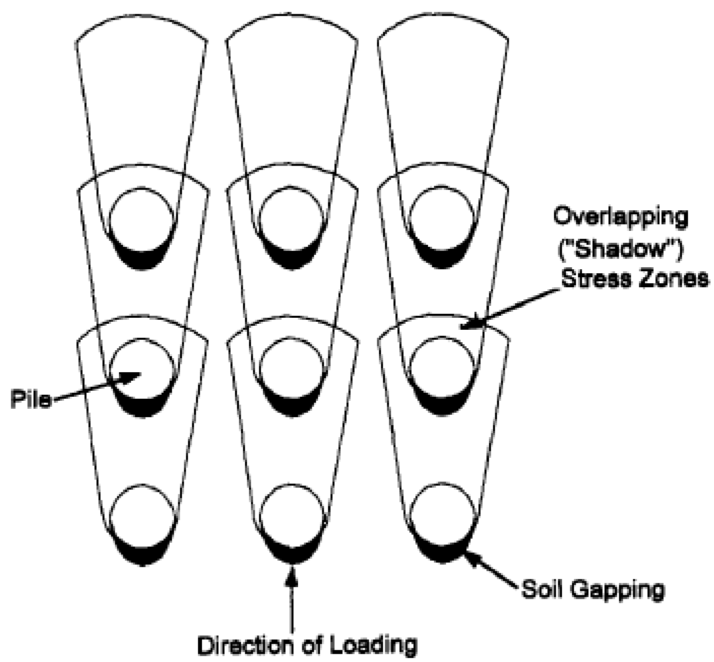


Figure 7-1: Illustration of the shadowing effect showing overlapping zones of mobilized lateral load resistance of a group of piles.

The shadowing effect is illustrated in Figure 7-1. In a group of deep foundation elements, the shear zones that are mobilized in response to a lateral load overlap, resulting in a reduction of the overall lateral load resistance that can be mobilized for the group compared to the sum of the individual pile resistances.

The cap is a relatively stiff block that connects all the deep foundation elements in a particular group. However, not all groups of deep foundation elements will have a cap; for example, a group of drilled shafts for landslide stabilization may not have a cap, and a line of piles for a pile and lagging wall or a noise wall may not have a cap. However, many groups of deep foundation elements, especially for column, pier, bent, abutment, or conventional retaining walls will have a cap that connects the individual deep foundation elements together. Under lateral and axial loading, the cap rotates and displaces, mobilizing the axial and lateral resistance of individual elements in the group; the load demand at any deep foundation element is determined not only by the type and magnitude of loads applied to the group, but also by the arrangement and spacing of the elements in the group and the specific position of the element within the group. These group effects, and the methods for analyzing the response of groups of deep foundation elements, are discussed in the remaining sections of this chapter.

7.2.1 Group Efficiency

Deep foundation elements in closely spaced groups behave differently from isolated deep foundation elements because the combined interaction of a foundation element with the soil, other piles, and with the cap are more complex than that of an isolated foundation element. In general, deflections of a closely spaced group of deep foundation elements are greater than the deflections of an individual deep foundation for the same foundation size, subsurface conditions, and load (on the individual foundation element). Likewise, maximum bending moments along a deep foundation element within a group will tend to be larger than those in an isolated deep foundation element. This results in an apparent reduction in resistance or reduction in efficiency of a group of deep foundation elements acting together compared to the sum of the same number of foundation elements if being analyzed individually in isolation. This reduction in efficiency is characterized by the term group efficiency, G_e, which quantifies the interaction effects in a group. G_e is defined as (Prakash and Sharma 1990):

$$G_e = \frac{(Q_g)}{n(Q_s)} \quad \text{(Equation 7-1)}$$

Where:

- Q_g = Lateral load resistance of the group.
- n = Number of deep foundation elements in the group.
- Q_s = Lateral load resistance of a single deep foundation element of the same design section and length as the elements in the group.

In Equation 7-1, it is assumed that all the deep foundation elements in the group have the same characteristics. In general, the lateral load resistance of the group is less than or equal to the sum of the lateral load resistances of the individual foundation elements, or $G_e \leq 1.0$. Pile group efficiency depends on various factors, including foundation element characteristics, group size, group arrangement, head fixity, subsurface conditions, and displacement of the foundation elements.

7.2.2 Load Distribution in a Group and the p-Multiplier Concept

Displacement and stress measurements in full-scale and model foundation groups indicate that the load is distributed non-uniformly to the individual elements within the group, with the load demand at any one deep foundation element depending on the location of the element within the group, as well as the spacing between the foundation elements, and other factors noted in Section 7.2.1. This trend is attributed to the shadowing effect depicted in Figure 7-1. In this case, the foundation elements in the leading or front row (the row closest to the zone of soil resistance) experience the least shadowing effect. Foundation elements in the trailing row (row immediately behind the lead row) experiences the shadowing from the front row. Because the shadowing effects overlap from one row to the next, each additional row experiences a compounding effect, making its lateral load resistance significantly less compared to an individual isolated deep foundation element that would not experience any shadowing effect.

In the p-y method of analysis, the shadowing effect is accounted for using a p-multiplier, P_m. The p-multiplier is a reduction factor that is applied to the entire p-y curve for a laterally loaded individual pile/shaft to account for the overlapping zones of influence in a group loading condition. The value of P_m used in practice is 1.0 or less. Figure 7-2 illustrates the concept of the p-multiplier for an individual p-y curve. Figure 7-3 illustrates the concept of the p-multiplier as it applies to a group of deep foundation elements, with each subsequent trailing row behind the first (leading) row experiencing a greater shadowing effect and therefore having a smaller p-multiplier and lower corresponding p-y curve for analysis.

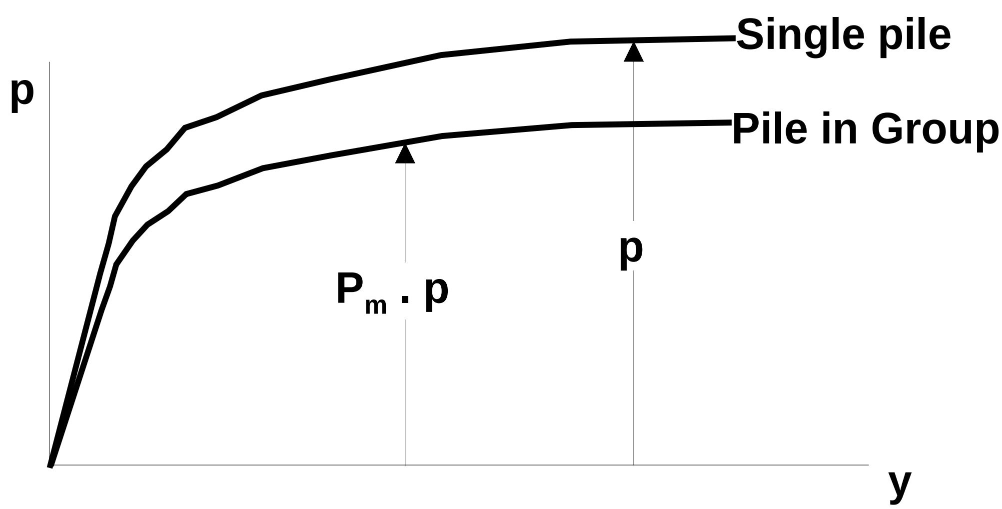


Figure 7-2: Illustration of the p-multiplier concept.

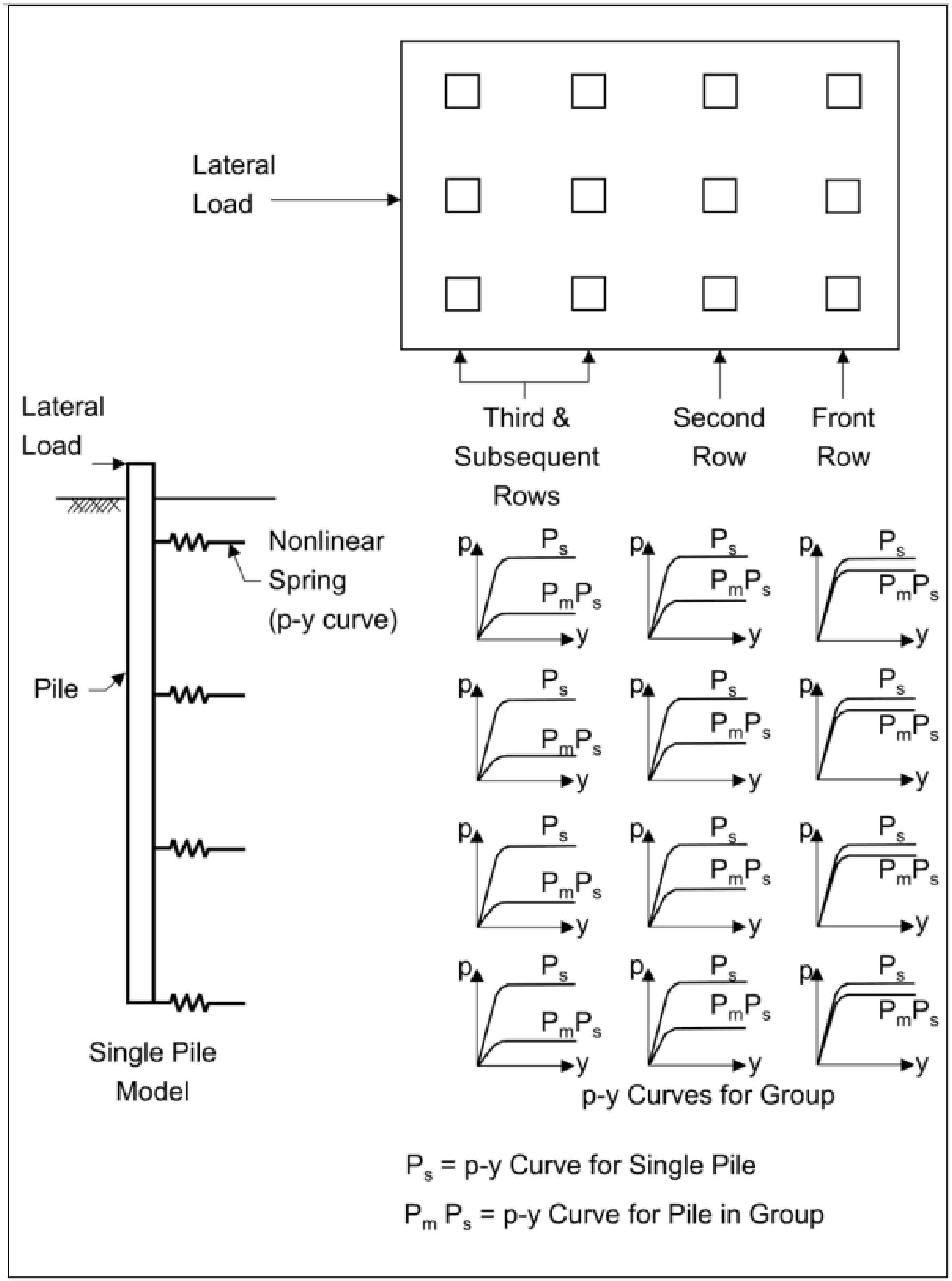


Figure 7-3: Illustration of the p-multiplier concept for rows of deep foundation elements in a group (from Hannigan et al. 2006).

7.2.2.1 Development of p-Multipliers

Brown et al. (1988), Ruesta and Townsend (1997), and Brown et al. (2001) conducted lateral loading tests on full-scale pile groups in loose to dense sands. McVay et al. (1995) performed centrifuge tests on a single pile and a series of pile groups in sand. Meimon et al. (1986), Brown et al. (1987), Rollins et al. (1998), and Rollins et al. (2006) conducted full-scale lateral load tests on pile groups in clay. Cox (1984), Rao et al. (1998), and Ilyas et al. (2004) performed model tests to examine the behavior of laterally loaded pile groups in clay.

Based on these and similar test programs, it is apparent that pile spacing, group arrangement, and group size have the most significant effect on group efficiency. Group efficiency decreases with an increase in group size (number of pile/shaft elements), as well as a decrease in pile/shaft spacing. The leading row develops the greatest soil resistance (attracting the largest lateral load demand) with decreasing contributions from each subsequent row, with the last row(s) developing the lowest soil resistance (and attracting the least lateral load demand); i.e., the leading rows have the highest p-multipliers and the last trailing row has the lowest p-multiplier. The most significant reduction in p-multiplier values occurs between the leading row and the trailing row immediately following the leading row. The reductions between subsequent trailing rows (i.e., between the second and third rows, or third and fourth rows) is relatively small compared to the reduction between the leading row (first row) and second row.

In some cases, the outer (side) elements appear to take more load than the interior elements. However, some experiments have observed approximately equal load distribution along a row of elements positioned perpendicular to the applied load. For analysis, p-multipliers are generally taken as the same for each row; i.e., equal load distribution along a row is generally assumed.

Pile group efficiency and p-multipliers decrease significantly when foundation element spacing decreases. In general, side by side interaction effects are not significant at a center-to-center spacing of 5B or more for sands or 3B or more for clays, where B is the width or diameter of the foundation element.

Sand density and clay strengths also have some effect on the p-multiplier values. McVay et al. (1995) compared the results from lateral load tests on a 3 × 3 pile group at a spacing of 3B in sand, and found that the load distribution was different for piles in dense sands compared to loose sands. In dense sands, a higher share of the load was taken by the leading row of piles compared to the trailing row, whereas in loose sands, the load share among the rows was more evenly distributed. Ilyas et al. (2004) found that interactions among piles in a group in normally consolidated clay are more significant than similar pile groups in stiff over-consolidated clay. However, overall, the effects of the soil type, density, and/or strength have a much less significant effect on the value of the p-multiplier values than the pile/shaft size, spacing, and layout relative to the applied load. In most cases, the type of soil and strength are not considered in the selection of p-multipliers for design, rather the p-multipliers are based on the size and layout of the foundation elements.

P-multipliers are somewhat dependent on pile deflection, but they are relatively constant at large pile head deflections, generally above about ¾ inch, as shown in Figure 7-4 (Rollins et al. 1998). This is consistent with results of a testing program reported by Caltrans (2003), which indicated that p-multipliers vary for displacements up to about 0.5 to 1.0 inches, but were generally constant for larger deflections up to 3 to 4 inches.

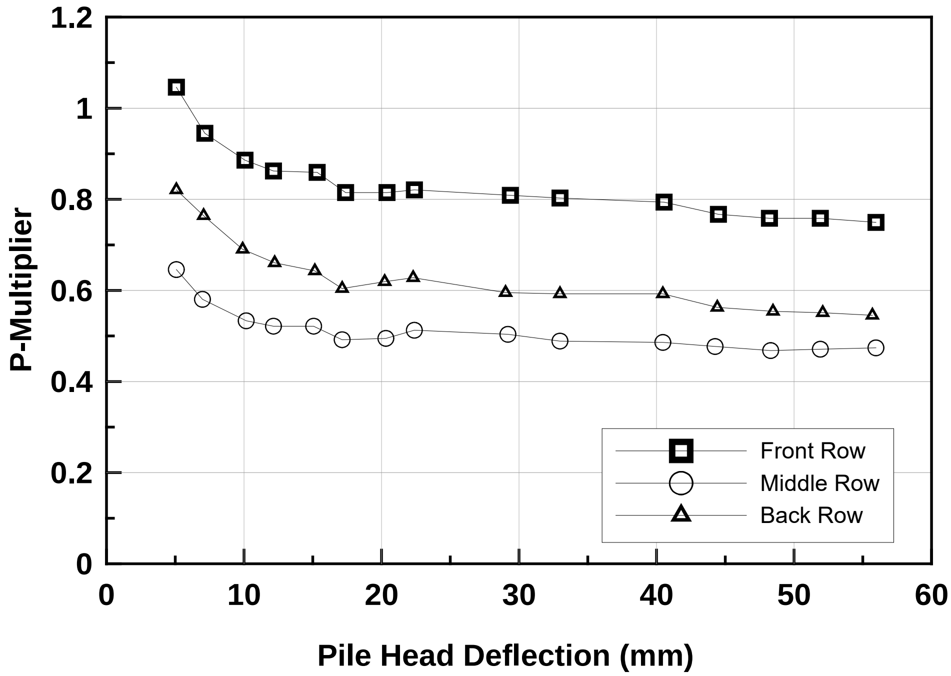


Figure 7-4: P-multipliers as a function of Lateral Pile Head Deflection (after Rollins et al. 1998).

Other studies have shown that installation methods may also impact the p-multipliers. Driven piles create a densifying effect on the surrounding soil compared to bored piles, which may loosen the surrounding soil. Comparisons of experimental programs on driven versus bored piles indicate that p-multipliers are generally higher for cases of driven piles compared to bored piles (Gandhi and Selvam 1997; Huang et al. 2001). However, in most projects, the effects of installation are not accounted for because the assumed densification effect would be difficult to estimate and quantify in design, and would have to be verified by in-situ testing or lateral load testing in construction.

Most test results have been from static lateral loading tests. However, the limited test data from pile groups loaded at velocities comparable to extreme events such as earthquakes or dynamic impacts indicate that p-multipliers from such dynamic loading are comparable to those from static load tests (Brown et al. 2010).

7.2.2.2 Recommendations for P-multipliers

AASHTO (2014) presents p-multipliers for analysis for several example group pile/shaft layouts. These p-multipliers are considered applicable for all subsurface conditions for the design of transportation facilities; i.e., these factors are applicable regardless of foundation type, geomaterial, or loading condition. As noted previously, the soil type, strength, and/or density do not have as significant an effect on the p-multiplier as the individual pile size, group size, and group layout. Example group foundation layouts for the application of the P_m. values in Table 7-1 are shown in Figure 7-5.

Table 7-1: P-multipliers for analysis of groups of deep foundation elements (from AASHTO 2014, originally modified from Hannigan et al. 2006).

Center-to-Center spacing of Deep Foundation Elements(1)	p-multiplier, P_m(2)(3) Row 1	p-multiplier, P_m(2)(3) Row 2	p-multiplier, P_m(2)(3) Row 3 and higher
3B	0.8	0.4	0.3
5B	1.0	0.85	0.7

Notes:
(1) Spacing is in the direction of loading; B = foundation element width/diameter
(2) P-multipliers are for vertical piles only
(3) Interpolation may be used for other spacing between 3B and 5B

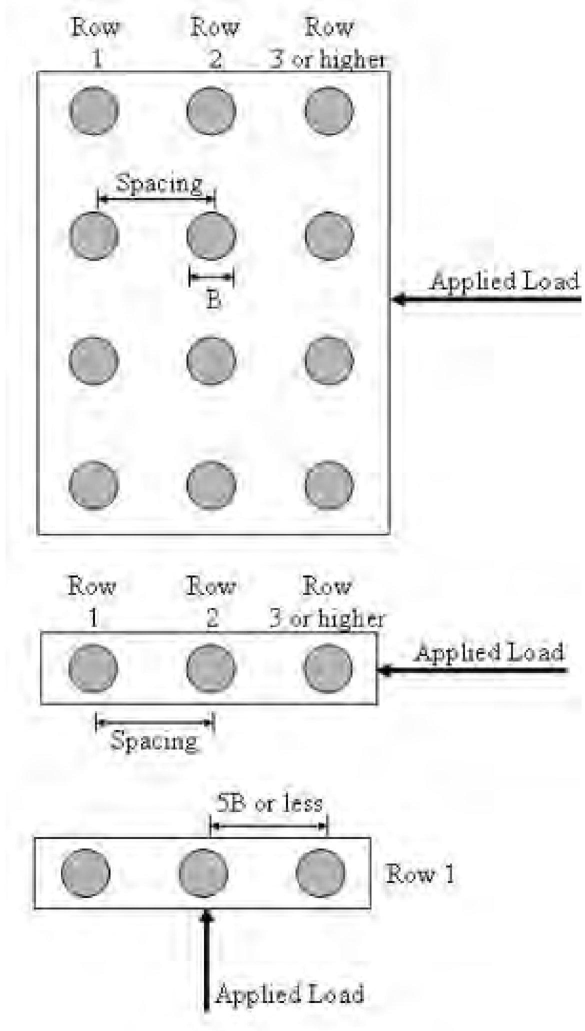


Figure 7-5: Loading direction and layout of deep foundation elements for p-multipliers in Table 7-1 (from AASHTO 2014).

The p-multipliers presented in Table 7-1 are recommended for design of laterally loaded deep foundation groups for transportation structures. Other p-multipliers are available in published sources and may be appropriate depending on the project size, location, local practice, stage of design, amount of construction testing, and other project specific considerations.

Some p-multipliers are available in commercial software programs. However, care must be exercised when using p-multipliers that differ from those in Table 7-1, even if included in software programs. Caltrans (2003) notes that the results of a full-scale group lateral load test program indicated that the p-multipliers that were automatically generated by one software program under-predict the group effect compared to the results of their field load tests, and that users of the program should consider specifying other p-multipliers.

The loading and subsurface conditions used for development of alternative p-multipliers should be carefully understood to ensure that the p-multipliers are appropriate for the particular design condition, as well as acceptable with regard to local practice and precedence. By contrast, the p-multipliers presented in Table 7-1 are published by AASHTO and are intended for wide use in the design of transportation structures.

7.2.2.3 Considerations for Group Effects for Other Methods of Analysis

The p-multiplier values presented in Table 7-1 are applicable only to the p-y method. The P_m values presented above and in the noted published references generally have not been calibrated to other analysis methods. The Broms method is applicable only to single piles and is not applicable to analysis of pile groups.

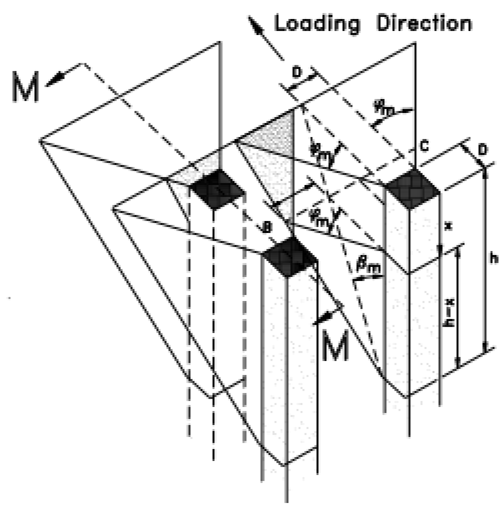


Figure 7-6: Pile group interaction in the SWM model (from Ashour et al. 2006, originally from Brown et al. 1988).

The SWM approach can be used to analyze pile groups. As discussed in Chapter 6, the SWM approach models the response of the soil as a wedge of soil resistance. In a group analysis, the wedges of different piles in the group overlap. The overlap of the SWM wedges decreases with depth, even in uniform soil, because the wedges vary with depth. This is illustrated in in Figure 7-6. The overlap of the wedges must be taken into account in the SWM group analysis (Ashour et al. 2006).

It is therefore possible to perform lateral analysis of pile groups using the SWM method. However, as discussed in Chapters 5 and 6, the p-y method for analysis is preferred and there are several commercially available programs for group analysis. The use of the SWM approach should be limited to cases where shallow pile behavior controls, or the method should be used as a supplemental analysis to p-y methods for complex or detailed design conditions, as discussed in Chapter 6.

7.3 LATERAL RESISTANCE CONTRIBUTION OF THE CAP

The cap for a group of deep foundation elements can provide a significant lateral resistance. Several studies have indicated that the cap can provide 40 to 50 percent or more of the overall contribution to the lateral resistance of a group of deep foundation elements (Beatty 1970;, Kim and Singh 1974; Rollins et al. 2000; Zafir and Vanderpool 1998; and, Mokwa and Duncan 2001). The lateral resistance of the cap is generated by passive pressures on the cap front, and shear resistances on the sides of the cap.

The base resistance under a cap is often disregarded because separation between the cap base and soil is likely due to soil settlement, particularly in soft ground conditions. Similarly, if scour or future excavations are likely to remove the soil in front of the cap, then the passive resistance and side resistance should not be relied upon. Soil shrinkage, frost action, or erosion may reduce the contact between the cap and the adjacent soil, which may reduce the side shear resistance or may increase the required deflection to mobilize the passive resistance. Use of passive pressure and side shear also requires verification of backfill placement and compaction around the cap to ensure that the soil is in full contact with the cap and properly compacted for the assumed strengths. For these reasons, the cap is often not relied upon for contribution to lateral load resistance of deep foundations. However, AASHTO does allow the effects of the lateral resistance provided by an embedded cap to be considered in the evaluation of horizontal resistance and displacements, but does not provide detailed guidance regarding this approach (2014).

Based on the findings from full-scale tests performed by Mokwa and Duncan (2001) and centrifuge tests conducted by McVay et al. (1995), the lateral resistance provided by a pile cap depends primarily on two factors:

1. Passive resistance in front of the cap, which is a function of the stiffness and strength of soil in front of the cap. The passive resistance that can be developed in front of a pile cap is directly related to the backfill strength and stiffness. The lateral resistance increases as the stiffness and strength of soil around the cap increases
2. Depth of cap embedment, as increasing cap thickness results in smaller lateral deflections at the same load (Mokwa and Duncan 2001). Additionally, increasing depth of cap embedment results in larger lateral resistance at the same deflection (McVay et al. 1995).

The inclusion of lateral resistance of the pile cap can be a complex problem to solve. The passive resistance in front of the cap is dependent on the amount of lateral deformation that occurs under the lateral load. For the Service Limit State, it is possible that passive resistance will not be fully mobilized. Considering that large ground displacements are typically needed to develop full passive soil resistance, the soil resistance used in design should be based on the results of a foundation displacement analysis and appropriate correlations between passive resistance and displacement.

The approach to analyzing the contribution of the pile cap involves consideration of loads and deformation, and an iterative process between p-y analysis of piles and assessment of deformations and reactions of the surrounding soils. This is a complex problem that is unlikely to be solved efficiently or accurately without the use of robust computational tools. However, software programs are available that can consider the contribution of the pile cap in addition to the pile-soil-pile interaction of the pile group (p-y analysis accounting for p-multipliers).

7.4 ANALYSIS OF GROUPS OF DEEP FOUNDATION ELEMENTS

The analysis of groups of deep foundation elements under lateral loads is a complex problem that requires computer analyses to perform effectively. As indicated previously, the p-y analysis method is the recommended method for analyzing deep foundations under lateral loading conditions, in particular for foundation elements that will behave as long piles subject to bending rather than shallow rotational failure. In most cases, groups of deep foundation elements are needed where loads (axial and/or horizontal) are large and the deep foundation elements will behave as long piles, and therefore group analysis is most often and appropriately performed using the p-y method.

Several options for p-y analysis of foundation groups are available. It is possible to analyze a group of deep foundations using analysis methods for individual deep foundation elements, although the method has some limitations as discussed below.

7.4.1 Analysis of Deep Foundation Groups using Individual Pile Analysis

For relatively simple design problems, preliminary designs, or non-critical structures, it may be sufficient to account for the group effects just by accounting for the interaction of the deep foundation elements. The p-multiplier is specified as an input parameter to account for the pile-soil-pile interaction, and the individual foundation element is analyzed using the p-y method as discussed in Chapter 6. An example of an application of this approach would be a soldier pile and lagging wall with no connecting cap. The soldier piles can be designed as individual piles subject to lateral loads with appropriate p-multipliers to account for the interaction between the piles, depending on pile size and spacing (including concrete encasement or casings for the embedded soldier pile section).

For other structures with a cap, but where the lateral resistance of the cap is neglected or cannot be relied upon, the analysis can be performed by analyzing the group of deep foundation elements by using individual pile analysis for each row of foundation elements, with evaluation of the group as a whole based on average outputs. The following approach, modified from Hannigan et al (2006), presents a procedure for analyzing a group of deep foundation elements using single pile analysis:

1. Obtain factored lateral loads for each row of the group.
2. Develop p-y curves for single pile analysis and develop the p-y curves from either:
    a. Site specific lateral load tests for a single pile,
    b. Published correlations with soil properties, or
    c. Based on site specific in-situ test data.
3. Perform single pile p-y analysis.
    a. Analyze each row position within the group using the p-multiplier, P_m, applicable for each row.
    b. Use the P_m values in Table 7-1 from AASHTO (2014), or other values as appropriate based on local DOT requirements.
    c. Determine the shear load versus deflection behavior for a single pile in each row and plot the load versus pile head deflection results as shown in Figure 7-7(a). Figure 7-7 is based on an assumed pile group consisting of 4 rows of piles.
4. Estimate the group deflection under the lateral load.

a. Determine the average group response (deflection) from the average of the individual responses for each of the rows in the cap as shown in Figure 7-7(a).

b. Divide the lateral load to be resisted by the entire group by the number of piles in each group to determine the average lateral load per pile.

c. Using the load deflection graph (Figure 7-7(a)) with the average load per pile, determine the estimated average group deflection.

5. Evaluate pile structural acceptability

a. Plot the maximum bending moment versus pile head deflection for each row of piles as determined from the p-y analysis, as shown in Figure 7-7(b).

b. Check the pile structural adequacy for each deflection, *p*. Using the estimated average group deflection (determined in Step 4c) and the bending moment versus deflection curve for each pile row, determine the maximum bending moment for an individual pile in each row.

c. Determine the maximum pile stress from the p-y analysis output corresponding to the maximum bending moment for each pile row.

d. Compare the maximum pile stress with the pile yield stress to assess structural acceptability.

6. Perform refined pile group evaluation that considered superstructure and substructure interaction. Brown et al. (2010) indicates that an alternative and simpler approach to the procedure described above is to use a weighted average p-multiplier value based on all foundation elements in the group. Based on experimental data and analyses, this simpler approach captures the overall group stiffness with respect to lateral load resistance with a sufficient level of accuracy for design compared to the uncertainties inherent to the design. The use of a weighted average p-multiplier also allows analysis of multi-directional loading with a single model, rather than having to adjust the calculation based on load orientation (Brown et al. 2001).

For this alternative approach using the average p-multiplier, the calculated maximum bending moment based on the average p-multiplier value may be less than the actual bending moment in a particular row, especially the leading row. To account for this, a simple overstress allowance can be applied to increase the maximum bending moment based on the average p-multiplier value. The overstress allowance is based on spacing of the deep foundations as follows (Brown et al. 2001; Brown et al. 2010):

- Foundation elements spaced 3B center to center, $M_{max} = 1.2 * M_{max,\ average}$
- Foundation elements spaced 4B center to center, $M_{max} = 1.15 * M_{max,\ average}$
- Foundation elements spaced 5B center to center, $M_{max} = 1.05 * M_{max,\ average}$

Where B is the width or diameter of the foundation element. All foundation elements are then designed with the same structural design based on the M_{max} value; i.e., all piles have the same size and section or all drilled shafts have the same reinforcing.

The methods described above for group analysis based on an individual p-y analysis can be used where more robust software for group analysis is not available, or for preliminary analyses, or for simple, routine, or non-critical structure analyses. This method is based on use of individual pile analysis and therefore has a number of shortcomings compared to the more robust software programs specifically designed for analysis of deep foundation groups. Examples of such shortcomings are that this method does not account for the cap and resulting pile cap effects that may influence pile head deflections and load distribution, or the potential for the inclusion of battered piles.

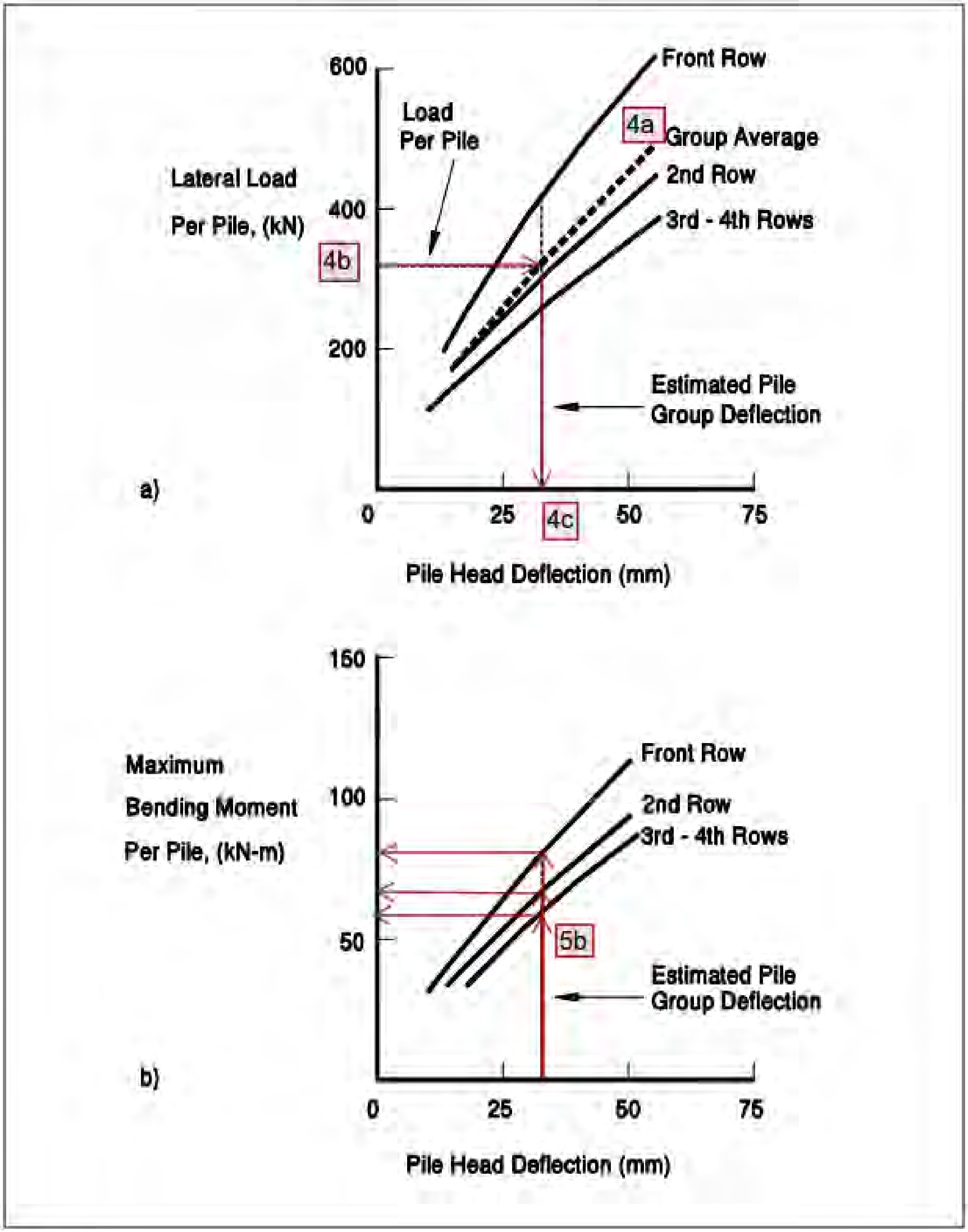


Figure 7-7: Example plots of load versus pile head deflection pile and bending moment versus pile head deflection for pile group analysis using single pile p-y analysis (modified from Hannigan et al. 2006, originally adapted from Brown and Bollman 1993).

7.4.2 Combined Lateral and Axial Loads from Frame Action

Design of groups of deep foundation elements requires analysis of the group behavior to determine the distribution of forces to the individual foundation elements from the combined axial, lateral, and overturning loads applied to the group. An efficient group will distribute loads to the foundation elements such that all of them are effectively utilized for providing appropriate load resistance. Calculation of load distribution may be done using one of the following approaches (Brown et al. 2010):

7. Simple static equilibrium – This approach may be used for very simple group arrangements. The following conditions and assumptions apply:
    a. Applicable if the group is very simple and can be modeled as a determinate simple frame.
    b. Loads are calculated using the equations of static equilibrium.
    c. A weighted average p-multiplier is assumed to apply. The weighted average p-multiplier approach is discussed in Section 7.4.1
    d. Rotational restraint provided by the pile/shaft head to the rotation of the cap is ignored.
    e. The moment applied to the cap is assumed to be resisted solely by pile/shaft axial forces.
    f. Axial stiffness of all piles/shafts are all assumed to be the same.
    g. Applies to vertical foundation elements only.
8. Elastic Solution – Individual deep foundation elements are modeled as springs and the cap is considered to be rigid. The following conditions and assumptions apply:
    a. The axial, transverse, and rotational stiffness of each foundation element are modeled as simple elastic springs.
    b. Foundation elements can be modeled as having different stiffness, or can be assumed to have the same stiffness if a weighted average p-multiplier is used (as discussed in Section 7.4.1).
    c. The cap is assumed to deform as a rigid body.
    d. The rotational restraint of the foundation element head to rotation of the cap can be included in the model.
    e. Applies to vertical foundation elements only.
9. Nonlinear Solution – Requires a nonlinear computer code for group analysis of deep foundations.

If the simple static solution or simple elastic solution is used to determine the loads applied to each individual foundation element in the group, the foundation element is analyzed for these loads using the average p-multiplier in a p-y analysis, as described in Section 7.4.1. The simple static solution and simple elastic solution are described in detail, including equations for the simple elastic solution and a calculation example of each, in Brown et al. (2010).

The more widely used approach is to use the nonlinear solution, which requires use of specialty computer software analyses. In preliminary structural analysis of superstructures supported by deep foundations it is commonly assumed that the foundation cap is rigid. However, the cap rotates around two orthogonal horizontal axes and one vertical axis, and displaces along two orthogonal horizontal axes and vertically. This trend causes the load to be redistributed non-uniformly among individual deep foundation elements within the group. As the pile cap rotates, those foundation elements in the group located off the group center induce additional vertical displacement that affect the overall response of those piles/shafts. These effects are collectively termed "pile cap effects." The effects of pile cap displacement and rotation are not commonly calculated manually; instead, these are more efficiently estimated using computer programs that account for the cap.

7.4.3 Finite Element Programs

Comprehensive finite element programs are also capable of modeling the soil around a group of deep foundation elements as a continuum and for modeling its nonlinear behavior. Such programs also include the provisions for gapping and slippage that may occur at the foundation/soil interface. Therefore, these programs not only capture the nonlinear behavior of the soil and deep foundation element, but also model the pile-soil-pile interaction.

The disadvantage of using general purpose FEM code programs is that they require a significant effort to establish a three-dimensional finite element mesh to model a foundation group. The constitutive modeling of the soil can pose technical challenges for most practitioners as well. The use of the general purpose FEM codes is generally not cost-effective for routine analysis and applications, especially when other programs are available. FEM may be worth exploring on large or very complicated projects.

7.4.4 Commentary on the Use of Computer Programs for Group Analysis

Care must be exercised when using software to perform analysis of groups of deep foundation elements. The available programs have robust capabilities for analyzing complex problems. However, these programs have limitations based on the inherent assumptions, methods, models, and default parameters used in their development, as is true of any software program. It is the responsibility of the designer to understand potential limitations or issues with the use of available software tools. Some examples of areas where care needs to be exercised with these programs are discussed below.

- P-y Curves: Software programs generally do not have the same number of p-y curves available as some of the programs for individual pile analysis. In some cases, with unusual, complex, or region-specific geomaterials, the available p-y curves in the program may not match well with the actual geomaterials. The user may need to make some simplifying assumptions regarding which P-y curve is most applicable for the subsurface materials at the project site. In such cases, it may be appropriate to perform parametric studies using multiple p-y models to assess how significant an impact these simplifying assumptions may have on the results and how sensitive the results are to variations in the soil models.
- P-multipliers: Rollins et al. (2000) performed analysis of a laterally load tested pile group and found that when the default p-multipliers in a software program were used, the computed pile-soil-pile resistance was about 35 percent higher compared to the resistance computed with p-multipliers based on the full-scale test results. Default values of p-multipliers should therefore be used with caution. The recommended approach is to use the p-multipliers in Table 7-1, or to use values based on local practice or testing programs, rather than default values in the software.

- Passive resistance of the cap: Numerous methods, including the log-spiral, Rankine and Coulomb methods, and p-y method, are available to estimate the passive resistance against the cap. Rollins et al. (2000) concluded that the log-spiral method provided the best agreement with the measured resistance. Estimates of passive pressure using the Rankine method significantly underestimated the passive soil resistance acting on the cap; whereas the Coulomb method overestimated the passive resistance on the cap. Similar conclusions were made by Duncan and Mokwa (2001).
- Other aspects of analyses of groups of deep foundation elements not directly related to lateral foundation analysis, may need to be considered in the overall analysis of the group. Examples include:
  - Settlement potential, as these programs may not adequately account for long-term settlement potential and its resulting impacts on foundation design, including downdrag loads, changes in subsurface soil properties, loss of contact with the cap, potential for unsupported pile/shaft length, and potential buckling of piles.
  - Elastic shortening of the piles/shafts under axial loads may not be addressed directly by the computer codes. For very long piles/shafts, elastic shortening of the piles may need to be evaluated.

7.5 USE OF BATTER PILES

Batter piles are piles that are installed on an inclination from vertical. Typical batter configurations are 1H:12V up to 1H:3V. Shallower inclinations may be possible but are generally more difficult to construct and should therefore be avoided.

Batter piles are considered when large static lateral loads are expected or where structural rigidity is required. Batter piles can typically resist larger lateral loads with less deflection compared to the same pile size installed as a vertical (or plumb) pile since the stiffer axial resistance of the pile contributes to resisting the applied lateral load. Deep foundations that are commonly battered for lateral load resistance include driven piles and micropiles. Battering of drilled shafts generally involves casing in order to prevent hole collapse during drilling, and to maintain alignment of the drilling tools. It is generally hard to drive or install casing for larger diameter elements (drilled shafts versus micropiles) and therefore battered drilled shafts are difficult to install and rarely used. Refer to Brown et al. (2010) for additional discussion on concerns regarding battered drilled shafts.

7.5.1 Concerns Regarding the Use of Batter Piles

In general, battered piles can increase the stiffness of the system significantly, creating an uneven distribution of loads in a cap (i.e., vertical pile will tend to absorb a modest fraction of the total lateral load), and result in uplift forces in adjacent piles. The use of battered piles may not be appropriate for cases where the system is desired to have greater flexibility, such as for earthquake loading.

The inclusion of batter piles may increase the lateral load demand that the foundation must be designed for. During a seismic event, this can result in transmission of excessively large horizontal forces to the structure. Horizontal movements from seismic events generate load, and because force is a function of stiffness, the higher stiffness for a foundation group with battered piles results in larger horizontal forces being transmitted to the structure compared to a foundation group with only vertical piles. This was observed during the 1989 Loma Prieta earthquake (Hadjian et al. 1992). Therefore, in seismically active areas, the use of batter piles should generally be avoided.

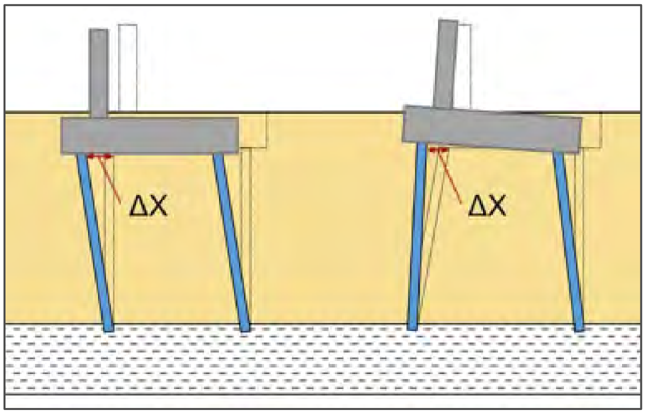
Batter piles should also be avoided in areas where significant settlement or downdrag is expected. Settlement that occurs after pile installation will increase the bending moment along the pile length and potentially damage the pile. In the event that batter piles must be included in areas with expected downdrag, the additional loads induced by downdrag must be incorporated into the structural analysis of the piles.

Construction of batter piles may also cost more compared to vertical piles, both in terms of the cost to install, the need for inspection to achieve the required pile location and batter. Bollman (1993) reported that the Florida DOT often uses only vertical piles to resist lateral loads, including ship impacts, because of the higher costs associated with batter piles.

The potential for batter piles to impact adjacent foundations, structures or facilities also needs to be carefully considered. The batter pile angle and potential deviations, as well as potential pile driving effects such as densification along the pile length, should be considered for impacts to adjacent properties or facilities. Batter piles may interfere with adjacent structures, especially underground structures such as basements, utilities, foundations, or tunnels. Coordination between temporary and permanent works must also be considered. Batter piles may interfere with temporary works such as support of excavation systems or utility relocations. Batter piles for temporary works, such as a temporary causeway or trestle for access, may interfere with existing or proposed facilities or structural elements, such as proposed permanent foundations.

7.5.2 Loads in Batter Piles

The lateral load response of a foundation group with batter piles differs from that of a foundation group with only vertical piles. For a pile group consisting of only vertical piles, the response to an applied lateral load and overturning moment results in the pile group translating horizontally and deflecting downward as shown in Figure 7-8(a). For a pile group with battered piles (outwardly battered piles in the direction of the lateral load and moment), the response to the same lateral load and overturning moment results in the pile group translating horizontally and displacing vertically upward as shown in Figure 7-8(b). Therefore, the inclusion of battered piles will influence the overall design of the foundation group and the loads and reactions for each pile (Wilson et al. 2006).



(a) (b)

Figure 7-8: Variation of foundation response for vertical and batter piles (modified from Wilson et al. 2006).

The additional lateral load resistance from a batter pile, compared to a vertical pile, comes from the horizontal component of the axial load in the batter pile. The horizontal component of the axial compression load contributes to the overall lateral load resistance of the group when the piles are battered outward, away from the load. In cases where the piles are battered inward, towards the load, the horizontal component of the axial load will reduce or subtract from the lateral load resistance of the group.

The analysis for a pile group with battered piles is similar to that for a group of vertical piles as described previously, but with the following modifications as described by Wilson et al. (2006):

1. Assuming the cap is rigid, the axial load for each pile is determined from the vertical load at each pile and the batter angle. The horizontal component of the axial batter pile load is then determined for each batter pile. This resistance to this horizontal load component (for outwardly battered piles) will provide resistance to applied lateral loads.
2. The sum of all the horizontal batter pile loads is determined. If this sum is less than the applied lateral load, then the remaining lateral load component must be resisted by the piles through bending as determined from a p-y analysis. If the sum of horizontal load components is greater than and in the opposite direction to the applied load, then the net horizontal load in the piles will result in displacement and bending towards the applied load.

It is important to carefully assess the applied horizontal loads and vertical load distribution within the pile group when determining the location and orientation of batter piles to ensure that the batter piles are used efficiently (i.e., contribute to lateral load resistance through horizontal axial components as well as bending). Considering this and the concerns indicated previously for batter piles, when designing a pile group, it is recommended to design the group first using vertical piles only. If the lateral load resistance cannot be fully addressed with only vertical piles, or if this approach requires a significant increase in the number of piles then batter piles can be incorporated into the design, with careful consideration of the pile arrangement to maximize the efficiency of the batter piles through axial loads and bending.

7.5.3 P-y Analysis of Batter Piles

Batter piles can be analyzed using the p-y method and with the inclusion of a modifying constant based on the direction of applied lateral load relative to the orientation of the pile batter. The modifying constant for batter ranges from 0 to 2 and modifies the p_{ult} value, which causes the rest of the p-values in the p-y analysis to be modified accordingly. The value of the modifying constant varies from 0 to 1 if the pile head is inclined toward the applied lateral load. This reduces the p value, which reduces the deflection, in effect increasing the lateral resistance of the pile compared to a vertical pile. The modifying constant varies from 1 to 2 if the pile head is inclined away from the applied lateral load, which results in the pile deflecting more than a vertical pile under the same loading conditions. The variation of the modifying constant with batter angle is shown in Figure 7-9.

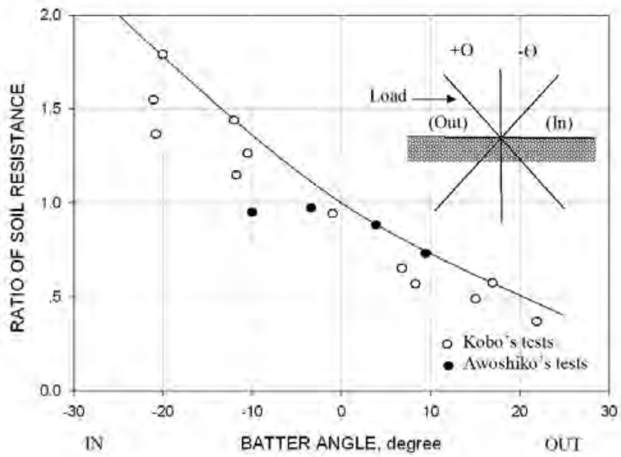


Figure 7-9: Modifying constant for p-y curves for batter piles (from Reese et al. 1974).

The modifying constant values are based on models (Awoshika and Reese 1971) and full-scale load tests (Kubo 1964). The modifying constant is therefore empirical in nature. Most computer software programs for the p-y method generally include the modifying constant within the software capabilities. The data for the basis of the constant agree well for outwardly battered piles (pile head inclined toward the load) compared to inwardly battered piles (pile head inclined away from the load).

7.6 OTHER CONSIDERATIONS FOR GROUPS OF DEEP FOUNDATION ELEMENTS

Analysis of deep foundation groups must consider a number of other aspects that are beyond the scope of this manual. This includes axial design of the individual foundation elements as well as consideration of the vertical resistance for the foundation group. This includes aspects of group axial design such as load transfer through the cap and structural connections to the piles and columns. Considerations relative to axial group loads, such as bearing resistance, settlement, the potential for block failure, and axial uplift or tension design considerations must also be addressed.

Settlement may cause a loss of contact between the ground and pile cap, which should be considered in the lateral resistance acting on the pile cap (lack of friction contact on the base of the cap). Settlement can also result in downdrag forces, which must be accounted for in the design of the deep foundations.

The spacing between deep foundation elements with regard to interaction and constructability issues must also be considered. In general, a minimum center to center spacing of 3 pile/shaft diameters, with a minimum of 3 feet, is recommended to provide more efficient pile group resistance and reduce the risk of constructability issues. Driving of piles may cause settlement in the area immediately around the pile in cohesionless soils. In cohesive soils, pile driving may result in heave of the surrounding ground and previously installed piles, or cause excess pore pressure build-up and a short-term decrease in group axial and lateral efficiency in cohesive soils. A wider spacing will reduce pile-soil-pile interaction from the standpoint of lateral load resistance, and will further reduce the potential for construction impacts between foundation elements.

A detailed discussion of these considerations is beyond the scope of this manual. Refer to Brown et al (2010) and Hannigan et al (2016) for further discussion on the design and construction of drilled shaft and driven pile foundations, respectively.

8 DESIGN FOR EXTREME EVENTS

8.1 INTRODUCTION

Extreme Event Limit States involve events with a low probability of occurrence. Such events are considered to be unique events whose return period may be significantly greater than the design life of the structure (AASHTO 2014). The Extreme Event Limit State is intended to ensure the survivability of the structure during such an event. This Limit State is to protect against collapse of the structure and loss of life; some damage or loss of functionality may be acceptable in such a case.

The resistance factors for extreme events when analyzing lateral resistance are 0.80 as discussed in Chapter 4 (Brown et al. 2010). The load factor is 1.0 (AASHTO 2014).

Extreme Event Limit States for design of deep foundations include the following:

- The check flood for scour
- Earthquakes/seismic events
- Loading from ice
- Vessel collision
- Vehicle collision

Note that the check flood is an extreme event, or super flood, with an estimated return period of approximately 500 years. It is not to be confused with scour from the 100-year flood, which is evaluated as part of the Strength and Service Limit States.

Arneson et al. (2012) provides detailed information regarding scour development near bridge piers and abutments. Kavazanjian et al. (2011) presents detailed information regarding seismic analysis and design of foundations for transportation facilities. AASHTO (2014) design specifications provide additional detail regarding the determination of vessel impact, vehicle impact, and ice loads.

8.2 EXTREME EVENT SCOUR (CHECK FLOOD)

Structures that are constructed over or adjacent to bodies of water and are subject to scour must be designed for the extreme scour event or check flood. This typically corresponds to a 500-year event return period. Arneson et al. (2012) provides details on design assessments of the scour conditions. AASHTO (2014) indicates that the foundation must be designed for the applied loads on the structure as well as any debris loads during the flood event.

Based on AASHTO, (2014), there is no separate load combinations that is applicable to the extreme event scour condition. However, load factors specific to extreme event conditions apply, and potentially deeper or wider scour conditions must be considered. The design for the extreme scour condition differs from the 100-year scour condition as follows:

1. The scour prism for the extreme flood condition is typically deeper and wider than for the 100-year flood condition. Referring to the process outlined in Chapter 5 for design of laterally loaded deep foundations, this will require updating the geotechnical conditions for analysis (Block 5).
2. The load factors for the structural loads are different for the Extreme Limit States compared to the Strength and Service Limit States. In general, load factors for Extreme Limit States are 1.0, whereas

load factors for Strength and Service Limit States may be greater than 1.0. Refer to Chapter 4 for the specific load factors and load combinations for each Limit State.

3. Resistance factors differ for the axial resistance for the Extreme Event Limit State. A resistance factor of 1.0 is applicable for axial and lateral resistance and 0.8 for uplift resistance for Extreme Event Limit States. Although a resistance factor of 1.0 is also used for p-y analyses for other Limit States, the often greater factored loads for the extreme Limit State may impact the design of the deep foundations in terms of foundation type, length or size. Also, as discussed in Chapter 4, other considerations may be applicable to the selection of resistance factors, and a value of less than 1.0 may be considered for the Extreme Event Limit States.

Deep foundations should be designed such that they are deep enough to account for the loss of resistance during the extreme scour event. As outlined in Chapter 5, the design for lateral loads should include a “pushover” analysis to check the overall stability of the deep foundation elements and verify that the foundations are embedded deep enough to avoid a pushover failure and behave as ductile elements under lateral loads, even under extreme conditions.

Additional considerations for extreme scour conditions with regard to load combinations and development of the scour profile can be found in Arneson et al. (2012). Specific considerations for scour related to drilled shafts can be found in Brown et al. (2010) and for driven piles in Hannigan et al. (2016).

8.3 SEISMIC

8.3.1 Equivalent Static Seismic Force

Structures must be designed for seismic loads and liquefaction potential. AASHTO (2014) presents a site-specific procedure for approximating seismic loading by using a dimensionless elastic seismic response coefficient. This coefficient depends on the acceleration coefficient, site effects, and the predominant period of vibration of the structure. From this coefficient, the equivalent static horizontal seismic force, $P_e(x)$ can be determined depending on the equivalent weight of the superstructure (Hannigan et al. 2016). Alternatively, more complex methods can be used for critical structures located in areas of high seismic risk. The following paragraphs present a description of the procedure for estimating seismic loads and guidance on the analysis procedures used in the seismic design of deep foundations.

The first step is to define the site ground coefficient and spectral coefficients based on site conditions. The site peak ground acceleration coefficient, PGA, short period spectral coefficient, S_s, and long period spectral coefficient, S_1, are determined from USGS contour seismic maps, presented in AASHTO (2014) Section 3.10.2.1. The USGS website has a tool for determining these coefficients based on location and site classification (Hannigan et al. 2016).

Results of the subsurface investigation can be used to determine the site classification using methods outlined in AASHTO (2014) Section 3.10.3.1. Site factors corresponding to the zero-period, short-period and long-period ranges of acceleration are specified by site class for various PGA, S_s, and S_1 coefficient values in AASHTO (2014) Section 3.10.3.2. Using values determined in the previous steps, the design five-percent-damped-design response spectrum can be created using procedures in AASHTO (2014) Section 3.10.3.3.

The elastic seismic response coefficient can then be determined as follows:

For periods less than or equal to, the elastic seismic coefficient for the *m*th mode of vibration, C_{sm}, shall be taken as:

$$C_{sm} = A_s + (S_{DS} - A_S)(\frac{T_m}{T_0}) \qquad \text{(Equation 8-1)}$$

Where:

$A_s = F_{pga}PGA$.

$S_{DS} = F_aS_s$.

PGA = Peak ground acceleration coefficient on rock (Site Class B).

S_s = Horizontal response spectral acceleration coefficient at 0.2-sec period on rock (Site Class B).

T_m = Period of vibration of mth mode(s).

T_0 = Reference period used to define spectral shape = $0.2T_s$(s).

T_s = Corner period at which spectrum changes from being independent of period to being inversely proportional to period = S_{D1}/S_{DS}(s).

For periods greater than or equal to T_0 and less than or equal to T_s, the elastic seismic coefficient, C_{sm}, shall be taken as:

$$C_{sm} = S_{DS} \qquad \text{(Equation 8-2)}$$

For periods greater than T_s the elastic seismic coefficient, C_{sm}, shall be taken as:

$$C_{sm} = S_{D1}/T_m \qquad \text{(Equation 8-3)}$$

Where:

$S_{D1} = F_vS_1$.

S_1 = Horizontal response spectral acceleration coefficient at 1.0 sec period on rock (Site Class B).

The elastic seismic response coefficient can then be used to determine the equivalent static force using Equation 8-4 (Hannigan et al. 2016).

$$P_e(x) = C_{sm}W \qquad \text{(Equation 8-4)}$$

Where:

$P_e(x)$ = Equivalent static horizontal seismic force acting on superstructure.

C_{sm} = Elastic seismic response coefficient (dimensionless).

W = Equivalent weight of the superstructure.

Once the equivalent static force is determined, the structural engineer applies the force to the superstructure following the procedure described in AASHTO (2014) Section 4.7.4.3.

Table 8-1 presents the seismic zone of a bridge depending on the coefficient S_{D1} (Hannigan et al. 2016). For multispan bridges with a seismic zone of 2 through 4, a liquefaction assessment is required.

The factored loads resulting from the seismic analysis should be applied to the foundation and analyzed as outlined in Chapter 5. Methods described in Chapters 6 and 7 can be used as appropriate for individual and group analyses.

Table 8-1: Seismic zones (Hannigan et al. 2016).

Acceleration Coefficient, $\boldsymbol{S_{D1}}$	Seismic Zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

8.3.2 Liquefaction

Ground failure due to liquefaction and/or seismic settlement beneath and near structures supported by deep foundations can cause significant damage to these structures. Liquefaction may occur for relatively large magnitude earthquakes in loose, saturated sands. However, other materials (e.g., some gravels and silts) can also liquefy. Several factors affect the liquefaction potential, including: (i) geologic age and origin of deposit; (ii) fines content and plasticity index; (iii) degree of saturation of deposit; (iv) depth below ground surface; and (v) soil penetration resistance. The consequences to a deep foundation due to liquefaction include loss of lateral and vertical capacity and lateral spreading or lateral flow.

Liquefaction potential should be addressed for structures where the peak earthquake acceleration will be greater than 0.1g (Hannigan et al. 2016). Foundations should be designed to accommodate loss of resistance, increased loads, displacements, and drag force resulting from liquefaction. An alternate option is to mitigate liquefaction potential through ground improvement techniques.

Where liquefaction potential exists, foundations should be designed to extend below the zone of liquefaction with adequate resistance in the underlying deposits. Residual strength properties can be assigned to liquefiable layers to evaluate compression and uplift resistances during an earthquake. Both axial and lateral loads should be analyzed using the residual strength of liquefiable soils. Residual strength can be estimated by applying an equivalent clean sand blow count, determined using Equation 8-5, to Figure 8-1 (Hannigan et al. 2016).

$$(N_1)_{60-e} = (N_1)_{60} - N_{corr} \qquad \text{(Equation 8-5)}$$

Where:

$(N_1)_{60\text{-}e}$ = Equivalent clean sand blow count.

$(N_1)_{60}$ = SPT N value corrected for energy and overburden stress.

N_{corr} = Correction for percent fines (see Figure 8-1).

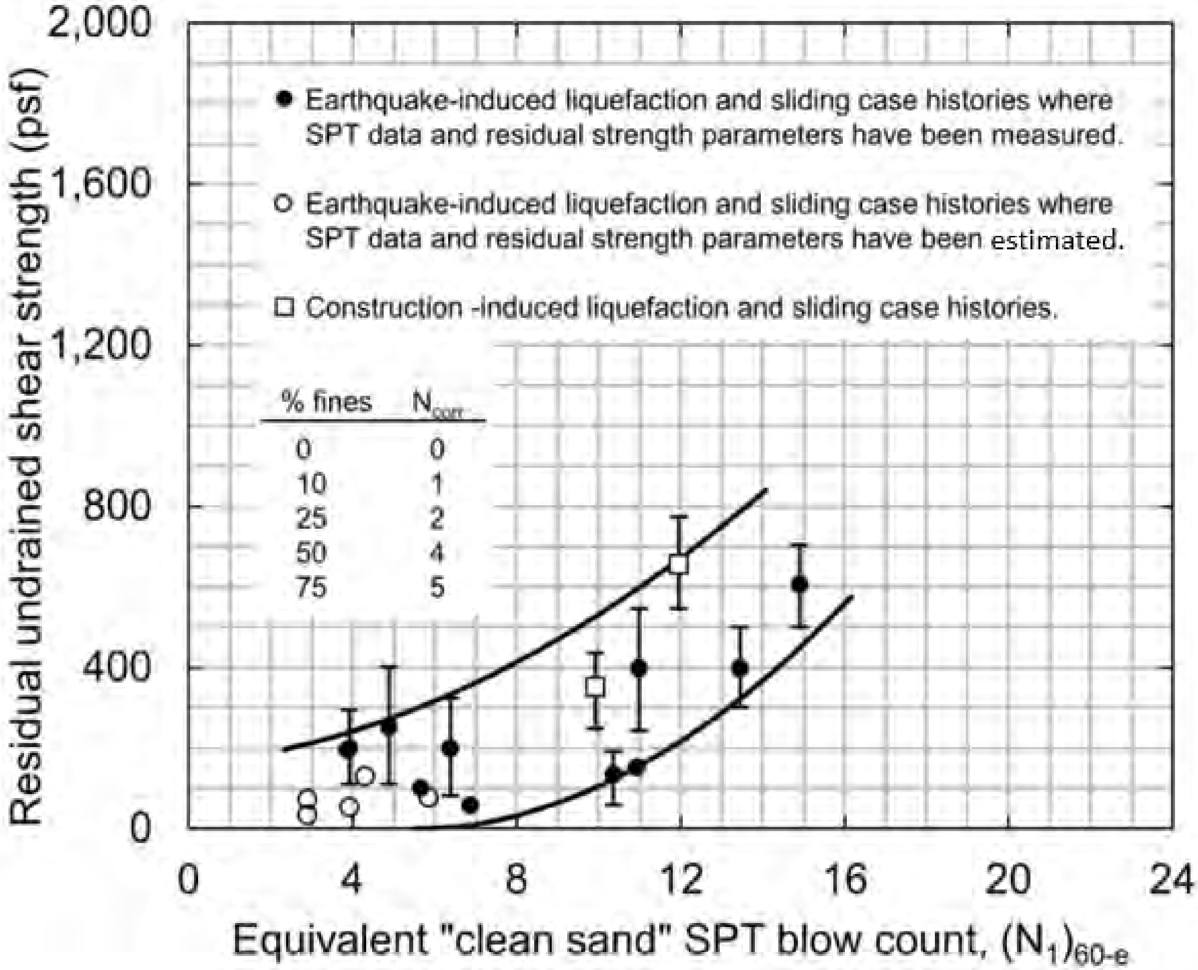


Figure 8-1: Corrected blow count versus residual undrained shear strength (Brown et al. 2010).

Liquefaction may also result in settlement, where the liquefied layer will generally consolidate as pore water pressure dissipates (settlement may also occur due to soil loss from sand boils to the ground surface.) If liquefaction occurs below the neutral plane, the axial compression load in the pile will increase along with additional pile settlement. The pile foundation must be structurally capable of supporting the increased drag force. A check should also be performed to ensure any additional settlement is within the structure's performance criterial. Settlement magnitude can be estimated using procedures in Kavazanjian et al. (2011).

Liquefaction can also cause lateral deformations due to lateral spreading or lateral flow as discussed in Kavazanjian et al. (2011). If there is potential for sliding, lateral spreading or lateral flow, foundations should be designed to accommodate the associated lateral loads. P-y curves can be adjusted to evaluate the bending moments and deformations. Reese's soft clay p-y curve with low residual shear strength and high ε_{50} values in the liquefiable layer is often used to evaluate the maximum bending moment (Hannigan et al. 2016). Hannigan et al. (2016) also recommends using p-multiplier values of 0.05 in loose sand and 0.30 in dense sand to model foundation p-y response in fully liquefied granular layers.

8.3.3 *Time-History Analysis*

For bridges classified as Seismic Zone 4, or for bridges that are geometrically complex or close to an active fault, a time-history structural analysis of the bridge is required by AASHTO (Brown et al. 2010). This analysis involves development of a site-specific time history that is used in structural modeling and analyses as discussed in Kavazanjian et al. (2011). This type of analysis is complex and must be performed by a specialist.

8.4 DESIGN FOR ICE AND COLLISIONS

Bridges and other structures may need to be designed to withstand the force effects of ice and collision of vehicles or vessels (ships or barges). These are addressed under Extreme Event II conditions in AASHTO (2014) and are treated as independent events, although they may be treated in tandem with other loading conditions, such as wind or water loads.

These loads are typically applied to the structure, either the superstructure, columns/piers, or the pile/shaft cap. Unlike the scour and earthquake extreme event cases, these cases do not result in changes to the geotechnical conditions or resistance. Structural analyses are performed to resolve these loads into force effects at the top of the deep foundation elements for analysis of the individual foundation elements and foundation group. A detailed discussion of such structural analyses is beyond the scope of this manual but can be found in AASHTO (2014). Once the force effects are determined at the top of the foundation element, lateral load analysis is performed as discussed in Chapters 6 and 7 for individual foundation elements and groups.

Within the LRFD design framework, the equation for the Extreme Event Limit State II is as follows:

$$\sum \gamma_i Q_i = \gamma_p DL + 0.5LL + 1.0WA + 1.0FR + 1.0(IC \ or \ CT \ or \ CV) \qquad \text{(Equation 8-6)}$$

Where:

γ_i = Load factor.

Q_i = Force effect.

γ_p = Load factor for permanent loads (from Chapter 4).

DL= Dead load.

LL= Live load.

WA = Water load.

FR= Friction load.

IC= Ice load.

CT = Vehicular collision load.

CV= Vessel collision load.

The last term in the equation indicates that the loads from ice, vehicular collision, and vessel collision are treated as separate extreme events and not combined. However, the equation indicates that these extreme events are considered in conjunction with other loads, such as dead loads, live loads, water loads, and friction loads.

The resistance factor for lateral load resistance under extreme loads is 1.0, unless site or project specific considerations apply as discussed in Chapter 4.

8.4.1 Ice Loads

AASHTO (2014) includes consideration for ice loads on bridge piers in fresh water lakes and rivers; saltwater ice loading conditions requiring specialized analysis, are beyond the scope of the AASHTO specifications and this manual.

The expected ice forces are assumed to act directly on the bridge piers and are expected to include the following modes of action:

- Dynamic pressure resulting from moving sheets of ice or ice flows carried by stream, wind, or currents.
- Static pressure due to thermal movement of ice sheets
- Pressure resulting from hanging dams or jams of ice
- Static uplift or vertical loads resulting from adhering ice in fluctuating water levels.

AASHTO (2014) Section 3.9 provides a detailed discussion of the development of both lateral and vertical ice loads. Lateral ice loads may be dependent on the thickness of the ice flow, the crushing strength of the ice and the potential failure mechanism of the ice flow, the width of the pier, the inclination of the pier nose, the size of the stream, and the angle of the pier relative to the ice flow, among other factors.

The forces from ice may be treated as dynamic forces or static forces according to the list above. Dynamic forces for impacts of ice sheets or ice flows should be considered in a dynamic analysis of the bridge model. Such an analysis should consider the potential for resonance between the bridge pier and the ice forces, and should consider the damping coefficient that is applicable to the piers. AASHTO (2014) indicates that Montgomery et al. (1980) found that flexible piers and foundations may cause considerable amplification of dynamic ice forces due to resonance between the ice and the structure at low levels of structural damping. Brown et al. (2010) indicate that drilled shafts may offer an advantage in such a situation compared to driven piles due to relatively high structural rigidity and higher damping than piles.

Static loads due to static pressure from thermal ice sheet movement or hanging dams or jams are treated as static loads acting directly on the bridge piers. These loads are incorporated into static structural analyses to determine the applicable force effects acting on the foundation group and elements. Once the loads on the foundation are determined, the analysis for lateral loads proceeds as a conventional lateral load analysis according to the procedure in Chapter 5 and the analysis methods in Chapters 6 and 7.

Vertical and uplift forces due to ice must also be considered, but are beyond the scope of this manual. These are discussed in more detail in Brown et al. (2010) for drilled shaft and Hannigan et al. (2016) for driven piles, with detailed procedures for determining structural loads presented in AASHTO (2014).

8.4.2 Vehicular Collision Loads

AASHTO (2014) requires that bridge abutments and piers located 30 feet from the edge of roadway be investigated for vehicle collision loads. Collision loads may be addressed by either providing crash protection or by providing structural resistance to the load. Crash protection may be provided by an embankment, or a structurally independent crash-worthy ground mounted barrier. Refer to AASHTO (2014) for additional requirements for crash protection barriers, including height and offset distance requirements, loading conditions, annual frequency of impact, etc.

For cases where the vehicular crash loads are resisted through structural resistance, the crash load is modeled as an equivalent static force of 600 kips acting at a direction of zero to 15 degrees with the edge of pavement in a horizontal plane at a distance of 5 feet above the ground. This load is based on full scale crash tests using tractor trailers. For individual columns, the load should be considered as a point load. For wall piers or abutments, the load may be distributed over a suitable area, but the area should be no larger than 5 feet wide by 2 feet high (AASHTO 2014).

The requirements outlined above are for bridges impacted by trucks. Other vehicular impacts, such as trains, should be addressed on a case by case basis and in accordance with applicable codes or design standards, such as the AREMA Manual for Railway Engineering or local transit agency guidelines.

As indicated above, the vehicle collision force (CT) is applied as a load to the structure. A structural analysis is required to resolve the force effects of the collision force to reactions at the head of the foundations element(s). Once the applied loads at the top of the foundation elements are determined, a lateral load analysis is performed as previously described. Because the CT load is applied as an equivalent static load, there are no additional special considerations in the foundation analysis; i.e., no dynamic or cyclical load considerations.

8.4.3 Vessel Collision Loads

Collision of a ship or barge with a bridge structure can be a severe loading condition that may result in collapse of the bridge. AASHTO (2014) requires that all bridge components in navigable waterways, located in water depths greater than 2 feet, be designed for vessel impact. The requirements in AASHTO (2014) have been adapted from the AASHTO Guide Specifications and Commentary for Design for Vessel Collision Design of Highway Bridges (1991) using the Method II risk acceptance alternative, and modified for the second edition (2009).

Vessel collision loads are determined based on the selection of a design vessel and considerations of the vessel and waterway relative to the bridge, such as the waterway geometry, the waterway depth, the size, type, loading, and frequency of vessels in the waterway, the vessel speed and direction, and the structural response of the bridge to the collision. The determination of the design vessel involves probabilistic analyses and risk assessments, and generally requires a multi-disciplinary team. These analyses are beyond the scope of this manual; refer to AASHTO (2014) for detailed information on the determination of the design vessel. Once a design vessel is selected, the design weight tonnage (DWT) is determined and the head-on ship collision impact force on a pier can be determined as follows:

$$P_s = 8.15V\sqrt{DWT} \qquad \text{(Equation 8-7)}$$

Where:

P_s = Equivalent static vessel impact force (kips).

DWT = Deadweight tonnage of vessel (tonne).

V = Vessel impact velocity (ft/sec).

Since the design load is treated as an equivalent static force, the lateral loading analysis for the foundation is performed using static loads rather than dynamic or cyclical loads. Two design cases are assessed for substructure design once the equivalent static force is determined (AASHTO 2014):

- 100 percent of the design impact force is applied in a direction parallel to the alignment of the centerline of the navigable channel
- 50 percent of the design impact force in the direction normal to the direction of the centerline of the channel.

All components of the substructure exposed to physical contact by any portion of the vessel are to be designed to resist the applied loads. The assessment should consider the geometry of the vessel in determining the portions of the substructure that may be in contact with the vessel, as well as crushing of the bow of the vessel.

The impact force is applied as follows (AASHTO 2014) for each of the two design cases described above:

- For overall stability, the design impact force is applied as a concentrated force on the substructure at the mean high water level (MHL) of the waterway as shown in Figure 8-2.
- For local collision forces, the design impact force is applied as a vertical line load equally distributed along the vessel's bow depth as shown in Figure 8-3. The vessel's bow is considered to be raked forward in determining the potential contact area of the impact force on the substructure.
- For a barge impact, the local collision force is taken as a vertical line load equally distributed on the depth of the head block as shown in Figure 8-4.

The foundations are designed for the force effects calculated for all the above conditions (parallel and normal to centerline, for concentrated and distributed force applications). The most critical case governs the design.

Additional vessel impact collision loads may include collision with the bow, deckhouse, or mast with the superstructure. As these loads are directly applied to the superstructure, a structural analysis is required to develop the force effects at the head of the deep foundation elements. Once the force effects on the foundations are determined, a lateral loading analysis can be performed according to the procedures previously outlined in this manual.

Protection against vessel impact loads can be provided by inclusion of physical protection systems such as fenders, pile clusters, dolphins, islands, or other measures. Such measures may reduce or eliminate the vessel collision forces applied to the bridge structure and its foundation. The design of such protective systems usually involves an iterative process to evaluate the energy absorption capacity of the system (including flexure, torsion, shear, and displacement of the system components) versus the kinetic energy of the vessel (AASHTO 2014). This type of analysis is beyond the scope of this manual.

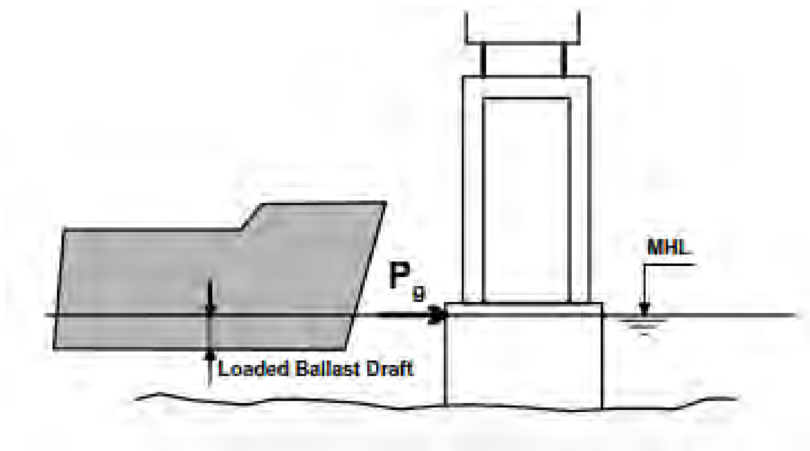


Figure 8-2: Ship impact concentrated force on pier (after AASHTO 2014).

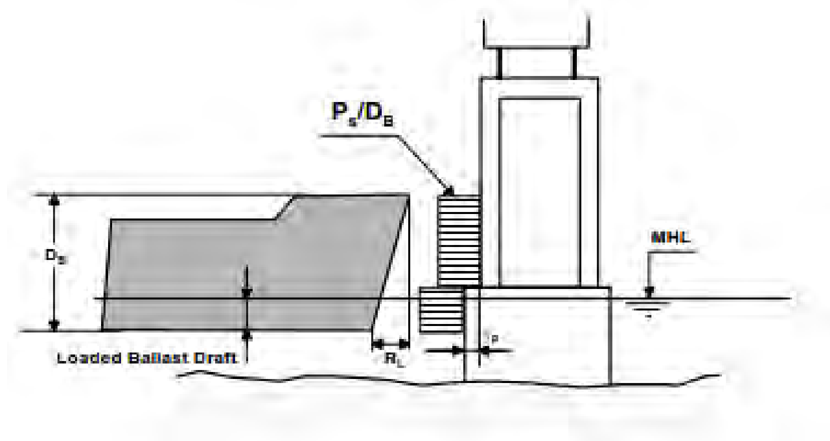


Figure 8-3: Ship impact line load on pier (after AASHTO 2014).

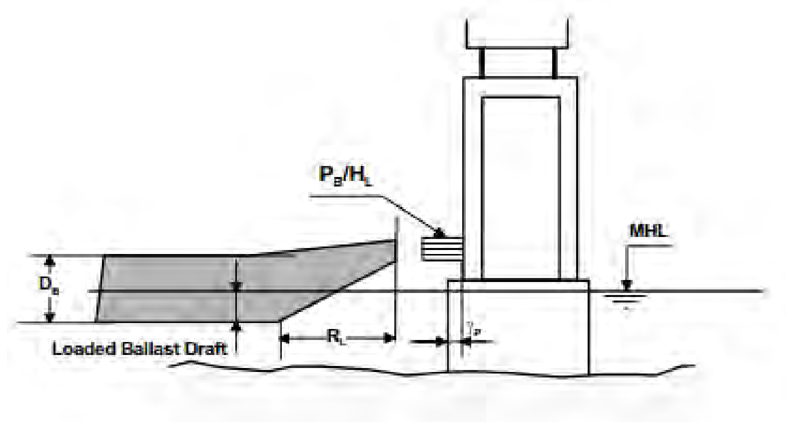


Figure 8-4: Barge impact force on pier (after AASHTO 2014).

8.5 COMBINATIONS OF EXTREME EVENTS

AASHTO (2014) provides two Extreme Event combinations. Extreme Event I is a combination of scour with earthquake loading, and Extreme Event II is a combination of scour with ice, vehicle and vessel collision loads, and hydraulic loads. As noted in Arneson et al. (2012), the Extreme Event I combination has a low occurrence probability for both the check flood and earthquake loading. Therefore, scour for the mean discharge or normal non-flood flow may be applied to this event combination. For the Extreme Event II combination, research is ongoing to assess the probability of joint loading conditions during the check flood, and judgement should be used based on site-specific factors. If ice or debris jams near the structure dictates the use of a more extreme flood event than the check flood, this may be used to assess the extreme event limit (Hannigan et al. 2016).

9 DESIGN FOR EARTH RETENTION STRUCTURES

9.1 OVERVIEW

The subject of this chapter is limited to non-gravity cantilevered walls with discrete vertical wall elements, such as drilled shafts, driven piles, or drilled-in piles, embedded below finish grade to provide passive resistance to the retained earth. Often referred to as top-down walls, soldier pile walls, or beam and lagging walls, the supporting drilled shafts or piles are spaced center-to-center generally at least three effective diameters apart, with fascia lagging, precast concrete panels, or temporary lagging with a cast-in-place concrete facing, positioned within the interval between the above grade portion of the vertical elements, as shown by Figure 9-1. Center-to-center spacing of vertical elements less than three diameters may result in overlapping passive stress influence between the supporting elements. Methods of analysis for continuous vertical wall elements (secant pile or tangent pile walls) are discussed in Brown et al. (2010).

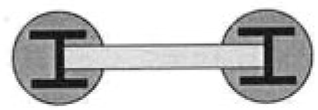


Figure 9-1: Soldier pile wall.

9.2 EARTH PRESSURES

Prior to a performing a p-y analysis for the discrete supporting elements, active earth pressures imposed above the base level of the fascia lagging or panels must be resolved into a resultant force, inclusive of any surcharge loads acting above the wall. AASHTO Section 3.11 describes the un-factored resultant active earth pressure (Pa) for design of permanent non-gravity cantilevered walls as follows:

$$\mathrm{Pa} = \frac{Ka\,\gamma'\,l\,H^2}{2} \qquad \text{(Equation 9-1)}$$

acting at H/3 above the design base grade, defined by the bottom level of the panel or lagging below the finish grade in front of the wall, where

H = Total wall height measured from the panel or lagging base level

γ' = Effective unit weight of the soil retained by the wall

l = Contributory length of wall relative to each vertical element (center-to-center spacing)

The active earth pressure coefficient (Ka) in Equation 9-1 is derived from Coulomb earth pressure theory, including wall friction as follows:

$$k_a = \frac{\sin^2(\theta + \emptyset'_f)}{r[\sin^2\theta\,\sin(\theta - \delta)]} \qquad \text{(Equation 9-2)}$$

In which:

$$r = \left[1 + \sqrt{\frac{\sin(\emptyset'_f + \delta)\sin(\emptyset'_f + \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2$$

(Equation 9-3)

Where:

- δ = Friction angle between backfill and wall.
- β = Angle of backfill with respect to horizontal.
- θ = Angle of back face of wall to horizontal (normally 90°).
- Φ'_f = Effective angle of internal friction of the retained soil.

The geometric variables of Equation 9-3 are illustrated by Figure 9-2.

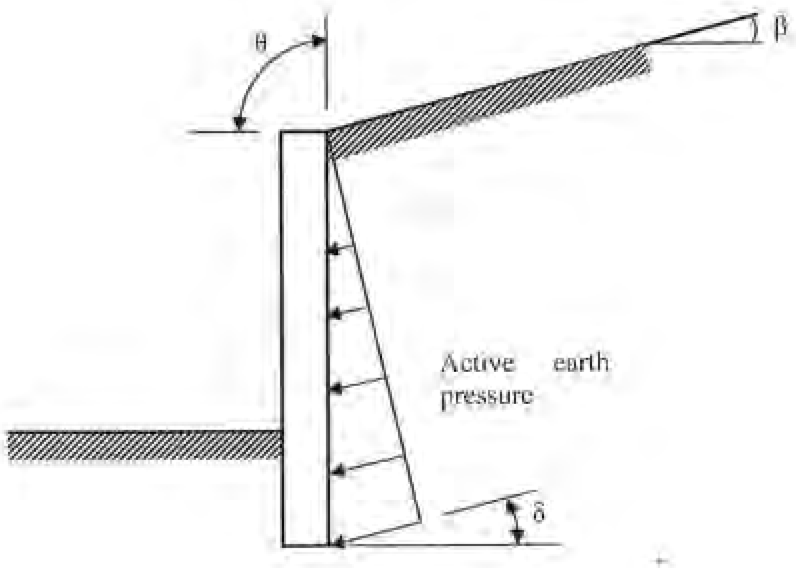


Figure 9-2: Geometry for active earth pressure.

Suggested values for friction angles provided in AASHTO Table 3.11.5.3-1 from the U.S. Department of the Navy (Design Manual 7.02, 1986) for various wall interface materials are listed in Table 9-1.

Table 9-1: Friction Angle for Dissimilar Materials.

Interface Materials	Friction Angle (δ°)	Coefficient of Friction, Tan δ
Mass concrete against the following materials:		
▪ Clean sound rock	35	0.70
▪ Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
▪ Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
▪ Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
▪ Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
▪ Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
▪ Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Steel piles against the following soils:		
▪ Clean gravel, gravel sand mixtures, well-graded rock fill with spalls	22	0.40
▪ Clean sand, silty sand-gravel mixture, single size hard rock fill	17	0.31
▪ Silty Sand, gravel or sand mixed with silt or clay	14	0.25
▪ Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete against the following soils:		
▪ Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
▪ Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
▪ Silty sand, gravel or sand mixed with silt or clay	17	0.31
▪ Fine sandy silt, nonplastic silt	14	.25

The values cited above are intended for mass concrete, steel, or precast concrete placed against the various materials listed. Some ground movement beyond that which is required to develop active earth pressure may be required to realize the full benefit of wall friction; ground movement will be influenced by the construction techniques used to install the lagging or panels between the vertical support elements.

Long-term earth pressures from stiff clays and plastic silts acting on permanent structures will generally be controlled by the effective stress strength properties. For temporary wall applications in cohesive soils, the active pressure can be determined based on total stress methods and undrained shear strength parameters; however, the active pressure shall not be less than 0.25 times the effective overburden pressure at any depth, or 0.035 ksf/ft of wall height, whichever is greater (AASHTO 2014). For soft to medium-stiff clays, lateral loads are typically governed by the undrained (total stress) strength properties as outlined within AASHTO Section 3.11. Information regarding evaluation of effective and total stress strength properties of cohesive soils is included in GEC-5 (Loehr et al. 2016).

If hydrostatic pressure behind the wall cannot be relieved using a drainage medium behind the lagging or panel fascia, water pressure must be added to that of earth pressure (with submerged unit weights for earth pressure).

9.3 DETERMINATION OF EMBEDMENT DEPTH

Once the resultant active earth pressure has been determined above the design grade level (base of lagging or panel fascia), p-y analysis software can be used to assess the embedment requirements for the vertical foundation elements, either by direct application of the factored resultant force on the pile/shaft or, depending upon the limitations of the program, by resolving the factored resultant force to an applied shear and moment at the design grade level. Design grade for passive resistance should be taken at the bottom of the fascia panel (below finish grade in front of the wall) to model the construction condition. A lower level should be considered to account for other potential disturbance during the service life of the wall, such as trenching for installation of utilities. Note that limit equilibrium methods can also be used to assess the embedment requirements for the vertical foundation elements. Following the design process in Chapter 5 of this manual (Section 5.1, Block 9.1), the required embedment can be determined based upon a geotechnical strength limit analysis for individual foundation elements, using the factored strength limit load combinations per AASHTO Section 3.4.1 and a geotechnical resistance factor (Φ) of 0.75 for passive resistance of the embedded vertical piles or drilled shafts, as per AASHTO 11.5.6-1.

9.4 EVALUATION OF DEFORMATION

After the pile or drilled shaft embedment and Structural Strength Limit State (refer to Chapter 11 of this manual) have been satisfied, the service load combinations, per AASHTO 3.4.1 can be used in a p-y analysis to assess deflection. The method of analysis for assessing deflection is discussed in Chapter 5 of this manual (Section 5.1, Block 9.3). If the p-y analysis is performed based upon resolved shear and moment at the design grade elevation, the free-head lateral translation and angular distortion computed result at the design grade level will need to be projected to the top of the wall.

10 DESIGN FOR SLOPE STABILIZATION

10.1 OVERVIEW

Methods for stabilizing slopes with deep foundation elements, independent of wall structures that buttress the slide force, include in-slope installation of: (i) drilled shaft, driven pile, or drilled-in pile systems to add shear resistance across the failure plane or a potential slip surface; and (ii) battered driven pile or micropile systems to essentially tie the soil mass of an existing slide together to the more competent ground below the plane of failure. Both design methods rely upon a comprehensive assessment of the existing slope stability, followed by modelling of the interaction between the soil and the installed deep foundation elements.

The analysis and design of laterally loaded deep foundation elements for slope stabilization applications involves the evaluation of: (i) the geotechnical resistance factor (ϕ) of the slope with the deep foundation elements installed; and (ii) the loads for structural design of the foundation elements. The geotechnical resistance factor that applies after the slope has been stabilized with deep foundation elements is not well defined because the earth pressures applied to the elements are dependent on the relative movement of the soil and the foundation elements, which depend on the geotechnical resistance, as well as the stiffness of the pile/shaft elements used to stabilize the slope.

This chapter provides a methodology for analysis and design of deep foundation elements for slide stabilization applications.

10.2 EXISTING SLOPE STABILITY

10.2.1 Data Gathering

Existing slope conditions should be assessed by review of available information and collection of geotechnical data through a subsurface exploration, including geophysics, and laboratory testing program. Data gathering typically includes the following:

- Topography – Topographic information is essential to development of cross-sections for the slope stability analysis, identification of critical areas, and establishment of limits for a landslide mitigation.
- Landslide Extent – Observations of known points of shear failure, as evidenced by head scarps, toe bulges, and surface slumps, should be carefully mapped for comparison with the topographic data.
- Subsurface Profile – Borings should be advanced to a depth suitable to gather information on soil indices and strength, stratigraphy above and below the plane of failure, and the prevailing groundwater conditions. For rapid assessment of overall conditions under emergency slide mitigation circumstances, open-hole borings may be supplemented by in-situ testing methods, such as cone penetration test (CPT) soundings. For failed slopes, laboratory strength testing may need to include development of residual soil strength data within the failure surface strata. For additional guidance on planning and execution of subsurface exploration programs for slopes, refer to Mayne et al. (2002) and Loehr et al. (2016).
- Failure Plane Identification -- Preliminary analysis of failed slopes can often be based upon the visual observation of shear failure surface features in combination with topographic data and available pre-failure subsurface information. However, to accurately assess the position of the failure surface, installation of inclinometer casings should be included in the subsurface exploration program, with

provisions for subsequent periodic monitoring. In addition, piezometers and groundwater monitoring wells can be installed to better define the groundwater conditions.

10.2.2 Geotechnical Resistance Factors for Slope Stability

Slope stability is evaluated at the AASHTO (2014) Service I Load Combination relative to geotechnical resistance factors that are the inverse of the factor of safety (FS) computed by the various software available for slope analysis. In practice, the target geotechnical resistance factors (ϕ) of 0.75 and 0.65, as referenced in 11.6.2.3 of AASHTO (2014), are equal to a factor of safety (FS) of 1/Φ, or FS 1.33 and 1.53, respectively. For consistency with the literature, analyses referred to herein are based on the use of estimated soil strength values and a factor of safety (FS) Initial Geotechnical Resistance Assessment

Using the information from the data gathering phase described in Section 10.2.1, analyses of the overall slope may be performed using a limit equilibrium approach such as the Modified Bishop, Simplified Janbu, or Spencer methods, as available in several different geotechnical analysis software. As discussed in Section 10.3, selection of a computer software that allows for evaluation of forces acting upon individual slices within the overall slope model is particularly useful in the subsequent analyses of slope stability with deep foundation elements installed to supplement the available shear resistance on the failure plane.

If the existing slope is failing, the computed factor of safety should approximate 1.0, comparable to a geotechnical resistance factor of 1.0 for the Service Limit State, Should the computer simulated surface of failure differ significantly from the estimated shear failure surface based on surface observations and inclinometer data, the engineering properties, soil stratification and/or pore pressures within the slope should be adjusted in iterative “back-analyses” until the output from the computer analysis conforms to the observed conditions. A back-analysis that produces a geotechnical factor of safety of 1.0 (geotechnical resistance factor 1.0), but includes a calculated failure surface that is inconsistent with field observations should not be relied upon. All relevant parameters need to be consistent with observations.

10.3 DRILLED SHAFT ANALYSIS

10.3.1 Background

Numerous examples have demonstrated the successful utilization of drilled shafts to stabilize a slope, e.g., Fukumoto (1972 and 1973), Sommer (1977), Ito et.al. (1981 and 1982), Nethero (1982), Morgenstern (1982), Gudehus and Schwarz (1985), Reese et.al. (1992), Rollins and Rollins (1992), and Poulos (1995 and 1999). Past research relevant to the analysis of drilled shaft stabilized slopes include work by Reese (1992), Ito and Matsui (1975), Hassiotis et.al. (1997) Poulos (1999), and Liang and Zeng (2002).

The Liang and Zeng (2002) method presented herein was selected because it relies on a detailed formulation based on limit equilibrium theory, incorporates soil arching effects, generates a general slip surface, handles complex slope geometry and soil layers, and allows optimizing the shaft location and spacing. The general principle of soil arching as used by the Liang and Zeng (2002) method is depicted by Figure 10-1, and detailed procedures for its application are presented in the following sections of this manual.

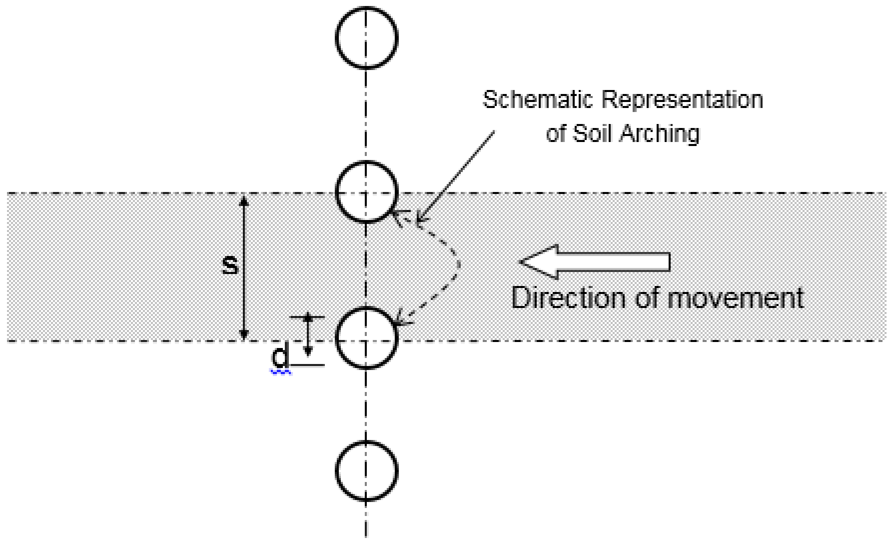


Figure 10-1: Direction of movement and soil arching (after Liang and Zeng 2002).

Whereas the Liang and Zeng (2002) method is based upon calculation of a lateral earth force to be resisted by the drilled shafts, Brown, et al. (2010) presents alternative approaches where: a.) the shear resistance and bending moments in the shafts are evaluated as a function of lateral movement above the slip plane, and b.) shafts are laterally analyzed relative to soil masses subject to liquefaction-induced instability.

10.3.2 LRFD Analysis for Slope Stabilization

Using the method of slices, as depicted in Figure 10-3, the Liang and Zeng (2002) method develops a resultant net force ($F_{net\text{-}shaft}$) that can be distributed in an equivalent loading diagram along the shaft length above the slip surface, as shown by Figure 10-5, to compute shear, moment, and lateral deflections with a laterally loaded pile p-y analysis software. The force analysis is based on the target geotechnical resistance factors as discussed in Section 10.3.3 for stabilization of the slope. Upon determination of the net resisting force, $F_{net\text{-}shaft}$, required to stabilize the slope, the distributed resultant can be used in a p-y analysis to assess the Geotechnical Service Limit State for the drilled shafts. Thereafter, $F_{net\text{-}shaft}$ is factored to confirm adequate geotechnical resistance at the Strength Limit State, as discussed in Section 10.3.4 and to design the shaft reinforcement, as discussed in Section 10.3.5.

10.3.3 Liang and Zeng (2002) Method

In the Liang and Zeng (2002) method, the slope is divided into “n” slices, as shown by Figure 10-2, to facilitate the computation of internal forces and the calculation of the factor of safety, which at 1.0 is the same as the geotechnical resistance factor. The resultant interslice force is assumed to be parallel to the inclination of the base of the previous slice with respect to the direction of slope movement, as shown by the method of slices in Figure 10-3. Development of a permanent arching effect is essential to this method, which may be inhibited by liquefiable sand that can flow around the drilled shafts, very soft soil that can squeeze through the shaft spacing, or shafts that are too widely spaced to develop arching. As the program is based upon a 2-dimensional finite element method (FEM) model, limitations include 3-dimensional effects and the uncertain validity of extrapolation to model shaft diameters larger than about 6-feet or cohesion values in excess of about 6 psi.

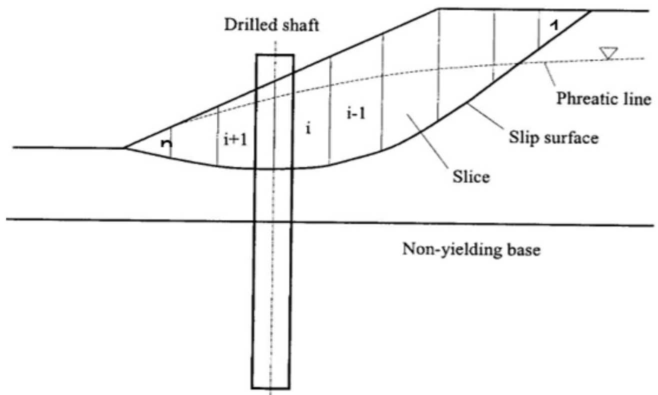


Figure 10-2: Slope divided into "n" slices (after Liang and Zeng 2002).

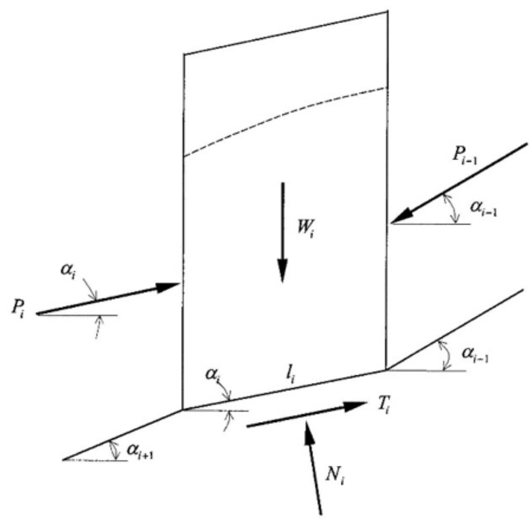


Figure 10-3: Forces acting on slice (Liang and Zeng 2002).

Dividing the slipping mass into *n* slices (method of slices) and applying force equilibrium results to each slice *i*, results in the following relationship:

$$P_i = W_i \sin \alpha_i - \frac{1}{FS}[C'_I l_i + (W_i \cos \alpha_i - u_i l_i) \tan \emptyset_i] + K_i R P_{i-1} \qquad \text{(Equation 10-1)}$$

Where:

$i = 1, 2, \ldots, n$

W_i = Weight of slice i.

α_i = Inclination with respect to the horizontal of the base of each slice.

FS = Factor of safety.

$C_i'l_i$ = Cohesion intercept at the slip surface.

l_i = Length of the base of the slice.

u_i = Pore pressure at the base of the slice.

ϕ = Friction angle of the soil at the slip surface.

K_i = Coefficient.

R = Reduction factor.

$P_i l_i$ = Interslice force.

K_i is obtained as follows:

$$K_i = \cos(\alpha_{i-1} - \alpha_i) - \sin(\alpha_{i-1} - \alpha_i)\frac{\tan\phi_i}{FS} \qquad \text{(Equation 10-2)}$$

The reduction factor R is a factor that considers the soil arching effect. When the shaft spacing (s) is comparable to the shaft diameter (d), i.e., the extreme case where s/d = 1, R = 1, the arch effect is largest, and the entire driving force of the slipping mass is transmitted to the shafts. Conversely, when s is much larger than d (i.e., s/d >> 1), the arch effect is negligible. At this point, R approaches R_p, defined as the percent of residual soil pressure acting on the soil mass between shafts. In the extreme case that $s/d \rightarrow \infty$, $R = R_p$. A general expression can be used to obtain R:

$$R = \frac{1}{s/d} + \left(1 - \frac{1}{s/d}\right)R_P \qquad \text{(Equation 10-3)}$$

Values of R_p were obtained in a 2-D finite element method (FEM) parametric study (Liang and Zeng 2002), included in Table 10-1 (a) through 10-1 (c) for shaft diameters of 1, 2, and 3 feet, respectively.

Equations 10-1, 10-2, and 10-3 are applied to each slice, resulting in a recursive formula for determining P_i with initial value P_o. First, an initial value of FS is assumed. For example, FS = 1/0.75 = 1.33 or FS= 1/0.65 = 1.53 for LRFD resistance factors ϕ = 0.75 or 0.65 per AASHTO (2014) 11.6.2.3. The FS value is used in the iterative formula. If Equation 10-1 results in tension for P_i (i.e., $P_i < 0$) at any computational step, P_i should be set to zero in the next step to calculate P_{i+1} because stability should not rely on the typically small tension resistance of soils.

Figure 10-4 shows that two cases of possible pressure distributions that need to be evaluated, because of the possible relative movement between the shaft and soil above the shear surface. In Case I, the earth pressure down-slope of the shaft is equal to the residual value, i.e., the upslope interslice force (Pi), which is reduced for the presence of the shaft per Equation 10-1, multiplied by the percent of residual soil pressure (Rp) occurring between the shafts per Table 10-1(a) through 10-1(c); Case II arises when the earth pressure down-slope of the shaft is equal to the at-rest value. Equations 10-4 and 10-5 are used to calculate the net shaft force for Cases I and II, respectively.

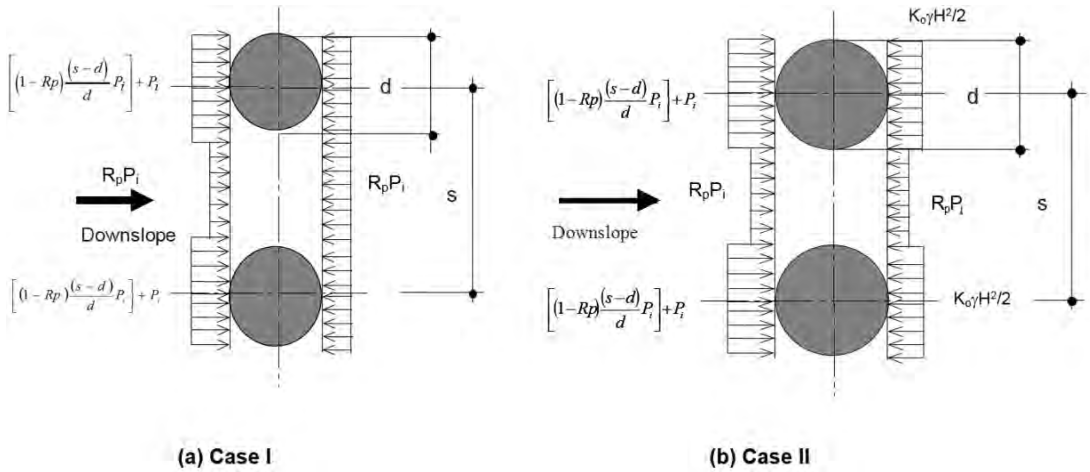


Figure 10-4: Reduction factor concept in distribution of forces (after Liang and Zeng 2002).

The net resisting force that keeps the drilled shaft in place is the resultant of the driving force on the left of shaft and the resisting force on the right of shaft (Figure 10-4 shown in opposite Figures 10-2 and 10-3 direction of movement), where H is the depth to the slip surface at the drilled shaft and K_o is the at-rest earth pressure coefficient, as follows:

$$F_{Shaft}^{Net} = (1 - R_P)P_iS \text{ Case I} \quad \text{(Equation 10-4)}$$

$$F_{Shaft}^{Net} = (1 - R_P)P_i(S - d) + \left[P_i - \frac{K_o\gamma H^2}{2}\right] d \text{ Case II} \quad \text{(Equation 10-5)}$$

The friction angle (ϕ), cohesion (C), shaft spacing (S), and shaft diameter (d) are required to determine the percent of residual load (R_p) acting on the soil mass between two adjacent drilled shafts. In Table 10-1(a) through 10-1(c), the variation in internal friction angle has a significant influence on the arching effect, particularly for soils with lower cohesion values. Also, soils with a higher friction angle are more likely to interlock and to develop stronger arching, which leads to a reduced R_p and thus a higher load on the shafts. It is possible to extrapolate the R_p values included in Table 10-1(a) through 10-1(c) for shaft diameters larger than 3 feet. However, the design engineer must be cautious in extending the extrapolation too far from the calculated range because the R_p increase with shaft diameter trend has not been confirmed. It is recommended that for purely cohesionless soils, the R_p values up to d = 3 feet be selected for larger shaft diameters. Interpolation or extrapolation can be exercised to determine cohesion values outside the range in Table 10-1(a) through 10-1(c), but it is important to perform numerical simulations to confirm the extrapolated values. Diameter extrapolation can be performed up to d = 6 feet.

A step-by-step procedure is described below in accordance with the set of equations presented above:

1. Compile available information for the current site conditions, including soil borings, shear strength parameters, and the shape and location of failure surface.
2. Select drilled shaft diameter, spacing, and location within the slope.
3. Assume an initial *FS* for the slope with drilled shafts using the limit equilibrium method with slices. Select the minimum factor of safety for slope stability.
4. Use Table 10-1(a) through 10-1(c) to evaluate the percent of residual pressure acting on the soil mass between adjacent shafts. Find the value of R_p corresponding to the soil shear strength parameters and the selected *s/d*.
5. Calculate the reduction factor, *R*, using Equation 10-3.
6. By applying the method of slices and Equations 10-1, 10-2 and 10-3 iteratively, calculate interslice forces at each slice and the corresponding *FS* with drilled shafts. Use the reduction factor (*R*) calculated in Step 5 in Equation 10-1 only for the slice just behind the shaft on the upslope side. For the rest of the slices, *R* is set to 1.0.
7. Calculate the net force that is transferred to the drilled shaft by substituting the percent of residual pressure, R_p, calculated in Step 4 and the interslice force for the slice just behind the shaft (upslope side) into Equation 10-4 (Case I) or Equation 10-5 (Case II).
8. Confirm the Geotechnical Service Limit State and Strength Limit State resistance for the selected shaft configuration in steps 3 through 7 in accordance with Section 10.3.4.
9. Perform structural design for the drilled shafts in accordance with Section 10.3.5.

Table 10-1(a): Percent of residual load on soil mass between shafts [d = 1 foot].

ϕ (Degree)	s/d	Rp (percent) C (psi) 0.0	Rp (percent) C (psi) 1.0	Rp (percent) C (psi) 2.0	Rp (percent) C (psi) 4.0	Rp (percent) C (psi) 6.0
0	2	100.00	39.10	10.64	10.47	10.46
	3	100.00	61.55	21.66	14.21	13.44
	4	100.00	71.73	39.21	21.02	15.35
10	2	64.09	11.25	10.44	10.36	10.36
	3	76.72	37.71	14.27	14.11	13.41
	4	81.26	54.85	20.61	15.32	15.01
20	2	34.92	10.83	10.68	10.31	10.31
	3	56.86	21.77	14.07	13.93	13.37
	4	64.72	40.72	14.88	14.21	13.35
30	2	16.48	10.56	10.50	10.44	10.35
	3	47.71	15.03	14.06	13.91	13.34
	4	59.29	27.69	14.81	14.14	13.33
40	2	16.32	10.47	10.41	10.34	10.31
	3	37.50	15.79	14.03	13.89	13.31
	4	54.51	29.32	14.74	14.02	13.27

Table 10-1(b): Percent of residual load on soil mass between shafts [d = 2 feet].

ϕ (Degree)	s/d	R_p (percent) C (psi) 0.0	R_p (percent) C (psi) 1.0	R_p (percent) C (psi) 2.0	R_p (percent) C (psi) 4.0	R_p (percent) C (psi) 6.0
0	2	100.00	39.92	10.52	10.23	9.98
	3	100.00	65.41	25.91	15.05	14.52
	4	100.00	76.51	45.45	22.67	17.12
10	2	69.03	16.54	10.26	10.14	9.89
	3	83.00	43.19	15.53	15.15	14.68
	4	86.68	60.05	26.52	16.51	16.37
20	2	44.15	12.91	10.31	9.86	9.74
	3	67.36	24.37	15.23	15.03	14.66
	4	76.76	48.10	16.38	15.35	14.62
30	2	28.95	10.29	10.11	9.73	9.63
	3	55.81	15.74	14.94	14.80	14.42
	4	68.42	36.01	15.81	14.96	14.36
40	2	24.59	10.27	9.78	9.75	9.66
	3	46.60	18.08	14.80	14.68	14.30
	4	59.75	32.87	15.17	14.80	14.25

Table 10-1 (c): Percent of residual load on soil mass between shafts [d = 3 feet].

ϕ (Degree)	s/d	R_p (percent) C (psi) 0.0	R_p (percent) C (psi) 1.0	R_p (percent) C (psi) 2.0	R_p (percent) C (psi) 4.0	R_p (percent) C (psi) 6.0
0	2	100.00	40.13	10.44	10.07	9.65
	3	100.00	67.98	28.75	15.61	15.24
	4	100.00	78.18	48.50	23.30	17.91
10	2	72.33	20.06	10.14	9.99	9.57
	3	85.33	45.68	16.01	15.52	15.18
	4	90.85	63.69	30.02	17.38	17.33
20	2	49.80	14.18	10.09	9.57	9.39
	3	71.90	25.38	15.62	15.39	15.12
	4	81.48	50.98	16.86	15.68	15.02
30	2	35.66	10.13	9.89	9.34	9.24
	3	60.58	16.16	15.46	15.32	15.06
	4	74.02	41.09	16.43	15.46	15.00
40	2	31.26	10.02	9.34	9.32	9.21
	3	53.54	19.54	15.32	15.18	14.94
	4	64.82	35.14	15.46	15.31	14.87

10.3.4 Geotechnical Resistance of Drilled Shafts

In a computer-based p-y analysis, the net shaft force calculated in Step 7, $F_{net\text{-}shaft}$, is distributed along the shaft length from the top to the slip surface with an equivalent triangular loading diagram as shown in Figure 10-5. Adaptation of the Liang and Zeng (2002) method in Geotechnical Bulletin GB7 (2014) by the Ohio Department (ODOT) allows that despite the complexity of loading, the triangular distribution is a close enough approximation of the actual condition to develop a realistic calculation of distributed shear, moment, and displacement of the drilled shaft. More conservatively, the triangular distribution of load can be converted to an equivalent trapezoidal load diagram in units of pounds per inch (lb/in) of shaft length above the slip surface to determine the lateral deflections, shear forces, and bending moments along the shaft length. Boundary conditions at the shaft head within the p-y analysis should be set based upon the appropriate freedom to move both laterally and rotationally, with a value of zero (0) input for both the shear and the moment at the head.

10.3.4.1 Service Limit State

Even though the primary objective of slope stabilization is to achieve a minimum factor of safety against shear failure, the stiffness of the shaft elements must be adequate to inhibit movement from occurring along with the moving soil mass. In the process of checking the Service Limit State, apply an (unfactored) vehicular live load surcharge (LS) equal to two feet of soil with a unit weight of 125 pcf to the computation of $F_{net\text{-}shaft}$, as per AASHTO (2014) 3.11.6.4. to assess the shaft deflection if the traffic surcharge is within the failure zone above the drilled shafts, or the horizontal distance between the drilled shafts and traffic loading is less than or equal to half the depth to the shear surface at the location of the drilled shafts. Geotechnical Service Limit State design is further discussed in Chapter 6.

10.3.4.2 Strength Limit State

The computed net force ($F_{net\text{-}shaft}$) must be factored to assess the Geotechnical Strength Limit State for determining shaft penetration beneath the slip surface and to verify adequate soil resistance below the slip surface. For the Strength Limit State analysis, use a load factor of Y_{LS} = 1.75 for the vehicular live load surcharge (LS) and a load factor of Y_{EH} = 1.50 for the horizontal earth pressure (EH), in accordance with Section 3.4.1 of AASHTO (2014). Geotechnical Strength Limit State design is further discussed in Chapter 6.

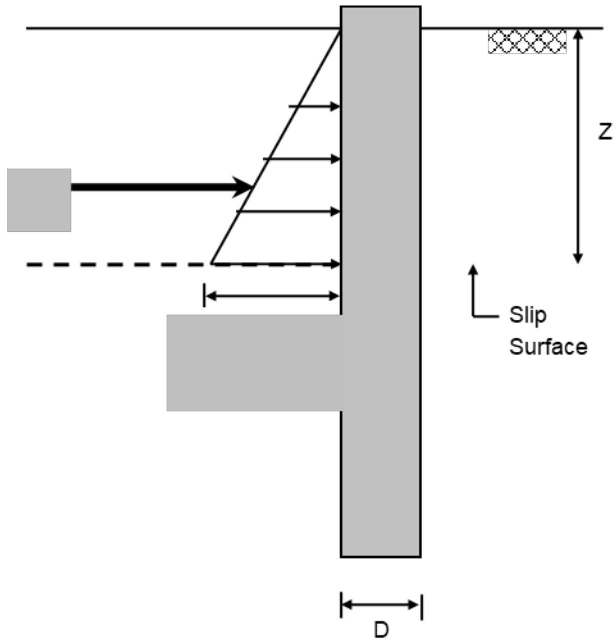


Figure 10-5: Loading for design of a drilled shaft in slope stabilization.

10.3.5 Drilled Shaft Reinforcement Design

The structural design of drilled shafts for slope stabilization should be in accordance with AASHTO (2014) using factored loads and resistance to design the reinforcement for flexure and shear. As per the Geotechnical Strength Limit State analysis described in 10.3.4, use a load factor of $\gamma_{LS} = 1.75$ for the vehicular live load surcharge (LS) and a load factor of $\gamma_{EH} = 1.50$ for the horizontal earth pressure (EH), per AASHTO (2014) Section 3.4.1 with a free-head boundary condition. Check the flexural resistance and shear resistance of the drilled shaft in accordance with AASHTO (2014) Sections 5.7.3 and 5.8.3, respectively, based on structural resistance factors per AASHTO (2014) Section 5.5.4.2. A detailed discussion regarding the structural design of drilled shafts is presented in Brown, et al. (2010).

10.3.6 Computer Applications

The Liang and Zeng (2002) procedure has been adopted by the Ohio Department of Transportation (ODOT) in a computer program (UA Slope) for drilled shaft landslide stabilization. The methodology for using this program can be found in Geotechnical Bulletin GB7 (2014). There are no known commercial applications of this method.

10.4 MICROPILE SLOPE STABILIZATION

Micropiles can be used to resist unstable slope forces through a combination of axial resistance developed at the grout-to-ground interface, both above and below the slip surface, and structural shear and bending resistance of the pile element. The shear resistance that an individual pile provides is assumed to correspond to the maximum shear force in the pile at its nominal bending resistance. The resistance of individual piles can be increased by designing the piles on a batter, whereby the upslope leg will be in tension and the downslope leg will be in compression. Piles are typically battered 30 degrees from vertical in an A-frame configuration with a reinforced concrete cap beam, as shown by Figure 2-11. If necessary, the resisting force can be supplemented by ground anchors, tied back to stable ground below the slip surface. Detailed procedures for designing micropiles for soil slope stabilization, along with an introductory discussion of the advantages and constraints associated with the design method, are included in Chapter 6 of this report.

11 STRUCTURAL DESIGN AND PERFORMANCE

11.1 OVERVIEW

This chapter provides general guidance for the structural design of deep foundation elements once the load effects imposed on the foundation elements have been determined through analysis. The structural design of the foundation elements is performed in accordance with the AASHTO LRFD Bridge Design Specifications (AASHTO 2014).

Laterally loaded piles and drilled shafts experience axial, bending and shear load effects simultaneously and are designed to account for the interaction of these load effects. The load effects are the result of load combinations applied to the superstructure and substructure which are ultimately transferred to the foundation elements. Load combinations are determined for each of the applicable Limit States (service, strength, or extreme events) in accordance with the load combinations dictated by the AASHTO specifications. For guidance regarding Limit States, loads, load factors and load combinations refer to Chapter 4 of this manual.

Once the appropriate applied force effects from the load combinations have been determined at the top of the foundation, analysis of the foundation element or group of foundation elements is performed to determine the force effects applied to the specific foundation element being designed. For guidance regarding the analysis of laterally loaded single piles/shafts and groups of deep foundation elements refer to Chapters 6 and 7 of this manual, respectively.

Force effects vary along the length of the foundation element. The foundation element can be designed to resist the governing force effects, or can have different designs along the length of the element. Transition zones between design sections should always be designed for the more severe combination of force effects.

Typical foundation elements are constructed from reinforced concrete (prestressed or conventionally reinforced) and structural steel. The design specifications for these materials are in AASHTO (2014) Section 5 – Concrete Structures, and Section 6 – Steel Structures, respectively.

Detailing of the connection between the foundation element and the substructure must be consistent with the analysis assumptions regarding fixity as this connection is used to determine the load effects in the foundation element. Much of the information in this section has been extracted directly and/or adapted from Hannigan et al. (2016) for driven piles, and from Brown et al. (2010) for drilled shafts.

11.2 STRUCTURAL DESIGN CONSIDERATIONS - GENERAL

The structural design of foundation elements must satisfy the basic LRFD equation as presented in AASHTO (2014) for all applicable Limit States:

$$\sum \eta_i \gamma_i Q_i \leq \emptyset R_n = R_r \quad \text{(Equation 11-1)}$$

Where:

γ_I = Load factor: a multiplier applied to force effects.

φ = Resistance factor: a multiplier applied to nominal resistance, as specified in AASHTO.

η_i = Load modifier: a factor relating to ductility, redundancy, and operational classification.

Q_i = Force effect.

R_n = Nominal resistance.

R_r = Factored resistance: φR_n.

Foundation elements subject to lateral loads are designed to take into account the simultaneous occurrence of shear, moment and axial load effects. The interaction between these load effects results in the foundation element being designed as a compression member with bending, i.e., as a beam-column. The typical interaction of load effects occurs in the following combinations: (i) axial load combined with moment, and (ii) axial load combined with shear. Designing for shear without considering the interaction with the simultaneously occurring axial load will result in a conservative design. The interaction between axial load and moment must always be considered.

Design is an iterative process wherein the applied factored load effects are compared to the structural resistance of the pile. If the computed factored maximum load effects exceed the nominal structural capacity of the foundation elements, then the design of the foundation element must be modified. This may include adding additional reinforcement in the case of concrete elements, using a heavier steel section in the case of steel piles and/or generally increasing the size (diameter, thicknesses or exterior dimensions) of the foundation element. If the size of the foundation element is increased, the Geotechnical Strength Limit State should be reviewed and re-analyzed to determine if the length can be reduced based on the increased size and/or stiffness of the foundation element. This iterative process requires coordination and communication between the geotechnical and structural design activities.

As stated above, a pile/shaft subject to lateral loads acts essentially as a beam-column. For some design purposes, the bending behavior of this beam-column can be represented by a constant, linear bending stiffness value E_pI_p. In this case, the non-linear effects due to concrete cracking (concrete pile/shafts) or plastic hinge formation (steel piles) are avoided. A linear assumption for the bending stiffness may be sufficient if the purpose of the analysis is to estimate the preliminary magnitude and distribution of moment and shear load effects along the pile/shaft and thereby obtain a preliminary value of the required area and distribution of longitudinal and transverse reinforcement in the case of concrete or the size of the steel section. Also, if the objective is to study the response of the pile/shaft under small deflections, a constant value for E_pI_p may be adopted.

Where:

E_p = Modulus of elasticity of the pile.

I_p = Moment of inertia of pile.

However, in many instances, the pile/shaft bending stiffness cannot be appropriately represented by a linear, constant value. When the loading scenario is such that the structural response causes non-linear effects in bending, the formation of yield moments (i.e., related to plastic hinge formation along the pile/shaft) must be considered. Therefore, the bending stiffness (E_pI_p) at each cross section must be determined as a function of the applied loading, and the yield or ultimate bending moment M_{ult} must be determined. Procedures for accomplishing a nonlinear bending analysis using commercially available general purpose structural analysis software can be found in the literature on this subject.

When non-linear bending is considered, the assumption normally made in concrete piles is that cracks will form where the net tensile stress exceeds the tensile strength of concrete anywhere along the pile/shaft. Nonlinear stress-strain curves are used for both steel and concrete. Per the common practice of reinforced concrete, it is assumed that failure of the concrete in compression occurs when the strain ε_c in concrete reaches approximately up to approximately 0.0038. For steel, yield is achieved when the strain in either tension or compression reaches a value defined as the ratio of the steel yield strength and the steel elastic modulus.

Refer to FHWA Report Number FHWA NHI-16-009: Geotechnical Engineering Circular Number 12 – Volumes 1 & 2 Design and Construction of Driven Pile Foundations for the evaluation of driving stresses in piles.

11.2.1 Effective Length and Buckling

Deep foundation elements are detailed and installed in two configurations. The element can either be completely underground, or a portion of the element may project above the ground surface. Any portion of the foundation element that projects above the ground surface is considered laterally unsupported unless structurally connected to a bracing member. The foundation element type, installation procedure, and ground conditions will all contribute to the distance below the ground surface at which point the foundation element can be considered continuously supported by the soil. This information is used to determine the unsupported length of the foundation element when checking the foundation for stability against buckling. The point of continuous lateral support is not the same as the point of fixity, as the point of fixity occurs below the point of lateral support.

From a structural view, foundation elements act as columns and therefore under axial and moment loads, an effective length could be considered for simplified frame analyses. The structural properties of the foundation element and the end conditions are used to approximate an effective length factor, K, as shown in Table 11-1, where the foundation element toe is generally assumed fixed for both translation and rotation (pinned). In the absence of sufficient bracing, (e.g., very soft soils, piles/shafts extended though water, large scour, etc.) the foundation element head may experience lateral displacement (sideways) and rotation, and therefore cannot be considered as fixed; for these conditions, the design value of K should be determined based on the anticipated head restrain condition. For example, if a foundation element extends above the ground surface and is connected to a rigid pier cap, rotation is generally prevented by the pier cap mass and stiffness, but the free-length of the element may result in lateral movement (reduced lateral bracing) in combination with existing loads. In this case, a fixed rotation and free translation condition may exist, as illustrated in Table 11-1. In pile bents, depending on the foundation's connection to the superstructure, the bent cap could allow rotation and translation perpendicular to the long axis of the bent cap, but free translation with fixed rotation along the long axis of the bent cap. To have a rotationally fixed foundation element top condition, Rollins and Stenlund (2010) observed that rather than defining a rule-of-thumb for minimum foundation element embedment length into the cap, the moment capacity of the cap to foundation element connection should be designed with foundation element embedment and cap reinforcement details such that the moment capacity of the connection exceeds the moment capacity of the foundation element.

Buckling is generally of concern when foundation elements extend through water or air, or for liquefaction, where an absence or reduction of confining stress is clearly recognizable. Very soft soils or peat are often considered to provide insufficient lateral support for providing resistance to buckling.

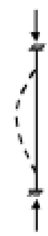
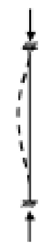
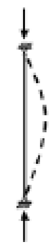
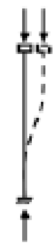
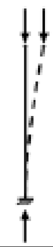
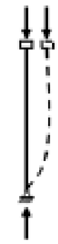
To characterize buckling resistance in soft soils, a load test program was performed by the Bethlehem Steel Corporation which suggested that even soft soils provide adequate support (Bethlehem Steel Corporation 1970). One such H-pile in this study extended through 31 feet of water and 29 feet of soft organic silt where the pile sank under its own weight. An applied axial load of 200 tons produced a gross settlement of 0.63 inches but no pile buckling occurred. In addition, Coduto et al. (2016) suggests that, "even the softest soils provide enough lateral support to prevent underground buckling in piles subject only to axial loads, especially when a cap is present and provides rotational fixity to the pile top."

A more conservative approach to this issue would be to determine the critical buckling load using computer software, such as LPILE. For this method, a foundation element-soil model is generated and incremental loads are applied to evaluate the resulting deflection. This method may provide the design engineer with a deflected pile shape to assess buckling for a given factored load in lieu of using prescriptive minimum soil strength values to characterize an unbraced length.

The unbraced length, l, or laterally unsupported length is defined by AASHTO (2014) as the distance between two braced points that resist buckling or distortion modes. For embedded foundation elements, the unbraced length is considered for scour and element stickup through air and/or water. For preliminary analysis, when lateral loads are applied, the effective length, *K*, for flexural resistance calculations is taken as the total unsupported length, plus an embedded depth to "fixity." If a lateral pile analysis with p-y curves for soil-structure interaction has been performed as discussed in Chapter 6, the depth to fixity concept is unnecessary—most software with lateral analysis also includes additional features to determine a pile's buckling capacity given the soil model and a pile model with the expected stick-up above the ground level.

For preliminary calculations, however, depth to fixity below the ground may be evaluated based on soil type and soil strength parameters as shown in Eq. 11-2 to Eq. 11-4 and discussed in Chapter 6. For sands, Table 11-2 contains the rate of increase in soil modulus, n_h, and should be used as applicable in the following depth to fixity estimates.

Table 11-1: Effective length factors, K (after AASHTO 2014).

Column Parameters	Column End Conditions					
Buckled shape of column is shown by dashed line						
Bottom Rotation Restraint	Fixed	Fixed	Free	Fixed	Fixed	Free
Bottom Translation Restraint	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
Top Rotation Restraint	Fixed	Free	Free	Fixed	Free	Fixed
Top Translation Restraint	Fixed	Fixed	Fixed	Free	Free	Free
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Design value of K when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.1	2.0

For preliminary calculations, however, depth to fixity below the ground may be evaluated based on soil type and soil strength parameters as shown in Eq. 11-2 to Eq. 11-4 and discussed in Chapter 6. Refer to FHWA Report Number FHWA NHI-16-009: Geotechnical Engineering Circular Number 12 – Volumes 1 & 2 Design and Construction of Driven Pile Foundations and FHWA Report Number FHWA NHI-10-016 for information regarding values for n_h.

For clays:

$$d_f = 1.4\left(\frac{EI_w}{E_s}\right)^{0.25}$$

(Equation 11-2)

$$E_s = 0.465s_u$$

(Equation 11-3)

For sands:

$$d_f = 1.8\left(E\frac{I_w}{n_h}\right)^{0.2}$$

(Equation 11-4)

Where:

d_f = Depth to fixity below the ground (ft).

E = Elastic modulus of the foundation element material (ksi).

E_s = Elastic modulus of clay soils.

s_u = Undrained shear strength of clay (ksf).

I_w = Weak axis moment of inertia of foundation element (ft^4).

n_h = Rate of increase of soil modulus with depth (ksi/ft).

11.3 PROCEDURES FOR REINFORCED CONCRETE SECTIONS

This section provides a description of the structural design considerations for foundation elements constructed from reinforced concrete. Foundation elements covered under this manual included bored piles, drilled shafts, concrete cylinder piles, and reinforced concrete piles (both conventionally reinforced and prestressed).

Although there are cases where the reinforced-column approach is unnecessary for concrete foundation elements due to zero or very low moment effects, concrete foundation elements must always be designed with the minimum reinforcement required for compression members as stipulated in Section 5 of AASHTO (2014) because of the potential for unforeseen loading. Detailing of the reinforcing and prestressing steel should be in accordance with the requirements of Section 5 AASHTO (2014).

Numerous commercially available computer programs are also available for the design of concrete compression members.

11.3.1 Material Properties - General

Unless otherwise noted, materials and material properties should be in accordance with the provisions of the current AASHTO LRFD Specifications (2014). Structural considerations of these material properties are discussed hereafter.

11.3.2 Concrete

Concrete foundation elements are generally designed with concrete having a specified compressive strength, f'_c, of 3.5 ksi to 5.0 ksi. Note that AASHTO (Section 5.4.2.1) prohibits the use of concrete with a specified compressive strength less than 2.4 ksi for structural applications (including foundation elements).

Concrete for foundation elements shall be normal weight. The modulus of elasticity for concrete, Ec, can be approximated by Equation 11-5:

$$Ec = 1820\sqrt{f_c'} \quad \text{(Equation 11-5)}$$

Refer to AASHTO (2014) Section 5.4.2 for additional requirements for concrete foundation elements concrete.

11.3.3 Reinforcing Steel

Reinforcing steel for concrete foundation elements will generally be AASHTO M31 (ASTM A615) Grade 60, with a minimum yield strength of 60 ksi. The use of reinforcing steel with yield strengths less than 60 ksi is not recommended. Bars with yield strengths less than 60 ksi should be used only with the approval of the owner.

The use of reinforcing steel conforming to ASTM A706, "Low Alloy Steel Deformed Bars for Concrete Reinforcement", should be considered where improved ductility is needed or where welding is required.

The modulus of elasticity, E_s, for reinforcing steel can be assumed to be 29,000 ksi.

Refer to AASHTO (2014) Section 5.4.3 for additional information on concrete foundation element reinforcing steel.

11.3.4 Casings

Casings are sometimes required for foundation element constructed using drilling techniques such as drilled shafts. When required, steel for permanent casings should generally conform to the values shown in Table 11-2.

Table 11-2: Minimum yield strengths for permanent steel casing.

Standard	Minimum Yield Strength (f_y)
ASTM A36	36 ksi
ASTM A242	50 ksi for thickness ≤ 0.5 inch 46 ksi for 0.75 inch < thickness ≤ 1.5 inch 42 ksi for 1.5 inch < thickness ≤ 4 inches
ASTM A252 Grade 2	36 ksi
ASTM A252 Grade 3	45 ksi

Note: The modulus of elasticity, E, for steel casings can be assumed to be 29,000 ksi.

The thickness of casings should be shown in the contract documents as "minimum". The minimum thickness of casings should be that required for reinforcement or for strength required during installation, whichever is greater. The latter is a function of both the site conditions and the method of installation. AASHTO Specifications (2007) require the contractor to furnish casings of greater than the design minimum thickness, if necessary, to accommodate the contractor's choice of installation equipment. Casings used for structural support in permanent foundation applications also need to consider potential corrosion. In such cases, the casing needs to be provided with protective coating and/or additional thickness to maintain the required structural thickness of the casing for the defined design life of the structure.

11.3.5 Minimum and Maximum Amount of Longitudinal Steel Reinforcement

AASHTO Section 5.7.4.2 (2014) specifies a range for the amount of steel reinforcement allowed in the cross-section of a concrete foundation element. The maximum allowable area of longitudinal reinforcing steel, As, is 8.0 percent of the gross cross-sectional area of the shaft Ag, or:

$$\frac{A_s}{A_g} \leq 0.08 \qquad \text{(Equation 11-6)}$$

In addition, AASHTO (2014) (Sections 5.10.11.3 and 5.10.11.4.1) limits the longitudinal reinforcement for Seismic Zones 2, 3 and 4 to not more than 6.0 percent. Typical amounts of reinforcement are between one and two percent but may be greater than 3 percent in high seismic zones. Construction of concrete foundation elements with longitudinal reinforcement greater than 4 percent is difficult, and should be avoided if at all possible. Difficulties with construction of concrete foundation elements with the higher percentages of longitudinal reinforcement include the flow of concrete through the rebar cage to the outside faces of the element. See Brown et al. (2010) for additional discussion of the amount of longitudinal reinforcement for drilled shafts.

The minimum amount of longitudinal reinforcement is affected by both the strength of steel and concrete. In the portions of the drilled shaft that behave as a column, defined as any portion of the shaft above the depth at which the shaft is laterally supported, the minimum longitudinal reinforcement amount is determined as:

$$\frac{A_s f_y}{A_g f_c'} \geq 0.135 \qquad \text{(Equation 11-7)}$$

In which f_y = yield strength of the longitudinal steel bars. Furthermore, the minimum longitudinal reinforcement area in the portions of the shaft that behave as a column should be not less than 1 percent of the gross concrete area of the shaft. Below the section where the drilled shaft behaves as a column (i.e., is laterally supported) the amount of reinforcement is typically governed by the moment demand along the length of the element. However, 0.5 percent of the gross concrete area of the pile is suggested as a practical minimum.

The longitudinal reinforcing bars should be evenly distributed among not less than 6 bars in a circular arrangement. The minimum size of longitudinal bars is No. 5 (AASHTO Section 5.7.4.2).

Per AASHTO Section 5.13.4.5.2, the clear distance between parallel longitudinal reinforcing bars shall be not less than 5 times the maximum aggregate size or 5.0 inches, whichever is greater.

However, recent research has indicated that, for drilled shafts constructed using tremie concrete, the proper flow of concrete through the rebar cage cannot be assured unless the clear spacing is equal to or greater than 10 times the maximum aggregate size. When necessary, vertical reinforcing bars should be bundled in order to maximize the clear space between vertical reinforcement bars. For drilled shafts constructed by the dry method, a clear spacing of 5 times the maximum aggregate size, with a minimum of 5.0 inches, is sufficient.

11.3.6 Minimum Amount of Transverse Steel Reinforcement

Transverse reinforcement in concrete foundation elements shall meet all the following (minimum) requirements:

- Shear design requirements are governed by AASHTO (2014) Article 5.8.
- Minimum requirements for transverse reinforcement determined in accordance with AASHTO (2014) Article 5.7.4.6. Note that this requirement applies to all Seismic Zones
- Minimum confinement requirements for seismic design is determined in accordance with AASHTO (2014) Articles 5.10.11.4.1d, 5.10.11.4.1e and 5.13.4.6. Note that the transverse reinforcement

requirements specified for Seismic Zones 3 and 4 under Sections 5.10.11.4.1d and 5.10.11.4.1e also apply to Seismic Zone 2, per AASHTO Article 5.10.11.3 (2014).

For all seismic zones, from the top of the drilled shaft to a depth of at least 3.0 diameters below the calculated depth of moment fixity, the minimum transverse reinforcement can be determined using Equations 11-8 and 11-9:

$$\rho_s \geq 0.45\left(\frac{A_g}{A_c}-1\right)\frac{f_c'}{f_{yh}}$$

(Equation 11-8)

Where:

ρs = Ratio of spiral or seismic hoop reinforcement to total volume of concrete core.

Ag = Gross area of column section (in^2).

Ac = Area of the concrete core measured to the outside diameter of the spiral (in).

f'_c = Specified strength of concrete at 28 days (ksi).

$f$yh = Specified yield strength of spiral or hoop reinforcement (ksi).

$$\rho_s \geq 0.12\frac{f_c'}{f_{yh}}$$

(Equation 11-9)

Where:

ρs = Ratio of spiral or seismic hoop reinforcement to total volume of concrete core.

f'_c = Specified strength of concrete at 28 days (ksi).

$f$yh = Specified yield strength of spiral or hoop reinforcement (ksi).

For Seismic Zones 2, 3 and 4, the maximum pitch of spiral reinforcement or spacing of seismic hoops shall be 4.0 inches down to a depth of at least 3.0 diameters below the depth of moment fixity and 9.0 inches below that depth (AASHTO Article 5.13.4.6.2b). For Seismic Zone 1, the maximum pitch of spiral reinforcement or spacing of seismic hoops shall be 6.0 inches down to a depth of at least 3.0 diameters below the depth of moment fixity and 12.0 inches below that depth (AASHTO Article 5.13.4.5.2). In all cases, spirals or seismic hoops shall not be smaller than #3 bars.

The clear distance between parallel transverse reinforcing bars should not be less than five times the maximum aggregate size or 5.0 inches, whichever is greater (AASHTO Section 5.13.4.5.2). In seismic zones this can be a challenge because high steel ratios are often dictated by the earthquake force effects. An effort should be made to meet the minimum 5-inch spacing requirement by bundling the longitudinal bars, as necessary.

11.3.7 Concrete Cover and Cage Centering Devices

Recommended minimum concrete covers to the primary (longitudinal) reinforcing steel for unprotected main reinforcing steel is given in Table 11-3. These values are extracted from Table 5.12.3-1 in the AASHTO (2014) LRFD Bridge Design Specifications:

Table 11-3: Minimum cover for unprotected main reinforcing steel (in).

Situation	Cover (in.)
Precast Reinforced Piles	
▪ Noncorrosive Environments	2.0
▪ Corrosive Environments	3.0
Precast Prestressed Piles	2.0
Cast-in-Place Piles	
▪ Noncorrosive Environments	2.0
Corrosive Environments	
▪ General	3.0
▪ Protected	3.0
▪ Shells	2.0
▪ Auger cast, tremie concrete, or slurry construction	3.0

The cover required for transverse reinforcement may be less than required for longitudinal bars by no more than 0.5 inch. Transverse reinforcement larger than 0.5-inch diameter would thus necessitate greater cover than specified above for longitudinal bars.

The above minimum concrete covers are for concrete with water-to-cementitious material ratios (W/CM) between 0.40 and 0.50. For W/CM ratios equal to or greater than 0.50, the cover requirements must be increased by a factor of 1.2. For W/CM ratios less than or equal to 0.40, the cover requirements may be decreased by a factor of 0.8. The modification factors of 1.2 and 0.8 are in recognition of the changes in concrete permeability resulting from higher and lower values of W/CM ratio. However, low W/CM ratios can lead to constructability problems because the flow characteristics and ability of concrete to pass through the rebar cage generally decrease at low W/CM ratios.

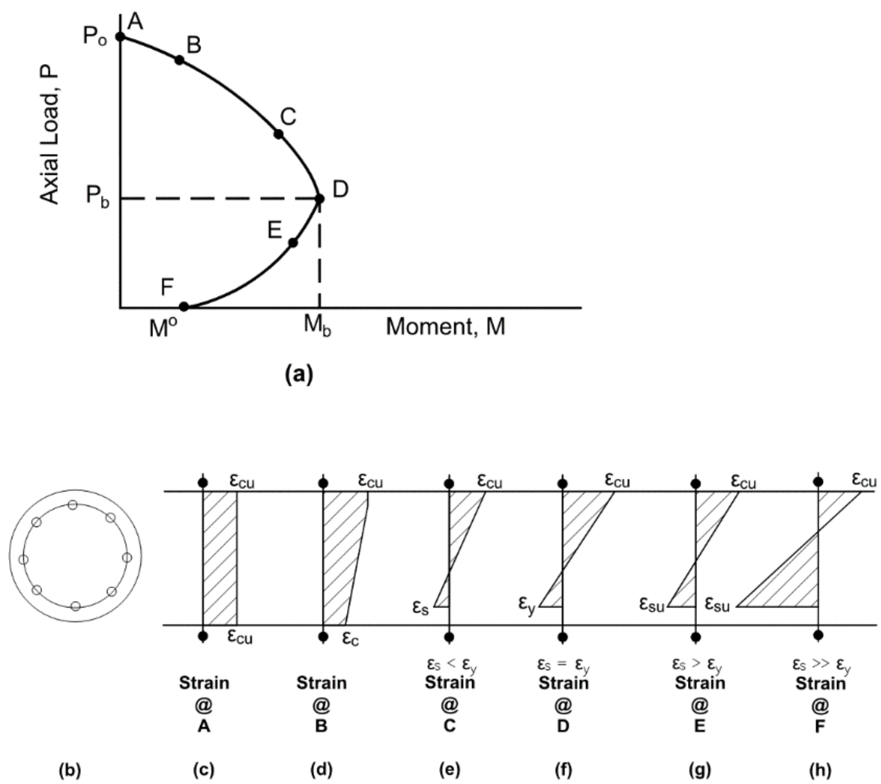
Centering devices must be used with drilled shaft and other drilled concrete foundation element construction to maintain alignment of the steel reinforcing cages and maintain the required minimum concrete cover. The centering devices are often plastic “wheels” installed around the transverse reinforcement. The wheels must be oriented such that they roll along the borehole wall without scraping into the soil.

11.3.8 Cases with Axial and Bending Moment (Linear Behavior)

The following structural design information generally follows the AASHTO LRFD Bridge Specifications (2014) design methods for concrete compression members. This manual uses a circular reinforced concrete section as the foundation element; however, the procedure can be adapted to any reinforced concrete cross section. When a cross-section of an axially loaded concrete foundation element is subjected to a bending moment from any source, there is a corresponding decrease in its axial structural resistance. The decrease can be explained by referring to Figure 11-1. The curve in Figure 11-1a shows the combinations of maximum axial load and maximum bending moment that the cross section of the foundation element can carry at the Structural Limit State. Points inside the curve, called an "interaction diagram", give combinations of axial load and moment that can be sustained; points on the curve, or outside of it, define a Structural Limit State. Interaction diagrams for a given cross section can be generated using several commercially available computer programs.

Although Figure 11-1 treats the case of combined axial compression and bending, the concepts presented in this section are equally valid and applicable to a reinforced concrete beam-column (concrete foundation element) subjected to combined axial tension and bending.

Figure 11-1b shows a schematic of a circular reinforced concrete cross section that is being analyzed to obtain the interaction diagram. The diagrams in parts (c) through (h) of Figure 11-1 illustrate the assumed distribution of strain in the cross section when it is subjected to different combinations of axial compressive load and bending moment, represented by the points on the interaction curve A, B, C, D, E and F, respectively. When failure occurs due to axial load only (P_o as at point A in Figure 11-1a), a uniform compressive strain ε_{cu} exists across the entire cross section (Figure 11-1c), where ε_{cu} is the compressive strain that causes crushing in the concrete (0.003). When failure occurs with a lesser axial load combined with a small amount of bending moment, as at point B, the strain distribution on the cross-section is no longer uniform. The top-fiber strain reaches the value of εcu whereas the bottom-fiber strain is reduced, but may still be compressive as in Figure 11-1d, if the moment is not large.



ε_{cu} = ultimate concrete strain in compression

ε_c = concrete strain in compression

ε_s = steel strain in tension

ε_y = yield strain of steel

ε_{su} = steel strain when concrete strain reaches ε_{cu}

Figure 11-1: Interaction diagram for a reinforced concrete column.

For a condition where bending moment is increased and axial load decreased, as represented by point C, part of the cross section is subjected to tension, which is taken by steel reinforcement if, for simplicity, it is assumed that the concrete is a material that cannot resist tension. This is a stage when sufficient tension is not developed to cause yielding of the steel, and the failure is still by crushing in the concrete. Proceeding to the state represented by point D, the failure combination of axial load and bending moment is such that the ultimate strain ε_{cu} in the concrete and tensile yield strain εy, in the steel are simultaneously reached. This stage is known as the balanced condition, and Mb and P_b are the moment and axial load resistances of the section at the balanced condition. At any failure combination between points A and D on the curve, failure is caused by crushing in the concrete before the steel yields.

Tensile yielding in the steel can occur with a lesser bending moment than that at the balanced condition if the compression is removed by decreasing the axial load. This stage is represented by the lower portion, DF, of the curve. Since the axial load is less, the steel yields before the ultimate concrete strain, ε_{cu} is reached. With further bending, the concrete compressive strain reaches ε_{cu} and failure occurs. At point F, the section is subjected to bending moment only (Mo), and failure occurs well after the steel yields.

Because the resistance of a cross section with given properties of steel and concrete depends upon the percentage of reinforcement and the position of the steel with respect to the centroidal axis, a set of interaction diagrams needs to be drawn for each drilled shaft cross section that is analyzed.

The nominal resistance interaction diagram, shown as the solid line in Figure 11-1 and Figure 11-2, should be obtained for all critical sections of the drilled shaft. Computer programs for lateral analyses typically include options for generating this interaction diagram for specified cross-sections. The factored resistance interaction diagram, illustrated as a dashed line in Figure 11-2, identifies the boundary in which factored force effects should reside. The method to determine the boundary is described herein.

The factored resistance interaction diagram (shown as the dashed line in Figure 11-2) is determined by multiplying the nominal moment and nominal axial resistances by the resistance factor φ (AASHTO 2014).

$$P_r = \varphi P_n \quad \text{(Equation 11-10)}$$

$$M_r = \varphi M_n \quad \text{(Equation 11-11)}$$

Where:

- P_r = Factored axial resistance.
- P_n = Nominal axial resistance.
- M_r = Factored moment resistance.
- M_n = Nominal moment resistance.
- φ = Resistance factor (see below).

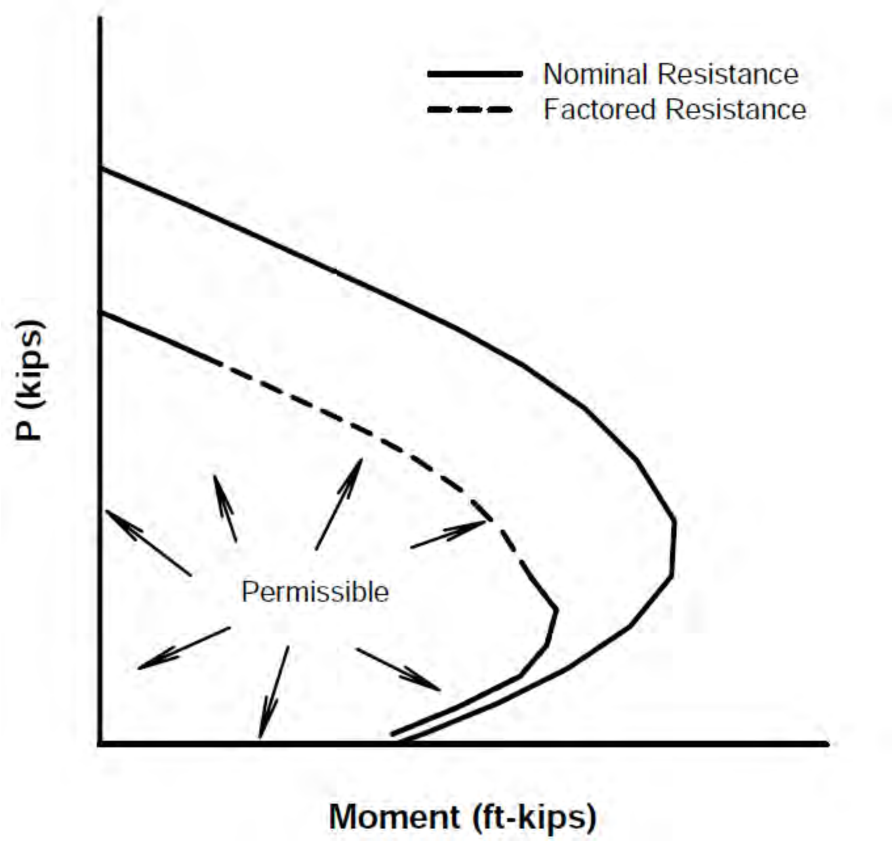


Figure 11-2: Nominal and factored interaction diagrams.

The interaction diagram uses a resistance factor (φ) that is variable and is determined by the strain conditions in the structural cross-section, at nominal strength. Therefore, resistance factors are different for compression-controlled and tension-controlled sections. Sections are considered “tension controlled” if the tensile strain (in the extreme tensile steel) at nominal strength is greater than 0.005. A value of 0.9 is used as φfor a tension-controlled section. A “compression-controlled” section uses a φof 0.75 and is defined as a cross-section for which the net tensile strain (εt) in the extreme tensile steel at nominal strength is less than or equal to the compression controlled strain limit of 0.003 (refer to AASHTO Articles 5.7.2.1 and C5.7.2.1 2014).

Linear interpolation is used to determine φ for sections that transition between tension-controlled and compression-controlled (see plot in Figure 11-3). The transition formula for φ can also be given by Equation 11-12:

$$0.75 \leq \varphi = 0.65 + 0.15\left(\frac{d_t}{c} - 1\right) \leq 0.9 \qquad \text{(Equation 11-12)}$$

Where:

c = Distance from extreme compression fiber to the neutral axis (in).

d_t = Distance from extreme compression fiber to the centroid of the extreme tensile element (in).

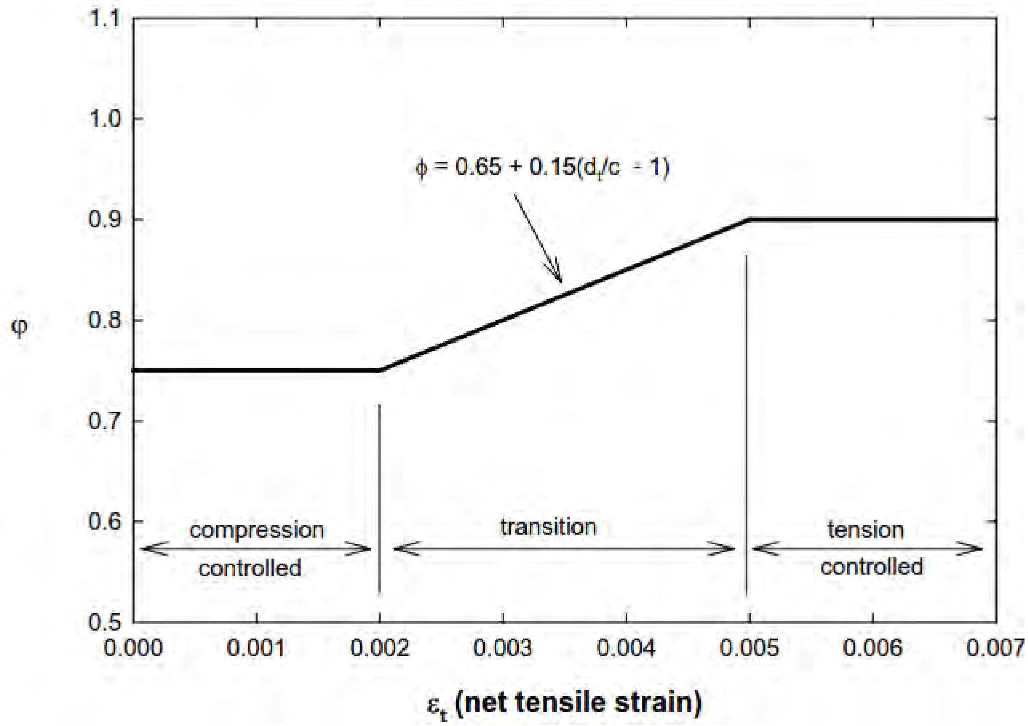


Figure 11-3: Variation of φ with net tensile strain, εt and dt/c for grade 60 reinforcement.

Cases involving combined axial tension and bending are analyzed by applying the same concepts described above for combined axial compression and bending. A notable difference would be that the Strength Limit State is always tension-controlled; therefore, the resistance factor is φ= 0.90.

11.3.9 Axial Compression and Biaxial Bending for Non-Circular Members

For axial compression loading, the factored Structural Limit State is taken. indicated in Equations 11-10 and 11-11.

To determine the nominal compressive resistance, a straightforward calculation is performed considering either spiral or tie reinforcement. As mentioned in the AASHTO (2014) commentary, reduction factors are placed on the respective equations to account for unintended eccentricity. Further details on axial resistance of concrete piles can be found in Article 5.7.4.4 of AASHTO (2014).

For members with spiral reinforcement:

$$P_n = 0.85\left[0.85 f_c'\left(A_g - A_{str} - A_{ps}\right) + F_{yr} A_{str} - A_{ps}\left(f_{pe} - E_{st}\varepsilon_{cu}\right)\right] \quad \text{(Equation 11-13)}$$

For members with tie reinforcement:

$$P_n = 0.80\left[0.85 f_c'\left(A_g - A_{str} - A_{ps}\right) + F_{yr} A_{str} - A_{ps}\left(f_{pe} - E_{st}\varepsilon_{cu}\right)\right] \quad \text{(Equation 11-14)}$$

Where:

P_n = Nominal compressive resistance (kips).

f_c'= Concrete compressive strength at 28 days, unless otherwise specified (ksi).

f_{pe} = Effective stress in the prestressing steel after losses (ksi).

F_{yr} = Yield stress of reinforcing steel (ksi).

A_g = Gross cross-sectional area (in^2).

A_{str} = Cross sectional area of longitudinal reinforcement (in^2).

A_{ps} = Cross sectional area of prestressing steel (in^2).

E_{st} = Elastic modulus of prestressing steel (in^2).

ε_{cu} = Failure strain of concrete in compression (in/in).

Biaxial flexural resistance must satisfy the following checks. Additional information may be found in Article 5.7.4.5 of the AASHTO (2014) specifications.

If $P_u \geq 0.10\varphi f_c' A_g$:

$$\frac{1}{P_{rxy}} = \frac{1}{P_{rx}} + \frac{1}{P_{ry}} - \frac{1}{\varphi P_0} \leq 1.0 \quad \text{(Equation 11-15)}$$

In which:

$$P_0 = 0.85 f_c' \left(A_g - A_{str} - A_{ps}\right) + F_{yr} A_{str} - A_{ps}\left(f_{pe} - E_p \varepsilon_{cu}\right)$$

If $P_u < 0.10\varphi f_c' A_g$:

$$\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad \text{(Equation 11-16)}$$

Where:

P_u = Factored axial load.

P_{rx} = Factored axial resistance determined on basis that only eccentricity, *ey*, is present (kips).

P_{ry} = Factored axial resistance determined on basis that only eccentricity, *ex*, is present (kips).

P_{rxy} = Factored axial resistance in biaxial flexure (kips).

M_{ux} = Factored flexural moment about x-axis (kip-in).

M_{rx} = Factored flexural resistance about x-axis (kip-in) (AASHTO (2014) Section 8.5.2.3).

M_{uy} = Factored flexural moment about y-axis (kip-in).

M_{ry} = Factored flexural resistance about y-axis (kip-in) (AASHTO (2014) Section 8.5.2.3).

φ_c = Resistance factor for axial compression (AASHTO (2014) Table 8-6).

f_c' = Concrete compressive strength at 28 days, unless otherwise specified (ksi).

f_{pe} = Effective stress in the prestressing steel after losses (ksi).

$F_{y\,r}$ = Yield stress of reinforcing steel (ksi).

A_g = Gross cross-sectional area (in^2).

A_{st} = Cross sectional area of longitudinal reinforcing steel (in^2).

A_{ps} = Cross sectional area of prestressing steel (in^2).

E_p = Elastic modulus of prestressing steel (in^2).

ε_{cu} = Failure strain of concrete in compression (in^2).

P_0 = Nominal axial resistance of a section at 0.0 eccentricity.

The analysis of the prestressed concrete section's response to a combination of an axial load and two orthogonal moments is complex. The concrete and the prestressing steel stress-strain relationships are assumed. For concrete, assume a maximum concrete strength of 0.85f'_c to include loading time effects on the concrete strength and all points on the stress strain curve are reduced to 85 percent of the short time values.

Bi-axial interaction diagrams are determined for each of an increasing set of axial loads up to the maximum axial strength condition. An illustration of one of these interaction diagrams for a particular axial load is shown in Figure 11-4. These diagrams are determined for the entire range of axial loads up to the axial failure case. With increasing axial load the maximum moment strength becomes smaller. A three-dimensional interaction diagram can then be constructed with the axial load on the vertical axis and a particular interaction diagram at each level of axial load. Imagine a stack of these interaction diagrams. Thus, a three-dimensional failure surface is defined. The equation of the failure surface can be generated by fitting a surface through the interaction diagrams at each level.

When the necessary failure surfaces are available, the analysis at a load level can be checked by examining whether the vector of the forces on the section (axial, *Mx* and *My*) falls within or outside the failure envelope. The deformations associated with the three applied forces make it possible to determine the displacements associated with the various load levels. This elegant and powerful analysis algorithm produces excellent results. Well-designed graphics make it possible for the foundation specialist to easily evaluate the results.

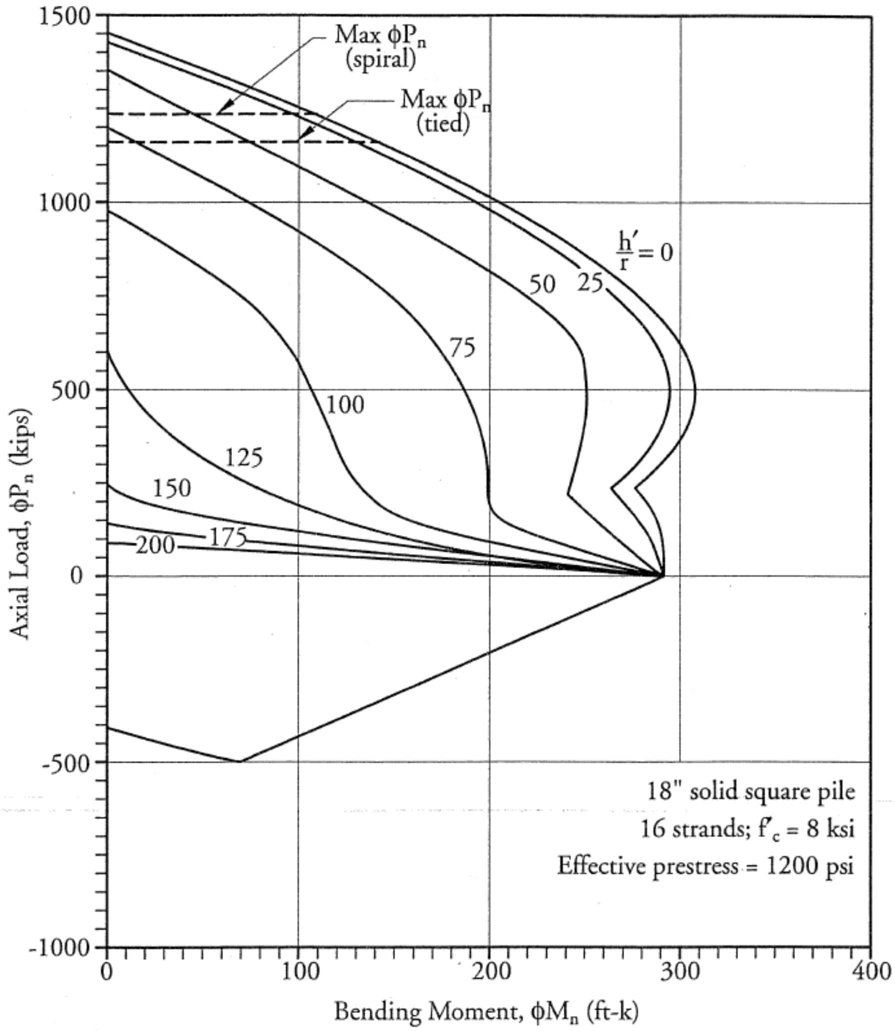


Figure 11-4: Moment interaction diagram.

11.3.10 Cases with Axial and Bending Moment (Non-Linear Behavior)

As the bending moment on a reinforced concrete section increases to the point at which it produces tensile stresses on one side of the shaft exceeding the tensile strength of the concrete, the section cracks, and a significant reduction in the E_pI_p of the section at that location will occur. The axial load (if compressive), when it has no eccentricity, produces a uniform distribution of compressive stresses in the section that superimposes with the bending- induced stress distribution. As a result, the behavior of the section is a function of the relative magnitude of both axial loads and moments.

The stress (σ) versus strain (ε) curves for concrete and reinforcing steel that are used by some of computer programs for design of laterally loaded deep foundations are shown in Figures 11-5 and 11-6, respectively (O'Neill and Reese 1999). The curve for concrete exhibits an initial almost linear response followed by a non-linear curve up to the peak stress (defined as f'_c), and a decreasing linear segment that ends at the maximum strain of concrete.

Referring to Figure 11-5:

$f''_c = 0.85\, f'_c$ (Equation 11-17)

E_c (initial slope of the stress-strain curve) = 4,730 $(f'_c)^{0.5}$ (Equation 11-18)

$$\boldsymbol{f_c} = \boldsymbol{f''_c}\left[\boldsymbol{2}(\boldsymbol{\varepsilon}/\boldsymbol{\varepsilon_0}) - (\boldsymbol{\varepsilon}/\boldsymbol{\varepsilon_0})^{\boldsymbol{2}}\right]$$
$(\varepsilon < \varepsilon_o)$ (Equation 11-19)

$$\varepsilon_0 = 1.7\, f_c'/E_c$$ (Equation 11-20)

$$f_r = 0.62(f_c')^{0.5}$$ (Equation 11-21)

Where: f'_c and E_c are the concrete compressive strength and elastic modulus, respectively. In these equations, the units of E_c, f'_c, f''_c and f_r are in MPa. Note that approximately 6.9 Mpa = 1 ksi.

In Figure 11-6:

$\varepsilon_y = f_y/E$ and E = 200,000 MPa (or 29,000 ksi) (Equation 11-22)

Where: f_y and E are the steel yield strength and elastic modulus, respectively. Most reinforcing steel used currently in piles/ shafts is Grade 420 (U.S. Grade 60), which has a nominal yield stress f_y = 420 MPa (60 ksi).

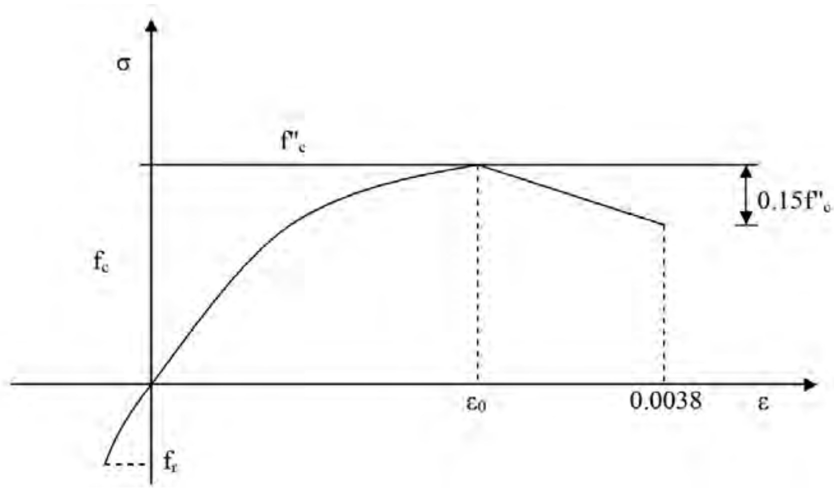


Figure 11-5: Assumed stress-strain relation for concrete (from O'Neill and Reese 1999).

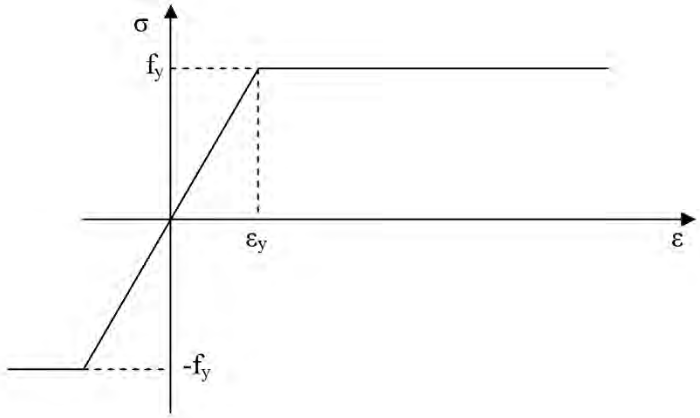


Figure 11-6: Assumed stress-strain curves for steel (from O'Neill and Reese 1999).

The derivation of the relation between bending moment, axial load, and E_pI_p proceeds by assuming that plane sections in a beam-column remain plane after loading. Therefore, when an axial load (P_x) and a moment (M) are applied to a section, it results that the neutral axis is displaced from the center of gravity of a symmetrical section. The equilibrium equations for such condition can be expressed as follows:

$$P_x = b\int_{-h_1}^{h_1} \sigma \, dy \qquad \text{(Equation 11-23)}$$

$$M = b\int_{-h_1}^{h_1} \sigma \, y \, dy \qquad \text{(Equation 11-24)}$$

Where:

$\Sigma = S$tress normal in the section.

y = Vertical coordinate in the section from the center of gravity of the section.

Other terms are defined in Figure 11-7 for a circular section. Note that integration above considers the forces caused by σ in each of the infinitesimal horizontal bands with width (b) and thickness (dy) shown in Figure 11-7.

The value of E_pI_p for reinforced concrete can be taken as that of the gross section. However, as the loading increases cracking of the concrete will occur, causing a significant reduction in E_pI_p. Further reductions occur as the bending moment further increases; therefore, a modification in E_pI_p may be needed for accurate computations, especially if deflection controls.

The numerical procedure for determining the relationship between axial load, bending moment, and E_pI_p of the section, considers the nonlinear stress-strain properties of the concrete and steel and the combined action of the (compressive) axial load and bending moment. The procedure, which is typically conducted in these computer programs, is summarized below.

- The dimensions of the section, as well the amount and distribution of longitudinal reinforcement are selected. Geometrical properties (areas, reinforcement spacing, section covers, etc.) must be selected.
- The neutral axis is selected. A strain gradient Φ_ε across the section about the neutral axis is also selected. Φ_ε is defined such that the product of Φ_ε and distance *y* from the neutral axis gives the strain at this specific distance from the neutral axis. Φ_ε, which has units of strain/length, is assumed to be constant, whether the section is in an elastic or inelastic state. This step defines the strain at every point in the section.
- With the strain distribution in the section and with the stress-strain relationships for the steel and concrete shown earlier, the distribution of stresses across the cross-section can be computed numerically.
- The resultant of normal stresses on the section is calculated with Equation 11-23. If the computed value is different to the applied axial load (P_x), the position of the neutral axis is moved and the computations are repeated. This process is continued until the computed value of P_x is equal to the applied value of P_x.
- The bending moment associated with this condition is then computed by summing moments from the normal stresses in the cross-section about a convenient point in the section (e.g., the centroidal axis or the neutral axis) using Equation 11-24.
- From beam theory, it can be shown that $E_pI_p = M/\Phi_\varepsilon$. Therefore, a unique relationship between P_x, M, and E_pI_p is found for a given section considering the selected amount and distribution of reinforcement, and the material properties. The process is repeated for different values of Φ_ε.
- The E_pI_p value for this combination of axial load P_x is then determined.

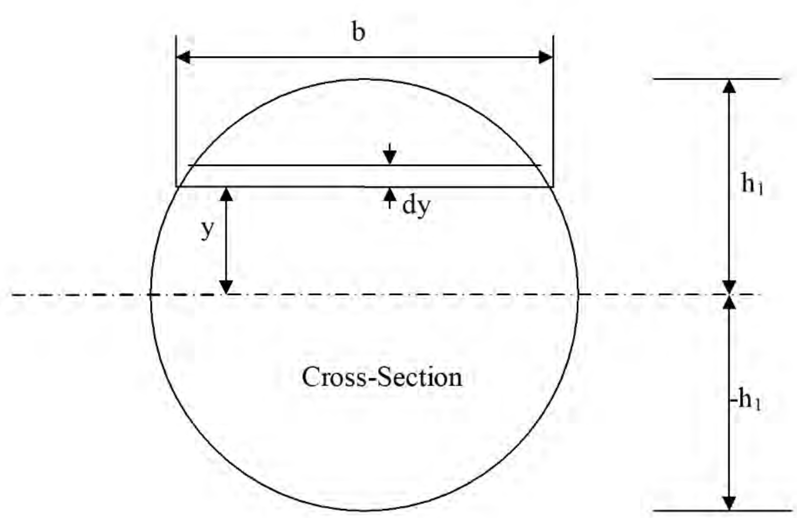


Figure 11-7: Bending in a circular section.

11.3.11 Pre-Stressed Concrete

Pre-stressed concrete piles have been widely used in deep foundation construction when the subsurface and environmental conditions are suitable for pile driving. A pre-stressed concrete pile has a configuration similar to a conventional reinforced-concrete pile except that the longitudinal reinforcing steel is replaced by pre-stressing steel. The pre-stressing steel is usually in the form of strands of high-strength wire that is placed inside a cage of spiral steel to provide lateral reinforcement. As the term implies, pre-stressing creates an initial compressive stress in the pile so the pile is more capable of providing bending resistance. Pre-stressed piles can usually be made lighter and longer than reinforced-concrete piles of the same size.

The application of a bending moment typically results in a reduction of compressive stresses rather than cracking as with conventional reinforced concrete members. Thus, there is a significant improvement in bending stiffness of the pre-stressed pile as compared to a conventionally reinforced pile. A further benefit is that the pre-load protects the pile from cracking due to tensile stresses developed during pile driving operations. The use of pre-stressing leads to a reduction in the ability of to resist normal compressive loads, a factor that is usually not critical in laterally loaded piles. Considerable bending moment may be applied to a pre-stressed pile before first cracking. Consequently, the pile-head deflection of the pre-stressed pile in the uncracked state is substantially reduced, and its performance under service loads is improved.

When analyzing a foundation consisting of pre-stressed concrete piles, the designer must define a value for the level of stress due to pre-stressing, after considering losses due to creep and other factors. The value will vary with manufacturers from region to region and will also vary with the geometry of the pile.

11.4 PROCEDURES FOR STRUCTURAL STEEL SECTIONS

This manual provides a description of the structural design considerations for foundation elements constructed from structural steel. Foundation elements covered under this manual included steel H-sections, steel pipe piles and fabricated steel sections.

11.4.1 Material Properties - General

Unless otherwise noted, materials and material properties should be in accordance with the provisions of the AASHTO LRFD Specifications (2014). Structural considerations of these material properties are discussed hereafter.

11.4.2 Material Properties – Structural Steel

The modulus of elasticity of all grades of structural steel can be assumed as 29,000 ksi.

Information on the type, grades and uses of structural steel can be found in Section 6 of AASHTO (2014).

11.4.3 Axial Compression

For axial compression loading, the factored Structural Limit State is taken as:

$$P_r = \varphi c P n \quad \text{(Equation 11-25)}$$

Where:

P_r = Factored compressive resistance (kips).

φ_c = Resistance factor.

P_n = Nominal compressive resistance (kips).

To determine the nominal compressive resistance however, pile strength and buckling failure should be considered, where a step-by-step procedure is presented as follows.

Step 1: Determine the equivalent nominal yield resistance, P_o:

The equivalent nominal yield resistance, P_o, is a function of the material yield stress, cross sectional area and slenderness reduction factor, if applicable. For non-slender piles in compression, the slenderness reduction factor, Q, is taken as 1.0. However, for slender piles, the full nominal yield strength under uniform axial compression is limited by local buckling. This reduction factor is governed by section buildup, pile dimensions and material properties, therefore, a further discussion of slender members and direction for calculating Q may be found in AASHTO (2014) Article 6.9.4.2.2.

$$P_o = QF_yA_g \qquad \text{(Equation 11-26)}$$

Where:

A_g = Gross cross-sectional area (in^2).

P_o = Equivalent nominal axial yield resistance (kips).

F_y = Yield stress of steel.

Q = Slender element reduction factor.

To satisfy the slender element requirement for local buckling, Equation 11-27 is used for H-piles while Equation 11-29 is used for unfilled pipe piles.

$$\frac{b_f}{2t_f} \leq 0.64\sqrt{\frac{k_cE_{st}}{F_y}} \qquad \text{(Equation 11-27)}$$

and:

$$0.35 \leq k_c \leq 0.76$$

in which:

$$k_c = \frac{4}{\sqrt{\frac{d_w}{t_w}}} \qquad \text{(Equation 11-28)}$$

Where:

b_f = Flange width (in).

t_f = Flange thickness (in).

F_y = Yield stress of steel (ksi).

E_{st} = Elastic modulus of steel (ksi).

d_w = Web depth (in).

t_w = Web thickness (in).

$$\frac{D}{t} \leq 0.11 \frac{E_{st}}{F_y}$$ (Equation 11-29)

Where:

D = Diameter of pipe (in).

t = Wall thickness (in).

F_y = Yield stress of steel (as per AASHTO).

E_{st} = Elastic modulus of steel (ksi).

Step 2: Determine the elastic critical buckling resistance, P_e.

In determination of the nominal compressive resistance, buckling may occur with a lack of sufficient bracing. AASHTO (2014) requires both flexural and torsional modes of buckling be checked if applicable. For fully embedded piles, the flexural buckling mode will be used. However, when the pile extends through water or air, doubly symmetric open section members (e.g., H-piles) must be evaluated for torsional buckling as well. The critical failure mode is the lesser buckling resistance, and is employed to define the nominal compressive resistance.

Flexural buckling:

$$P_e = \frac{\pi^2 E_{st} A_g}{\left(\frac{Kl}{r_s}\right)^2}$$ (Equation 11-30)

Where:

Pe = Elastic critical buckling resistance (kips).

Est = Elastic modulus of steel (ksi).

= Gross cross-sectional area (in^2).

K = Effective length in the plane of buckling (Table 11-1) (dimensionless).

l = Unbraced length in the plane of buckling, or laterally unsupported length plus d_f where d_f is the depth to fixity below the ground (in).

r_s = Radius of gyration about axis normal to plane of buckling (in).

Torsional buckling:

$$P_e = \left[\frac{\pi^2 E_{st} C_w}{(K_z I_z)^2} + GJ\right] \frac{A_g}{I_x + I_y}$$ (Equation 11-31)

In which:

$$C_w = \frac{I_y h^2}{4} \quad \text{(Equation 11-32)}$$

$$G = 0.385E_s \quad \text{(Equation 11-33)}$$

Where:

Pe = Elastic critical buckling resistance (kips).
Est = Elastic modulus of steel (ksi).
Cw = Warping torsional constant (doubly symmetric open sections) (in^6).
Kz = Effective length for torsional buckling (dimensionless).
lz = Unbraced length for torsional buckling (in).
G = Shear modulus (ksi).
J = St. Venant torsional constant (in^4).
Ag = Gross cross-sectional area (in^2).
I_x , I_y = Moments of inertia about the major and minor principal axes of cross section, respectively (in^4).
h = Distance between flange and centroids (in).

Step 3: Determine the nominal axial compressive resistance, P_n.

With the above resistances defined, the nominal resistance for axial compression may be evaluated using the following equations, which are provided in AASHTO (2014) Article 6.9.4.1.

If $\frac{P_e}{P_0} \geq 0.44$

$$P_n = P_0 0.658^{\frac{P_0}{P_e}} \quad \text{(Equation 11-34)}$$

If $\frac{P_e}{P_0} < 0.44$

$$P_n = 0.877P_e \quad \text{(Equation 11-35)}$$

Where:

P_n = Nominal compressive resistance (kips).
P_0 = Equivalent nominal yield resistance (Eq. 11-26) (kips).
Pe = Elastic critical buckling resistance (Eq. 11-30 or 11-31) (kips).

11.4.4 Flexure

For flexure, the factored Structural Limit State is taken as:

$$M_r = \varphi_f M_n \qquad \text{(Equation 11-36)}$$

Where:

M_r = Factored flexural resistance (kip-in).

φ_f = Resistance factor.

M_n = Nominal flexural resistance (kip-in).

The nominal flexural resistance is a function of pile shape as well as general pile properties. Steel piles are primarily H-piles or pipe piles, therefore the step-by-step procedure that follows will consider only these two steel pile types. If alternative sections are used, the engineer is referred to Article 6.12.2.2 of the AASHTO (2014) specifications. Steel H-piles and I-sections are treated equally for flexural resistance; therefore, part A of this procedure applies to both steel H-piles and miscellaneous I sections.

11.4.5 Step-by-Step Procedure for: "Nominal Flexural Resistance" for Linear Behavior

11.4.5.1 Steel H-Section

Step 1: Check flange slenderness ratio and limiting slenderness.

$$\lambda_f = \frac{b_f}{2t_f} \qquad \text{(Equation 11-37)}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_{yf}}} \qquad \text{(Equation 11-38)}$$

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_{yf}}} \qquad \text{(Equation 11-39)}$$

Where:

λ_f = Slenderness ratio for flange.

λ_{pf} = Limiting slenderness ratio for a compact flange.

λ_{rf} = Limiting slenderness ratio for a non-compact flange.

b_f = Flange width (in).

t_f = Flange thickness (in).

E_{st} = Elastic modulus of steel (ksi).

F_{yf} = Minimum yield strength of lower strength flange (ksi).

Step 2: Determine the nominal flexural resistance.

To determine the nominal flexural resistance, the above slenderness definitions should first be resolved. These functions serve to define the limiting flexural resistance. In the case where the limiting slenderness ratio of a compact flange is greater than the slenderness ratio, the plastic moment about the weak axis will limit resistance. For H-piles, Equation 11-37 can be used. Conversely, Equation 11-39 should be used when the slenderness ratio is greater than the limiting slenderness ratio of a compact flange.

If $\lambda_f \leq \lambda_{pf}$

$$M_n = M_p \qquad \text{(Equation 11-40)}$$

In which, for HP-sections about the weak axis:

$$M_p = 1.5F_y S_y \qquad \text{(Equation 11-41)}$$

If $\lambda_{pf} < \lambda_f \leq \lambda_{rf}$ the nominal flexural resistance about the weak axis is:

$$M_n = [1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_{yf}}}}\right)]f_{yf} Z_y \qquad \text{(Equation 11-42)}$$

Where:

M_n = Nominal flexural resistance (kip-in).

M_p = Plastic moment about the weak axis (kip-in).

S_y = Elastic section modulus about weak axis (in^3).

Z_y = Plastic section modulus about weak axis (in^3).

λ_f = Slenderness ratio for flange (Eq. 11-37, dimensionless).

λ_{pf} = Limiting slenderness ratio for a compact flange (Eq. 11-38, dimensionless).

E_{st} = Elastic modulus of steel (ksi).

F_y = Yield stress of steel (ksi).

F_{yf} = Minimum yield strength of lower strength flange (ksi).

11.4.5.2 Steel Pipe Piles

Step 1: Check diameter to thickness ratio

If the diameter to thickness ratio is sufficiently large, local buckling limits flexural resistance. To inspect whether the plastic moment or local buckling will govern flexural resistance, Eq. 11-43 should be applied. If Eq. 11-43 is satisfied, the plastic moment will yield the steel pile and Step 2a should follow. Conversely, local buckling will limit flexural resistance if Eq. 11-43 is not satisfied, and therefore Step 2b should follow.

$$\frac{D}{t} \leq 0.07\frac{E_{st}}{F_y} \qquad \text{(Equation 11-43)}$$

Where:

D = Outside diameter of pipe (in).

t = Pipe thickness (in).

Est = Elastic modulus of steel (ksi).

Fy = Yield strength of steel (ksi).

Step 2a: Determine nominal flexural resistance by plastic moment

The nominal flexural resistance can be taken as follows:

$$M_n = M_p = F_y Z_y \qquad \text{(Equation 11-44)}$$

Where:

M_n = Nominal flexural resistance (kip-in).

M_p = Plastic moment (kip-in).

Z_y = Plastic section modulus about weak axis (in^3).

F_y = Yield strength of steel (ksi).

Step 2b: Determine nominal flexural resistance by local buckling

Where local buckling will limit the nominal flexural resistance, the following checks apply.

If $\frac{D}{t} \leq 0.31 \frac{E_{st}}{F_y}$

$$M_n = \left(\frac{0.021\, E_{st}}{\frac{D}{t}} + F_y \right) S_y \qquad \text{(Equation 11-45)}$$

in which:

$$f_{cr} = \frac{0.33\, E_{st}}{\frac{D}{t}} \qquad \text{(Equation 11-46)}$$

Where:

D = Outside diameter of pipe (in).

t = Pipe thickness (in).

Es = Elastic modulus of steel (ksi).

Fy = Yield strength of steel (ksi).

Mn = Nominal flexural resistance (kip-in).

Sy = Elastic section modulus about weak axis (in^3).

fcr = Elastic local buckling stress (ksi).

11.4.6 Combined Flexure and Axial Compression

Combined axial and flexure checks are only applied to pile groups with vertical piles. At this time, AASHTO (2014) does not have a recommendation to include battered piles. For combined flexure and compression of vertical piles, AASHTO (2014) requires the factored Structural Limit State to satisfy the following Limit State checks.

If $\frac{P_u}{P_r} < 0.2$

$$\frac{P_u}{2.0\ P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$ (Equation 11-47)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$ (Equation 11-48)

Where:

P_u = Factored axial compressive load (kips).

P_r = Factored compressive resistance (kips).

M_{ux} = Factored moment about x-axis (kip-ft).

M_{rx} = Factored flexural resistance about x-axis (kip-ft).

M_{uy} = Factored moment about y-axis (kip-ft).

M_{ry} = Factored flexural resistance about y-axis (kip-ft).

11.4.7 Cases with Axial and Bending Moment (Non-Linear Behavior)

Various references on steel design provide tabulated values of the plastic modulus (Z_x) to obtain the plastic or yield moment, M_{ult} based on the distribution of stresses in the steel section shown in Figure 11-8 for a typical H-section. The computation of M_{ult} is then performed with:

$$M_{ult} = f_y Z_x$$ (Equation 11-49)

Where: f_y is the steel yield strength, and Z_x is the plastic modulus (bending about the x-axis of the section).

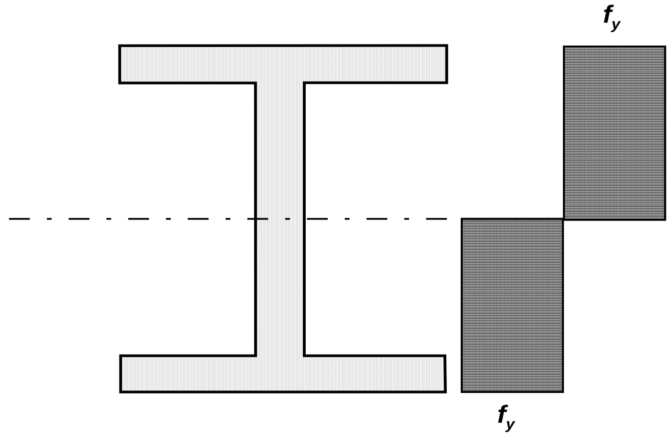


Figure 11-8: Sketch for computing plastic moment of steel H-section.

Figure 11-9 shows the reduction of the ultimate moment due to axial loading for H-sections (Horne 1971). The values in the horizontal axis in this graph are the applied axial load normalized by the (non-eccentric) compressive axial load capacity, P_u. The values in the vertical axis in this graph are normalized values of the yield moment.

P_u is obtained as follows:

$$P_u = f_y A \quad \text{(Equation 11-50)}$$

Where A = cross-sectional area of pile.

The relationship between the normalized load and normalized yield moment becomes slightly non-linear for normalized yield moments greater than about 90 percent. Therefore, the effect of axial loads causes a reduction in plastic moment capacity (as compared to a linear relationship) that is small when the applied axial load is approximately 20 percent or smaller than P_u.

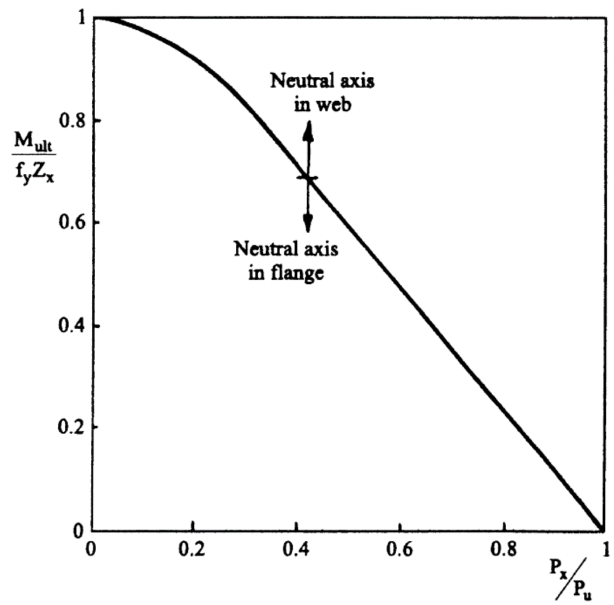


Figure 11-9: Effect of axial loading on plastic moment in steel H-section.

As a result, a constant, elastic value of the section bending stiffness, and therefore yield moment, for a steel section can be used for all ranges of normal loads without significant error. If desired, the equations may be modified to reflect the nonlinear behavior. However, reduced values of the modulus of the steel section will affect the computed value of the yield moment only slightly.

11.4.8 Steel Pipe Section

As for steel pipe section, the elastic bending stiffness can also be used without much error in computing the bending moment. The moment of inertia of a pipe section is computed with the following equation:

$$I_p = \frac{\pi(d_0^4 - d_i^4)}{64} \qquad \text{(Equation 11-51)}$$

where d_0 and d_i are the outer and inner diameters of pipe piles, respectively.

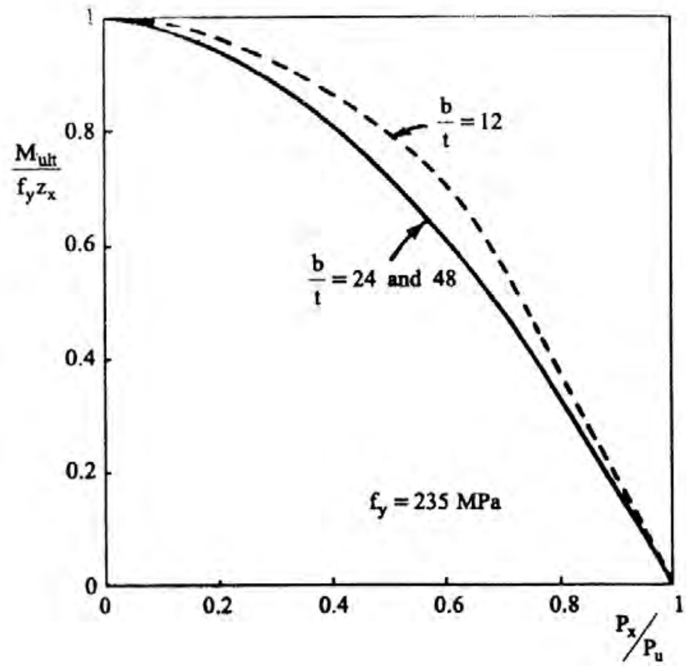
Considering the distribution of stresses in the pipe section as that shown in Figure 11-8, the ultimate bending moment is computed as:

$$M_{ult} = f_y Z_p \qquad \text{(Equation 11-52)}$$

Where:

$$Z_p = \frac{1}{6}(d_0^3 - d_i^3) \qquad \text{(Equation 11-53)}$$

The influence of the axial loading on the yield moment are shown in Figure 11-10 (Horne 1971) for f_y = 235 MPa (34 ksi) and ratios of diameter to wall thickness equal to 12 to 48. Note that in this figure, the diameter and wall thickness are denoted as *b* and *t*, respectively.



(Note fy = 234 MPa equivalent to 34 ksi)

Figure 11-10: Effect of axial loading on plastic moment in steel pipe piles.

Similarly to H-sections, the relationship between the normalized load and normalized yield moment becomes non-linear for smaller axial loads. This effect also causes a reduction in plastic moment capacity (as compared to a linear relationship).

11.5 PROCEDURES FOR STRUCTURAL COMPOSITE SECTIONS

Composite piles for structural applications are defined as concrete filled steel pipe piles. Guidance for other composite pile types is not provided for in AASHTO (2014) specifications, and therefore structural resistances over the length of alternative pile materials should be evaluated considering the primary section material.

11.5.1 Structural Resistance

11.5.1.1 Axial Compression

The behavior of composite sections is somewhat different than non-composite members and is therefore evaluated by alternate means. AASHTO (2014) provides evaluation methods to assess the Strength Limit State of composite sections, provided the following criteria are met.

1. The cross-sectional area of steel is at least 4 percent of the total cross-sectional area. If the cross-sectional area of steel is less than this limit, the pile is considered non-composite and should be evaluated following procedures in Section 11.3 Procedures for Reinforced Concrete Sections Concrete Piles.
2. The concrete compressive strength is between 3.0 ksi and 8.0 ksi. Commentary provided in AASHTO (2014) notes the lower limit is imposed to encourage use of good quality concrete.
3. The yield strength of longitudinal reinforcement to determine the compressive resistance cannot exceed 60.0 ksi.

For axial compression loading of concrete filled steel pipes, the factored Structural Limit State is taken as:

$$P_r = \varphi P_n \quad \text{(Equation 11-54)}$$

Where:

- Pr = Factored compressive resistance (kips).
- φ = Resistance factor.
- Pn = Nominal compressive resistance (kips).

11.5.2 Step-by-Step Procedure for: "Nominal Flexural Resistance"

Step 1: Determine the normalized column slenderness factor, λ.

The normalized column slenderness factor is to be evaluated using the following equation. However, if the pile is fully embedded, λ may be taken as 0 (AASHTO 2014).

$$\lambda = \left(\frac{Kl}{\pi r_s}\right)^2 \frac{F_e}{E_e} \quad \text{(Equation 11-55)}$$

In which:

$$F_e = F_y + C_1 F_{yr}\left(\frac{A_{str}}{A_{st}}\right) + C_2 f_c'\left(\frac{A_c}{A_{st}}\right) \quad \text{(Equation 11-56)}$$

$$E_e = E_{st}\left[1 + \left(\frac{C_3}{n}\right)\left(\frac{A_c}{A_{st}}\right)\right] \quad \text{(Equation 11-57)}$$

$$n = \frac{E_{st}}{E_c} \quad \text{(Equation 11-59)}$$

Where:

- λ = Normalized column slenderness factor.
- A_{st} = Cross sectional area of steel (in^2).
- A_c = Cross sectional area of concrete (in^2).
- A_{str} = Cross sectional area of longitudinal reinforcing steel (in^2).
- K = Effective length factor
- l = Unbraced length in the plane of buckling (in).
- r_s = Radius of gyration about axis normal to plane of buckling (in).
- F_e = Nominal compressive resistance of composite section (ksi).
- F_y = Yield stress of steel (ksi).
- F_{yr} = Yield stress of reinforcing steel (ksi).
- f'_c = Concrete compressive strength at 28 days, unless otherwise specified (ksi).
- E_e = Modified elastic modulus of steel for composite column (ksi).

E_t = Elastic modulus of steel (ksi).

E_c = Elastic modulus of concrete (ksi).

C_1 = Composite column constant 1 (1.00 for concrete filled pipes).

C_2 = Composite column constant 2 (0.85 for concrete filled pipes).

C_3 = Composite column constant 3 (0.40 for concrete filled pipes).

Step 2: Determine the nominal axial compressive resistance, P_n.

After determining the normalized column slenderness ratio, a relatively straightforward calculation of the nominal compressive resistance is made using either Equation 11-60 or Equation 11-61.

If $\lambda \leq 2.25$:

$$P_n = 0.66^{\lambda} F_e A_{st} \qquad \text{(Equation 11-60)}$$

If $\lambda > 2.25$:

$$P_n = \frac{0.88 F_e A_{st}}{\lambda} \qquad \text{(Equation 11-61)}$$

Where:

λ = Normalized column slenderness factor.

P_n = Nominal compressive resistance (kips).

A_{st} = Cross sectional area of steel (in^2).

F_e = Nominal compressive resistance of composite section (ksi).

12 LATERAL LOAD TESTS

The behavior of deep foundations under lateral loads is often difficult to predict accurately, especially with limited published material models that may not correlate well with local geologic materials. Lateral load tests can provide site-specific information that can be used to develop site-specific p-y curves, develop or verify a project design and/or be used in future designs in the same geologic formations. Lateral load tests reduce the uncertainty and potential risks associated with the design, and may result in more efficient (lower cost) foundations. Documenting and publishing lateral load test results also adds value to the overall industry by enabling other practitioners and researchers to benefit from the testing without having to incur the significant cost of implementing a test program

12.1 CONSIDERATION FOR PLANNING LATERAL LOAD TESTS

Lateral load tests can be used to fulfil three functions:

1. To develop site specific parameters/p-y curves and investigate performance for development of the foundation design,
2. To verify the adequacy of the foundation design during construction through proof testing, and
3. For research and documentation purposes, to further the state of practice.

This chapter has its focus on static lateral load testing of single full scale foundation elements for use by the designer. Guidance is provided on the planning and execution of static lateral load tests, including aspects of testing and monitoring, approaches to data reduction, and analysis and derivation of p-y curves for use in design. The p-y method is used for detailed analyses of laterally loaded deep foundations. Other more simplified methods (i.e., Broms method) would not require full scale load testing, and design methods of similar or more complex application (strain wedge method or numerical modeling) would be used in addition to the p-y method.

Much of the discussion, in particular with regard to test set-up and instrumentation, is based on testing of individual driven piles and drilled shafts, but the same principles apply to other types of deep foundations.

Testing of pile groups under lateral loads is rarely done in practice, given the significant costs that are not typically justified, even on major projects. Accordingly, this topic is not addressed in detail.

A guide specification for lateral load testing of a single deep foundation element is provided in Appendix D.

12.1.1 Lateral Load Tests for Design

A lateral load test is performed to measure the load-deflection performance of a deep foundation element (pile or drilled shaft) for the anticipated means and methods of construction, and for the ground and groundwater conditions of a site. For lateral load tests performed during design, preliminary analyses using available data, including p-y curves, is necessary in order to develop the load test program. Information needed to plan a lateral load test includes subsurface conditions, foundation element type, depth, planned test loads and loading intervals, performance criteria, and expected performance.

For structures where lateral loads govern the design, lateral load tests provide a means to investigate and confirm key issues or design inputs such as deflection versus load performance, required minimum depth, and depth and magnitude of maximum bending moment for reinforcement design. There may also be opportunities to optimize the design by developing site-specific p-y curves from the test that may demonstrate higher lateral capacities compared to the “standardized” p-y models available in published references or software programs. In addition, it may be possible to develop different p-y curves for Service Limit State and Strength Limit State design cases (which may be based on different limiting deflections), which may allow further refinement in the design.

The decision to implement a lateral load test program should consider the following issues:

- Engineering time and cost for planning and design of the test program, preparation of contract documents, monitoring the load test(s), and to reduce and evaluate the load test results;
- Cost for implementing the load test program, including equipment mobilization, materials, instrumentation equipment, and test frame materials and set-up;
- Cost of sacrificial foundation elements for testing (foundations for lateral load tests are typically not performed on production piles or shafts because they are likely to be loaded beyond the service limit deflections and possibly beyond the deflections used to define the strength limit);
- Cost of load reaction system, including the installation of reaction piles or drilled shafts;
- Costs for supervising and conducting the load tests, and professional services for instrumentation installation and monitoring during the load test, and for preparing the load test report;
- Costs of supplemental subsurface exploration and in-situ testing at the site of the lateral load tests to correlate with the p-y curves derived from the test; and
- The potential cost savings that may be achieved in the event that the lateral load tests result in stiffer p-y relationships than standardized relationships would suggest. (This task requires assumptions regarding a reasonable expectation of increased soil stiffness, and foundation analyses to estimate the potential savings in foundation arrangement and cost associated with this increased soil stiffness).

12.1.2 Lateral Load Tests in Construction to Verify the Design

Lateral load tests may be required in construction in order to verify the adequacy of the design. This may be done to reduce risk, particularly where there is greater uncertainty regarding design parameters. Performing lateral load testing in construction may be simpler from a procurement standpoint, especially for design-bid-build, because it does not require a separate contract or expense during the design phase.

If inadequate performance of the foundation is considered a significant risk or possibility, then the specifications should require that the lateral load testing be performed and results provided within a certain period of time prior to ordering or mobilizing material and additional equipment for production work. This will help mitigate some of the cost impacts that may result from changes in the foundation design. The risk of re-design during construction can be further mitigated by providing a more robust foundation design that considers less favorable site conditions rather than average conditions.

12.1.3 Considerations regarding Subsurface Characterization for Lateral Load Test Program

Subsurface exploration and testing for a lateral load test program should be planned and performed in accordance with approaches described in Chapter 3. Focus should be placed on characterizing the ground profile and engineering properties of the near surface soil (and rock) as it is in this zone that most of the resistance to lateral loads will need to be derived. That is not to say that the characteristics of the deeper strata can be ignored. These strata are also important for defining the p-y response and for assessment of foundation fixity. If the foundations are long, and rely on rock sockets at depth for lateral resistance, then the strength, stiffness and rock mass characteristics (e.g., RQD, orientation of discontinuities, nature of infill, presence of clayey gouge, weak laminated inter-beds, anisotropy) are particularly important.

If the lateral load tests are for research purposes, then it is recommended that a detailed program of advanced in-situ and laboratory testing be performed at the load test site to provide a means by which to correlate p-y curve responses to ground engineering parameters (e.g., Simpson and Brown 2003). If the lateral load test is performed as part of a project, then there must be an adequate level of investigation at both the test location and the production foundation locations to be sure that the subsurface conditions at the test site(s) are representative.

The groundwater conditions likely to prevail during the life of the structure need to be considered in planning the lateral load test. Depending on the time of year that the test(s) is done, the seasonal groundwater level may be lower than the elevation that will occur at other times of the year or during flood events. In conditions such as this, it is desirable to saturate the soil at the test location prior to the test, if practical to do so. An example of this was presented by Dunnavant and O'Neill (1989) who performed lateral load tests on open-ended steel pipe piles and drilled shafts in the over-consolidated Beaumont Clay of Houston, Texas. Five months prior to installation of the foundation elements, the test pits in which construction would occur were intentionally flooded with water. During the load tests, these pits were maintained in the submerged condition.

Any anticipated changes in the site conditions at the production foundation locations, such as placement of fill, cuts, or removal and replacement of unsuitable soils (soft soils, etc.) should be considered in the design of the test pile program and, where practical, such changes should be incorporated into the test program.

12.1.4 Considerations for Test Pile Location

The selection of the test location(s) should be made such that the ground profile is representative of the less favorable ground and groundwater conditions into which the production piles/shafts will be installed. In considering what is representative, emphasis should be placed on the upper strata, say within 5 to 10 diameters of the ground surface, excluding any depth of possible soil disturbance or scour. It is typically in this zone that a pile or drilled shaft will need to derive most of the lateral resistance to load, and it is also the zone in which bending moments will likely be highest. Unfortunately, the uppermost soil layers are often those least sampled and tested in subsurface exploration programs, and are typically very variable in terms of composition and consistency. Often it is the deeper bearing strata that receive most attention because it is in the deeper zone where foundation elements will achieve resistance for axial capacity. Specifications for subsurface exploration should require sampling in the near surface with appropriate laboratory testing to characterize the strength and stiffness properties of the ground for laterally loaded pile/shaft analysis. These near surface data are essential for correlating the p-y response from the load test to the site-specific conditions.

For rock socketed foundation elements, such as drilled shafts, the depth and quality of rock at the test location should be representative of the less favorable rock conditions that may be encountered at the production foundation locations. Testing rock that is less weathered, stronger, higher quality, shallower depth, or otherwise more favorable compared to production pile/shaft conditions may produce test results that are unconservative for design and may not assure required foundation performance at the different foundation locations. Rock sockets for test foundations should be similar to the socket lengths anticipated for production foundations.

Given the test piles or shafts will likely be sacrificial, it is important to locate them and the reaction system where they will not pose an obstruction to the proposed foundations of the permanent works, recognizing that the locations of the permanent foundations may not be known at the time of planning the test.

12.1.5 Considerations for the Design of the Test Pile/Shaft

Pile/shaft length may be governed by axial capacity or lateral capacity. For a lateral load test, the length of the test pile/shaft should be at least the minimum length estimated for the production foundations. It is recommended that an additional length be considered for driven test piles, beyond the minimum indicated by analysis, in the event the piles need to be driven deeper to achieve the required resistance to axial loads. If the pile is driven to the minimum design length, and that length is determined to be inadequate for axial loading considerations, the data from the lateral load test may not be representative of production pile installations. The additional length will be a matter of judgment and may vary based on the overall pile length and size, as well as subsurface conditions (depth to harder strata, cost of additional length, etc.)

It is important that the foundation element be long enough to exhibit bending and not fail through rotation. The rigid body rotation response of short stiff piles or shafts (say length to diameter ratios less than 10) is quite different to the bending and deflection exhibited by long flexible foundation elements; the test pile length should be sufficient to produce bending and deflection rather than rotation.

If the axial capacity of the foundation requires a greater length than required for lateral resistance, it may be sufficient to design the test pile/shaft based on the lateral resistance length and not necessarily the full length that is required for axial resistance, provided the response to lateral loading of the shorter element will be the same as for the full-length element.

The preferred approach is to construct the test pile/shaft to the same diameter as the production piles/shafts to provide consistency in the test results, interpretation of p-y curves, and calibration of soil models with the production piles. Smaller diameter or prototype piles or shafts will be more strongly influenced by soils at shallower depths compared to larger diameter production piles/shafts. Also, a smaller diameter pile or shaft may not have enough stiffness to transfer stresses to the depths that would be influenced by a larger pile/shaft because the flexural stiffness of a pile or shaft is proportional to the fourth power of the shaft diameter (Brown et al. 2010).

If lateral load tests are performed during the design phase with a view to optimizing the design of the foundations, or if the test foundations are for research purposes, it is recommended that the foundation element be designed with a high shear and moment capacity, possibly higher than what would be used for a production pile/shaft. In this way, the foundation element will be able to deflect significantly under loads that are likely to be high enough to mobilize the soil resistance, but without failing the structural element. This may be particularly applicable for CFA piles, drilled shafts, or micropiles, where additional reinforcement can be included in order to increase the shear and moment capacity of the test pile compared to a production pile. However, using a stiffer element may lead to smaller deflections at the service load.

Any other aspects of the design that will be included on production piles/shafts should also be included for the test piles/shafts. For example, coatings on piles for corrosion protection, which may also impact the frictional and shearing resistance between the pile and the adjacent ground, should be included on the test pile. If it is not possible to do so, the potential differences and risks of different behavior between the test and production piles should be assessed and possibly addressed through the use of a resistance factor.

12.1.6 Considerations for Test Pile Installation Methods

Ideally, the test piles/shafts should be installed using the same or similar means and methods for construction that are anticipated or required for the production piles/shafts. It is especially important to use methods for the test elements that will result in similar disturbance or lack thereof in the immediate area around the pile/shaft as compared to the production piles/shafts. For example, the installation of driven piles may result in a densification of the soils in the immediate vicinity of the pile; pre-drilling of test piles should be avoided unless predrilling is also required for the production piles. Similarly, the use of oversized surface casing for a drilled test shaft should be avoided unless such measures are intended for the production shafts as well.

During a design-phase test program, it may not be possible to anticipate and use the same means and methods for installation of the test elements as will be used during the subsequent construction contract. Similarly, if the test is performed early in a design-build contract, before the construction contractor finalizes his means and methods, there may be some inconsistency between installation of the test pile/shaft and production piles/shafts. The methods for lateral pile analyses do not account for different installation methods, but it is known that installation means and methods can have a significant influence on the test results. If the designer is comfortable that different methods will not compromise or alter the behavior of the subsurface materials under lateral loads, the data from the tests should be applicable. However, if, in the judgment of the designer, there exists some risk that the production pile/shaft behavior under lateral loads may be different than the test pile/shaft due to differences in the installation methods, it may be desirable to defer the test program until the means and methods of construction are defined, or plan the test program to model the likely more adverse condition that may be used in construction, or include more restrictive limits to the contractor’s means and methods in the contract specifications.

12.1.7 Coordination with Axial Load Tests

Lateral load tests can be performed on a test element used for axial load testing following completion of the axial load test. The lateral load test should not be performed before the axial load test as it may change the stress conditions and soil contact around the foundation element, which may impact the axial resistance (the lateral load test may require relatively high deflections in order to investigate the Strength Limit State). Axial load tests are discussed further in Hannigan et al. (2016), Brown, et al. (2010), Brown et al. (2007), and Sabatini et al. (2005) for driven piles, drilled shafts, CFA piles, and micropiles, respectively.

12.2 LATERAL LOAD TEST METHODS

Static lateral load tests are the most common form of lateral load test applied in practice. Other types of tests include the Rapid Load Test, also known as a Force Pulse lateral load test, and bi-directional load cell tests.

12.2.1 Static Lateral Load Tests

ASTM D3966 describes The Standard Method of Testing Deep Foundations Under Lateral Load. Several alternative systems for applying the lateral load to the deep foundation element and measurement movements are provided in this standard. Requirements for presenting the results of lateral load tests are also provided in this standard.

A conventional static lateral load test involves applying a load against a pile or drilled shaft, using a jack and reaction system. The load application can be by pushing or pulling, depending on the test set-up. Figure 12-1 below illustrates a typical set-up with a hydraulic jack and load cell bearing against the head of a drilled shaft. A curved concrete “saddle” is cast between the load cell and the surface of the shaft, in order to distribute the load evenly into the shaft and minimize rotation and torsion as the load is applied. This bearing surface may also be a hemi-spherical steel plate. Sometimes the head of the pile or shaft is made-up with a square or rectangular reinforced concrete pad or pedestal, against which the jack and load cell are placed. A quick setting high strength grout may be required between the foundation element and the bearing plate, to minimize stress concentrations.

An example of a one-way lateral load test set-up is shown in Figure 12-2. The hydraulic jack is bearing against steel channels and high tensile steel bars transfer the load into the reaction system (not shown). A load cell is located between the jack ram and the steel plate. Dial gauges are shown near the head of the casing, bearing against a reference beam. Cables from strain gauges are seen at the pile head, and the yellow pipe is an inclinometer casing for measuring deformations during the test.

Figure 12-1: Example of a two-way lateral load test set-up for a drilled shaft.

If a strut is used between the test pile and the reaction pile (e.g., in a pushing type test), it is good practice to install a pinned connection using a swivel joint (clevis) at one end of the strut to allow rotation without eccentricity of load application, as shown in Figure 12-3. Use of this type of connection also reduces the risk of the bearing plate or reaction frame components from becoming dislodged during the test, or worse "popping out" of the system, presenting a safety hazard as well as compromising the test data.

The jacks used to apply the load must function when oriented horizontally, without leaking of hydraulic fluid, and without piston malfunction. Although the load applied in lateral load testing of piles and drilled shafts is usually much less than axial tests, the displacements (and hence the amount of jack travel) will likely be greater. The amount of travel of the ram in the jack must be able to accommodate the anticipated magnitude of displacement. It is therefore important that during the planning of a lateral load test, a preliminary analysis is made so that the load capacity and lateral displacement are estimated so the capacity of the jack and load cell and the available travel of the ram and associated external instrumentation (see Section 12.6) can be selected appropriately. Jacks can also be controlled by compressed air. Some jacks are reciprocating, or double acting, so that reversals in load direction can be performed. An alternative to jacks are electric winches, pulley blocks and turn buckle devices.

Figure 12-2: Example of a one-way lateral load test set up on a deep foundation.

To reduce thermal effects on the reference beam (the reference point for measuring horizontal displacements), it is good practice to shield the test site from direct sunlight with a canopy and to protect the site from wind and frost action. Depending on location and season, a temporary enclosure around the test set-up may be required, with temporary heating to keep the ambient air temperature above 50ºF. The temporary enclosure may be constructed using framing, wood panels, canvas, tarpaulins and other materials. Ambient air temperature should be monitored during the test.

The reference beam, which is typically steel, should be robust so as not to deflect between its supports. Painting the beam white can help to reduce thermal induced movements in the beam. The load measurement device should be independent of the jack, although the pressure gauge on the jack itself is a useful backup device in the event that the load cell malfunctions during a test. Although rarely done, if combined axial and lateral load tests are performed (ASTM D3966 Procedure H), then an anti-friction bearing plate assembly is required between the axial load and the head of the pile or drilled shaft. Steel plates with roller bearings, or a PTFE polymer slide bearing on a stainless-steel plate can be used. The purpose of the plate is to enable lateral movement of the head of the pile or shaft without being restrained by the axial load above.

Given the expense and time associated with planning, mobilizing, pile/shaft installation and testing, it is advisable to require the testing contractor to be equipped with spare parts for the key electrical and hydraulic components in the jacking systems used, and to have on site a back-up system for the pumps, ram jacks, and load cell. Some economy can be achieved in a lateral load test program if the reaction piles or shafts from a conventional axial compression load test are used for lateral load testing <u>after</u> the axial test has been performed. If this approach is adopted, the locations and types of reaction pile/shaft need to be considered in the planning so that an appropriate lateral load test set-up can be constructed. It is also possible to test two adjacent piles/shafts simultaneously by pulling the piles together or pushing them apart (Figure 12-3). By adopting this approach, two independent data sets are obtained that may provide insights into the repeatability of the construction means and methods.

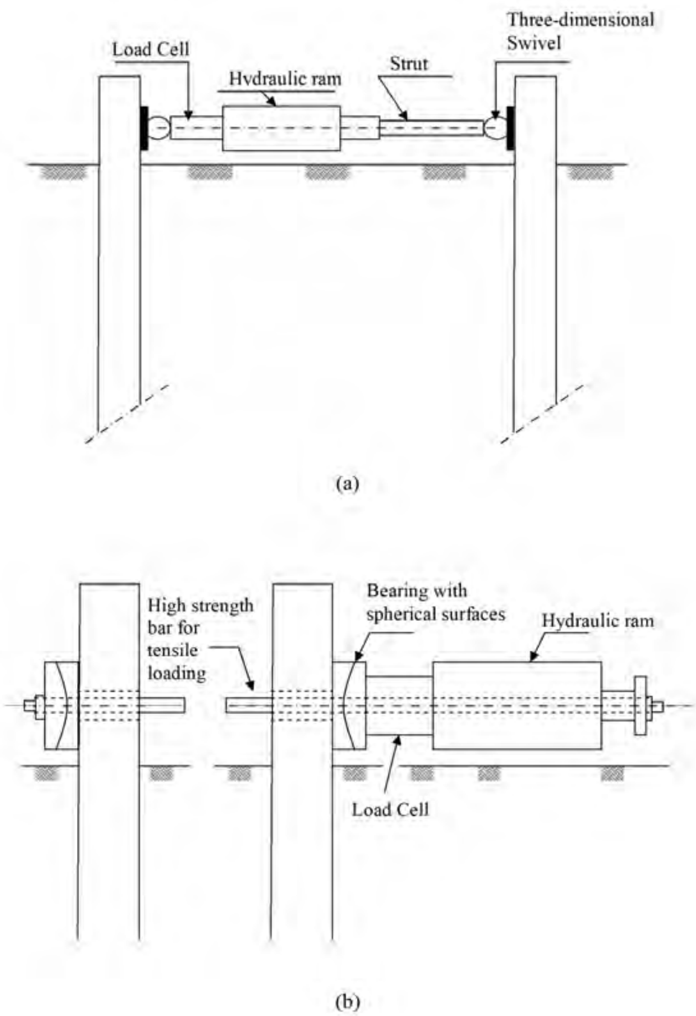


Figure 12-3: Arrangement for two-way lateral loading (a) compression; (b) tension.

Cyclic lateral load tests are sometimes performed to investigate the load-displacement response, when such issues are relevant to design, such as earthquake loads and wave impact loading. However, although reciprocating jacks allow load reversals, the frequency of load cycles cannot match the ground motions experienced during earthquakes.

When soil liquefaction is a concern, research tests have been performed in conjunction with blasting to liquefy the ground adjacent to the pile/shaft during the test. Rollins et al. (2001) reported the results of lateral load tests on deep foundations during blast induced liquefaction of granular soils on Treasure Island in San Francisco Bay, California, near the National Geotechnical Experimentation Site (NGES).

Static lateral load tests should be performed in accordance with ASTM D3966 Standard Test Method for Deep Foundations under Lateral Load; this test standard should be referenced for details on requirements of the test foundation preparation, apparatus for applying and measuring loads, apparatus for measuring movement, test procedures, equipment calibration, safety requirements, and items to be addressed in a lateral load test report. This test standard is recommended in the absence of owner specific requirements. The standard shows examples of various test set-up arrangements for both one-way and two-way loading, using a variety of reaction systems. The standard also provides examples of the lateral loading apparatus, including conventional hydraulic cylinder ram jacks, center-hole jacks, and single line and multi-line winch set-ups. Arrangements for a fixed head condition are also described.

ASTM D3966 has 8 loading schedules to choose from, denoted as Procedures A through H. Procedure A recommends a loading schedule that consists of 1) load increments to a maximum of 200 percent of the design lateral load, followed by 4 decrements of load. Hold periods vary from 10 minutes up to 60 minutes depending on the particular load increment. Assuming the test pile or shaft is not a production pile, then it is preferred that the loading increase until the element deflects enough that the geotechnical resistance at the ultimate Limit State is approached. The test standard has provisions for excess loading stages beyond the standard loading schedule, up to a maximum of 250 percent of the design load in what is termed Procedure B. However, the development of plastic hinges in the pile or drilled shaft should be avoided.

If cyclic loading is required, Procedure C should be followed. A further option provided in the standard is Procedure D, for surge loading that involves multiple loading cycles at any specified load increment. Surge loading can be implemented during the standard loading schedule or following completion of the standard schedule of loading. If reverse loading is required, then Procedure E is followed whereby loads are applied either by pushing, then pulling, or vice versa. Reciprocal loading is provided for under Procedure F. The lateral load increment is applied in one direction and then the other direction for a specified number of cycles. In Procedure G, the test is performed until a specified lateral displacement is measured. Last, Procedure H is for combined loading in which an axial load is applied to the test pile or shaft first, and then the lateral load is applied with the axial load held constant during the test.

12.2.2 Rapid Load Test

The Rapid Load Test (i.e., also known as the Force Pulse test) uses a controlled combustion of fuel in a confined pressure chamber to propel a piston against a pile or drilled shaft (Bermingham and Janes 1989). A heavy reaction mass is placed on the piston and this is forced to move as a result of the combustion. An equal and opposite reaction occurs against the pile or drilled shaft. The rapid load is applied momentarily (in around 120 milliseconds). The force on the foundation is measured with a built-in load cell and laser sensors measure the time-history of load and the displacement of the foundation. Venting of the combustion gases from the pressure chamber relieves the load on the foundation. Test loads in excess of 1000 tons can be applied with this test. The size of the load is controlled by the amount of fuel used and the size of the reaction mass deployed.

Depending on the orientation of the piston, a rapid load test can be performed vertically (axial compression), or laterally. In the latter case, a steel sled is used on which the reaction mass is placed. If performed over water, the sled slides on the deck of a barge. The benefits of this type of test over conventional static load tests are in the speed of the test, and there is no requirement for reaction piles. The reaction mass is considerably smaller than the equivalent load required for a static test. A Rapid Lateral Load test is a form of dynamic load, and in that sense, it is perhaps more applicable to investigation of the lateral load response of deep foundations to dynamic loading such as vessel impact, ice, wind and wave loads than static tests. For a similar load magnitude, a Rapid Lateral Load Test may be 25 to 50 percent lower in cost than an equivalent static load test.

Brown (2007) presented a method of analysis of the rapid lateral load test that considers inertia and rate of loading effects. He noted that it is preferable to conduct four successive and increasing load pulses as the best means to capture non-linear load deformation response. To reload, reassemble and reposition the equipment against the test pile or shaft takes about an hour between pulses. Double integration of the acceleration time history, measured via accelerometers, provides a deformation time history. A string of eight down-hole accelerometers was noted as adequate for defining the deformation time history at each instrument location. These instruments are positioned in an inclinometer casing, and then recovered after the test. Resistance type strain gauges at intervals along the pile are also used, from which to compute bending moments. Resistance type gauges, rather than vibrating wire strain gauges, are selected because of the frequency demands of the rapid test. A high-speed data acquisition system is required to capture the data at a sampling rate in the order of 1000 samples per second.

12.2.3 Bi-directional Lateral Load Test

Bi-directional testing has been applied to the lateral load testing of drilled shafts (O'Neill et al., 1997; Brown et al. 2010). The method uses conventional bi-directional load cells (e.g., hydraulically activated load cells) embedded within the shaft instead of a load test frame at the top of the drilled shaft. Instead of the load cells installed horizontally at or near the bottom of a reinforcement cage, as in the case with an axial compression load tests, they are turned 90 degrees, and used to jack the two halves of a drilled shaft apart. The load in the cells is divided by the length of the test interval to derive the lateral load applied to the ground. Lateral displacements are measured during the test with linear displacement variable transducers (LDVTs) that are embedded in the shaft concrete. This bi-directional test does not apply load at the top of the foundation, and therefore cannot be used to determine the bending and deflection of the shaft, but it does provide a means to directly evaluate the lateral resistance of a zone of soil or rock at depth. This test may therefore find application in design of drilled shafts in water where the structural strength and deformation of the free length of the shaft may limit the displacement that can be achieved below the mudline during a conventional lateral load test. The test may also be useful for determining p-y curves in rock sockets.

Two arrangements for a bi-directional test for drilled shafts are shown in Figure 12-4. The cell in the left photo was installed into a 6-ft diameter socket into sandstone at a depth of around 100-ft below the surface. The pair of cells in the photo at the right were installed into a socket which was 8-ft diameter and 15-ft long into a chalk formation approximately 60-ft below grade (Brown et al. 2010).

Figure 12-4: Bi-directional Lateral Testing Apparatus Using Embedded Load Cells (from Brown et al. 2010)

12.3 INSTRUMENTATION

Instrumentation is required for lateral load tests to collect data that can be used to verify performance as well as develop design parameters (p-y curves). Instrumentation can be considered external or internal to the foundation element.

12.3.1 External instrumentation

External instrumentation for lateral load tests includes the following:

- Linear displacement variable transducers (LDVTs)
- Long travel potentiometers
- Dial gauges
- Tilt-meters
- Calibrated load cells
- Calibrated jack gauge
- Theodolite

Such instruments are located at the head of the pile or drilled shaft to measure and record displacements (LVDTs, dial gauges, potentiometers, theodolite), rotations (tilt-meters) and load (calibrated load cell). The hydraulic pressure on the loading jack is also measured with a calibrated gauge as a useful backup device in the event that the load cell malfunctions.

The external instrumentation is essentially the same as that required for an axial compression test, but there are some important differences to consider. The loads applied in a lateral load test are usually much smaller than for an axial load test, but the displacements will likely be greater. It is therefore important to select a load cell with the appropriate accuracy and precision for the range of loads to be applied. The length of travel required on the loading jack ram will need to be sufficient for the displacement anticipated. The travel on dial gauges and LVDTs will also need to be selected with the anticipated displacement magnitude in mind. In planning the load test set-up, it is recommended to run a preliminary analysis for the site-specific ground conditions, foundation type and loads to gain a feel for the magnitude of displacement, such that an appropriate jack and instrumentation selection can be made. Further information on the selection and use of geotechnical instrumentation is available in Dunnicliff (1998) and Brown et al. (2010).

12.3.2 Internal instrumentation

Internal instrumentation for lateral load tests includes the following:

- Vibrating wire strain gauges
- Inclinometer casings
- In-place inclinometer arrays (down-hole accelerometers)
- Shape accelerator arrays (SAAs)

Strain gauges, typically of the vibrating wire “sister bar” type, are attached to the longitudinal reinforcement in drilled shafts in pairs, one gauge located on either side of the cage aligned parallel to the direction of applied load (Figure 12-5). One gauge in the pair will measure the compressive strain and the other gauge will measure the tensile strain. Data from strain gauges are plotted as micro strain versus load, and the data are used to compute bending curvature with depth, from which estimates of the bending moments in the drilled shaft can be computed, see Section 12.7. On steel piles (e.g., pipe piles, H-piles), the strain gauges are fixed to the face of the pile; for these applications, it is prudent to provide a protective shield to the gauges by welding on steel angles over the gauges to protect them from damage during pile installation.

To measure the bending curvature of the drilled shaft or pile accurately, from which p-y curves are subsequently derived, it is necessary to have multiple levels of strain gauge pairs. Typically, vertical spacing of strain gauge pairs is in the order of 5 to 10 feet. Closer spacing is preferred, although wider spacing may be acceptable in very long elements in relatively uniform strata, provided the intended minimum data coverage is met. The cable from each gauge is routed to the ground surface in an organized manner. Labeling the cables of each pair at the ground surface with colored tape to differentiate the different gauge depth and position is a convenient approach. On reinforcement cages, it is recommended that the cables be attached to the longitudinal steel using plastic cable ties (Figure 12-5). Some protection is provided if the cable is positioned in the corner joint between the longitudinal steel and the spiral reinforcement (e.g., Hayes and Simmonds 2002).

Conventional grooved inclinometer casings are a cost-effective addition for measuring lateral deformations along the foundation element in a lateral load test. The inclinometer probe is run down the casing prior to the load application, and baseline readings collected at intervals from the bottom upwards. It is preferable to ensure that the tip of the casing is sufficiently deep so that it will not deflect (i.e., below the deepest point of zero deflection of the foundation element). If this cannot be achieved, it is important to survey the horizontal position of the top of the casing as a baseline to which the subsequent data are then referenced. It is important that during installation of the inclinometer casing that one set of grooves be aligned in the direction of the applied load. It is also preferred to position the inclinometer casing close to the neutral axis, to minimize axial strain induced effects on the inclinometer casing. It is good practice to measure the casing cumulative deformations at the end of each load interval, although this can add significant time to each test. Lowering an inclinometer probe to the bottom of a typical drilled shaft and measuring deformations at conventional intervals in both groove sets of the casing may take 30 minutes to 1 hour depending on the length of the shaft and the number of “rounds” of readings taken. With 10 load increments in a standard loading schedule per Procedure A in ASTM D3966, it is apparent that the time required for a well instrumented and monitored lateral load test can increase significantly beyond the time required for application of the load increments alone.

If time is limited, then the inclinometer readings should be made at a minimum at the beginning of the test prior to the first load increment being applied, then again when the design load (service limit) has been applied (or when the service limit head deflection criterion has been reached), and finally when the maximum load is applied (or when the strength limit head deflection criterion has been reached). It is recommended that the readings be taken in both the A and B axes of the casing, and a spiral survey of the casing is also recommended. The cumulative deformation profile of the inclinometer casing is a useful check on the shape of the deflection profile that may be evaluated from the strain gauges. In the event that some strain gauges malfunction during the test, the inclinometer data serves as a useful and cost-effective back-up. Moreover, the data are useful for the identification of the depth of maximum moment. The deflection data from inclinometers may not be accurate enough to compute bending moments for derivation of p-y curves but it is certainly beneficial to utilize the data in the overall evaluation of the test, see for example Pincus et al. (1994) and Sinnreich and Ayithi (2014). The depth of maximum moment is important to know for reinforcement design.

Figure 12-5: A pair of sister bar strain gauges attached to a reinforcement cage.

An alternative to conventional inclinometer casings is to use in-place inclinometer arrays, or strings of in-place accelerometers. These instruments are attached to a continuously recording data logger system at ground level. These instruments enable deformations to be monitored during the load test, without having to stop the load test at intervals for manual, considerably reducing the time needed for testing.

An alternative to inclinometer casings and in-place inclinometers are Shape Array Accelerometers (SAA). This instrument can be embedded in the shaft concrete, or grouted with cross hole sonic logging tubes. Boeckmann et al. (2014) describe the use of SAAs for a lateral load test set-up for drilled shafts in shale. The SAA comprises a chain of sensors that measure tilt on a continuous basis during the test. This avoids the need to stop the testing at intervals to take manual readings of inclinometer casing. It also increases safety, by avoiding the need to make readings in close proximity to test elements under load.

Further information on the selection and use of geotechnical instrumentation is available in Dunnicliff (1998) and Brown et al. (2010).

12.4 DATA ANALYSIS

A key objective of instrumented lateral load tests is to derive p-y relationships for use in design (also see Appendix C). In order to achieve these relationships, numerical methods are required to convert the strain gauge data into bending moment profiles and to derive p-y design curves. The procedure used is summarized in the following steps:

Plot the profile of bending curvature (ϕ) with depth. The curvature is computed as the difference between the compression and tensile strains measured in each pair of strain gauges, divided by the horizontal distance between the two strain gauges in each pair.

1. The lateral deformation (y) with respect to depth is computed by double integration of the profile of bending curvature:

$$y = \int \left(\int \emptyset dz \right) dz \qquad \text{(Equation 12-1)}$$

Where:

z = depth below the top of pile or drilled shaft.

2. The bending moment (M) profile with depth is computed by multiplying the profile of bending curvature by the flexural stiffness of the pile or drilled shaft:

$$M = EI\emptyset \qquad \text{(Equation 12-2)}$$

Where:

E = Young's modulus of the reinforced concrete or steel element.

I = Second moment of area of the section.

3. The soil resistance per unit length of pile/shaft (p) is then obtained by double differentiation of the bending moment profile:

$$p = -\frac{d^2M}{dz^2} \qquad \text{(Equation 12-3)}$$

Double differentiation of the bending moment profile can amplify numerical errors, especially if the number of strain gauge pairs is limited or if inclinometer data is used for this purpose. It is therefore recommended that a numerical curve fitting procedure be applied to the raw instrumentation data, and then develop the p-y curves from a "smoothed" relationship. The numerical methods available generally involve polynomial curve fitting with varying degrees of numerical complexity. High order global polynomial curves (Reese and Welch 1975), piecewise polynomial curve fitting (Matlock and Ripperger 1956), cubic splines (Dou and Byrne 1996), weighted residuals (Wilson 1998; Yang et al. 2005), and B-splines (de Sousa Coutinho 2006) have been used to evaluate lateral load test instrumentation data. Yang and Liang (2006) provide a summary of the different mathematical approaches for evaluation of strain gauge data from lateral load tests. Of the various techniques described, they recommended the use of piecewise cubic polynomial curve fitting to achieve profiles of p with depth as it has been shown to accommodate the non-linear behavior of the foundation materials and p-y responses in layered soil profiles.

12.4.1 Deflections from Strain Gauge Data

A fifth order polynomial was used by Wilson (1998) to fit a curve to strain gauge data points from which a profile of bending curvature is then derived. The polynomial is in the form:

$$\emptyset = a + bz + cz^2 + dz^3 + ez^4 + fz^5 \qquad \text{(Equation 12-4)}$$

where *a, b, c, d, e*, and *f* are constants. This polynomial function is used to fit discrete curvature data points from each pair of strain gauges along the shaft depth, by using the least-squares method. The deflections (y) are found by double integration of the fitted curvature. Two boundary conditions are needed to obtain the two constants in the double integration. Yang (2005) and Dunnavant (1986) describe the following boundary conditions that can be applied:

1. y_0, y_{tip} = 0 for a long drilled shaft (i.e., length to diameter ratio, *L/D*, ≥ 10)
2. y_0, y_{fixity} = 0 for a short shaft (i.e., *L/D* < 10)

Where:

> y_0 = Measured deflection at the ground line.
>
> y_{tip} = Deflection at the drilled shaft tip.
>
> y_{fixity} = Deflection at a "fixity" point, defined where the deflection is approximately zero from inclinometer data.

When the boundary condition y_{fixity} = 0 is not available, then the condition θ_0 can be used instead for a short drilled shaft, where θ_0 is the measured shaft tilt at the ground line. This demonstrates why it is important to measure the deflection of the head of the pile or drilled shaft with dial gauges and/or LVDTs bearing against a reference beam. Using two gauge positions, one set vertically above the other at the head of the element provides a means to measure the tilt at the ground line. The importance of including an inclinometer or SAA in the test set-up is also apparent from study of these boundary conditions, because it may enable the depth of a "fixity" point to be defined.

12.4.2 Bending Moment Profiles

The flexural stiffness of reinforced concrete in drilled shafts exhibits non-linear strain degradation; it is not a constant. To reduce numerical errors in computing bending moment profiles from the profiles of bending curvature in a drilled shaft, it is therefore important to account for this non-linear flexural stiffness (Reese and Welch 1975). Put another way, when an elastic beam is loaded, the compressive and tensile strains measured by strain gauges positioned equal distances from the centerline will be equal in magnitude and opposite in sign, at least at small strain. However, for a concrete element, once the concrete cracks in bending, the sitffness is no longer constant, and the elastic beam idealization no longer applies. The neutral axis moves toward the compression side.

Field measured moment-curvature relationships for the concrete can be found from instrumentation at the ground line and at some distance above the ground line. Alternatively, non-linear flexural stiffness relationships for reinforced concrete are available (Wang and Reese 1993). In considering non-linear flexural stiffness of concrete, it is also important to account for other factors, including:

- Concrete compressive strength will depend on the mix, water cement ratio, age, cure time and conditions of curing;
- Concrete modulus will be different for the cracked and un-cracked sections;
- The as-constructed diameter of the drilled shaft at the strain gauge location may differ from the nominal design diameter;
- The as-constructed position of the reinforcement cage may be off-center;
- The actual tensile strength of the steel reinforcement is usually higher than the nominal grade; and,

- If temporary casing is left in place, it will influence the strength, stiffness, and composite action of the shaft concrete.

12.4.3 Net Resistance (p) Using Piecewise Polynomial Curve Fitting

Matlock and Ripperger (1956) and Dunnavant (1986) used piecewise cubic polynomial functions to fit discrete moment data. The approach is used to mitigate situations where the data is rather scattered, which is to be expected in natural layered soil profiles that exhibit non-linear behavior. This polynomial is then subjected to differentiation to obtain the net soil resistance *p*.

Figure 12-6 shows how the polynomial is fitted to five-point intervals along the shaft in a piecewise fashion. Least-square adjustments are used to establish the coefficients. Every five consecutive points along the shaft length where moment was calculated from the curvature found at strain gauge pair locations are fitted to one cubic polynomial curve. Double differentiation of the fitted polynomial curve with respect to the middle point of five provides the resistance *p* at that point. The resistance of the upper three points and the bottom three points is obtained from the smoothed local cubic polynomial moment curve using the top five points and bottom five points of the fitted equation, respectively. Also, a zero moment at the point of load application or a known moment value at the ground line should be included in the bending moment profiles that are used in this approach. The first polynomial M_1, is differentiated twice to evaluate *p* at the 3 moment levels closest to the surface (including the loading level). Other polynomials, such as M_2, are used to evaluate *p* at the group center point. The polynomial for the five points closest to the shaft tip is used to evaluate *p* at the three lowest points.

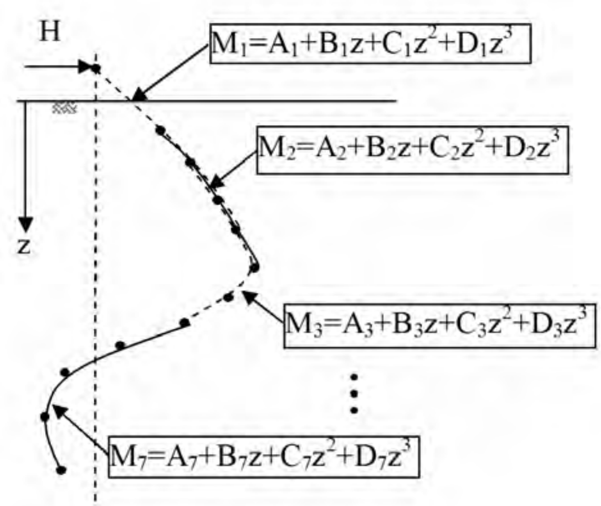


Figure 12-6: Piecewise polynomial curve fitting applied to moment profile (after Dunnavant 1986).

To use this technique, a minimum of five pairs of strain gauges along the shaft are required, and this should be considered when planning the instrumentation requirements. The more pairs of gauges used along the shaft, the greater the accuracy of the fitted curves, and the p-y relationships derived from them.

12.5 LATERAL LOAD TEST REPORTS

Lateral load testing programs should be documented in a report. Topics to be addressed in the documentation and reporting of lateral load tests include the following:

1. Background information on the project – bridge, wall, research program, etc.
2. Objectives of the test program – develop parameters for design, construction stage verification testing, research to investigate a particular aspect of lateral foundation analysis or develop design parameters, etc.
3. Background on the site
    a. Subsurface conditions – characterization of soil, rock, and groundwater conditions at the test site.
    b. Information as to why the site was selected, if available.
4. Details of the foundation construction – type, diameter or size, length, strength of materials, reinforcing, and other details as applicable regarding the construction of the foundation element(s) that are tested.
5. Test type and detailed description of test set-up.
    a. Include applicable ASTM references and owner specifications; note any deviations from standard specifications or approved modifications of testing set-up or procedures.
    b. Description, including details and photographs, of test set-up. Include relevant calculations as appropriate for determining adequacy of test apparatus.
    c. Test procedures.
6. Instrumentation and monitoring details
    a. Instrumentation layout including the types, number, and locations of instruments. Include drawing details and installation records.
    b. Data acquisition procedures.
    c. Instrumentation calibration data.
7. Test results and analyses
    a. Overall description and discussion of the performance of the test, whether the stated objectives were met or not, any issues or problems that occurred or were observed during the test.
    b. Numerical data and plots of data obtained from the test.
    c. Interpretation of test results, as applicable, including description of methods for data reduction and analyses.
8. Limitations of the tests or on the use of the test results and interpretations, as applicable.

The reporting of the test results and interpretations may be done in one comprehensive report by a single party, as may be the case for a research program, or may be documented in multiple reports by multiple parties. A construction verification test, for example, may be documented in terms of the test set-up and results by the testing contractor, but the interpretation of the results may be performed and documented by the design engineer. Often, for large projects with multiple test sites, a separate report may be prepared for each site to allow work to proceed at one site before all testing is completed.

In many cases, the test program will be prepared following specifications for a construction contract, either as part of a stand-alone contract for the testing or as part of a larger construction contract. Such a specification should require submittals of the proposed test set-up for approval prior to construction of the test foundation and test set-up. The approved submittals can serve as documentation, with any noted as-built changes, and can be included in the test report.

12.6 PUBLICATION OF LATERAL LOAD TESTS RESULTS

The analysis and design or laterally loaded deep foundations is an evolving area of practice, especially within the LRFD design framework. Recent advances in the state of the practice include development of additional p-y curves for local and regional materials as well as development of LRFD resistance factors.

When lateral load tests are performed for transportation projects, there could be a potential benefit to the local practice as well as the industry as a whole by making the results publicly available. Having high quality lateral load and deformation data, along with subsurface information, publicly available would enable other practitioners and researchers to benefit from the testing without having to incur the significant cost of implementing a test program. Even if designers for other projects must perform their own data reduction and analysis of the test data, the savings in time and cost of being able to use local or regional test results without incurring the time and cost of performing a test program could be significant for a smaller project.

Test reports could be published online on individual DOT websites or incorporated into larger industry databases. For example, a database of full scale lateral load tests, FindAPile.Com, is available online. The database is a collaborative effort between University of California, Irvine, the Deep Foundation Institute's (DFI) Seismic and Lateral Loads Committee, DFI's Drilled Shaft Committee, and the International Association of Foundation Drilling (ADSC). At the time of writing of this manual, the website contained records of over 30 lateral load tests on reinforced concrete and steel piles and drilled shafts performed over a period of 30 years from sites around the world. The amount of detail and available information for each project on the website varies; however, the industry recognizes the value and potential benefit of sharing such test data. There may be additional local or regional databases of lateral load tests as well.

To address concerns regarding the use of test data interpretation by designers for other projects, the load test reports can be broken into a data report (presenting factual data such as subsurface conditions and instrumentation data) and interpretive report (data analyses, assessment and development of p-y curves, etc.). In some cases, this may already be done if the testing is performed by one entity (a foundation testing specialist) and the interpretations and analyses performed by another entity (the designer of record). In this approach, the data report can be published for use by others, while the interpretive report remains a project document.

12.7 LIMITATIONS OF LATERAL LOAD TESTS

Lateral load tests are not without limitations, which need to be considered carefully when first considering the use of lateral load tests, during the planning of the test, and for applying the results in design. Limitations of lateral load tests include:

- Use of a "free head" condition in a lateral load test does not replicate the behavior of a "fixed head" condition that often represents the connection of piles/shafts to a pile cap;
- A load test on a single pile (or drilled shaft) does not replicate the behavior of piles/shafts in a pile group. p-multipliers need to be selected using judgment and code provisions (e.g., AASHTO Table 10.7.2.4-1). Lateral load tests on pile groups can address this limitation, but are expensive, and rarely, if ever, performed in practice;

- A lateral load test is unlikely to appropriately model a future scour condition. It is possible to model local scour with the use of an isolation casing, but the reduction in effective overburden stress due to general scour and contraction scour typically cannot be accounted for in the test set-up;
- Lateral load tests performed during construction typically come too late to benefit foundation design; lateral load tests during construction typically serve as “proof tests” to verify design parameters and evaluate the influence of construction means and methods;
- Lateral load tests performed during the design phase of a project require costly mobilization for installation of a few test piles/shafts; such costs may not be justified;
- Lateral load tests done during the design phase of a project may not be representative if different means and methods are used to install the subsequent production piles/shafts. Except for a design-build delivery method, the test program contractor will likely be different from the contractor engaged to install the permanent foundations;
- A few load tests may not capture the variability in the near surface soils at a project site, particularly when a thick layer of fill is present;
- Tests piles/shafts rarely, if ever, model axial loads in combination with lateral loads, yet this is the condition most likely to occur when the structure is in service and results in different pile/shaft stiffness and p-delta effects.
- Pile/shaft performance may be governed predominantly by the structural properties of the foundation element rather than the ground, particularly for cases with large unsupported pile/shaft length and column lengths (e.g., tall bridge bents; deep foundations installed through water); and
- Pile/shaft deformations during the test may not be sufficient to define the full shape of the p-y curves (e.g., piles/shafts with large unsupported length may fail structurally before mobilizing the full resistance of the soil).
- Lateral load test may not be cost effective or needed if pile lateral resistance is controlled at the strength limit state by the structural capacity and not the geotechnical capacity.

12.8 ALTERNATIVES TO LATERAL LOAD TESTS

Lateral load tests may not be appropriate for all projects. For example, typical simply supported bridges with short spans for highway projects may not require lateral load tests, not only because the lateral loads may not govern the design, but the costs involved may not be justified for a small number of foundation elements. Alternatives to lateral load tests include the use of more in-depth investigations including the use of in-situ testing or use of available p-y curves from other projects in the same or similar geology.

In-situ testing methods can be used to develop site-specific p-y curves. Several in-situ test methods can be used in a variety of materials as discussed in Chapter 3. An extensive in-situ testing program may still be significantly less expensive than a full scale lateral load test program, and may therefore be a more effective use of resources. Such in-situ testing can be performed during the design phase, and therefore the project can benefit from having this data at an early stage. Site-specific data can also be compared and correlated with similar types of data from other areas where actual lateral load tests have been performed.

Local experience may be available for typical bridges in similar ground conditions that have exhibited satisfactory performance during service. If local practice or precedent has a well-established basis for design, lateral load tests may not be required. In some areas, lateral load testing may have been performed at other projects or as part of research programs. Several research programs have been performed to investigate lateral load testing of drilled shafts in local conditions (refer to Chapter 4 for examples). The results of such research programs or other load tests in the project area or geologic formations can be used if considered consistent with the project conditions. In areas where there is limited local experience, reference to lateral load tests in similar subsurface conditions from case histories published in the engineering literature may provide insights that are helpful to a designer.

It is important to note that when using p-y curves or data from other tests, locations, or geologic conditions, there is an element of risk or uncertainty that is introduced in the design. Even using in-situ testing to develop p-y curves should be considered to have greater uncertainty or risk than a full scale lateral load test. Therefore, when using alternatives to lateral load tests, consideration should be given to using input soil/rock properties for design that are less than what would otherwise be used if lateral load tests are specified to verify foundation performance. Any adjustment to input parameters will rely on the judgement of the designer, considering the type materials at the site and the variability in their geotechnical properties, among other considerations. This approach should result in a more robust design that provides the required foundation performance in variable or uncertain ground conditions. Often, this approach will have little or no impact to the foundation arrangement or cost.

13 CONSTRUCTION CONSIDERATIONS

13.1 CONSTRUCTION MANAGEMENT AND INSPECTION

Successful construction of foundation elements to meet the design objectives for lateral load resistance depends on a) achieving the predetermined minimum embedment established by the designer, b) avoiding disturbance of the geo-materials around the foundation element that are relied upon for providing resistance to lateral loads, and c) constructing the foundation elements with the required structural strength and integrity. Knowledgeable supervision and inspection is essential, and equally important is the recognition of, and response to, unanticipated site conditions that could adversely impact the performance of the foundation to lateral loading. Inspection roles and responsibilities are discussed in detail in Hannigan et al. (2016) and Brown et al. (2010) for driven piles and drilled shafts, respectively. Construction management and inspection considerations with specific bearing upon laterally loaded foundations are discussed in the following sections of this chapter.

13.2 CONSTRUCTABILITY REVIEW

Prior to construction, the design should be reviewed to verify that the design is constructible with available means and methods. Elements of construction that may pose risks should be identified and mitigated if possible.

Considerations for a constructability review may include:

1. Are the foundation size and type appropriate?
    a. Are pile sizes and lengths appropriate for the type of pile (diameter or dimension, steel section, steel thickness, etc.)? Will piles require splicing and will that impact the schedule?
    b. Are drilled shaft diameters reasonable given the project size and expected contractor pool? Large diameter shafts may be inappropriate on relatively small local projects that will only attract local contractors.
2. Can foundations of this size and depth be installed given the site constraints, such as working area, slopes, headroom, access, equipment support, etc.?
3. Can the proposed depths be realized, i.e., can the bearing strata be penetrated to the required depth with the proposed deep foundation size and type? Are there risks of obstructions, rock, or other impediments to reaching the design length or otherwise installing the proposed foundation size and length?
    a. For driven piles, a driveability analysis should generally be performed.
    b. For drilled shafts, the need to change tools or use specialty tools to penetrate hard layers or remove obstructions should be considered.
4. Are there risks regarding installation such as settlement due to vibration, soil instability in excavations (drilled shafts, CFAs), contaminated spoils, etc.?
5. Are there risks to third parties, risks of damage to nearby structures, risks of public complaints or community impacts, etc.?
6. What will the construction sequence be and is it feasible (both technically feasible as well as economically feasible)?

7. If load tests are required, is there an area to perform such tests that is also representative of the subsurface conditions of the site as a whole (for applicability to the production piles/shafts).
8. If there are potential risks or foreseeable problems, are there ways to effectively and efficiently resolve issues related to those risks should they occur?
9. If there is more than one viable alternative, develop cost estimates and consider constructability for comparison. The lowest cost alternative is not necessarily the best, especially if there is a higher risk potential associated with this alternative; if the potential problem occurs during construction, the cost of the apparent lower cost alternative may increase significantly and may end up being well above the cost of the other, lower risk alternative.

Constructability considerations will vary by project size, location, local conditions, local practice, and other aspects of the project. The list above is intended to provide some examples, but it is by no means expected to be a comprehensive list for a complete constructability review. Such a review should be performed by staff with appropriate construction experience.

13.3 DESIGN CONSIDERATIONS AND CHANGES IN CONSTRUCTION

As construction occurs, considerations, impacts, or changes to design may include:

1. Proposed design changes based on data available before construction: value engineering or alternative design proposals by the contractor. This may require an entirely new design which would follow the design process above, or this may be a change in one aspect of the design (i.e., pile type, drilled shaft size, etc.). The requirements for lateral load design, such as design specifications, Strength and Service Limit State deflections, should be clearly communicated to the contractor to ensure that the proposed design will meet project criteria. Also, this should include required or suggested resistance factors (in some cases resistance factors may be required by the agency or design specification, in other cases it will be a matter of judgment of the designer).
2. Consideration of means and methods of construction: review and approval of means and methods of construction is typically part of the construction process, especially if aspects of the foundation performance may be impacted by the means and methods that are selected. This may be more of an issue for axial loading, but the potential impacts of the lateral loading capacity should be considered as well. For example, pre-excavation for removal of obstructions may reduce the lateral resistance of the soils within the depth of pre-excavation.
3. Design changes due to data received in construction: data regarding subsurface conditions is received in construction through additional explorations as well as actual construction operations including installation of deep foundations, excavation, and groundwater control.
    a. In some cases, additional field investigation and testing are performed in construction.
        i. In a design-bid-build approach, this may be a requirement for the construction contractor, such as performing borings at certain locations that were inaccessible prior to construction. For example, performing a boring at every drilled shaft location in order to verify the quality of rock within the socket depths. It is worth noting that deferring part of the subsurface investigation until the construction phase in a design-bid-build contract introduces a significant risk element into the construction contract. If conditions in the construction phase borings are found to be significantly different than assumed in the design, the design may need to be revised, which will likely adversely impact the construction schedule and costs.

Inclusion of means to pay for construction phase investigations and a means to adjust the payment of the foundations should be included in the contract terms for this scenario.

ii. Borings may also be performed by the contractor to investigate conditions for design of temporary works, a value engineering design, or for data regarding a possible differing site condition.

iii. For a design-build approach, additional investigations are usually performed to develop a design, to finalize a preliminary design shown in the bid documents, and to investigate opportunities to optimize the design, in addition to the purposes stated above. Typically, in design-build contracts, the final subsurface investigation program must be deferred until the locations of the bridge foundations are determined by the design-build contractor's design consultant.

b. Deep foundation design is often impacted by data received during construction of the foundations. Deep foundation design inherently includes some aspect of the observational method because the ground conditions at each particular foundation site may vary from the conditions or the behavior expected, and such variations are typically not apparent until construction inspection and/or verification testing is performed.

i. An example of this would be where the ground is weaker than anticipated. The lateral capacity may need to be reviewed, re-evaluated, and the design possibly adjusted. Construction testing, including lateral load tests, may be needed. These conditions may include conditions where weak strata are thicker than anticipated or geotechnical resistance is less than anticipated, as evidenced by investigations, testing, or by construction inspection records (pile driving, etc.).

ii. Another example could be where rock is higher or lower than anticipated. If rock is higher than the anticipated tip depth of deep foundations that were not designed to be socketed into rock (driven piles, CFA piles, or drilled shafts in soil), then a change in the design or construction means and methods may be necessary. A higher rock elevation would result in a shorter pile or shaft length which may not satisfy the design requirements for lateral loading. A change in pile size or a requirement to add rock sockets or pre-drilling into rock may be required. Where rock is lower than expected and the foundations were to be socketed into rock, the condition is similar to that described above where the weaker material overlying rock is thicker than anticipated and may impact the lateral capacity of the foundations.

c. Design changes may be made as a result of construction load testing. If lateral load testing is performed, either as a requirement of the contract or at the election of the contractor to optimize the design (more common for design-build but could also be part of a value engineer design or for claim resolution for design-bid-build), the results of the lateral load test will provide data that can be used for verification or updating the design of the foundations. Design updates may be to accommodate actual performance that is better than or worse than the anticipated performance.

13.4 DRIVEN PILE, DRILLED SHAFT, AND BACKFILL CONSIDERATIONS

Specific construction considerations related to driven piles, drilled shafts, and backfill and grading are discussed in the subsequent sections.

13.4.1 Pile/Shaft Position and Alignment

Laterally loaded vertical foundation elements, either within a group or spaced at linear intervals, as for a noise wall or top-down wall, are designed in consideration of overlapping passive influence zones using p-y multipliers determined from the spacing of the foundation elements. It is therefore important that the foundation elements be installed at their design location. To reduce the risk of piles/shafts being installed out of plan location or vertical alignment, care should be taken to clear the foundation locations of potential obstructions, including abandoned foundations, tree roots, and other obstructions. Buried pipes and other utilities that have not been accurately identified or relocated in advance should be field located relative to the planned pile/shaft positions to identify potential conflicts that may require redesign. All field adjustments to planned foundation positions should be approved by the engineer-of-record. Equipment accessories, such as pile leads, pre-bore augers, casing templates, or special tools, e.g., rock chisels, boulder breakers, etc., as needed to maintain piles and drilled shafts plumb during the foundation installation should be considered in advance of construction, consistent with requirements within the project specifications. Any foundation elements installed out of the specified tolerance for location or plumbness, should be promptly communicated to the engineer-of-record for resolution.

13.4.2 Driven Pile Installations

The utility of driven piles for lateral load resistance can be adversely affected by the selected means and methods of installation. Whereas a broad range of construction considerations for driven piles are discussed in detail in Hannigan et al. (2016), the following considerations are specific to laterally loaded pile foundations.

13.4.2.1 Equipment Selection

Wave Equation Analysis (WEA) drivability studies are very useful in assessing the suitability of impact hammers to install piles to at least the minimum tip elevation necessary to achieve the required lateral resistance defined in the foundation design. WEA drivability assessments should be conducted by a knowledgeable foundation specialist, matching the contractor’s proposed driving system with the specific pile type, ground conditions and performance requirements of the design. Additional details regarding the WEA analysis can be found in Hannigan et al. (2016).

13.4.2.2 Sequence of Driving

When driving piles on slopes, such as a river bank, the sequence of driving must be considered. Driving the piles sequentially from the bottom to top of the slope typically results in displacement, tilting and possibly bending of previously installed piles. To avoid this situation, pile installation on a slope should progress sequentially from top to bottom.

Similar to slope installations, ground displacement during driving favors driving the interior piles within a pile group first to reduce the risk that densification during the driving process will preclude driving following piles in the group to plan depth. Inability to install piles to plan depth may impact the functionality of laterally loaded piles in bending.

13.4.2.3 Driving Refusal

For various reasons, such as the presence of hard layers, the presence of boulders, an unanticipated higher bedrock elevation, or soil densification from previously installed piles, piles may achieve driving refusal above the plan minimum tip elevation specified for lateral resistance. Whenever such a condition is encountered, the engineer-of-record should be notified to assess the acceptance of the pile in question and to determine if measures need to be taken to achieve the minimum penetration for subsequent piles in the group. Such measures may include predrilling through a hard layer, spudding to break up obstructions, changing the driving sequence for the remaining piles, or modifying the number or arrangement of the piles in the group.

13.4.2.4 Splicing

Circumstances may be encountered where piles need to be driven deeper than the original pile order length to meet axial resistance requirements. Splices for steel piles should be full penetration groove welds to develop the full structural capacity of the pile section, and welding should be accomplished by a qualified and experienced certified welder. Mechanical splices with less than full strength in bending should be avoided unless the need for the splice can be anticipated, and thereby located deep within the ground where bending moments are relatively small. Field splices, if required, should be approved by the engineer-of-record.

Splices should generally be avoided for precast concrete piles since typical pile splices may not develop the full structural capacity of the pile and may be damaged by continued driving of the pile.

13.4.2.5 Jetting and Pre-boring

Hard driving or the necessity to control driving vibrations may prompt consideration of jetting or pre-boring from the surface to a limited depth. These techniques are sometimes precluded by the project specifications for reasons such as the unknown reduction in side friction resistance or, more specific to jetting, the potential settlement or undermining of adjacent piles or structures. Where allowed, pre-boring is usually limited by depth and by the hole diameter relative to the pile dimensions, e.g., Hannigan et al. (2016) recommend limiting the size of the pre-bored hole to not more than 6 inches than the largest dimension of the pile. Both jetting and pre-boring are known to reduce axial pile resistance, as noted in Hannigan et al. (2016), and may also reduce the available lateral resistance due to loosening of the soils around the pile. Jetting or pre-boring should be avoided, unless specifically allowed by the project documents or approved by the engineer-of-record.

13.4.3 Drilled Shafts

Numerous construction considerations related to drilled shaft installations are depicted in Brown et al. (2010). Construction considerations of particular importance to the performance of laterally loaded drilled shafts are discussed below.

13.4.3.1 Pre-Drilling and Surface Casing

Pre-drilling, either with or without removal of material, is sometimes used to facilitate the installation of temporary or permanent casing at the beginning of drilled shaft installation. If the auger diameter is larger than the diameter of the casing, pre-drilling may result in loosened material outside the completed shaft. This is not a concern if the depth of pre-drilling is limited to the shaft cut-off level or to the design scour depth. In other cases, the diameter of the auger used for predrilling should generally be not more than the diameter of the completed shaft. To further reduce the risk of soil disturbance outside the completed shaft, predrilling in such cases should be limited to just loosening the soil rather than removing it. If an oversized pre-drilled hole extends below the drilled shaft cut-off level, the contractor should be required to fill any voids around the casing with tamped granular backfill, or with grout, before continuing with shaft drilling operations.

The use of an oversized surface casing to stabilize and support the soil near the top of the shaft can result in loosened material in the annular zone between the completed shaft and the surface casing. To mitigate this condition, the contractor can fully remove all soil within the surface casing prior to extending the shaft below the bottom of the surface casing, and then allow the shaft concrete to flow into the annular void as the temporary casing is removed; with this approach, the contractor must prevent drilling spoil from falling into the annular void prior to concrete placement. If permanent casing is installed within the surface casing, the annular void should be backfilled with tamped granular backfill or with grout after the permanent casing is installed.

13.4.3.2 Structural Integrity

A major concern associated with construction of drilled shafts is the structural integrity of the concrete in the completed shaft. Poor concrete placement procedures or inappropriate concrete mixes can result in structural defects in the completed drilled shaft that can reduce its stiffness and lead to greater displacement during lateral loading. Such defects can also greatly reduce the structural integrity of the shaft for axial loading. Common defects may include necking of the shaft, honeycombing, soil intrusion, segregation of concrete aggregates, bleed-water channels, low strength concrete, concrete laitance, and cold joints. Non-destructive testing (NDT) methods, such as Crosshole Sonic Logging (CSL) per ASTM D6760 and Sonic Echo / Impulse Response (SE/IR) methods per ASTM D5882, are typically specified to verify the structural integrity of the shaft concrete. Other, less commonly used methods include Gamma-Gamma Logging (GGL) per California Department of Transportation Test No. 233, and Thermal Integrity Profiling (TIP) per ASTM D7949. These methods have advantages and disadvantages, as discussed in Brown et al. (2010); however, they offer a practical approach to identifying anomalies (i.e., potential defects) within the shaft that may warrant further investigation by concrete coring or, when anomalies are detected near the top of the shaft, by visual inspection of the exposed top and sides of the shaft. When such investigations confirm the presence of a defect in the shaft that will significantly impact the performance of the shaft, remediation measures, as discussed in Brown et al. (2010), can be performed to correct the defect, or one or more additional shafts can be installed to replace or supplement the defective shaft.

13.4.3.3 Rock Sockets

Drilled shafts and other drilled-in foundation types, such as micropiles, can develop high lateral resistance as a result of fixity within a rock socket. However, to achieve this high resistance the designer must verify that the assumed top of rock elevation and rock quality is consistent with design assumptions. Complicating this issue is the fact that, for some geologic settings and in some rock formations, the rock surface elevation and the quality of the rock can vary dramatically over short distances. When bedrock conditions are encountered that differ appreciably from the conditions that could reasonably be expected based upon the construction documents, the work could be delayed as the engineer-of-record evaluates the impact of these changed conditions on the performance of the foundations. To reduce the risk of delays and costly design changes during construction, an appropriate subsurface investigation should be performed during the design phase of the project, as discussed in Chapter 3, to define the rock conditions at the site. In addition, projects requiring highly loaded drilled shafts or non-redundant drilled shaft foundations should require a rock core boring at each shaft location to define rock depth and rock quality in advance of drilling operations to allow time for the designer to confirm or adjust the rock socket depth and length, and time for the contractor to fabricate the reinforcement cage needed to suit the site-specific shaft and socket lengths. These additional core borings are typically included in the construction contract and performed prior to initiating shaft installation.

13.4.4 Backfill and Grading

When foundations are installed though a new embankment, it is often specified that the embankment be constructed before installation of the foundation elements to facilitate placement and compaction of the fill, and to avoid potential damage or displacement of the foundation elements during fill placement. Where driven piles must be installed through compacted embankment fill, it is common to require pre-boring through the fill to facilitate driving and to maintain pile alignment.

When foundations must be installed prior to embankment construction, such as behind a Mechanically Stabilized Earth (MSE) wall, the embankment fill will need to be placed in smaller lifts and compacted using hand operated equipment to achieve the required fill compaction without damaging or displacing the foundation element.

Embankment fill placed around the foundation elements should be sloped to drain away from the foundation elements. To the degree possible, backfill should be placed in contact with the foundation to avoid loose soil zones around the pile or drilled shaft, and to prevent ponding or infiltration of water that can lead to soil softening, or erosion, and a resulting degradation or loss of lateral support.

At sites where soft, compressible soils underlie the proposed embankment at a foundation location, it is usually recommended to preload the site to eliminate, or substantially reduce, the amount of ground settlement prior to installation of the foundation elements. This sequence of construction may not reduce the design downdrag loads on the foundation (since small relative settlement can cause full downdrag load to develop), but will protect the foundation elements from lateral soil displacements, with related pile/shaft deformation and bending stresses, that may occur near the perimeter of the preload embankment during the preload period.

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APPENDIX A: EXAMPLE P-Y CURVES AND PARAMETERS FOR VARIOUS SUBSURFACE CONDITIONS BASED ON AVAILABLE PUBLISHED SOURCES

This appendix provides example p-y curves and parameters for various subsurface conditions, and is not intended to present a comprehensive list of all p-y curves available in literature. Instead, this appendix presents the most commonly used p-y curves in geotechnical practice, and discusses the equations that are used to construct such curves. The p-y curves presented in this appendix are also included in the computer programs LPILE, GROUP and FB-Multipier.

With this appendix, the user should be able to understand the main parameters affecting the p-y curves construction, and how the p-y curves vary for different site conditions.

The following p-y curves are presented:

- P-y Curve for Soft Clay with Free Water (Matlock 1970)
- P-y Curve for Stiff Clay with Free Water (Reese et al. 1975)
- P-y Curve for Stiff Clay with No Free Water (Reese and Welch 1975)
- P-y Curve for Sand (Reese et al. 1974)
- P-y Curve for Weak Rock (Reese 1997)
- P-y Curve for Liquefied Sands (Rollins et al. 2005)
- Sloping Ground

A.1 P-Y CURVE FOR SOFT CLAY WITH FREE WATER (MATLOCK 1970)

Matlock (1970) proposed a p-y criterion for soft clay under water based on results of pilot tests. Soft clays are those with undrained shear strength (S_u) ranging from 250 to 500 psf.

Figure A-1(a) shows the main characteristics of the static p-y curve for soft clay. In this relationship, the soil resistance (p) is normalized by the ultimate soil resistance (p_u) and is expressed as a function of the lateral deflection (y), which is also normalized by the deflection that occurs at 50 percent of the ultimate resistance (y_{50}). Under static loading conditions, the resistance of soft clay increases monotonically as a function of deflection until the ultimate resistance is reached.

To reflect the strength degradation, a correction is applied to the p-y curve for short term, monotonic loading of Figure A-1(a). Under cyclic loading conditions, the resistance achieves a maximum value and then decreases as the normalized deflection increases. The curve for cyclic conditions also depends on the depth that is considered: for soils at a depth (z) greater than a critical depth (x_r), it is considered that the clay flows around the pile and that $p = 0.72\ p_u$ for $y/y_{50} > 3$; for $x < x_r$, p decreases from $0.72\ p_u$ at $y/y_{50} = 3$, to a smaller value, $0.72\ p_u\ x/x_r$, at $y/y_{50} = 15$, as indicated in Figure A-1(b) and presented below.

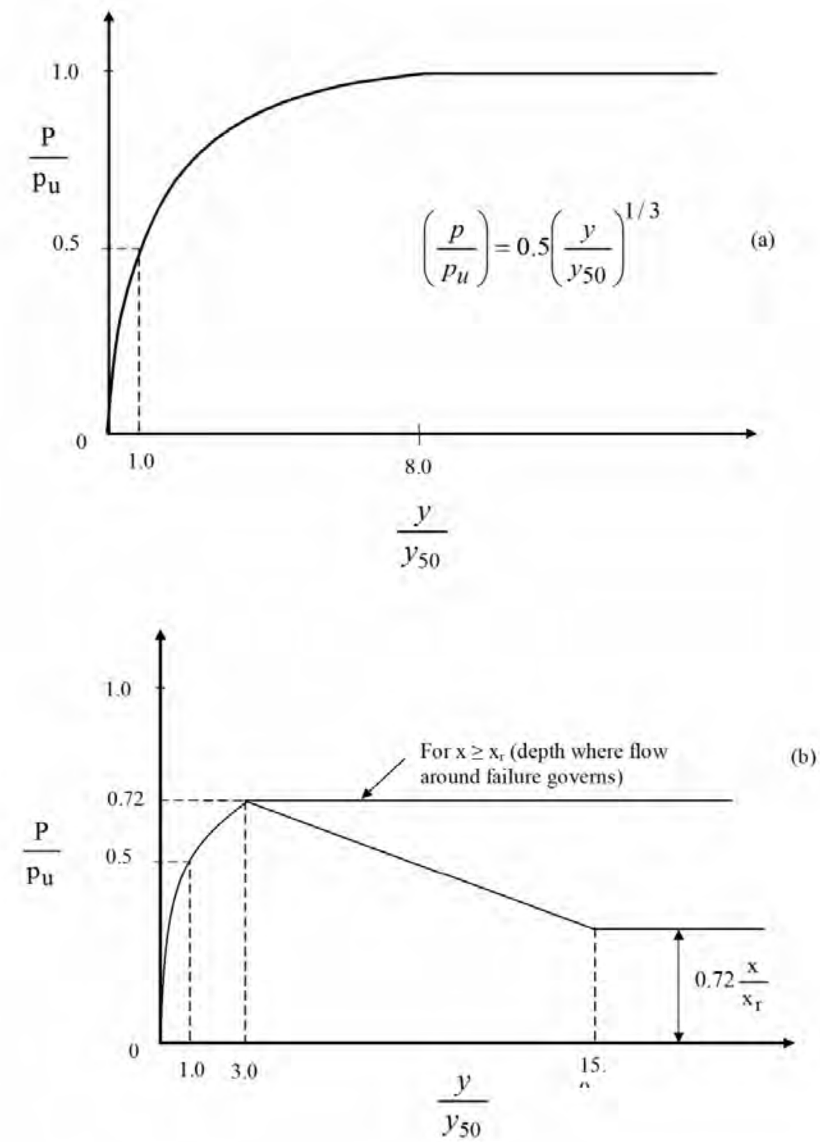


Figure A-1: P-y curves for soft clay with free water: (a) static loading and (b) cyclic loading (after Matlock 1970).

The required parameters to construct this p-y curve include: profiles of the undrained shear strength (labeled in various publications presenting p-y curves as c_u or S_u), unit weight (γ), and the axial strain (ε_{50}), which occurs at 50 percent of the maximum principal stress difference as measured in a triaxial testing.

The following steps can be used to construct the p-y curve for soft clay with free water (above water surface).

Static Loading

1. Obtain profiles with depth of undrained shear strength (C_u), effective unit weight (γ'), and strain at 50 percent of maximum deviatoric stress as measured in a triaxial test (ε_{50}). Use recommendations made in Chapter 4.

2. Select pile/shaft diameter *D*.
3. Compute the ultimate resistance (p_u) of soil per unit length of pile. Use the smaller of the following values calculated as:

$$p_u = \left(3 + \frac{\gamma'}{C_u}z + \frac{J}{D}z\right)C_U D \qquad \text{(Equation A-1)}$$

$$p_u = 9C_U D \qquad \text{(Equation A-2)}$$

where γ' = average effective unit weight between the ground surface to the depth *z* under consideration, C_u = undrained shear strength at depth *z*, *D* = diameter or width of pile/shaft, and *J* is an experimental parameter that depends on the clay consistency. Matlock (1970) recommended to use *J* = 0.5 for soft clay and *J* = 0.25 for medium clay.

4. Compute the deflection, y_{50}, that occurs at 50 percent of the ultimate soil resistance as follows:

$$y_{50} = 2.5\varepsilon_{50}D \qquad \text{(Equation A-3)}$$

5. Construct the p-y curve using the following relationship:

$$\frac{p}{p_u} = 0.5\left(\frac{y}{y_{50}}\right)^{1/3} \qquad \text{(Equation A-4)}$$

This curve is delimited to $p = p_u$ for $y \geq 8y_{50}$.

<u>Cyclic Loading</u>

For cyclic loading, follow the steps indicated below to construct the p-y curve shown in Figure A-1(b):

1. For $p \leq 0.72\ p_u$ (i.e., for $y / y_{50} = 3$) use Equation A-1 to construct the first part of the p-y curve for cyclic loading.
2. Find the critical depth z_r. According to Matlock (1970), the critical depth z_r represents what is in reality a rather indefinite point of transition from a condition of incomplete vertical restraint to one where plastic flow is confined to horizontal planes.

   If γ' and C_u are homogeneous in the upper zone, z_r can be calculated as:

$$z_r = \frac{6C_U D}{(\gamma' D + JC_U)} \qquad \text{(Equation A-5)}$$

   If the γ' and C_u profiles are non-homogeneous, calculate z_r by solving simultaneously Equations A-1 and A-2 at depths where the p-y curve is applied and using the corresponding soil properties at these depths.

3. If a depth *z* at which the p-y curves is applied results $z \geq z_r$, then use $p = 0.72\ p_u$ for deformations $y \geq 3y_{50}$. In this case, the p-y curves for cyclic and static conditions coincide.
4. If the depth *z* at which the p-y curves is applied results $z < z_r$, then calculate *p* as a line decreasing from $p = 0.72\ p_u$ at $y = 3\ y_{50}$ up to a value defined by:

$$p = 0.72 p_u \frac{z}{z_r} \qquad \text{(Equation A-6)}$$

at y = 15 y_{50}. For $y \geq 15y_{50}$, p remains constant as given by Equation A-6.

A.2 P-Y CURVE FOR STIFF CLAY WITH FREE WATER (REESE ET AL. 1975)

Reese et al. (1975) developed a criterion to be used for stiff clay with water based on field tests. Stiff clays are those with undrained shear strength (S_u) ranging from 1,000 to 2,000 psf).

Figures A-2 through A-5 show the main characteristics the original and modified p-y curves under static and cyclic loading conditions.

It must be recognized that the presence of free water does not necessarily translate in conditions below the groundwater table. In fact, the presence of free water refers to the submerged conditions of the pile that was tested during the experimental studies to develop the p-y curves. The tests used to develop the criterion for stiff clay in the presence of free water were performed using cyclic loading at a site of stiff fissured clay in a submerged condition. During the application of the cyclic loading, an annular gap developed between the soil and the pile after deflections at the ground surface of about 0.4 inch. The soil response was observed to rapidly degrade with multiple cycles of load due to this localized scour adjacent to the pile, and the criterion developed for static loading also exhibits significant strain-softening behavior. This criterion will result in a substantial reduction in mobilized soil resistance compared to that of Welch and Reese (1972), which does not include such strain softening. This reduction is only appropriate for situations where stiff clay is exposed to free water at or near the ground surface, where degradation similar to that observed in the load test experiment can occur. In conditions where the groundwater surface is at depth and free water is not present at or near the ground surface, the Welch and Reese criterion is more appropriate, even below groundwater. Similarly, stiff clay strata at depth below a sand stratum would normally not be subject to degradation due to free water (unless scour removed the overlying sand).

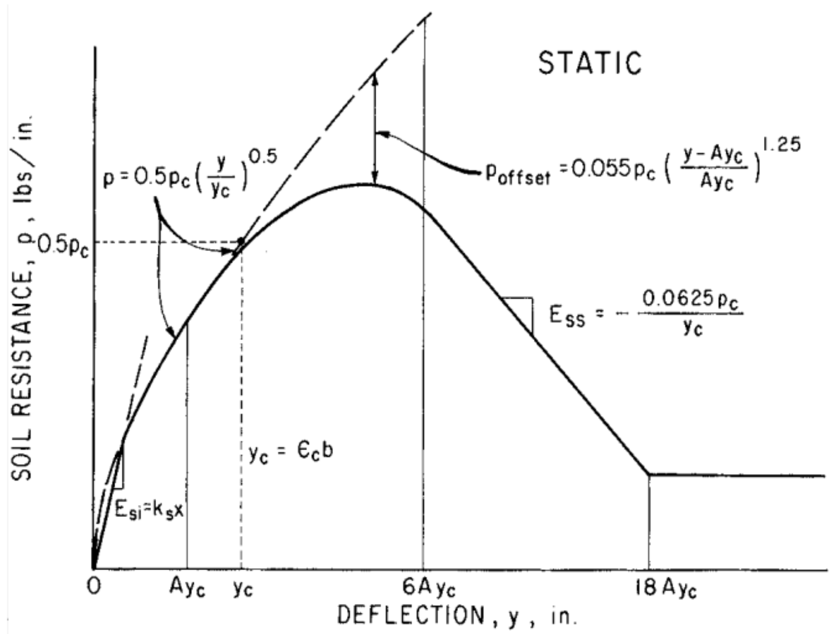


Figure A-2: P-y curve in stiff clay with free water - static loading (after Reese et al. 1975).

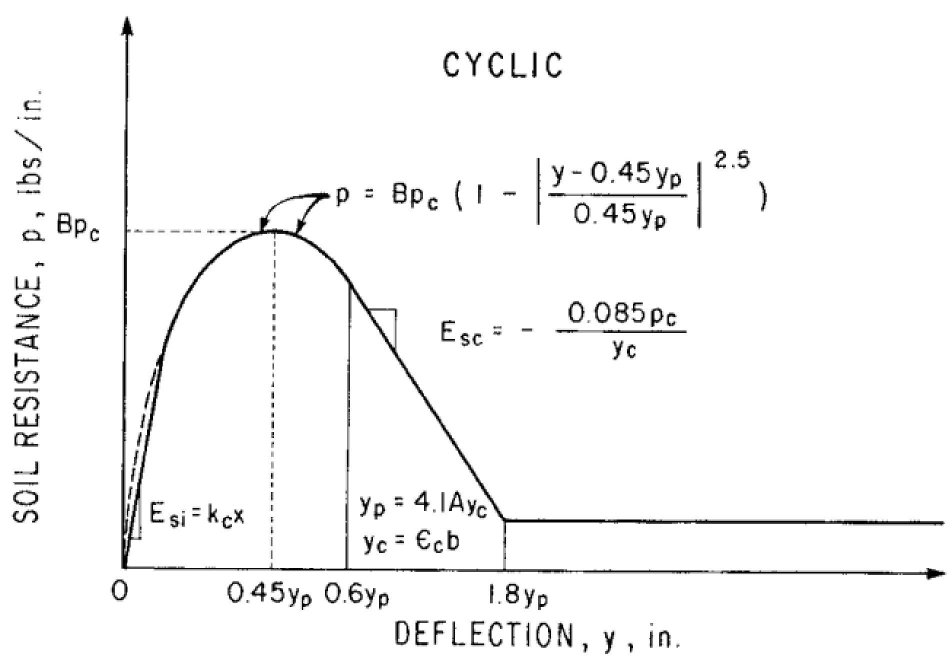


Figure A-3: P-y curve in stiff clay with free water - cyclic loads (after Reese et al. 1975).

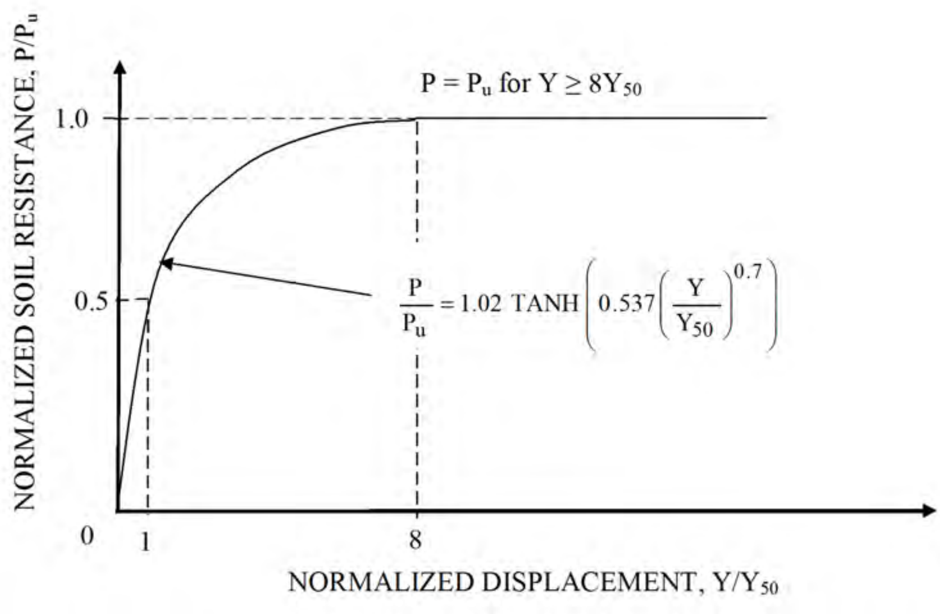


Figure A-4: Normalized p-y curve for stiff clay with free water – static loads (after Dunnavant and O'Neill 1989).

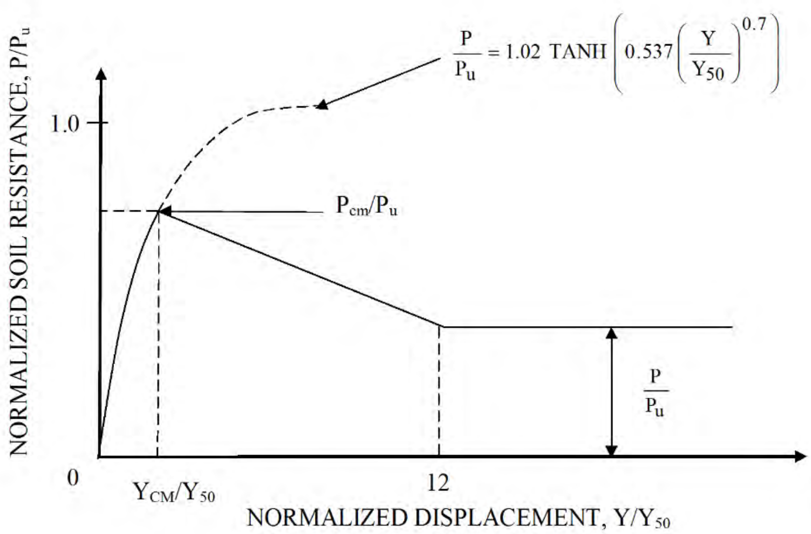


Figure A-5: Normalized p-y curve for stiff clay with free water – cyclic loads (after Dunnavant and O'Neill 1989).

The following steps can be followed to construct the p-y curve in stiff clays with free water at depth z.

<u>Static Loading</u>

For static loading, follow the steps indicated below to construct the p-y curve as shown in Figure A-2:

1. Obtain the profiles of undrained shear strength (C_u) and effective unit weight (γ').
2. Select pile/shaft diameter *D*.

3. Compute the average undrained shear strength (C_a) over the depth z.
4. Compute the ultimate resistance (p_u) of soil per unit length of pile. Depending on the type of failure mechanisms that is formed (near ground surface or well below ground surface), two values can be calculated for the ultimate resistance, p_{ct} and p_{cd}, respectively. Use the smaller of these values, which are computed as:

$$p_{ct} = 2C_a D + \gamma' Dz + 2.83 C_a z \qquad \text{(Equation A-7)}$$

$$p_{cd} = 11 C_u D \qquad \text{(Equation A-8)}$$

5. Construct the initial linear portion of the p-y curve as follows:

$$p = (kz)y \qquad \text{(Equation A-9)}$$

where k is a proportionality coefficient with units of F/L^3. This coefficient can be assigned to be k_s (static loading) or k_c (cyclic loading). Values for k_s or k_c can be selected from the recommended values presented in Table A-1.

Table A-1: Representative values of k for stiff clays (Reese et al. 1975).

Loading Condition	Coefficient k_s or k_c (pci) Average Undrained Shear Strength, C_a (ton/ft^2) 0.5 - 1	Coefficient k_s or k_c (pci) Average Undrained Shear Strength, C_a (ton/ft^2) 1 - 2	Coefficient k_s or k_c (pci) Average Undrained Shear Strength, C_a (ton/ft^2) 2 - 4
Static	500	1,000	2,000
Cyclic	200	400	800

6. Calculate y_{50} as:

$$y_{50} = \varepsilon_{50} D \qquad \text{(Equation A-10)}$$

ε_{50} can be obtained from the results of lab tests or from Table A-2.

Table A-2: Representative values of ε50 for stiff clays (Reese et al. 1975).

ε_{50} (-) Average Undrained Shear Strength, C_a (ton/ft^2) 0.5 - 1	ε_{50} (-) Average Undrained Shear Strength, C_a (ton/ft^2) 1 - 2	ε_{50} (-) Average Undrained Shear Strength, C_a (ton/ft^2) 2 - 4
0.007	0.005	0.004

7. Construct the first portion of the non-linear part of the p-y curve using the following 2nd- degree equation:

$$p = 0.5 p_c \left(\frac{y}{y_{50}}\right)^{0.5} \qquad \text{(Equation A-11)}$$

Where p_c is the smaller value calculated from Equations A-7 and A-8.

8. Obtain factor A_s to be used in to construct the non-linear part of the p-y curve. Obtain A_s from Figure A-6 for the selected normalized depth *z/D* (note that in Figure A-6, the variable *x* coincides with depth *z*).
9. If the curves defined by Equations A-11 and A-9 intersect in the deformation range $0 \leq y \leq As\, y_{50}$, the straight line defined by Equation A-9 is maintained. If these curves do not intersect, Equation A-11 controls the p-y curve and the 2nd- degree equation is extended to y = 0, while the linear portion is discarded.
10. Establish the second portion of the non-linear part of the p-y curve as follows:

$$p = p_c \left[0.5 \left(\frac{y}{y_{50}} \right)^{0.5} - 0.055 \left(\frac{y - A_s y_{50}}{A_s y_{50}} \right)^{1.25} \right] \qquad \text{(Equation A-12)}$$

The equation above defines the portion of the p-y curve in the range $A_s y_{50} \leq y \leq 6 A_s y_{50}$.

11. Establish the next straight line portion of the p-y curve as:

$$p = 0.5 p_c \sqrt{6A_s} - 0.411 p_c - \frac{0.0625}{y_{50}} p_c (y - 6A_s y_{50}) \qquad \text{(Equation A-13)}$$

The equation above defines the portion of the p-y curve in the range $6A_s y_{50} \leq y \leq 18 A_s y_{50}$.

12. Establish the final straight line portion of the p-y curve.

$$p = 0.5 p_c \sqrt{6A_s} - 0.411 p_c - 0.75 p_c A_s \qquad \text{(Equation A-14)}$$

The equation above defines the portion of the p-y curve in the range $18 A_s y_{50} \leq y$.

Cyclic Loading

For cyclic loading, follow the steps indicated below to construct the p-y curve.

1. Follow Steps 1 - 6 for the p-y curve for static loading.
2. Obtain factor A_c to be used in to construct the non-linear part of the p-y curve. Obtain A_c from Figure A-6 for the selected normalized depth *z/D* (note that in Figure A-6, the variable *x* coincides with depth *z*).
3. Calculate:

$$y_p = 4.1 A_s y_{50} \qquad \text{(Equation A-15)}$$

4. Construct the parabolic portion of the p-y curve as follows:

$$p = A_c p_c \left[1 - \left| \frac{y - 0.45 y_p}{0.45 y_p} \right|^{2.5} \right] \qquad \text{(Equation A-16)}$$

The equation above defines the portion of the p-y curve between the point of intersection of the initial straight line and the curve defined by Equation A-16 and y 0.6 y_p. If there is no intersection. Equation A-16 controls.

5. Establish the next straight line portion of the p-y curve as follows:

$$p = 0.936A_c p_c - \frac{0.085}{y_{50}} p_c (y - 0.6y_p)$$ (Equation A-17)

The equation above defines the portion of the p-y curve in the range $0.6y_p \leq y \leq 1.8y_p$.

6. Establish the final straight line portion of the p-y curve as follows:

$$p = 0.936A_c p_c - \frac{0.102}{y_{50}} p_c y_p$$ (Equation A-18)

The equation above defines the portion of the p-y curve in the range $1.8y_p < y$.

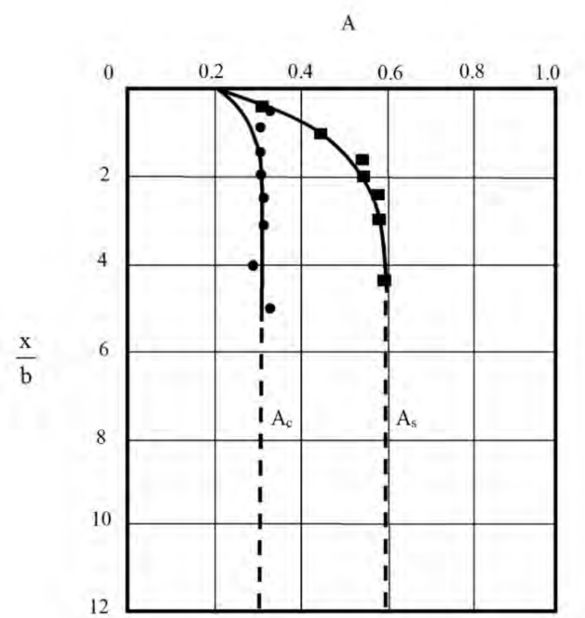


Figure A-6: Values of As and Ac (after Reese et al. 1975).

A.3 P-Y CURVE FOR STIFF CLAY WITH NO FREE WATER (REESE AND WELCH 1975)

Welch and Reese (1972) and Reese and Welch (1975) presented a p-y criterion for stiff clay with no water based on results of field tests. Figure A-7 and Figure A-8 show the main features of the p-y curves for static loads and cyclic loads, respectively. For a given *p* under static loads, the deflection under cyclic loads (y_c) is modified to account for the load cycles (*N*) and load magnitude (*p*). The ultimate pressure, p_u, depends on the undrained shear strength (S_u).

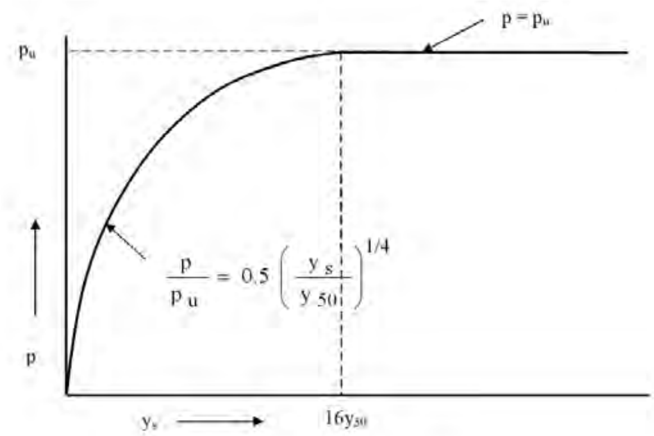


Figure A-7: P-y curve in stiff clay with no free water – static loading (after Welch and Reese 1972).

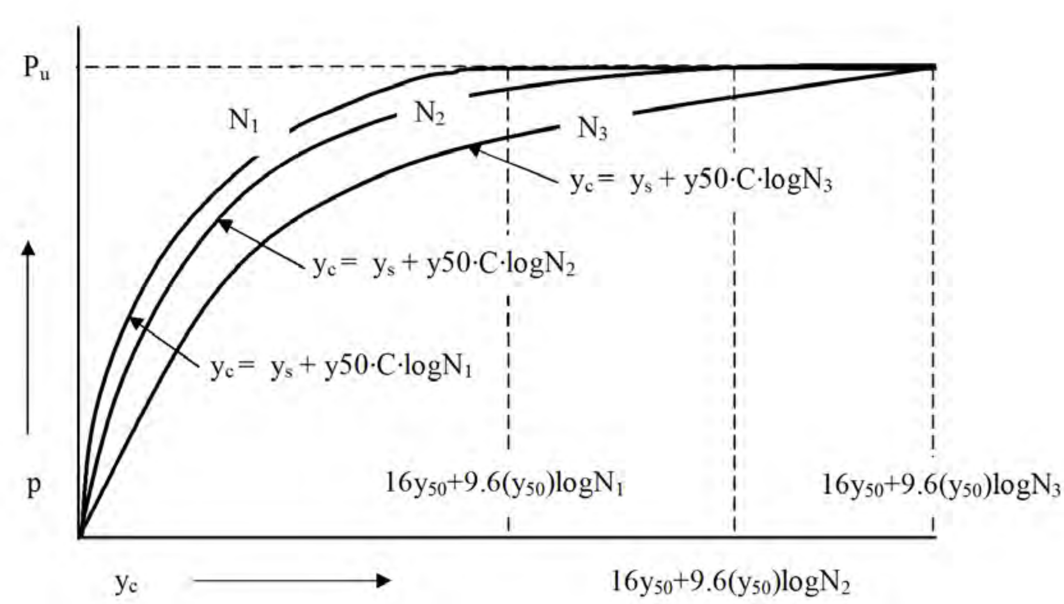


Figure A-8: P-y curve in stiff clay with no free water – cyclic loading (after Welch and Reese 1972).

<u>Static Loading</u>

For static loading, the following procedure can be used to construct the p-y curve for stiff clay with no free water (below water surface) as shown in Figure A-7:

1. Obtain input parameters, including the undrained shear strength C_u, effective unit weight γ' with depth, and pile diameter *D*. Obtain the value of ε_{50} from stress-stain curves or from Table A-2.
2. Calculate the ultimate soil resistance per unit length of pile (p_u) as the smaller value calculated from Equations A-1 and A-2. When using Equation A-2, the average shear strength from the surface to the depth under consideration should be used. Take *J* = 0.5.

3. Calculate y_{50} using Equation A-3.
4. Construct the p-y curve using the following equation.

$$\frac{p}{p_u} = 0.5\left(\frac{y}{y_{50}}\right)^{1/4} \qquad \text{(Equation A-19)}$$

5. For $y \geq 16y_{50}$, p is equal to p_u for all values of y.

Cyclic Loading

For cyclic loading, the following procedure can be used to construct the p-y curve for stiff clay with no free water as shown in Figure A-8:

1. Construct the p-y curve for short-term static loading using the previously described procedure.
2. Determine the number of cycles of load application, *N*.
3. For several values of p/p_u, obtain the parameter *C*, which characterizes the effect of repeated loading on deformation. Welch and Reese (1972) developed the following relationship from laboratory tests that should be used, in the absence of additional information, to estimate the parameter *C* as:

$$C = 9.6\left(\frac{p}{p_u}\right)^4 \qquad \text{(Equation A-20)}$$

4. For the same *p* values used to calculate p/p_u in Step 3, compute the deflection y_c for cyclic loading conditions with the following expression:

$$y_c = y_s + y_{50} C \log N \qquad \text{(Equation A-21)}$$

   where y_c is the deflection after *N* load cycles; y_s is the deflection upon initial loading; and *C* and *N* as defined previously.

5. Obtain the p-y curve the soil response after *N* cycles of load.

A.4 P-Y CURVE FOR SANDS (REESE ET AL. 1974)

Reese et al. (1974) developed a p-y criterion for sand based on results of field tests, as described by Cox et al. (1984). These p-y curves are based on the power functions and are widely used in various computer programs, including FBMultipier and LPILE. Figure A-9 shows the characteristic shape of the p-y curves for short-term static loading. The p-y curve shows four segments: (i) an initial, elastic segment (up to point “k”); (ii) a non-linear transition (from point “k” to point “m”); (iii) a linear part (from point “m” to point “u”); and (iv) an ultimate, constant part.

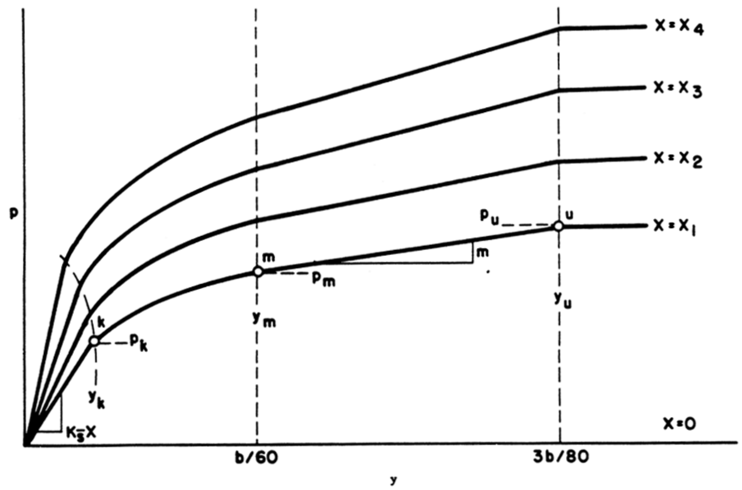


Figure A-9: P-y curve for static and cyclic loading in sand (after Reese et al. 1974).

The p-y curve for sand shown in Figure A-9 can be obtained using the following procedure:

1. Establish the soil friction angle ϕ, soil unit weight γ, and pile diameter *D*.
2. Calculate the following parameters for subsequent calculation as follows:

$$\alpha = \frac{\emptyset}{2}; \beta = 45 + \frac{\emptyset}{2}; K_0 = 0.4; and\ K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right)$$ (Equation A-22)

3. Calculate the ultimate resistance per unit length of pile/shaft using the smaller of the values given by the following equations:

$$p_{st} = \gamma z\left[\frac{K_0 z \tan\emptyset \sin\beta}{\tan(\beta - \emptyset)\cos\alpha} + \frac{\tan\beta}{\tan(\beta - \emptyset)}(D + z\tan\beta\tan\alpha) + K_0\tan\beta(\tan\emptyset\sin\beta - \tan\alpha) - K_a D\right]$$ (Equation A-23)

$$\boldsymbol{p_{sd} = K_a D\gamma z(\tan^8\beta - 1) + K_0 D\gamma z\tan\emptyset\tan^4\beta}$$ (Equation A-24)

For sand below the water table, the submerged unit weight (γ') should be used.

4. In Step 3, find the depth z_t where there is an intersection of Equations A-23 and A-24.
5. Select a depth at which a p-y curve is desired.
6. Establish y_u as 3*D*/80. Compute p_u as follows:

$$p_u = \overline{A_s} p_s \ or\ \ p_u = \overline{A_c} p_s$$ (Equation A-25)

Use appropriate values for $\overline{A_s}$ or $\overline{A_c}$ from Figure A-10, for the particular non-dimensional depth, and the static or cyclic case.

7. Establish y_m as *D*/60. Calculate p_m as follows:

$$p_m = B_s p_s\ or\ p_m = B_c p_s$$ (Equation A-26)

Use appropriate values for B_s or B_c from Figure A-11, for the particular non-dimensional depth, and for the static or cyclic case.

8. Establish the initial straight line portion of the p-y curve.

$$p = (kz)y \qquad \text{(Equation A-27)}$$

Use the appropriate value of *k* from Table A-3.

Table A-3: Representative values of k for sand under static and cyclic loading.

Condition	Relative Density Loose *k* (pci)	Relative Density Medium *k* (pci)	Relative Density Dense *k* (pci)
Submerged	20	60	125
Above Water	25	90	225

9. Establish the parabolic section of the p-y curve as follows:

$$p = \bar{C}y^{1/n} \qquad \text{(Equation A-28)}$$

Where:

$$\bar{C} = \frac{p_m}{y_m^{1/n}} \qquad \text{(Equation A-29)}$$

$$n = \frac{p_m}{m y_m} \qquad \text{(Equation A-30)}$$

$$m = \frac{p_u - p_m}{y_u - y_m} \qquad \text{(Equation A-31)}$$

Point "*k*" in Figure A-9 signals the end of the linear segment of the p-y curve. Its y-coordinate is determined as follows:

$$y_k = \left(\frac{\bar{C}}{kz}\right)^{\frac{n}{n-1}} \qquad \text{(Equation A-32)}$$

10. For y>= 3D/80, p is equal to the ultimate resistance per unit length of pile/shaft calculated in Step 3.

If Equations A-27 and A-28 have no intersection, Equation A-27 defines the p-y curve until there is an intersection with the p-y curve branch defined in Step 10.

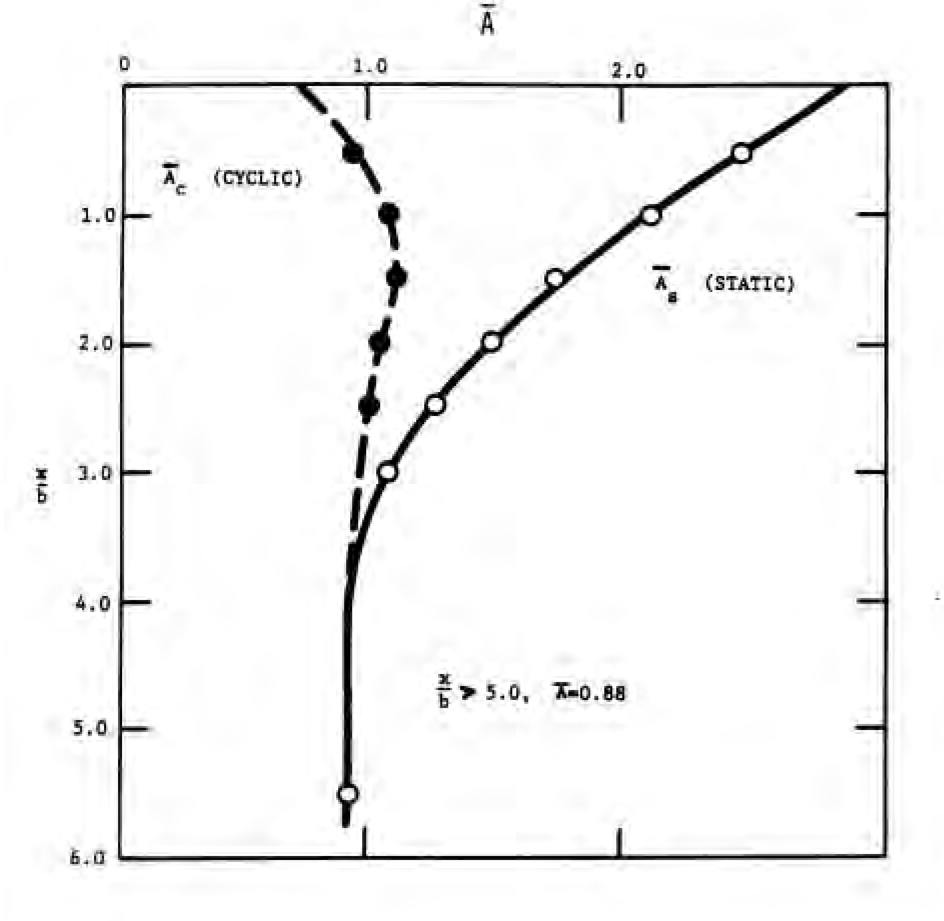


Figure A-10: Values of coefficients $\overline{A_s}$ and $\overline{A_c}$ (after Reese et al. 1974).

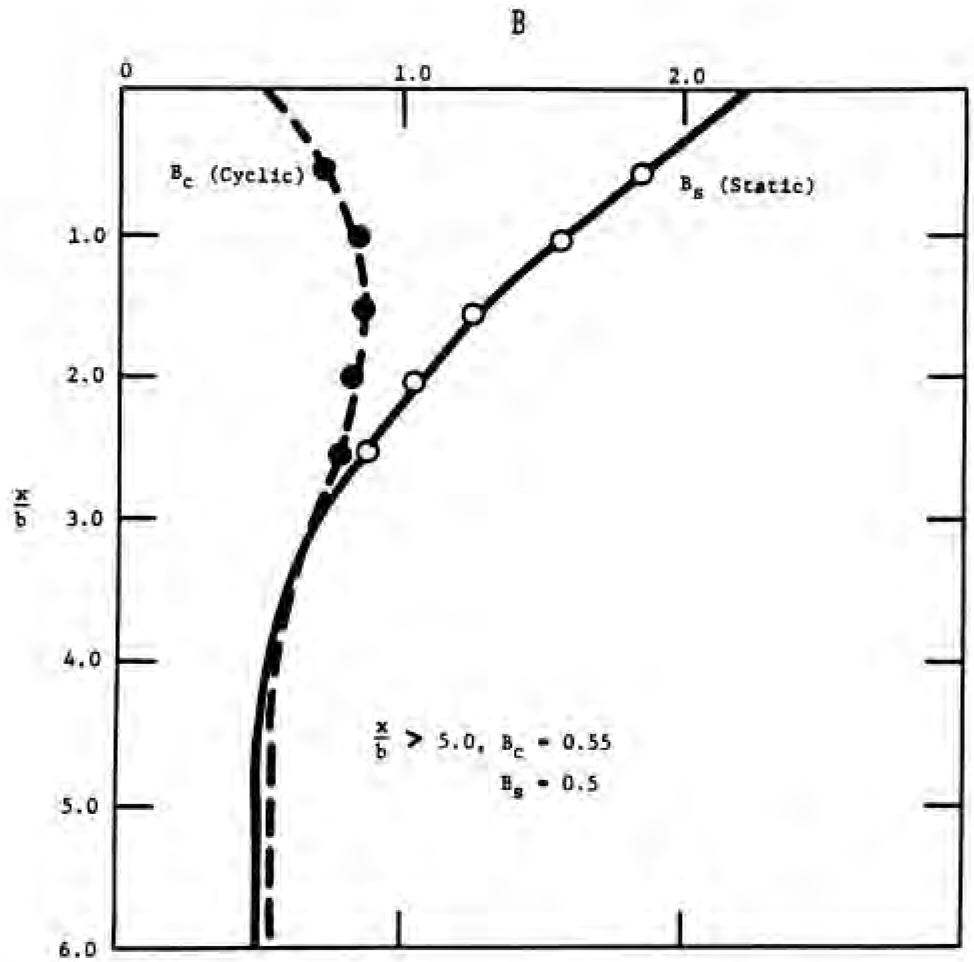


Figure A-11: Values of coefficients B for soil resistance vs. depth (after Reese et al. 1974).

A.5 P-Y CURVES FOR WEAK ROCK (REESE 1997)

Reese (1997) proposed a p-y criterion for drilled shafts socketed in weak rock, defined as having an unconfined compressive strength 0.5 MPa ≤ q_u ≤ 5 MPa (5.2 tsf ≤ q_u ≤ 52 tsf). In this criterion, the ultimate resistance of the weak rock is a function of the uniaxial unconfined compressive strength of intact rock, the drilled shaft diameter, rock quality designation (RQD), and depth.

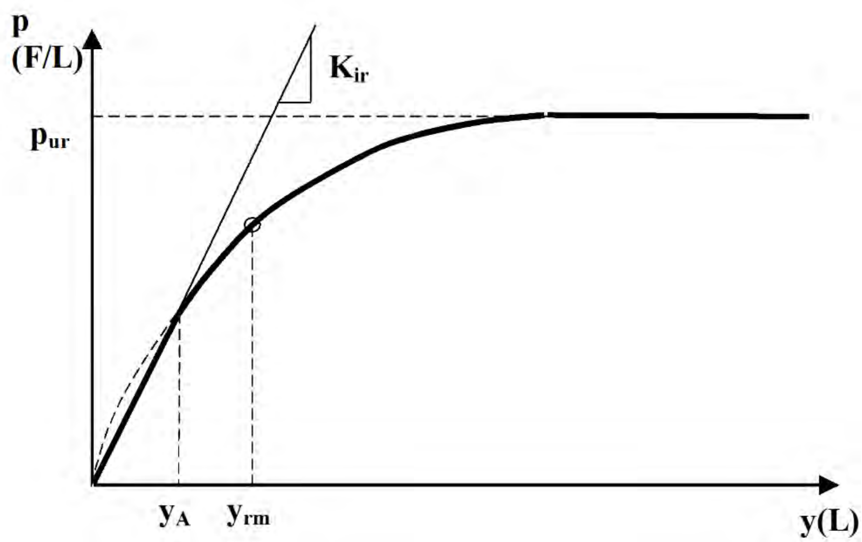


Figure A-12: Basic shape of p-y curve for weak rock (Reese 1997).

The shape of the Reese (1997) p-y curve is shown in Figure A-12. The ultimate reaction, p_u (F/L), of rock was given by:

$$p_u = \alpha_r \sigma_{ci} D \left(1 + 1.4 \frac{z_r}{D}\right) \text{ for } 0 \le z_r \le 3D \qquad \text{(Equation A-33)}$$

$$p_u = 5.2 \alpha_r \sigma_{ci} D \text{ for } z_r \ge 3D \qquad \text{(Equation A-34)}$$

where σ_{ci} is the uniaxial unconfined compressive strength of intact rock; α_r is the strength reduction factor, which is used to account for fracturing of rock mass; D is the diameter of the drilled shaft; and z_r is the depth below rock surface. α_r is assumed to be 0.33 for RQD = 100 percent and to increase linearly up to 1.0 for RQD = 0.

The slope of the initial portion of the p-y curve was given by:

$$K_{ir} = k_{ir} E_m \qquad \text{(Equation A-35)}$$

where K_{ir} = initial tangent to p-y curve; E_m = deformation modulus of rock masses, which could be obtained from a pressuremeter or dilatometer test; and k_{ir} = dimensionless constant. The expressions for k_{ir}, derived by correlation with experimental data, are as follows:

$$k_{ir} = \left(100 + \frac{400 z_r}{3D}\right) \text{ for } 0 \le z_r \le 3D \qquad \text{(Equation A-36)}$$

$$k_{ir} = 500 \qquad \text{(Equation A-37)}$$

A complete description of the interim p-y criteria may be summarized as follows:

<u>First segment</u>:

$$\boldsymbol{p = k_{ir} y; y \leq y_A}$$ (Equation A-38)

<u>Second segment</u>:

$$\boldsymbol{p = \frac{p_u}{2}\left(\frac{y}{y_{rm}}\right)^{0.25}; y \geq y_A \ and \ p \leq p_u}$$ (Equation A-39)

<u>Third segment</u>:

$$\boldsymbol{p = p_u; p \geq p_u}$$ (Equation A-40)

Where:

$$y_m = k_{rm} D$$ (Equation A-41)

$$y_A = \left[\frac{p_u}{2(y_{rm})^{0.25} K_{ir}}\right]^{1.333}$$ (Equation A-42)

in which, k_{rm} is the strain at 50 percent of the ultimate load, which ranges from 0.0005 to 0.00005.

A.6 P-Y CURVES FOR LIQUEFIED SANDS (ROLLINS ET AL. 2005)

Based on experiments, Rollins et al. (2005) developed a concave p-y criterion that increases the stiffness as pile deflection increases, as shown in Figure A-13, for liquefied sand. The proposed p-y curve is only a function of depth and the unit weight of the sand.

The p-y criterion for liquefied sand proposed by Rollins et al., 2005 is given by:

$$p = A(By)^C$$ (Equation A-43)

Where: *A*, *B*, and *C* are coefficients defined below; *p* is the soil resistance (in kN/m); *y* is the lateral deflection of the pile/shaft (in mm); and *z* is the depth (in m). The coefficients are defined as:

$A = 3\,(10^{-7})\,(z + 1)^{6.05}$;

$B = 2.80\,(z + 1)^{0.11}$; and

$C = 2.85\,(z + 1)^{-0.41}$

The use of Equation A-43 should generally be limited to conditions comparable to those from which it was derived, specifically for soil resistance *p* of approximately 85 lb/in. or less, pile deflections of 6 in. or less, depth of 20 ft or less, and sands with initial relative densities of about 50 percent.

The equation above was developed from a site where the groundwater was near the ground surface. For cases with different subsurface conditions, the depth (z) may be selected so as to obtain initial vertical effective stresses that are consistent with those of the study by Rollins et al. (2005), for which the effective unit weight of the sand was approximately 61.8 pcf.

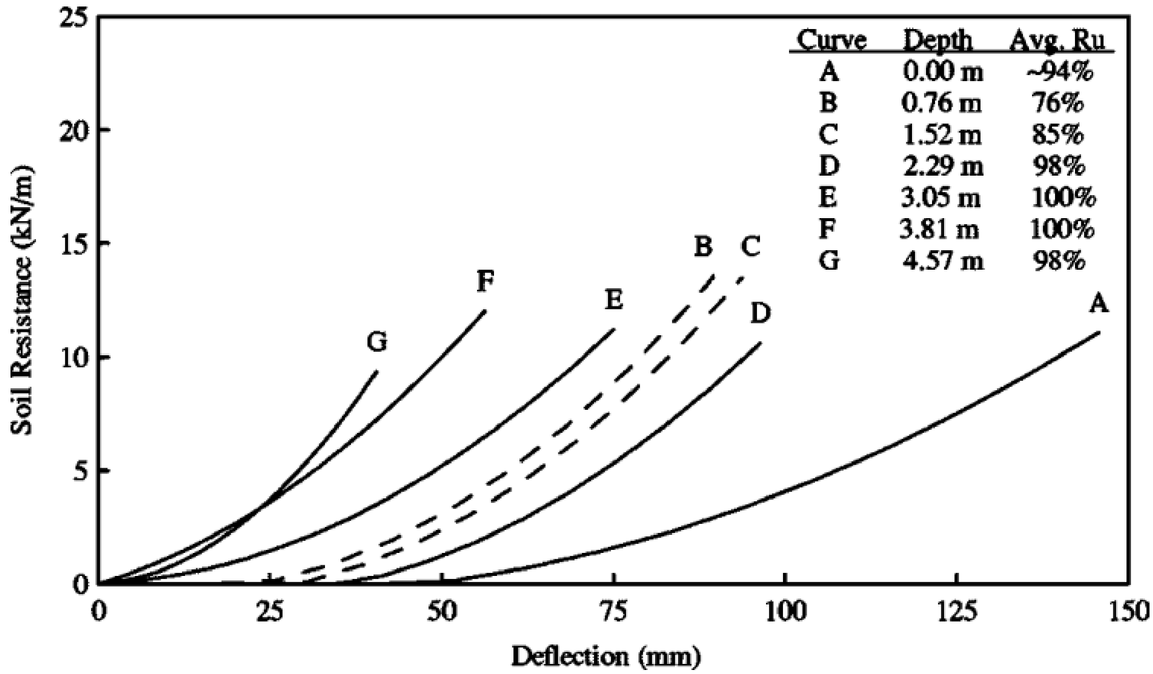


Figure A-13: Example p-y curve for liquefied sand (after Rollins et al. 2005).

Note that Equation A-43 is valid only for a pile diameter D = 324 mm (12 in.). For other pile diameters, the p values should be corrected. The effect of diameter on the p-y curves developed by Rollins et al. (2005) was studied by Weaver et al. (2001) who proposed a modification factor for correcting Equation A-43 as follows:

$$P_d = 3.81 \ln D + 5.6 \qquad \text{(Equation A-44)}$$

Where: D is the diameter of the pile in meters. The value of p calculated using Equation A-43 is multiplied by P_d to obtain p-y curves of piles for diameters other than the reference value. However, Equation A-44 may not be appropriate for piles/shafts with diameters significantly smaller than approximately 0.3 m (12 in.) or larger than 1 m (39 in.).

A.7 SLOPING GROUND

A.7.1 Ultimate Soil Resistance

Reese (1958) developed the following equations to estimate the soil ultimate resistance near the surface in sloping ground in clay:

$$p_u = (2c_a D + \gamma DH + 2.83 c_a H)\frac{1}{1+\tan\theta}$$

for soil at the front of the pile (Equation A-45)

$$p_u = (2c_a D + \gamma DH + 2.83 c_a H)\frac{\cos\theta}{\sqrt{2}\cos(45+\theta)}$$

for soil at the back of the pile (Equation A-46) where c_a is the average undrained shear strength, θ is the angle of the slope measured from the horizontal, and H = depth from ground surface to where the soil resistance is calculated.

For sand, the following equations presented by Reese et al. (2005) can be used to modify p_u of soil near surface except that the horizontal surface needs to be adjusted as a slope. The soil resistance at the front of the pile is:

$$p_u = \gamma z \left[\frac{K_0 \tan \emptyset \sin \beta}{\tan(\beta - \emptyset) \cos \alpha}(4D_1^3 - 3D_1^2 + 1) + \frac{\tan \beta}{\tan(\beta - \emptyset)}(DD_2 + z \tan \beta \tan \alpha D_2^2) + K_0 z \tan \beta (\tan \emptyset \sin \beta - \tan \alpha)(4D_1^3 - 3D_1^2 + 1) - K_A D\right]$$

(Equation A-47)

Where:

$$D_1 = \frac{\tan \beta \tan \theta}{\tan \beta \tan \theta + 1}$$

(Equation A-48)

$$D_2 = 1 - D_1$$

(Equation A-49)

$$K_A = \cos \theta \frac{\cos \theta - \sqrt{\cos^2 \theta - \cos^2 \emptyset}}{\cos \theta + \sqrt{\cos^2 \theta - \cos^2 \emptyset}}$$

(Equation A-50)

The soil resistance at the back of the pile is:

$$p_u = \gamma z \left[\frac{K_0 z \tan \emptyset \sin \beta}{\tan(\beta - \emptyset) \cos \alpha}(4D_3^3 - 3D_2^2 + 1) + \frac{\tan \beta}{\tan(\beta - \emptyset)}(DD_4 + z \tan \beta \tan \alpha D_4^2) + K_0 z \tan \beta (\tan \emptyset \sin \beta - \tan \alpha)(4D_3^3 - 3D_3^2 + 1) - K_A D\right]$$

(Equation A-51)

Where:

$$D_3 = \frac{\tan \beta \tan \theta}{1 - \tan \beta \tan \theta}$$

(Equation A-52)

$$D_4 = 1 + D_3$$

(Equation A-53)

APPENDIX B: EXAMPLE PROBLEMS AND/OR CASE HISTORIES

This appendix contains two detailed examples that illustrate the application of the recommended design methods presented in this manual. The example designs were developed for the following applications:

- Single Pile Lateral Analysis for the Design of an Intelligent Transportation System (ITS) Pole
- Pile Group Lateral Analysis for Design of a Bridge Pier

B.1 SINGLE PILE LATERAL ANALYSIS FOR THE DESIGN OF AN INTELLIGENT TRANSPORTATION SYSTEM (ITS) POLE

An ITS pole needs to be constructed as part of a large project in open terrain. For this type of structure, a single drilled shaft is considered feasible to support the ITS pole. This example is supported by analyses performed with the computer program LPILE. The ITS pole and its foundation system are shown in Figure B-1.

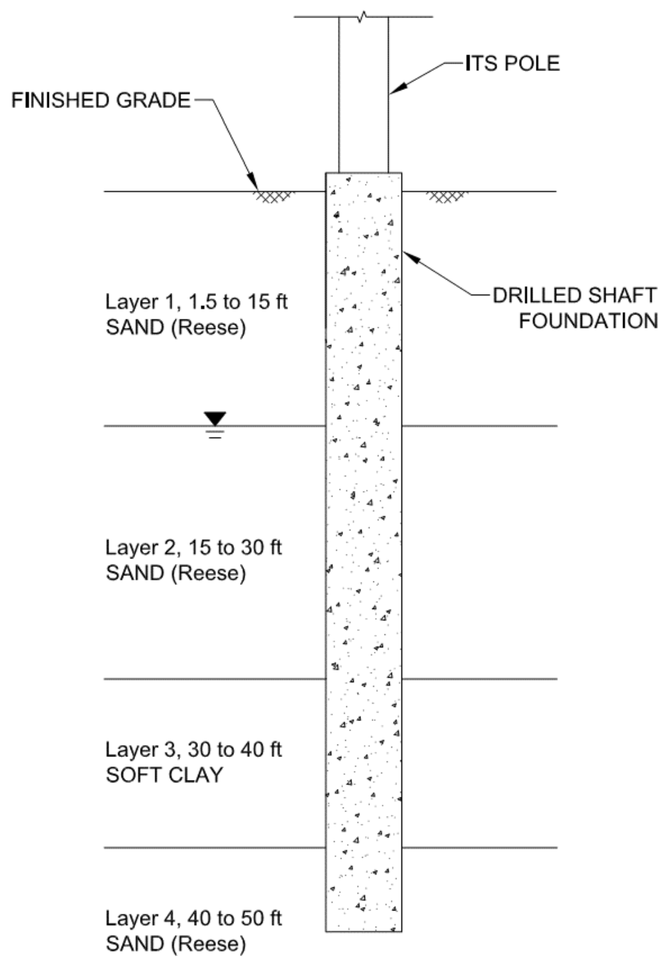


Figure B-1: Example problem for ITS pole.

Step 1: Determine Idealized Soil Profile and Geotechnical Design Parameters

The first step consists of the determination of an idealized soil profile based on the results of the subsurface investigation program. Presenting a detailed evaluation of the subsurface investigation program goes beyond the scope of this manual, and therefore is not presented in detail herein. Figure B-1 shows the assumed idealized soil profile.

The assumed geotechnical design parameters are summarized in Table B-1.

Table B-1: Interpreted soil parameters.

Layer No.	Soil Type Model	Depth* (ft)	Φ (deg)	Su (psf)	γ' (pcf)	k_s** (pci)	ε_{50}** (-)
1	Sand (Reese)	1.5 - 15	30	-	115	90	-
2	Sand (Reese)	15 - 30	30	-	52.6	60	-
3	Soft Clay	30 - 40	-	300	47.6	-	0.02
4	Sand (Reese)	40 - 50	30	-	57.6	80	-

* Depth is measure from the top of pile

** Refer to Chapter 3 for discussion on estimating soil parameters

The top of pile is 1.5 feet above the ground surface, and groundwater is located 13.5 feet below the ground surface (15 feet below top of pile).

Step 2: Obtain Preliminary Structural Design

The lateral response of the pile depends on the stiffness properties of the pile itself, as well as on the subsurface soil conditions. A preliminary structural design (drilled shaft diameter, number of size of reinforcement, etc.) needs to be established before computing deflections, bending moment and shear diagrams. For this example, the following preliminary structural design is selected:

- Drilled shaft diameter = 54 inches
- Concrete Compressive Strength = 4,000 psi
- 20 #10 single bars (Fy = 60 ksi)
- Concrete cover to edge of bar distance = 3 inches
- No permanent steel casing
- Drilled Shaft Length = 45 feet (5 feet embedment into Layer No. 4)
- #3 bars for Ties

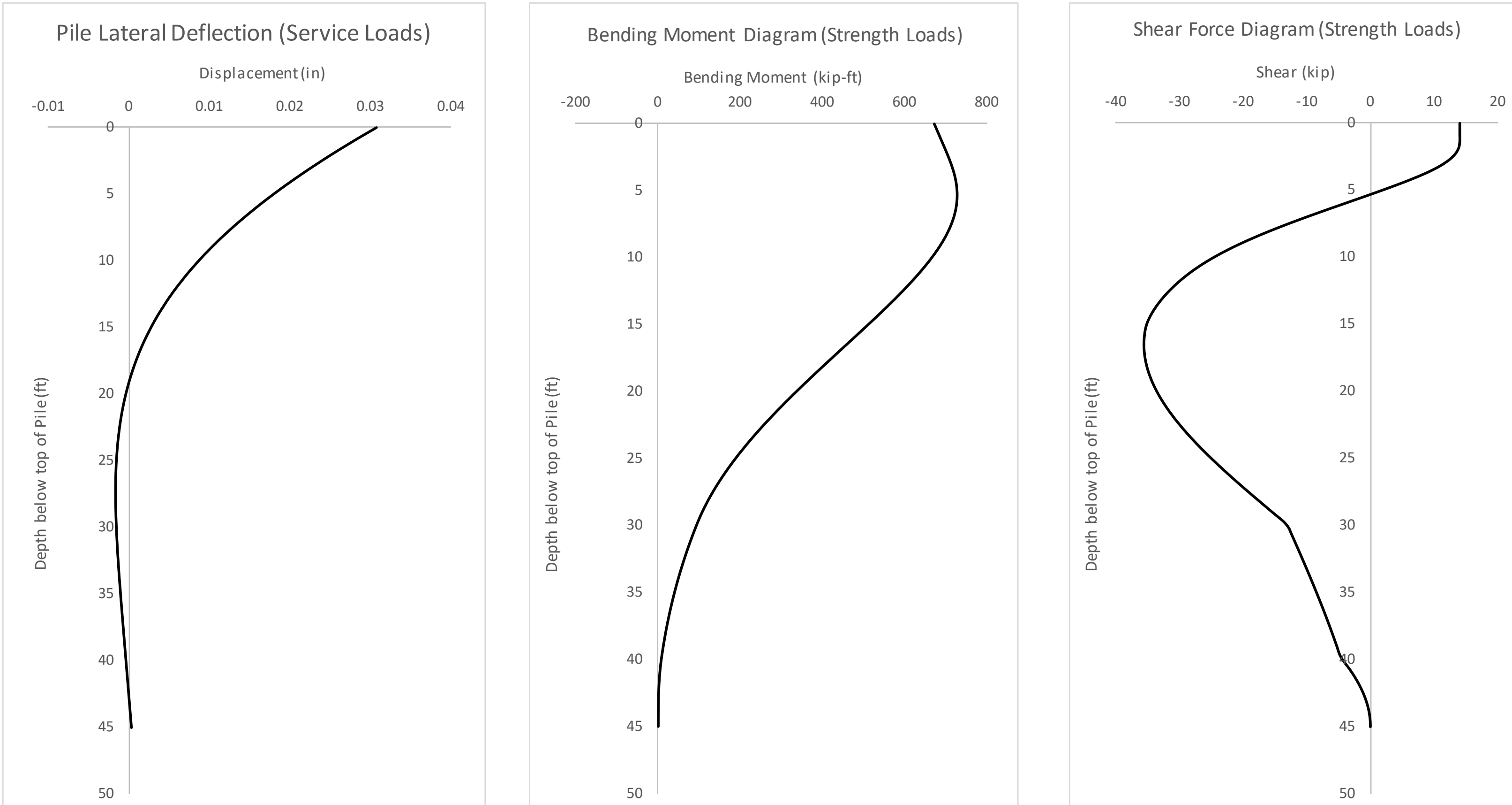
Step 3: Determine Factored Loads

The actual and detailed determination of the factored loads is beyond the scope of this example. In general, at minimum, factored loads for Strength and Service Limit States should be determined and analyzed. As applicable, other Limit States such as Extreme I (earthquake) should also be considered. For simplicity, this example will only analyze Strength and Service Limit States. The Strength factored loads are used to assess the structural integrity of the pile, while the Service factored loads are used to estimate the lateral deflection profile of the pile.

Table B-2 presents the factored loads used in this example. All loads are applied at the top of the pile.

Table B-2: Factored loads.

Limit State	Moment (kip-ft)	Shear (kip)	Axial (kip)
Service	144	3	9
Strength	672	14	12



(a) (b) (c)

Figure B-2: Pile top lateral deflection (a), bending moment (b), and shear diagrams (c).

Step 4: Obtain Bending Moment, Shear, and Lateral Deformation Profiles

The program LPILE was used to compute bending moment, shear and lateral deformation profiles.

Note that in accordance with LRFD guidelines, the pile lateral deflection is computed using the Service Loads, while bending moment and shear diagrams are evaluated using the Strength Loads. In summary:

- Pile top lateral deflection = 0.03 inch
- Maximum bending moment = 727 kip-ft
- Maximum shear = 35.5 kip

The values reported above are computed using the appropriate factored loads, and are used in the next step to assess the pile structural integrity.

Step 5: Assess Pile Structural Integrity

The engineer must check the structural integrity of the selected section by constructing an interaction diagram for the effect of combining the Bending Moment (M) and Axial Load (P). The P-M interaction diagram is based on an equilibrium and strain compatibility approach for biaxial flexure and compression.

If the section is non-circular, under special circumstances, the AASHTO Article 5.7.4.5 allows designers to use an approximate method to evaluate biaxial bending combined with axial. In lieu of the simplified or rigorous analysis, designers could compute their P-M interaction diagram using commonly available software such as spColumn (Structure Point, 2016) or WinYield (Caltrans 2014b), keeping in mind the material of the section and the standards that govern the calculation.

The single reinforced-concrete pile example was computed using spColumn v5.50 (by Structure Point, LLC), following ACI 318-14 and AASHTO Articles 5.5.4.2.1 and 5.7.4.4.

The values reported are well within the P-M interaction diagram (acceptable structural capacity). If desired, the section could be optimized by modifying the pile structural section and restarting from Step 2. This optimization process is not presented in this example.

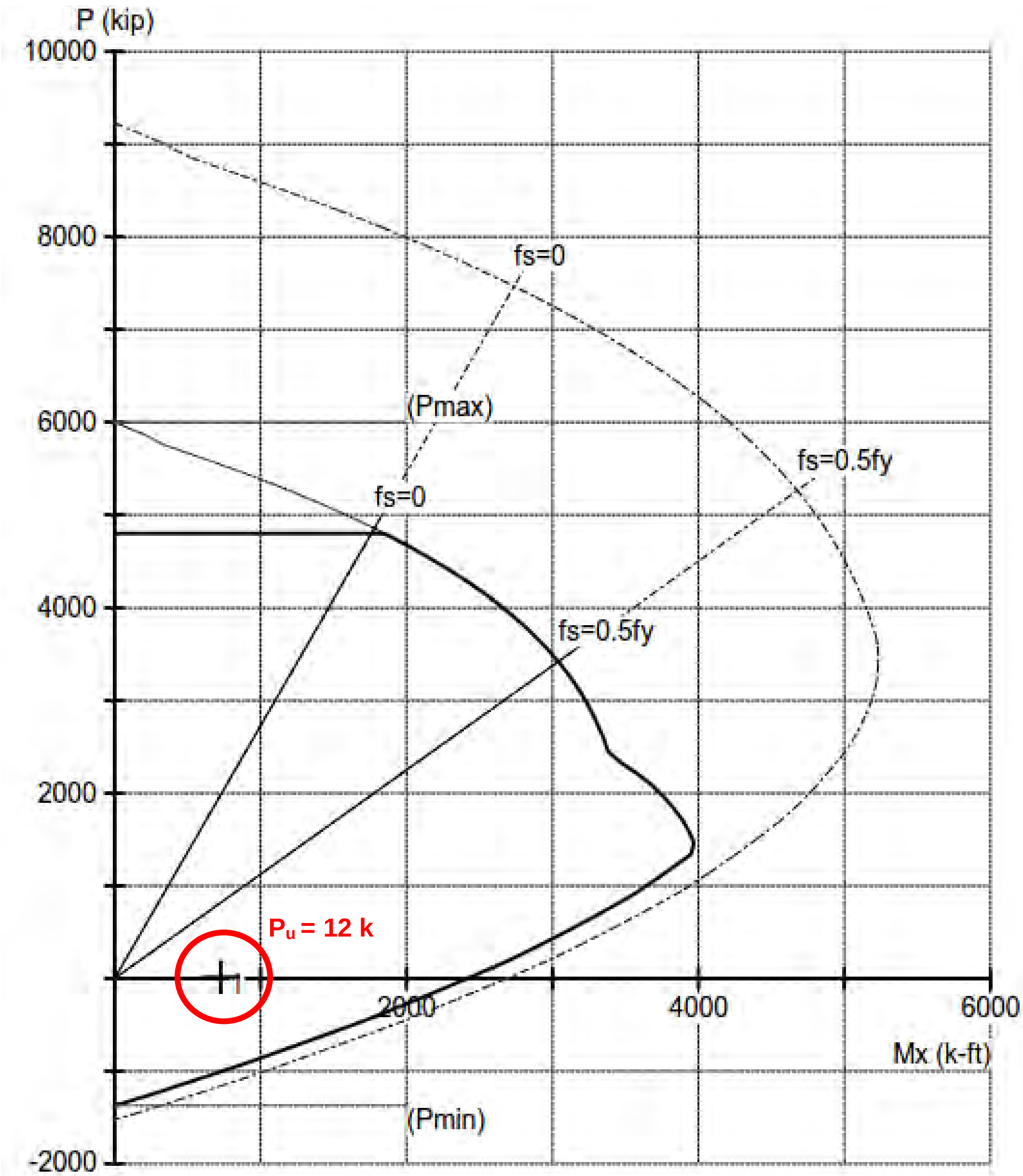


Figure B-3: Unfactored and factored bending moment and axial force interaction diagram with resulting ultimate loads from lateral analysis.

Step 6: Final Design

This step consists of the evaluation of the data collected in previous steps, and in the assessment of any potential design optimization.

First, the results of the structural assessment (Step 6) should be used to determine whether a change in the pile preliminary structural design (diameter, reinforcement, etc.) is needed. If any element affecting the pile lateral response is modified, the analyses should restart from Step 2 based on the new proposed structural design. If the structural assessment is considered adequate, and no modification to the pile structural section is needed, then a check on the pile length should be performed. This can be verified by analyzing the top pile lateral deflection for different trial pile lengths, and constructing a plot relating pile top lateral deflection versus total pile length. This plot can automatically be generated by available programs such as LPILE. For the example being analyzed, Figure B-4 shows such relationship. Note that Service loads should be used to generate this graph since it is built by calculating a displacement.

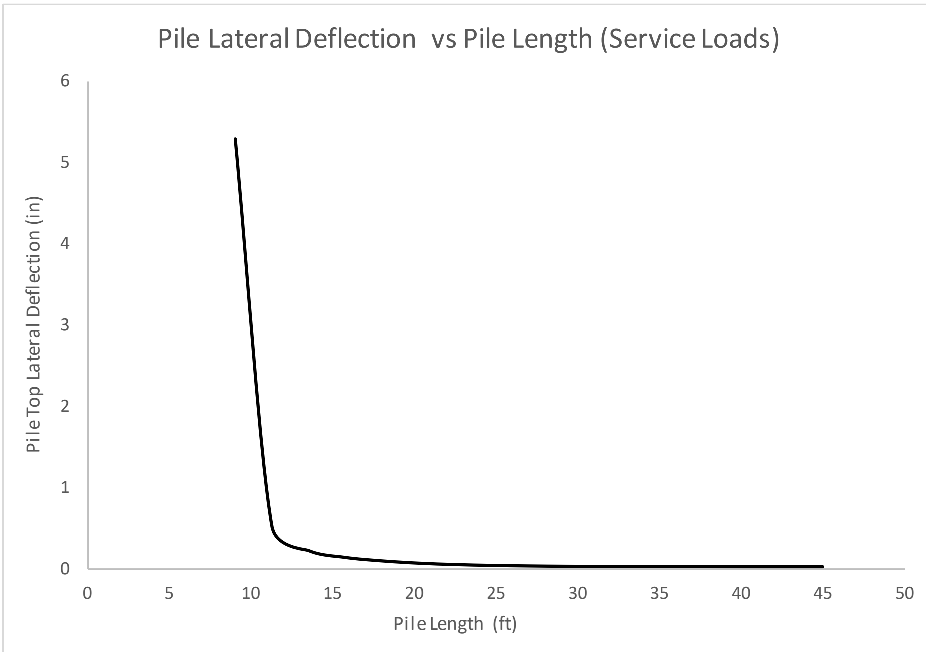


Figure B-4: Pile top lateral deflection vs. pile length.

The plot shown in Figure B-4 shows that for this example the pile top lateral deflection is relatively constant once the pile length exceeds about 20 feet. From this perspective, a shorter pile could be selected and re-analyzed. Other considerations such as settlement of the soft clay layer, pile axial resistance, and other owner constraints which are not discussed in this example may still require the pile to extend below the bottom of the soft clay layer (Layer 3). It is important to consider that a reduction in the pile length will have an impact on the pile bending moment and shear. A similar plot to the one shown in Figure B-4 could be constructed relating pile length to maximum bending moment and maximum shear (Strength Loads shall be used for this plot as bending moment and shear are being evaluated), as shown in Figure B-5. Nevertheless, if a reduced pile length is selected, the analyses should restart from Step 2 utilizing the reduced pile length and any other structural modification to the pile structural section as selected by the structural engineer. This additional process is not presented in this example, but could be carried out to optimize the pile design.

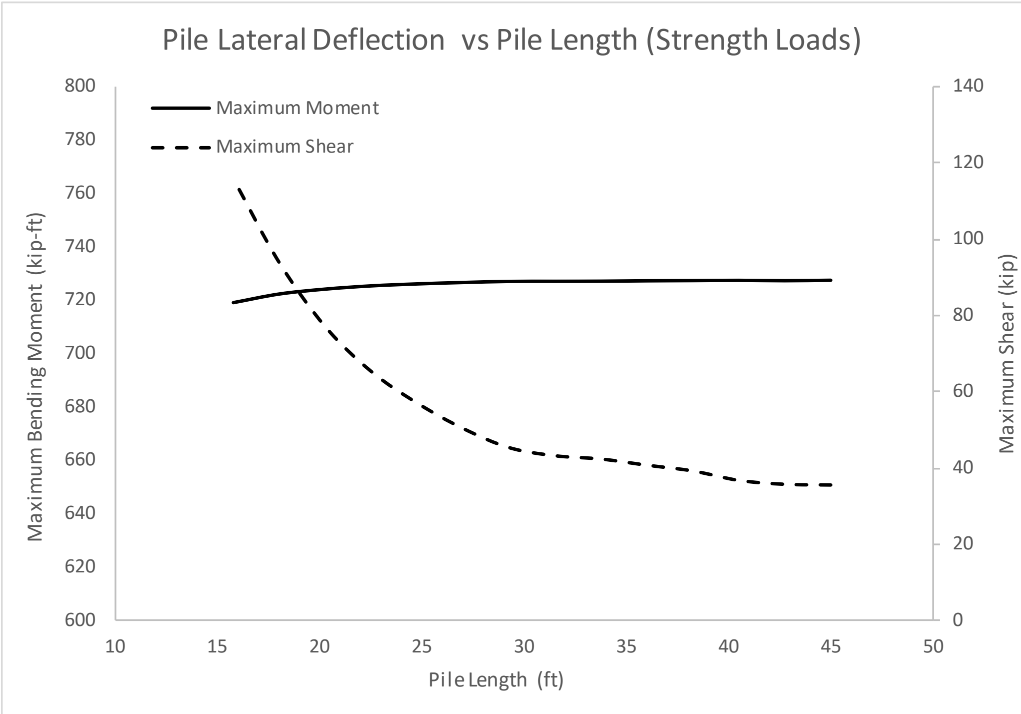


Figure B-5: Maximum bending moment and shear vs. pile length.

B.2 PILE GROUP LATERAL ANALYSIS FOR DESIGN OF A BRIDGE PIER

This examples presents a lateral analysis of a bridge pier foundation consisting of a group of 4 2-foot diameter driven close ended pipe piles filled with concrete. This example is supported by analyses performed with the computer program GROUP. The bridge pier and its foundation system are shown in Figure B-6. Due to the nature of the loading, a three-dimensional analysis (3D) is performed (longitudinal and transverse directions).

Step 1: Determine Idealized Soil Profile and Geotechnical Design Parameters

The first step consists of the determination of an idealized soil profile based on the results of the subsurface investigation program. Presenting a detailed evaluation of the subsurface investigation program goes beyond the scope of this manual, and therefore is not presented in detail herein.

The assumed idealized soil profile and geotechnical design parameters are summarized in Table B-3.

The top of pile and groundwater level are 7 feet below the ground surface.

In this example, it is determined that no potential for liquefaction, lateral spreading, or lateral flow exists, therefore the same parameters shown in Table B-3 can be used to assess all the Limit States being considered (Service, Strength and Extreme).

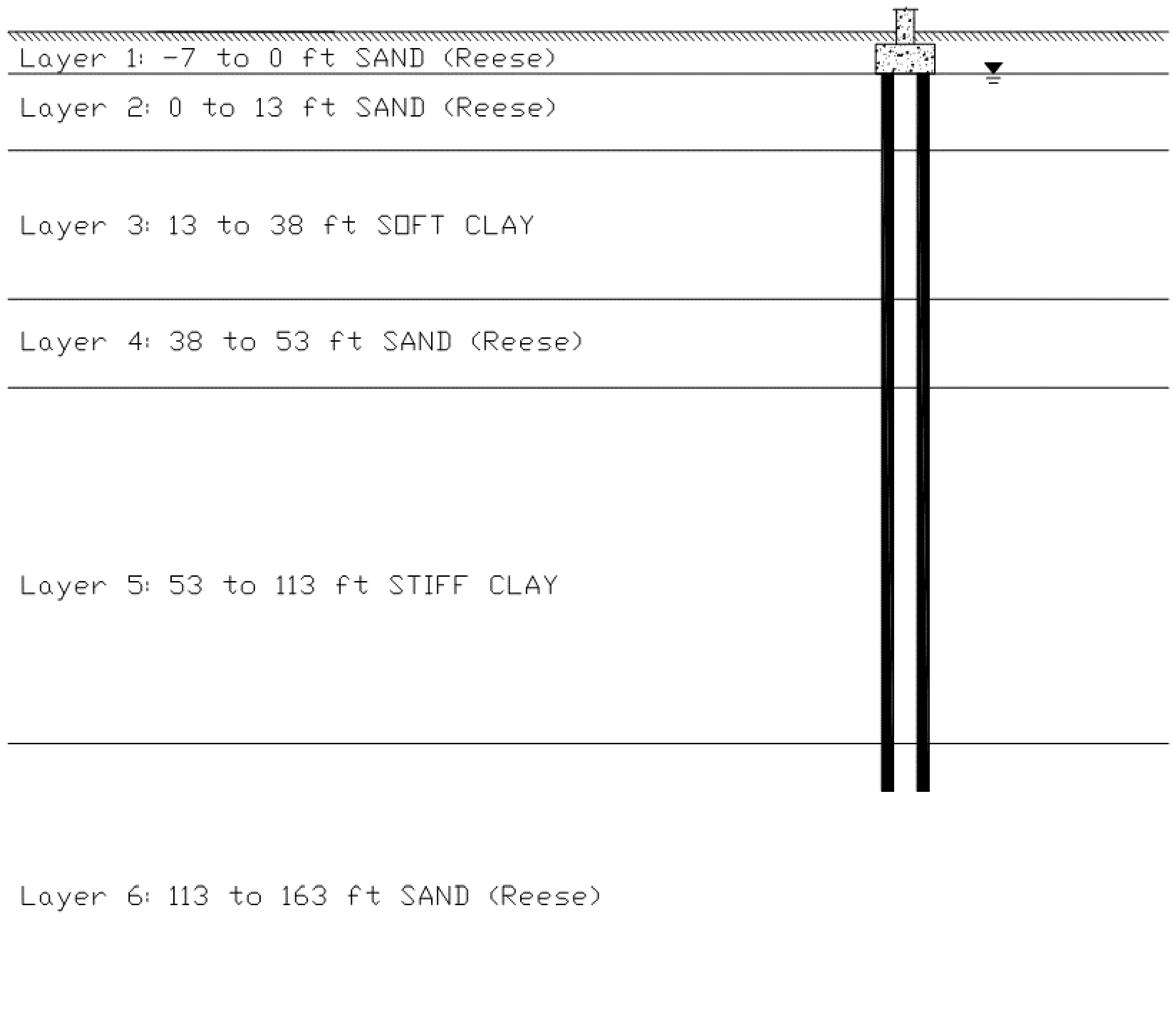


Figure B-6: Example problem B2.

Table B-3: Interpreted soil parameters.

Layer No.	Soil Type Model	Depth* (ft)	**Φ (deg)**	Su (psf)	**γ' (pcf)**	k_s** (pci)	**ε_{50}**** (-)
1	Sand (Reese)	-7 - 0	30	-	115	50	-
2	Sand (Reese)	0 - 13	30	-	52.6	35	-
3	Soft Clay (Matlock)	13 – 38	-	400	37.6	-	0.02
4	Sand (Reese)	38 - 53	36	-	57.6	95	-
5	Stiff Clay	53 – 113	-	1500	57.6	500	0.007
6	Sand (Reese)	113 - 163	40	-	57.6	150	-

* Depth is measure from the top of pile. Negative values represent distances above the top of pile.

** Refer to Chapter 3 for discussion on estimating soil parameters

Step 2: Obtain Preliminary Structural Design

A preliminary structural design needs to be established before calculating deflections, bending moment and shear. For this example, the following preliminary structural design is selected:

- Close Ended Pipe Pile Diameter = 24 inches
- Concrete Compressive Strength = 4,000 psi
- No steel rebars
- Pile Wall Thickness = 1 inch
- Pile Length = 120 feet (5 feet embedment into Layer No. 6)
- Pile head is assumed to be fixed
- Steel grade = 36 ksi (A36)
- Piles Center to Center Spacing = 6 feet

Step 3: Determine p-multipliers

The group effect due to closely spaced piles is typically accounted by utilizing a p-multiplier value which reduces the soil resistance “p” in the p-y curves by a factor equal to the p-multiplier. The selection of p-multipliers is discussed in more detail in Chapter 7.

For this example, the procedure highlighted in AASHTO LRFD Paragraph 10.7.2.4 is used. For each pile, a p-multiplier in the longitudinal and transverse direction must be calculated. Referring to Figure B-7 with the piles ID shown, the p-multipliers shown in Table B-4 are calculated.

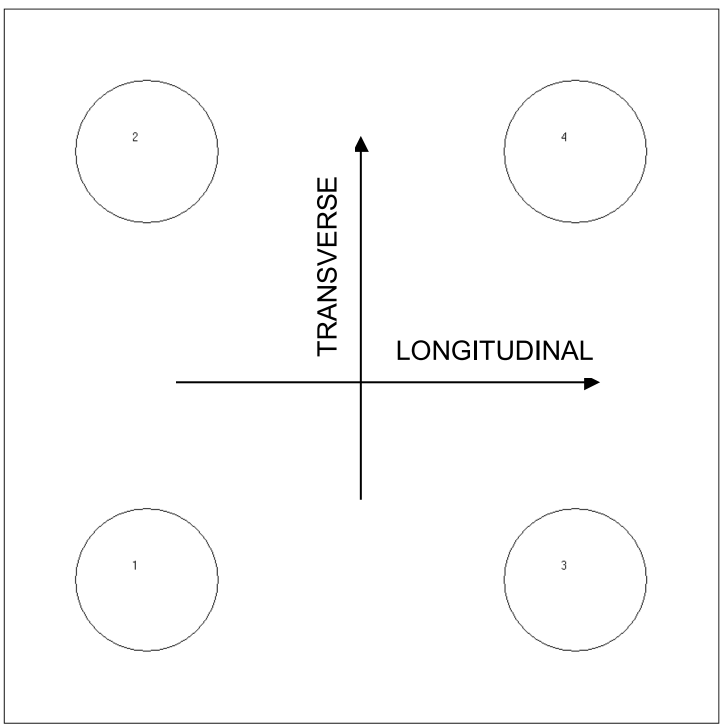


Figure B-7: Piles arrangement and ID.

Table B-4: P-multipliers.

Pile No.	p-multiplier (longitudinal)	p-multiplier (transverse)
1	0.4	0.4
2	0.4	0.8
3	0.8	0.4
4	0.8	0.8

Step 4: Determine Factored Loads

The determination of the factored loads is beyond the scope of this example. In this example, Strength, Service, and Extreme Limit States are provided by others as a result of structural design analyses.

Table B-5 presents the factored loads used in this example. All loads are applied at the bottom of the pile cap.

Table B-5: Factored loads.

Limit State	Moment Longitudinal (kip-ft)	Moment Transverse (kip-ft)	Moment Torsion (kip-ft)	Shear Longitudinal (kip)	Shear Transverse (kip)	Axial (kip)
Service	250	80	8	15	25	1200
Strength	400	110	10	40	65	1600
Extreme	800	1000	150	120	90	900

Step 4: Obtain Bending Moment, Shear, and Lateral Deformation Profiles

The program GROUP was used to compute bending moment, shear and lateral deformation profiles. Figure B-8 shows the pile group model.

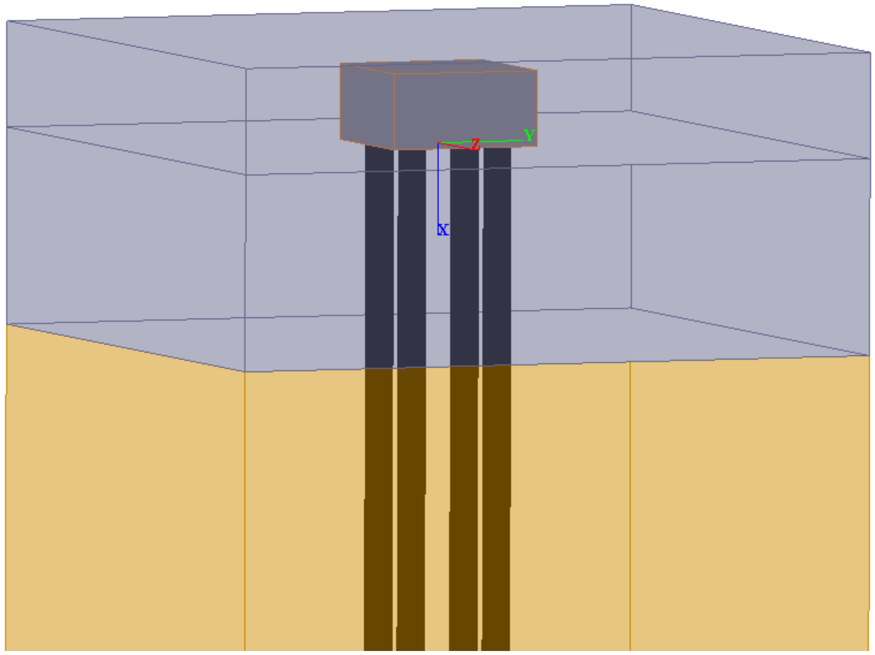


Figure B-8: Pile group model (from GROUP program).

Note that the in accordance with LRFD guidelines, the pile lateral deflection is computed using the Service Loads, while bending moment and shear diagrams are evaluated using the Strength Limit State Loads and Extreme Loads. In summary:

- Pile top maximum lateral deflection = 0.04 inch
- Bottom of pile cap displacement (longitudinal) = 0.01 inch
- Bottom of pile cap displacement (transverse) = 0.03 inch
- Maximum bending moment (STRENGTH) = 48 kip-ft
- Maximum shear (STRENGTH) = 19 kip
- Maximum bending moment (EXTREME) = 114 kip-ft
- Maximum shear (EXTREME) = 38 kip

The values reported above are combined values in the longitudinal and transverse directions, and are used in the next step to assess the pile structural integrity.

Step 5: Assess Pile Structural Integrity

The structural capacity of the pile can be verified utilizing the same procedure presented above for the single pile example. Given that the piles are closed ended composite piles, the design should follow AISC and AASHTO 6.5. This example is also computed with spColumn v5.50 (by Structure Point, LLC), and shows both Strength and Extreme Load Cases. The results are shown in Figure B-9.

The values reported are well within the P-M interaction diagram (acceptable structural capacity). If desired, the section could be optimized by modifying the pile structural section and restarting from Step 2. This optimization process is not presented in this example.

Step 6: Final Design

This step consists of the evaluation of the data collected in previous steps, and in the assessment of any potential design optimization.

First, the results of the structural assessment (Step 6) should be used to determine whether a change in the pile structural section (diameter, wall thickness, etc.) is needed. If any element affecting the pile lateral response is modified, the analyses should restart from Step 2 based on the new proposed structural design. If the structural assessment is considered adequate, and no modification to the pile structural section is needed, then a check on the pile length should be performed. This procedure is discussed in the previous example, and will not be repeated here.

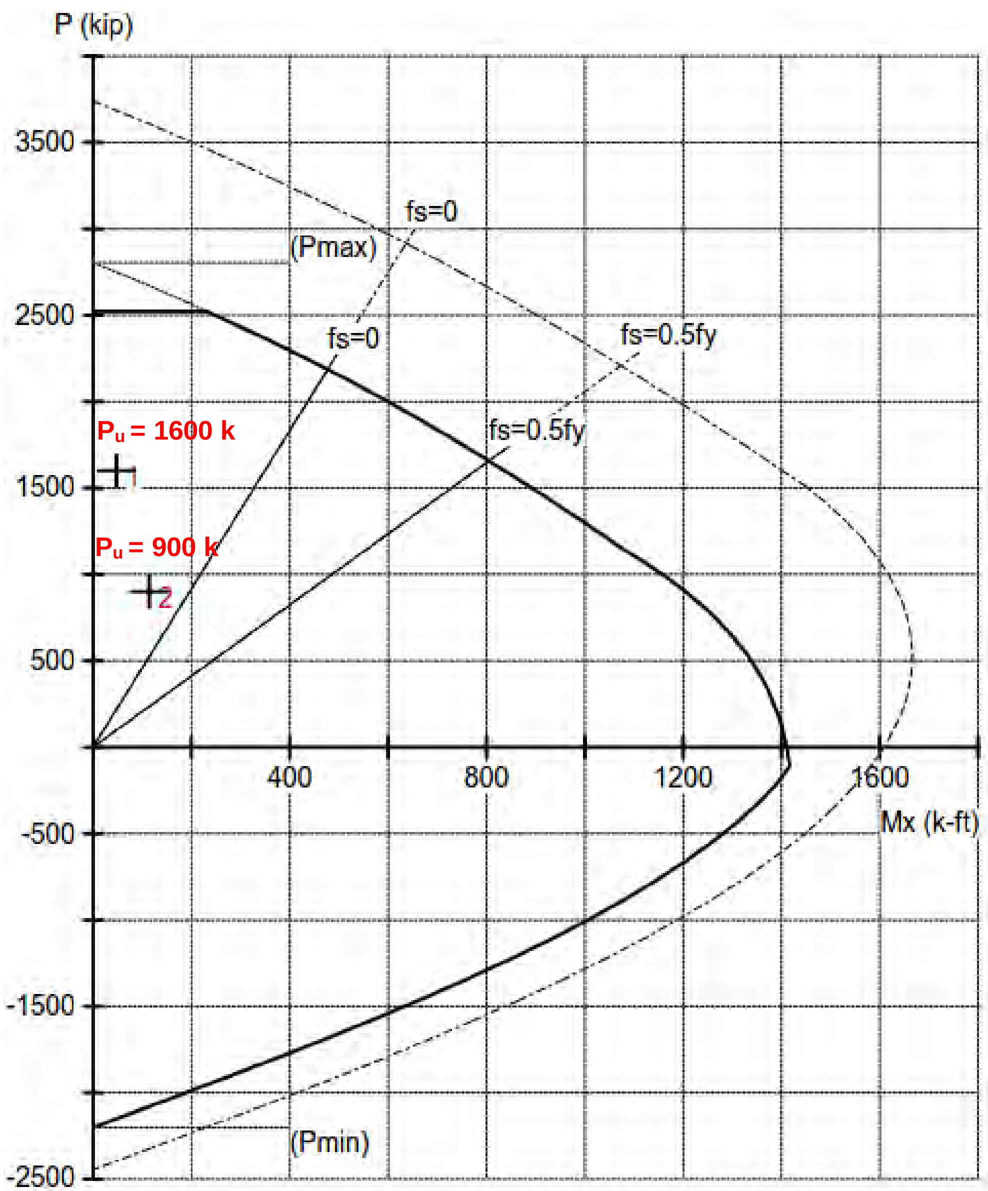


Figure B-9: Unfactored and factored bending moment and axial force interaction diagram with resulting load demands from lateral analysis (Step 4).

APPENDIX C: EXAMPLE LOAD TEST RESULTS AND INTERPRETATION AND DETERMINATION OF P-Y CURVES

An instrumented lateral load test program was reported by Nusairat et al. (2006) for design of drilled shafts in rock for a two-span reinforced concrete rib arch bridge project on the Stillwater River in Dayton, Ohio. The results of that test are used as an example for p-y curve generation. Information from the test is summarized herein as needed to support the example p-y curve generation; refer to the report by Nusairat et al. (2006) for full details of the test program and results.

C.1 SUBSURFACE CONDITIONS AT TEST SITE

The surficial soils at the test site are Quaternary glacial deposits that are generally less than 30 feet in thickness. The underlying bedrock comprises limestone and shale of the Richmond Formation. Figure C-1 illustrates the logs from borings B-2 and B-4 that were drilled near the site of the lateral load test. Boring B-2 is the closer of the two borings to test site. Fill was found to a depth of 3.5 feet and 8 feet at B-2 and B-4 respectively. The fill was underlain by natural fine grained soils, namely sandy silt, silt and clay, and silty clay. Auger refusal occurred at depths of 13.5 feet and 24 feet in borings B-2 and B-4 respectively. The rock below the refusal depth was cored to depths of 28.5 feet and 40 feet, respectively. The rock is soft to medium gray shale inter-bedded with hard gray limestone layers less than 1 foot in thickness. The shale is slightly weathered to decomposed, weakly calcareous, thinly laminated, and broken to very broken, becoming massive near the completion depth of the borings.

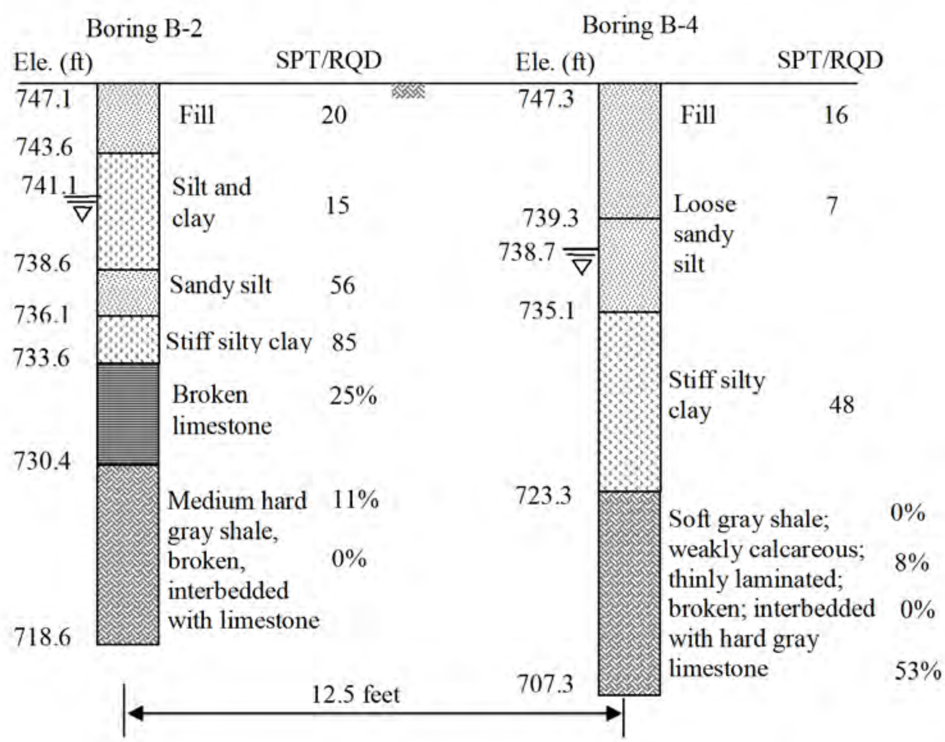


Figure C-1: Soil and rock layer profiles at Dayton test site.

Drilled shafts were the chosen foundation type. However, the lateral load resistance of the drilled shafts in the weathered rock was not known with reasonable engineering certainty. Lateral load testing was therefore proposed.

C.2 LATERAL LOAD TEST SET-UP

Drilled shafts for lateral load testing were constructed at the site, each embedded entirely within the shale inter-bedded with limestone. The embedment length in rock for each shaft was 18 feet. The shafts were 6 feet in diameter, and each was reinforced with a cage comprising 36 #11 longitudinal steel bars. A spiral consisting of #6 bar with a pitch of 2 to 3 inches provided the shear reinforcement. The shafts had a center to center spacing of 18 feet (i.e., 3 diameters) and each was instrumented with an inclinometer casing and an array of strain gauge pairs, spaced at 2 to 3 feet vertical intervals, per Figure C-2.

The drilled shafts were pushed apart in the test using a hydraulic jack and steel strut between the shafts. A load cell was installed to measure the applied lateral load (Figure C-2). Lateral loads were applied in increments of 50 or 100 kips, up to a maximum load of 1126 kips. Each load was held until the rate of deflection at the top of the shaft was less than 0.04 inches per minute. Inclinometer readings were taken just prior to the application of each load increment.

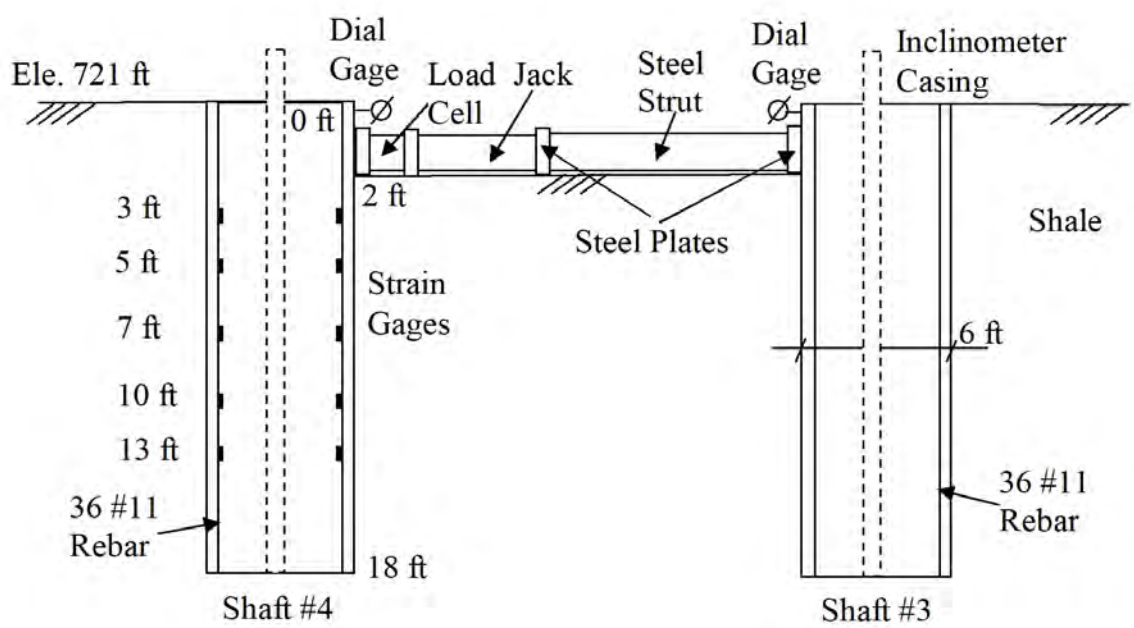


Figure C-2: Drilled shaft lateral load test set-up and instrumentation, Dayton, Ohio.

C.3 LATERAL LOAD TEST RESULTS

The lateral load test results obtained from simultaneously pushing the two shafts apart include load-deflection curves for the tops of the two shafts, profiles of deflection versus depth from the inclinometers at each increment of load, and the strain gauge readings at each load increment.

Figure C-3 presents the load-deflection measured at the top of each shaft during each increment of load application. The deflections plotted are averaged from three dial gauge readings at the head of Shaft #4 and two dial gauge readings at the head of Shaft #3. The response in both shafts is similar, indicative of a consistent response of the rock mass at the two locations spaced 18 feet apart. It is also apparent that the response is non-linear, even at small deflections. A permanent deflection of 0.05 inches was measured at the top of the shafts on unloading from the maximum test loads.

Figure C-4 and Figure C-5 show the lateral deflection versus depth profiles for drilled shafts #3 and #4 respectively, that were derived from the inclinometer data. These plots illustrate that deflections occurred along the entire shaft lengths, even in the early stages of the test.

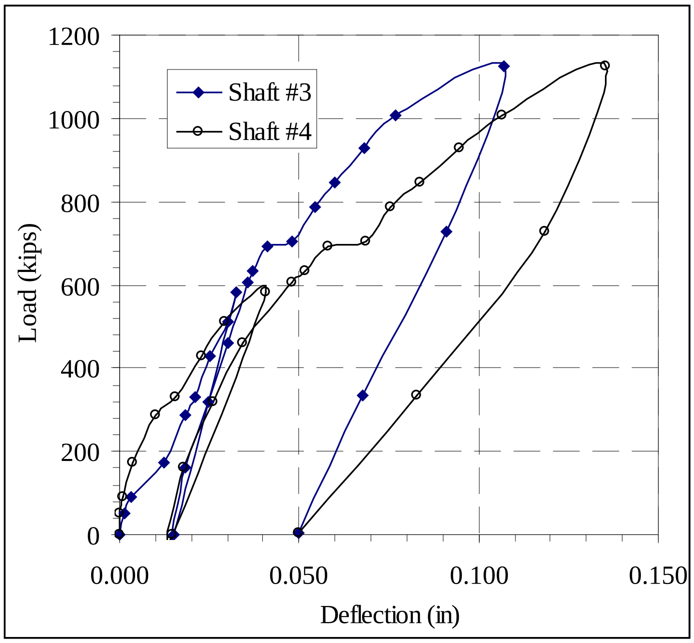


Figure C-3: Load deflection response at the top of each shaft.

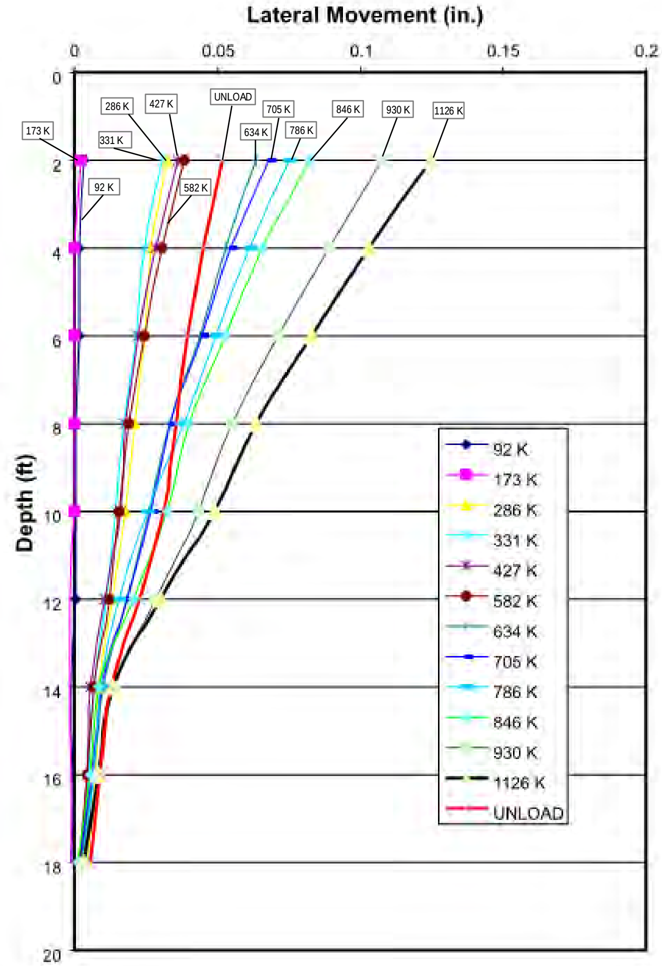


Figure C-4: Lateral deflection versus depth profiles for drilled shaft #3.

The compressive and tensile strains measured in drilled shaft #4 are presented in Figure C-6 and Figure C-7, respectively. A large increase in tensile strain was recorded at the depth of 8 feet when the load increased from 510 kips to 582 kips (Figure C-7). It is inferred that this marks the onset of concrete cracking. One of the outcomes of this test for use in design would therefore be to recognize the requirement for additional steel reinforcement in this upper zone to control the deflection and limit cracking of the concrete.

By using the piecewise polynomial curve fitting technique, the p-y curves at two depths (3 feet and 11 feet) for Shaft #4 are derived (Figure C-8). Commercially available software is used to predict the deep foundation load versus deformation response for a particular "standard" p-y relationship, or user defined p-y relationship where site specific data from tests exist. With a p-y relationship derived from a lateral load test, the software can be used to evaluate the computed load deflection response compared to the measured response. A practitioner may then adjust the soil parameter values input to the software to calibrate the soil model with the test derived p-y curve relationship. In this way, the designer has a predictive tool to evaluate the foundation design under other load cases and geometries, knowing that the soil parameters for the site in conjunction with the p-y model in the software are replicating the performance of the test. Figure C-9 provides an example plot of a load versus deflection prediction for the top of the drilled shaft. The curve obtained from the lateral load test data is in close agreement with the results using the Reese (1997) p-y criterion.

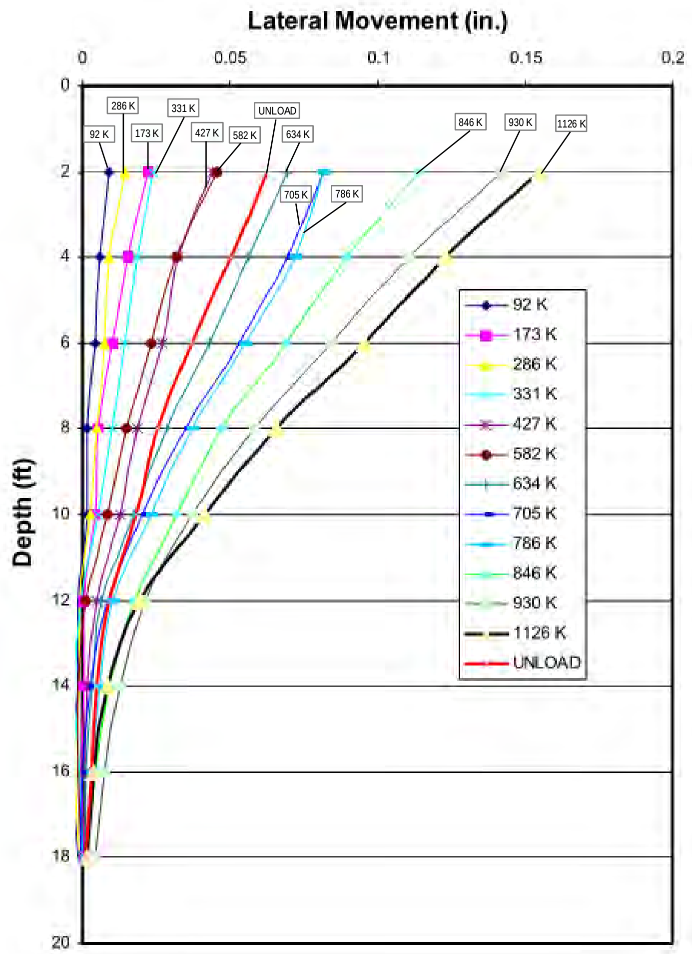


Figure C-5: Lateral deflection versus depth profiles for drilled shaft #4.

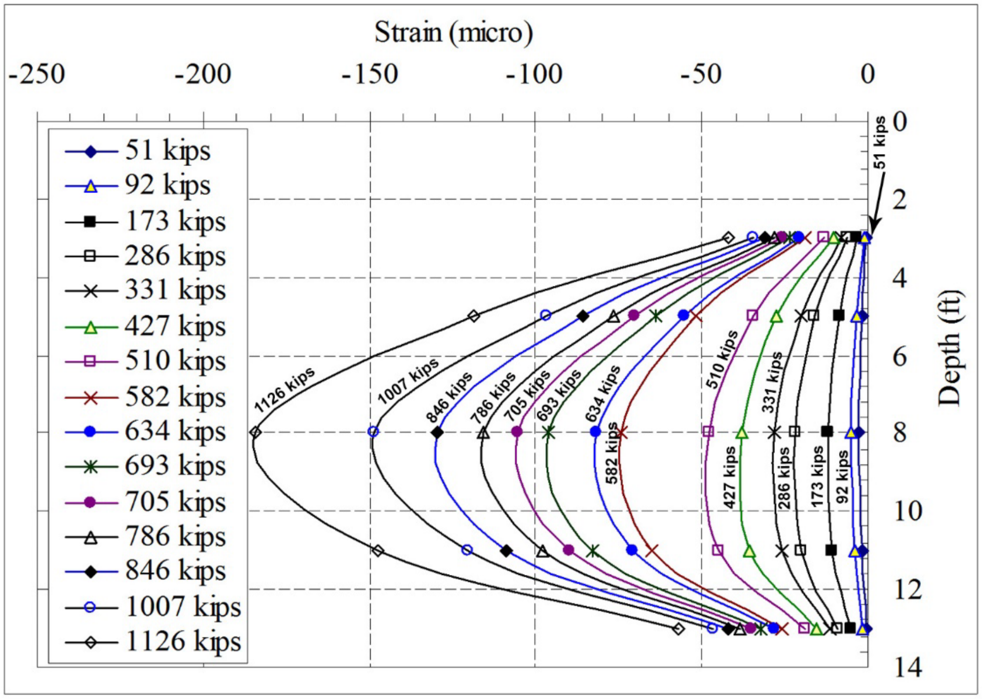


Figure C-6: Compressive strain versus depth in drilled shaft #4.

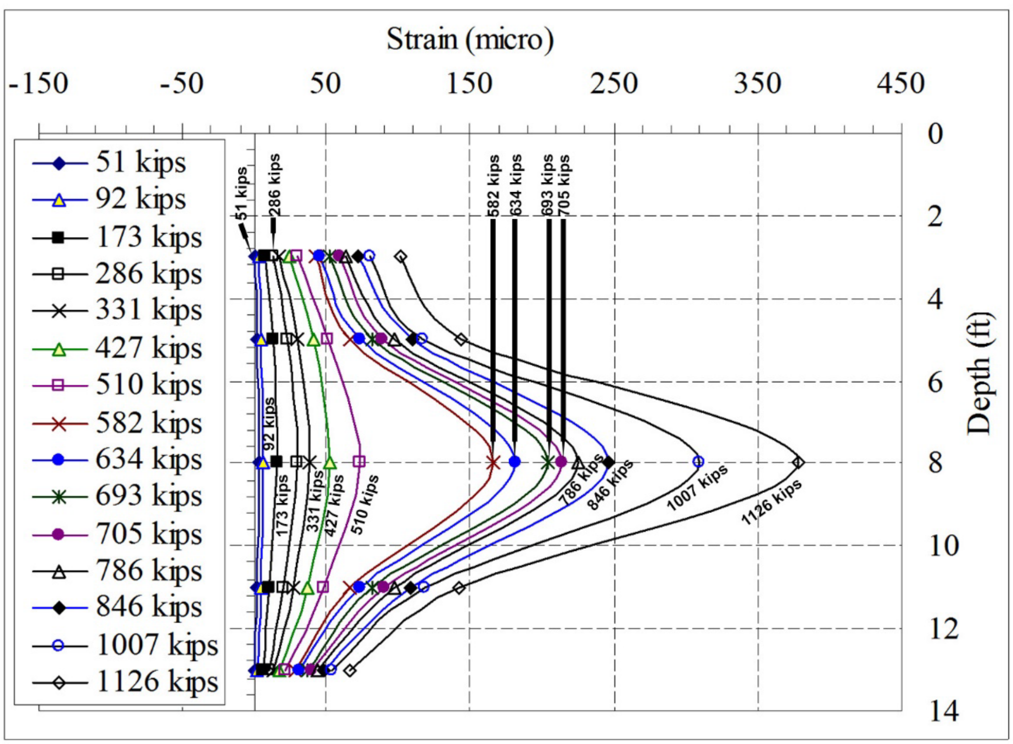


Figure C-7: Tensile strain versus depth measured in drilled shaft #4.

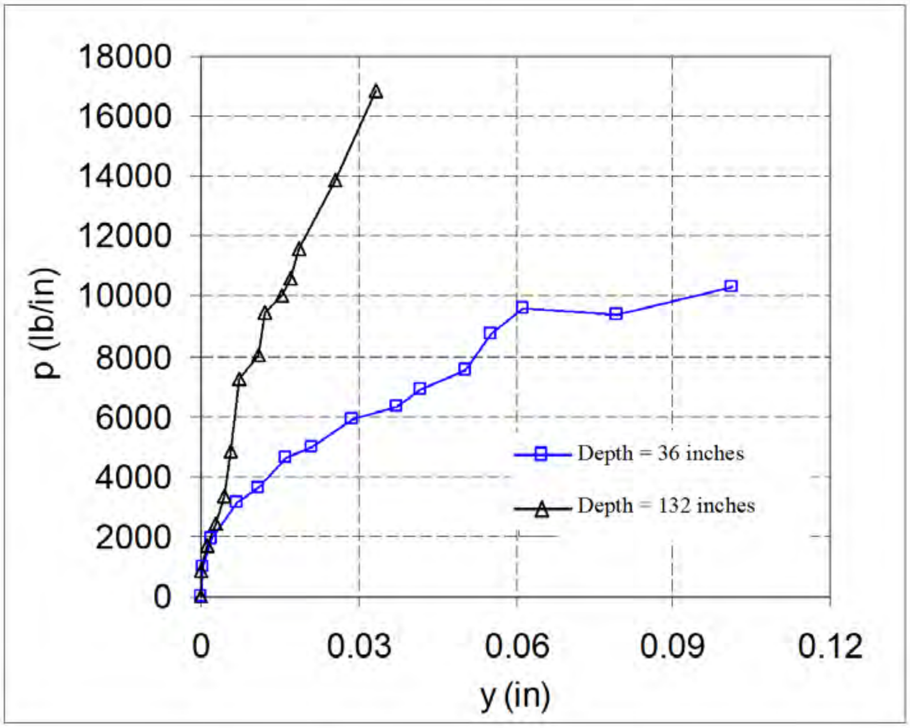


Figure C-8: P-y curves for drilled shaft #4 derived from load test.

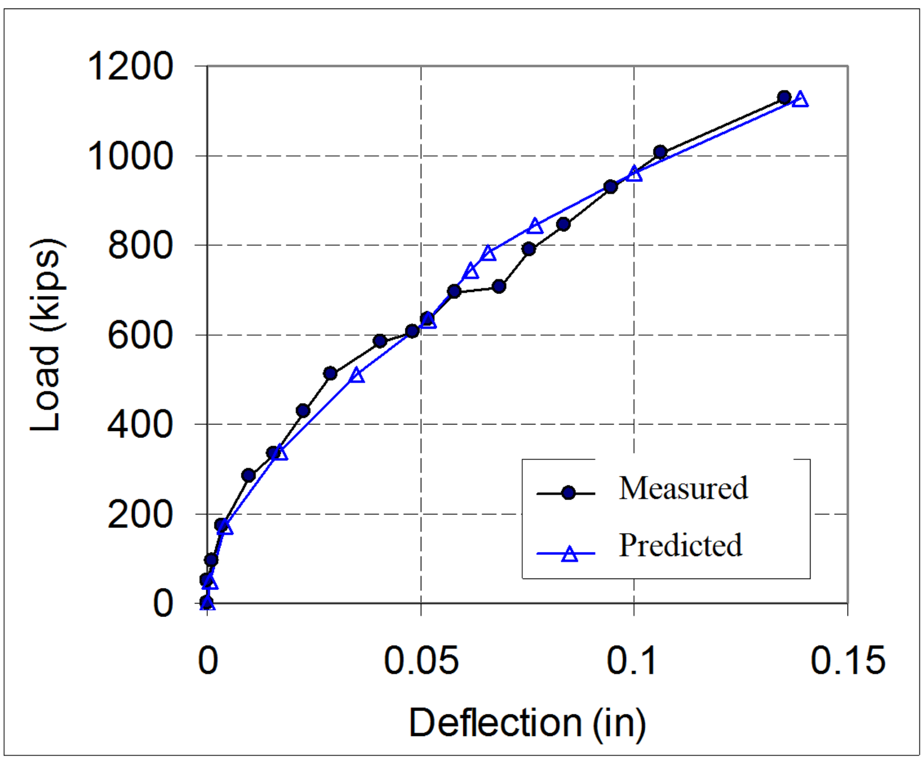


Figure C-9: Load-deflection response for drilled shaft #4 using experimental p-y curves.

APPENDIX D: GUIDE SPECIFICATION FOR LATERAL LOAD TESTS

This appendix provides a guide specification for the performance of lateral load testing of deep foundations. This guide specification covers the lateral load testing only. Other specifications will be needed for installation of the foundation elements and other testing, such as dynamic pile testing, integrity testing, axial load testing, proof testing or other types of tests. Refer to Hannigan et al. (2016), Brown et al. (2010), Brown et al. (2007), and Sabatini et al. (2005) for guide specifications for driven piles, drilled shafts, CFA piles, and micropiles, respectively.

This specification does not address measurement and payment terms as those will vary by owner, project, and/or contracting approach. Lateral load tests may be paid individually, included as part of an overall testing program (may include multiple lateral load tests or axial load tests as well), or may be included in the Lump Sum price of a Design-Build contract. Except in a Design-Build contract, lateral load tests would typically be paid on a unit price basis per test. The construction of the foundation test elements is typically paid for a separate pay item, per each, since the effort and time required to install these individual elements is usually much greater than the installation of the same elements in a production operation. Alternatively, the cost for the foundation test element, including reaction system, can be included in the price for the lateral load test.

Lateral load testing of deep foundations should be performed in accordance with ASTM D3966, Standard Test Method for Deep Foundations under Lateral Load. The specification below is intended to be a guide and is not intended to be used directly for project applications. Commentary in the specification below is provided in italics.

1.0 DESCRIPTION

This work shall consist of furnishing all design, materials, labor, tools, equipment, services, and incidentals necessary to perform the testing of piles under lateral loads in accordance with the Contract Documents and this Specification.

(Commentary: It is assumed that there is a specification for the installation of the foundation elements; accordingly, pile construction is not addressed herein. Similarly, there may be a separate test specification for axial load testing that should be coordinated with this section.

The following requirements relate to the lateral load tests but should be included in the foundation construction specification:

- *All lateral load testing should be completed and the testing results evaluated by the Engineer prior to installation of production foundation elements, unless authorized by the Engineer.*
- *The same installation procedures should be used for production foundation elements as for the test foundation elements. Any changes in the procedures must be evaluated by the Engineer for potential impacts on lateral load capacity. The Engineer may require additional lateral load tests to evaluate the impact of changes to installation procedures, at no cost to the owner.*

The scope of work could also include subsurface investigation borings at the load test sites if design-phase borings are not available at these locations. Site-specific borings are needed to interpret the load test results and to correlate lateral resistance to the soil/rock conditions at the test site.

Terminology:

1. *The word "pile" is used here and throughout, but the specification should reference the actual foundation type used (driven pile, drilled shafts, micropiles, or CFA piles, as applicable).*
2. *The term "Contract Documents" is used throughout. This term is intended to cover the requirements of the contract such as plans, specifications, special provisions, technical provisions, etc. This term may be replaced by "Plans and Specifications," "Contract Drawings and Specifications," or similar terms as appropriate.*
3. *The term "Engineer" is used throughout. This term may be adjusted depending on the contract type. For a design-bid-build contract, the Engineer is typically the Construction Manager and, indirectly, the Designer. For a design-build contract, the Engineer is typically the Engineer of Record.)*

1.1 Related Specifications

(Commentary: Engineer to indicate related specification sections here. At a minimum, this will likely include specifications that govern the installation of the foundation elements (driven pile, drilled shaft, CFA, or micropile specification section), the ASTM standard for lateral load tests, and the design specifications for the project. Additional sections should be included as appropriate.)

1.1.1 Specification Sections:

(Insert reference to foundation construction specifications and axial load testing, if required))

1.1.2 Reference Standards:

ASTM D3966 Standard Test Method for Deep Foundations Under Lateral Load

AASHTO LRFD Bridge Design Specifications, Seventh Edition, U.S. Customary Units, 2014.

(Commentary: Other standards, such as the AASHTO Bridge Construction Specifications, DOT Specifications, and reference documents can be added here as applicable.)

1.2 Submittals

1.2.1 Qualifications

At least 45 days prior to the start of testing, the load testing contractor shall submit proof of qualifications and experience. The load testing Contractor shall be experienced in the performance of lateral load testing of piles and shall have successfully performed at least 5 lateral load tests in the last 5 years of similar test type as those required in this Contract. Submit a project reference list including a project name, location, description of number and type of test piles, maximum test load, test dates, and Owner or client's contact name and phone number. Submit an example lateral load test report from one of the projects.

Submit a personnel list that identifies the load test designer (if applicable) and the supervising engineer assigned to the project. Include a summary of each individual's experience sufficient for the Engineer to determine the required qualifications are met. Both individuals should be licensed engineers in the state in which the project is located.

(Commentary: It is generally required that the lateral load test program and test report be signed by a registered professional engineer in the state where the project is located.

General provisions of the contract should cover aspects such as the time for the Engineer to review and return comments or approval on submittals. If not, add to this section that the Engineer will review the submittals within a specified period; 10 working days or two calendar weeks is suggested.)

1.2.2 Lateral Load Testing Plan

At least 30 days prior to the start of testing, submit a Lateral Load Testing Plan that includes detailed step-by-step description of the planned test pile construction, instrumentation, load testing and monitoring procedure including personnel, testing and equipment to assure quality control. Provide sufficient detail to fully describe all elements of the test program, to the satisfaction of the Engineer. Include detailed plans for lateral load tests including shop drawings, details, structural computations and apparatus to measure test loads as well as pile top movement and displacements and strains with depth. Include the required instrumentation and monitoring described in this Specification and shown in the Contract Plans. The Lateral Load Testing Plan shall also comply with the requirements of ASTM D 3966. Include calibration reports for each test jack, pressure gauge, master gauge and related calibrated monitoring equipment dated within 60 calendar days of submission. Include proposed start date, order of materials, and schedule for installation.

(Commentary: The Lateral Load Testing Plan provides an opportunity to review the proposed test set-up, procedures, and instrumentation for data collection. The details of the plan will vary based on the reasons for the testing; i.e., a test during design for developing site-specific geologic models for p-y curve development will have more instrumentation and may be tested under more loads or deflections than a proof or verification test performed in construction to verify the lateral resistance of the foundation element.

Some tests may be monitored by the Engineer, others may be monitored by a consultant working for the Contractor, such as a testing firm or an instrumentation and monitoring specialist.)

1.2.3 Test Pile Construction Inspection Records

Submit construction inspection records for the test pile(s). Test pile construction inspection records shall contain all required construction data as required in Section XXXX Driven Piles (or Drilled Shafts, Micropiles, or Continuous Flight Auger Piles). In addition, test pile construction logs shall include details regarding the installation of instrumentation, including locations within the test pile. Any deviations in the construction of the test pile from the Lateral Load Testing Plan shall be noted and the reasons for the deviations provided. The test pile inspection records shall be submitted within 24 hours of completion of the test pile and prior to the start of the lateral load test.

(Commentary: The as-built condition of the test pile should be documented and submitted. It is assumed that the installation specification for the foundation elements will cover minimum reporting criteria for as-built conditions. All required testing, including integrity testing, should be performed and documented as part of the inspection; it is assumed that this is covered under the foundation construction specification.

This section is based on the assumption that the quality control inspection is performed by the Contractor. Some projects will have the quality control inspection performed by the Engineer, Owner, or by a third-party inspection firm working directly for the owner. If the inspection is not performed by the contractor, other arrangements should be made to ensure that the construction of the test piles, including instrumentation installation, is properly documented and provide to the Engineer.)

1.2.4 Lateral Load Testing Report

Submit a Lateral Load Testing Report within 10 days following completion of the lateral load test. The report shall include details regarding the test pile construction including the test pile installation logs, final tip elevation, pile length, description of unusual installation conditions, and testing results (PDA testing for driven piles, CSL testing for drilled shafts, or similar testing as appropriate.) The Lateral Load Test Report shall also include details of the test including a description of the test set-up, any changes or deviations from the Lateral Load Test Plan and the reasons for such changes, a description of the testing conditions including weather conditions and any nearby activities that may have influenced the test data, records of the test data collected, plots of the test data including plots of the deformation versus load increments, and a discussion of whether the test objectives were met and any relevant observations from the test.

(Commentary: If the owner's Engineer is performing the inspection and data collection of the test, this provision can be deleted from the construction specification.)

1.3 Pre-construction meeting

The Contractor shall schedule a pre-construction meeting prior to the start of test pile construction to be attended by the owner's representative, Engineer, Contractor, and specialty contractor(s), testing firm, and instrumentation specialist, as applicable. The purpose of the meeting will be to review and clarify planned test pile construction procedures, coordinate the construction schedule and activities, delineate responsibilities, and review reporting requirements. At a minimum, the items of review will include excavation at test locations, anticipated subsurface conditions, test pile installation and testing, survey control, geotechnical instrumentation and monitoring requirements, data reduction and analysis procedures, reporting requirements, and schedule of submittals.

(Commentary: This pre-construction meeting can be combined with other pre-construction meetings, such as for installation of foundation elements, as appropriate.)

2.0 MATERIALS

Not Applicable

(Commentary: The materials for foundation construction should be covered under the relevant specification for those foundation elements. The materials for the instrumentation should be covered in the Lateral Load Test Installation Plan. There may also be an instrumentation specification section that may cover minimum requirements for instrumentation.)

3.0 EXECUTION.

3.1 Test piles shall be installed at the locations shown in the Contract Plans unless approved by the Engineer. Test Piles shall be constructed in accordance with the requirements in the Contract Documents and the approved Lateral Load Test Installation Plan. Perform all required inspection and testing and submit applicable records, as specified herein, prior to initiating the lateral load testing.

3.2 The load testing apparatus shall be installed in accordance with the Contract Documents and the approved Lateral Load Testing Plan

3.3 Lateral load test shall be performed in accordance with ASTM D3699 except as modified herein. Load tests shall be performed prior to the start of production pile driving (or construction of production drilled shafts, micropiles, of CFA piles). No piles shall be driven within 200 feet of a pile undergoing lateral load testing.

(Commentary: It is possible that there may be different areas of a project site under construction at the same time. The operations from one area should not disturb testing at another area, hence the restriction on pile driving noted above. Other foundation types, such as drilled shafts, do not produce the same amount of ground disturbance due to vibration. However, the same concept would be applicable, namely that no production elements or other operations that involve ground disturbance, such as installing casing by driving or vibratory hammer, or excavation blasting, be performed within a certain distance or at the same time as the lateral load testing.)

3.4 Selected test piles shall be load-tested in conformance with the applicable provisions of ASTM D 3966 including Standard Loading Procedure (Procedure A) and Standard Measuring Procedure, or as otherwise directed by the Engineer.

(Commentary: Although Procedure A is typically used, this needs to be assessed for each project.)

3.5 The Contractor shall install reference beam(s) and dial gauge(s) to measure lateral movement. Dial gauges shall have a precision of one thousandth (0.001) of an inch and shall provide for a travel of three (3) inches.

3.6 Supports for the reference beam(s) shall be firmly embedded in the ground and at a clear distance of at least seven (7) feet from the test pile. Utilize a transit or other surveying equipment to determine if either the pile or reference beam move during the test.

(Commentary: The supports for the reference beams must be a minimum distance away from the test pile to not be influenced by the movement of the ground during the lateral loading. The distance indicated above is based on judgment and should be adjusted as appropriate based on project specific conditions and foundation design.)

3.7 Structures for the load test shall be constructed in accordance with Apparatus for Applying Loads and Apparatus for Measuring Movements of ASTM D 3966 and approved Lateral Load Testing Plan. Test arrangement shall be designed so that loads are applied to the pile accurately and without eccentricity by means of certified, electric load cell, calibrated hydraulic jacks, pressure regulating devices and tanks, and hand jacks, so that there will be a constant load maintained under increasing movement. The load cell shall be calibrated to an accuracy of not less than one (1) percent of the total test load and shall be equipped with an approved swivel plate. Design structures and test apparatus to safely carry the test loads.

3.8 Protect reference beams, test piles, load cell, swivel plate, jacks, compensators and pressure tanks from the sun. In addition, provide a portable shelter to shield the survey instruments from the sun, wind, and precipitation, and provide necessary lighting and suitable heat to conduct the load testing during the hours of darkness.

3.9 The total test load to each test pile shall be as indicated in the Contract Plans. Loading and unloading sequence will be established by the Engineer. The Contractor shall furnish all labor to regulate the loads.

3.10 The Contractor shall take all optical survey and dial gauge readings of pile movement.

(Commentary: If the Engineer or an independent inspection firm working for the owner performs the inspection, then this section should be modified to indicate that the Engineer (or inspection firm) will take all optical survey and dial gauge readings, and that the Contractor shall assist the Engineer (or inspection firm) in taking such readings.)

3.11 The jack shall be positioned at the beginning of the test such that the unloading and repositioning of the jack during the test will not be required.

3.12 The testing of the piles shall be performed under lateral load increments per Table XXXX– 1:

Table XXXX – 1
Standard Loading per ASTM D3966 Procedure A

Load Increment	% of Design Load	Hold Time (Minutes)
1	0	--
2	25	10
3	50	10
4	75	15
5	100	20
6	125	20
7	150	20
8	170	20
9	180	20
10	190	20
11	200	60
12	150	10
13	100	10
14	50	10
15	0	--

(Alternate Table XXXX-1)

Table XXXX – 1
(Proof test and test to failure)

Load Increment	Percentage of Design Load (%)	Load (kips)	Hold Time (Minutes)
1	0	(To be calculated based on maximum test load)	--
2	25		10
3	50		10
4	75		15
5	100		20
6	125		20
7	150		20
8	170		20
9	180		20
10	190		20
11	200		60
12	150		10
13	100		10
14	50		10
15	0		10
16	50		10
17	100		10
18	150		10
19	200		10
20	210		15
21	220		15
22	230		15
23	240		15
24	250		15
25	Etc. to a maximum of 1" of movement		30
26	75% of maximum		10
27	50% of maximum		10
28	25% of maximum		10
29	0		--

(Commentary: Two sample load increments tables are provided above; it is only intended to include one in the test specification.

The first table includes the loading sequence per ASTM D3966 Procedure A as indicated. This is a design verification test sequence that will test the pile up to 200 percent of the design load. The second table includes the proof test sequence followed by a sequence to test the pile to a maximum of 1" deflection. The intent in the second procedure is to test the pile until the maximum deflection is met rather than simply 200 percent of the design load. The maximum deflection can be modified for the test. Other load sequences are presented in the ASTM specification and the table above can be modified and load sequences added, as appropriate to satisfy the test objectives.

The specification can present a table of the load sequence, as indicated above, or can simply refer to the designated load test sequence in the ASTM specification.)

3.13 Readings of all instrumentation shall be performed for each load increment. Record readings of time, load, and movement immediately before and after the application or removal of each load increment. Additional readings shall be performed at 5 minute and at 15 minute intervals while the test load is applied and at 15 and 30 minute intervals after the test load is removed.

(Commentary: The frequency of readings can be adjusted to fit the test objectives. Some readings, such as gauge readings on the displacement gauges, are easy to take, and can be done electronically with LVDT gauges and data recorders. Others, such as inclinometer readings, are time consuming and would only be expected to be performed once per load increment. Frequent readings during load increments, especially high load increments, will help detect creep or the onset of failure.)

3.14 If pile failure occurs before one inch of movement, a reading shall be taken immediately before removing the first load increment.

3.15 Following completion of the lateral load test and recording of all data readings, the Contractor shall perform site restoration at the test location in accordance with the requirements of the Contract Documents.

APPENDIX E: LITERATURE REVIEW

This appendix contains a literature review report that was prepared in October 2015 as an initial step in the development of this manual. The intent of the literature review report was to conduct a search of available literature regarding the design of laterally loaded deep foundation systems for transportation structures, summarize the current state-of-the practice, and identify gaps in the literature or state of the practice. The literature review was also intended to assist with identifying how AASHTO LRFD procedures are addressed by different state DOTs as well as how international standards address the topic of laterally loaded deep foundations.

GEC No. 9: Design and Analysis of Laterally Loaded Deep Foundations

Literature Review

1. INTRODUCTION

The purpose of the GEC is to present guidelines for design procedures that can be applied to laterally loaded deep foundation elements based on best available state of the practice information relative to highway facility applications. These applications include single and groups of deep foundations for bridge loads, excavation support, landslide repairs, retaining structures, noise walls, sign and signal foundations, vessel impact mitigation, and seismic event resistance in both vertical and mixed alignment configurations (vertical and battered). Deep foundations include such elements as concrete, steel, and timber piles, micropiles, drilled shafts, and auger-cast piles.

2. SCOPE OF RESEARCH

This document presents the results of a literature review, prepared as part of the initial research for drafting GEC 009. The literature review is not intended to present an exhaustive summarization of all available information sources, but rather to provide a representative gauge for the existing state of the practice. As such, the research discussed herein concentrates on published manuals from various state departments of transportation in the U.S., some foreign countries, and other public or private organizations. These resources, taken together, provide a reasonable overview of the existing state of the practice.

Other sources of information that were sought included case histories, test program documentation, and research experience that would tend to validate one or more of the published design procedures. Individual test results were found to be available on websites and in published case histories, but generally without adequate documentation on a comparison of prediction methods to the actual field test results. As such, the literature review focused on published critiques comparing the accuracy and applicability of select design methodologies.

The literature review contained herein is divided into four main sections based on the following sources:

- State transportation departments, herein referred to as DOTs for simplicity
- Other US-based sources
- International standards
- Case history summary

Due to language and accessibility limitations, international practices were given less emphasis. Nevertheless, we have contacted engineers within our international organization in an attempt to assess worldwide practices as compared to the U.S.

3. BACKGROUND

For the purposes of this document the below terminology and classifications should be understood prior to reading the following summary:

Broms Method: Broms (1964a & b) for estimating lateral capacity and deflection based on the strength properties of the soil and the structural stiffness. Short piles and long piles are analyzed differently based on formulas employing the soil subgrade reaction modulus and pile/shaft properties.

Reese Method: FHWA-IP-84-11 for piles and drilled shafts (1984). This method is based on the p-y methodology and presents the “critical penetration” concept, more commonly referred to as critical depth, defined as the depth at which increasing pile length does not result in decreasing head deflection. See Figure E- 1.

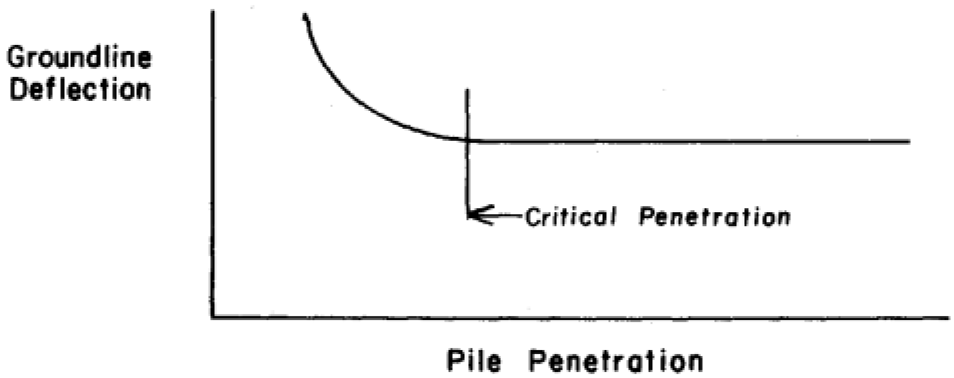


Figure E- 1: Influence of embedment depth versus head deflection (Reese 1984).

Deep Foundation Elements: Long, vertical, or near-vertical foundation elements with significant length to width ratio, generally consisting of piles, drilled shafts, auger cast piles, micropiles, or similar elements. For the purposes of this document, the terms drilled shaft and pile are referenced as appropriate according to the source material, but the concepts are considered universal, with a dependence on the diameter and relative stiffness of the element, regardless of the actual type of element or installation method.

P-y Curve Methodology: Represents soil-pile interaction as a series of non-linear one-dimensional springs.

Strain Wedge Model (SWM) Method: Uses conventional soil strength parameters to characterize the resistance of a three-dimensional passive soil passive wedge and develops p-y curves for analysis as a one-dimensional “beam on elastic foundation.”

4. STATE DOT RESEARCH METHOD

As part of the literature review, all 50 DOT websites were visited to find available documents related to design of bridges, noise barriers, luminaries, and other transportation related structures. Specific documents considered relevant consisted of bridge design manuals and geotechnical manuals. Each website was searched for these documents. A bridge design manual or guidance document was located for 35 states, but only 15 had a geotechnical manual or guidance regarding geotechnical design. Additionally, the websites were searched for keywords commonly associated with lateral design of deep foundations. This additional search resulted in 10 design recommendations or memoranda specific to the topic under consideration that have been issued by the DOT but not incorporated into specific design manuals as of the time of the research. The level of detail and guidance provided in the manuals and other documents varies. It is recognized that related documents from the various states may exist, but were not revealed by the method of investigation, or that some of the documents that were found may be obsolete, superseded or in the process of being replaced. Nonetheless, based on the quantity of information revealed online, we believe the basic intent to document the state of DOT practices has been met based on the data set investigated. Additional information may exist in offline DOT sources, but would not be expected to differ significantly from the information summarized herein. In addition, unpublished or offline sources may still be under review or revision and may not be final, and therefore may not reflect the current state of the practice.

Additionally, the results of a 2007 national survey of State DOTs were provided by the FHWA, including brief descriptions of the respective State's design approach for lateral analysis and displacement limits for deep foundations. Puerto Rico and Federal Lands were included in that survey, but were not included in the web-based research.

5. SUMMARY OF STATE DOT RESEARCH

Within the manuals and design memoranda/directives investigated the approaches to design vary considerably, from no guidance to detailed design procedures. The division of responsibility between the geotechnical engineer and structural engineer also varies considerably among the available manuals and published procedures. Some manuals limit the geotechnical engineer's responsibility to supplying p-y curves or soil parameters, whereas other manuals direct the geotechnical engineer to perform the majority of the design, with the structural engineer only responsible for verifying structural capacity. Deflection limits, when supplied, ranged considerably and varied between service and Strength Limit States. Given the foregoing variation in state DOT practices, differences are summarized in the following sections.

5.1 DESIGN METHODOLOGIES

Figure E- 2 depicts the design methodologies referenced within the DOT manuals. Note that some DOT's provide guidelines for or permit more than one methodology for various reasons including:

- Preliminary versus final design
- Projects with low versus high anticipated lateral loads
- Piles/shafts classified as short versus long
- Size and complexity of the project

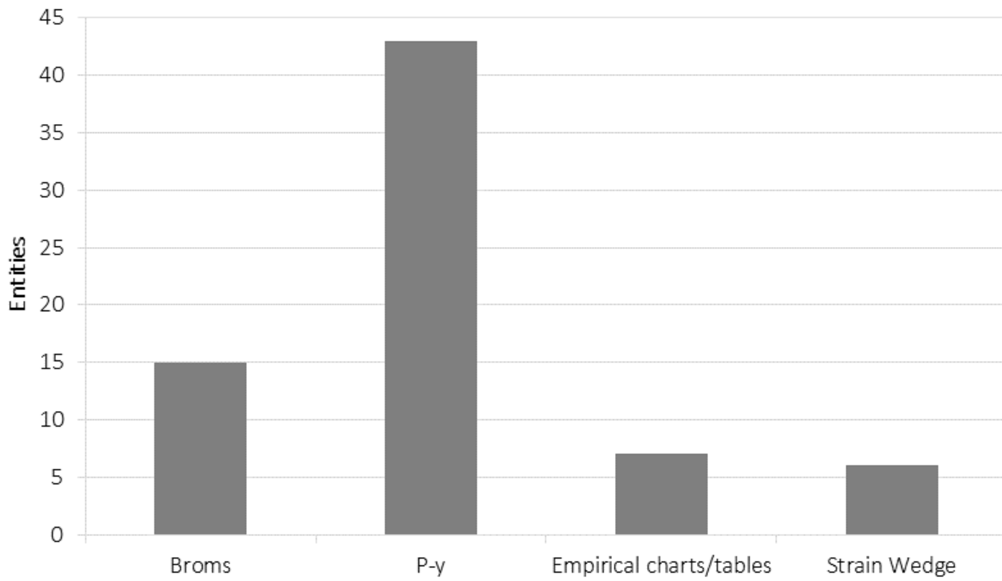


Figure E- 2: State DOTs referencing various design methodologies.

In the more thorough design manuals, Broms method is normally only used for preliminary design purposes. However, in some cases, such as projects with relatively low lateral loads, small scale, or low complexity it may be the only method employed. Other DOTs simply list the methods, inclusive of Broms method, but do not offer written guidance on which may be appropriate for various design stages or project complexity.

The most prevalent design methodology involves the use of p-y curves. It is listed in nearly all the manuals that include sections related to resistance to lateral loading. In the vast majority of manuals, there is no specific guidance beyond mentioning the analysis method. There is often no discussion regarding what defines the design length, head fixity condition, service or strength state, or deflection criteria.

A few of the manuals include empirical charts or tables based on general “rules of thumb” or analyses conducted by the DOT or a third party related to common pile/shaft sizes and loading conditions. The use of these items is limited to specific design cases and is indicated as such. No similarities could be ascertained based on a cursory review of the information obtained.

The strain wedge model is only specifically mentioned by six DOTs. It is recognized that this method is another version of Soil Structure Interaction (SSI), similar to the p-y method, such that the methods could be used interchangeably. However, because the strain wedge model is not as widely used as the p-y method, this document only includes cases where the specific method or software is mentioned. Deep Foundation System Analysis Program (DFSAP), a strain wedge program, is no longer recommended or supported by the Washington State DOT due to “lack of usage by the greater bridge community, other DOTs and consultants, compatibility with software upgrades, lack of agreement with AASHTO LRFD revisions and lack of conformity with WSDOT IT system and maintenance support” (WSDOT Memorandum dated March 26, 2014). Other software packages still exist and AASHTO LRFD 2014 does allow use of the strain wedge model for the Strength Limit State in the comments section. Additionally, Report S2-R19B-RW-1 by the Transportation Research Board includes mention of strain wedge model in the main text for consideration in future AASHTO LRFD revisions.

5.2 OTHER DESIGN TOPICS

Other concepts and the number of DOTs mentioning each topic are presented in Table E-1, followed by a brief discussion regarding each.

Table E-1: DOTs referencing various design topics regarding lateral loading.

Topic	Number of DOTs
Fixity Depth	16
Critical Depth/Reese (1984, 1985)	12
Group Multipliers	12
Head Fixity	4
Deflection Limits	22
Seismic	10
Design Procedure	6
Engineer Responsibilities	18
Resistance Factor	8

5.2.1 Fixity Depth

The concept and definition of fixity depth, or point of fixity, is often interpreted differently between the structural and geotechnical disciplines and/or between different geotechnical practitioners. The structural definition for preliminary analysis per by Davisson and Robinson (1965) is where the fixity depth of a partially unsupported foundation element represents the point at which the element can be considered a cantilevered beam, with no lateral support and a fixed end condition. The Davisson and Robinson method is referenced in AASHTO 10.7.3.13 for preliminary design, and tends to provide a quick means to compare foundation alternatives with respect to the requirements to resist bending. For more rigorous strength analysis, AASHTO 6.15.2 defines fixity based upon the second point of zero deflection at the factored load based on a P-Δ analysis. The geotechnical discipline typically uses a definition based on the lateral head deflection at a defined depth of embedment based on a soil structure interaction analysis, whereby fixity is defined as the first or second point of zero deflection or the point of maximum negative deflection, but is dependent on the load case being analyzed (strength/service/extreme). Two of the investigated documents added 5 feet to the calculated point. The depth, as defined by the geotechnical discipline infers the minimum design length to safely resist lateral loads relative to deflection and soil strength (push-over analysis). The quantity in Table E-1 makes no distinction between definitions, but simply presents the number of documents using the fixity depth terminology. Some variation of the geotechnical definitions described above is generally included in five of the documents, three of which use the second point of zero deflection. The other two instances reference the first point of zero deflection and the point of zero deflection, with no mention of first or second point. The load combinations employed for the analyses differ between the documents and do not employ strictly Strength or Service Limit States.

5.2.2 Critical Depth

Critical depth, as described in Section 3, can also be used to define the design length to resist lateral loads. The number of DOTs referencing critical depth in Table E-1 specifically refer to the term or to the FHWA's manuals by Reese (1984, 1985), which include critical depth as part of the design procedure.

5.2.3 Group Multipliers

Pile group modifiers, also referred to as P-multipliers (P_m), are commonly mentioned in the investigated documents. Due to the passive 3D wedge used in strain wedge model, these modifiers do not apply to the strain wedge model.

5.2.4 Head Fixity

Head fixity, as related to modeling procedure in SSI software, is not discussed in most referenced documents. When discussed some manuals recommend:

- Supplying both free and fixed conditions
- Supplying full head fixity and 50 percent head fixity
- Calculating cap embedment for head fixity

A literature review with regard to pile embedment in the pile cap and the degree of fixity is presented in the commentary to the Iowa LRFD Bridge Design Manual. A summary of that review is presented in Table E-2.

Table E-2: Summary of head fixity literature review.

Reference	Pile Size	Results for 12 in embedment into pile cap (1.2 * Diameter)	Notes
Castilla (1984)	HP 10x42	61 to 83% fixity	Based on computer modeling
Wasserman and Walker (1996)	HP 10x42	Strong axis, full plastic moment	
Hughes et al. (2007)	HP 10x42	50% fixity	
Rollins and Stenlund (2008)	HP 10x42	Weak axis, full moment capacity Strong axis, not full moment capacity	Based on testing
Rollins and Stenlund (2008)	HP 14x89	-	5 inches embedment, 25 to 66% ultimate moment capacity

5.2.5 Deflection Limits

Head deflection for deep foundation elements were listed by 22 DOTs ranging from 0.25 to 2 inches for the Service Limit State. A chart showing the distribution is included as Figure E-3. Note that if the Limit State was not specifically indicated, the Service Limit State was assumed, consistent with LRFD requirements for assessing deflection.

In a few cases, Strength and Extreme Limit States were defined as correlating with specific maximum values of deflection rather than ultimate geotechnical capacity. Five limit values were reported among the DOTs for use as a defining criteria for strength and extreme limits cases as follows:

- 1.0 inches (Strength)
- 1.5 inches (Extreme)
- 2 inches (Strength/Extreme for assessing rotation)
- 3 inches for prestressed concrete (Strength)
- 6 inches for steel (Strength)

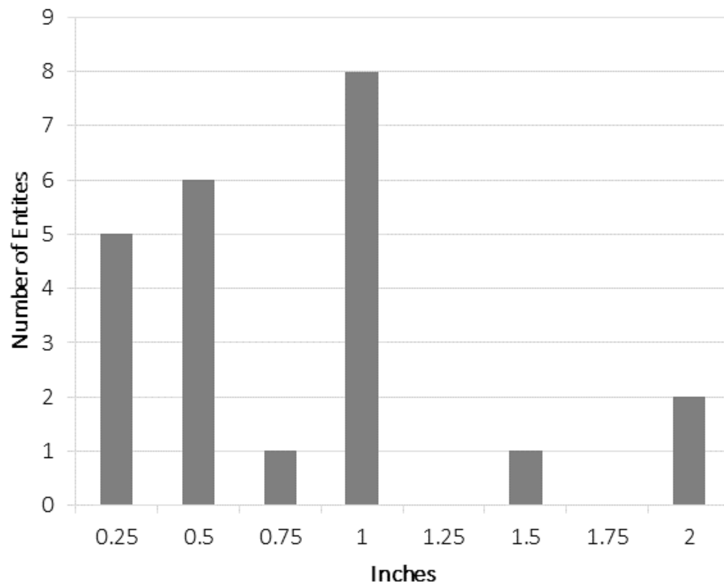


Figure E-3: Number of DOTs referencing various head deflection (service).

5.2.6 Seismic

Assessing lateral resistance related to seismic events is mentioned in ten documents. Some manuals simply mention the use of liquefied p-y curves, others recommend an iterative procedure to identify loads due to lateral spreading, and others offer opinions and discuss the various methods available.

5.2.7 Design Procedure

A staged design procedure is only detailed in six documents. These include; Arizona, California, Louisiana, Massachusetts, Pennsylvania, and Washington. The details of each procedure will not be discussed herein, as the steps are sufficiently different except for the inclusion of a deflection analysis with computer software.

5.2.8 Engineer Responsibilities

The respective responsibilities of the structural engineer and geotechnical engineer are discussed in 18 of the documents. The most common approach, in twelve of the 18 documents, is to have the geotechnical engineer supply soil parameters or p-y curves and loads associated with liquefaction (when appropriate) to the structural engineer. The structural engineer then performs the structural and lateral analysis. Some of these publications indicated that the results are to be reviewed by the geotechnical engineer for verification. Another, less common, published approach, in three of the 18 documents, has the geotechnical engineer develop the parameters or p-y curves and liquefaction loads and perform the lateral load analysis. In this case, the structural engineer performs only the structural design of the pile. In the remaining three documents, the p-y curves are stated to be provided by the geotechnical discipline, but there is no specific statement that the structural discipline is to perform the lateral analysis.

5.2.9 Resistance Factor

The use of a resistance factor (Φ) is only mentioned in eight of the state transportation agency documents, possibly due to codification of resistance factors within current AASHTO LRFD specifications. Even though AASHTO (2014) indicates that p-y curves are not factored since they already represent the ultimate condition (C10.7.3.12), it allows that p-y parameters should be reduced for extreme event seismic liquefaction (10.7.4). The use of resistance factors varies and includes defining specific resistance factors for specific cases, as is done by Idaho, or indicating that a resistance factor of 1.0 should be used for lateral analyses (consistent with AASHTO design specifications), or indicating that no resistance factor should be used. From an analysis standpoint, the use of a resistance factor of 1.0 is mathematically the same as not including a resistance factor because it does not change the resistance used in the analysis. The Idaho Transportation Department states two different values—1.0 for the Service and Extreme Limit States and 0.90 for the Strength Limit State. The use of resistance factors is further discussed in Section 0, regarding design methodology for drilled shafts from FHWA GEC 010 (2010).

6. OTHER US-BASED SOURCES

Included in the following section are basic summaries of various references which are considered important in the historic progression of lateral analysis for deep foundations. A full summary is not presented, but rather a brief, chronological summary of the salient concepts as introduced into the state of the practice.

6.1 BROMS (1964A, B)

As described in Section 6.5, the Broms method provides a means of estimating lateral capacity and deflection of deep foundation elements in homogeneous clay or sand with a level ground surface using the strength properties of the soil and the stiffness of the structural element. The linear load displacement method distinguishes between long and short piles using a ratio which employs the soil subgrade reaction modulus. Short piles are then analyzed for a failure mode commonly described as “fence-posting,” where the tip is not fixed and the entire element rotates. Long piles are assumed to have fixity at the tip and are assessed based on possible exceedance of the maximum moment (development of plastic hinge).

6.2 DAVISSON AND ROBINSON (1965)

As stated previously, the main design concept presented by Davisson and Robinson (1965) is development of the point of fixity, as it relates to structural analysis using the equivalent cantilever method. This manual is only applicable to long piles as defined by the ratio presented by Broms (1964). It is referenced in AASHTO LRFD C10.7.3.13.4 (2014) as the recommended method for determining the depth to fixity for preliminary design. The limits for differentiating between long and short piles are not included in AASHTO.

6.3 REESE (1984, 1985)

Documents FHWA-IP-84-11: “Handbook on Design of Piles and Drilled Shafts Under Lateral Load” (Reese, 1984) and FHWA-IP-85-106: “Behavior of Piles and Pile Groups under Lateral Load” (Reese 1985) present Broms method, but also recommend a method of using p-y curves to design laterally loaded piles. The method determines failure loads by iteration of increasing loads until the maximum bending moment is exceeded. At this point the allowable service loads are established by dividing the input loads by the desired safety factor. The allowable service loads are then input and deflection assessed. The concept of critical penetration is introduced, which has been adopted by some DOTs and is commonly called critical depth, as described in Section 3. The documents recommend significant interaction between the structural and geotechnical engineers during the design process, but do not define specific roles and responsibilities.

6.4 COM624P MANUAL (1993)

The COM624P manual, authored by Wang and Reese, focuses on the usage of the referenced software which is based on the p-y methodology. It offers no specific design guidance but, similar to Reese (1984, 1985), discusses the concept of critical penetration. The reference notes that there is no discernable difference between analyses where more than two points of zero deflection are present. It states that to save computation time the pile can be shortened so that there are two or three points of zero deflection.

6.5 AMERICAN PETROLEUM INSTITUTE (API) (2000)

API (2000) presents methods for determining the ultimate lateral bearing capacity for soft clay, stiff clay, and sand and for generating p-y curves for the same soil types. It additionally recommends that due to API-sponsored studies that multiple methods should be employed to assess group effects of lateral loads and that upperbound and lowerbound soil parameters be used.

6.6 FHWA NHI-10-016: DRILLED SHAFTS: CONSTRUCTION PROCEDURES AND LRFD DESIGN METHODS (2010)

Presents a detailed design process which includes:

Geotechnical Strength Limit State (“pushover analysis”): uses p-y method to analyze linear elastic shaft with factored loads and a factored resistance to define the Limit State based on a head deflection that is less than 10 percent of the shaft diameter.

Structural Strength Limit State: uses p-y method to confirm that nominal axial, shear, and flexural resistance exceed the factored axial, shear, and bending moments using the non-linear flexural stiffness (“cracked section”) of the shaft.

Service Limit State: uses the p-y method and service loads to confirm that deflection is acceptable using the non-linear flexural stiffness of the shaft (cracked section).

For the first stage listed above, resistance factors (Φ) are presented for various design conditions ranging from 0.40 to 0.80. The resistance factors are not applied directly to soil strengths, but instead as an inverse (1/Φ) applied to the factored overturning forces. The use of resistance factors for lateral analysis is more conservative than the AASHTO design specifications.

6.7 FHWA-NHI-11-032: LRFD SEISMIC ANALYSIS AND DESIGN OF TRANSPORTATION GEOTECHNICAL FEATURES AND STRUCTURAL FOUNDATIONS (2011)

The manual presents the state of the practice for seismic analysis and design at the issue date in 2011. It recognizes that the there is no clear consensus on some subjects, such as the p-y curve selection for liquefied soils, and that the evaluation methods and estimated magnitudes are uncertain. Therefore, design professionals should be conscience of ongoing research.

The suggested procedure for analysis of flow failures and lateral spreading or lateral flow are summarized below:

1. Slope stability analyses are performed for the liquefied condition, using the residual shear strength parameters for the affected layers, to determine the post-liquefaction yield acceleration and the associated failure surface.
2. Newmark sliding block analysis is conducted using the post-liquefaction yield acceleration from Step 1 to estimate displacements of the soil-pile system.
3. Based on Step 2, the forces on the structure and foundation due to lateral spreading or lateral flow movements are calculated.
4. Determination of plastic hinge mechanisms that are likely to develop in the deep foundation elements. This can be assessing moment and shear induced in the foundation using a soil-pile interaction analysis.

The reference describes two main methodologies for p-y curve representation of liquefied soils; 1) Soft cohesive soil using the undrained residual shear strength as the cohesion, 2) Liquefied sand curves by Liu and Dobry (1995).

6.8 LPILE TECHNICAL MANUAL (2012)

The manual interprets load and resistance factors by using lowerbound and upperbound values for important soil parameters. It employs a procedure similar to that presented in Reese (1984, 1985), which iterates analyses until failure moment/shear occur and then uses a safety factor to determine working loads. The element is then analyzed for behavior under working stresses for acceptability. The concept of critical length is included, stating that "the designer will normally select a pile for a particular application whose length is somewhat greater than L_{crit}." A chapter for using vertical piles to stabilize slopes is presented, but no specific guidance is given on the applied load above the failure surface.

6.9 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS (2014)

Although no new concepts are introduced in the manual compared to other publications, a brief summary of the highlights related to lateral deep foundation design are included in this section. In Figure C-6.15.2.1 "fixity" relative to flexural strength of the foundation element is defined by a graphic as the second point of zero deflection from a P-Δ analysis, above which a resistance factor of 1.0 is recommended for flexure and 0.70 for axial for steel piles. Only structural analysis relative to axial loading is needed for piles exceeding this depth.

In section C.10.7.3.13.4 the formulas for calculating the depth to fixity for preliminary structural design are included, which are referenced from Davisson and Robinson (1965). The estimation of nominal lateral resistance is discussed in section 10.7.3.12 which recommends using p-y curves, group effects accounted for using P-multipliers, and a resistance factor of 1.0. Resistance of the pile cap is allowed if it is to be embedded. Section 10.7.2.4 allows the use of strain wedge model for large diameter, relatively short piles or shafts, but notes that P-multipliers are not applicable due to the overlap of wedge zones. Minimum penetration to obtain fixity is relative to the applied lateral loads at the Strength Limit State as indicated in section 10.7.6.

The majority of State DOT practice is consistent with the guidance presented in AASHTO LRFD (2014), with the DOTs occasionally providing additional guidance. In some cases, that DOT guidance is more conservative, such as using resistance factors, and in other cases the DOT guidance is simply different, such as using Broms method for final design or employing critical depth instead of fixity depth. A few of the other recurring differences include:

- Defining the head deflection limits
- Different definition of the fixity depth
- Recommendations regarding head fixity
- Defining geotechnical and structural discipline responsibilities

7. INTERNATIONAL RESEARCH METHOD

As stated in Section 2, the literature review focused less attention on international source documents due to language differences and lack of access to foreign documents. The methods employed included an internet search for readily available documents in English, along with contacting experts possessing experience with international codes.

8. SUMMARY OF INTERNATIONAL PRACTICE

8.1 EUROCODE AND UNITED KINGDOM

The Euronorm (EN) Eurocodes (EC) are a suite of design codes that were introduced across European member states using a Limit State design (LSD) framework. Each member state (country) is permitted to publish its own National Annexes (NA). This enables the member state to define the specific values of partial factors applied to loads and resistances and define which of several Design Approaches (DA) are to be used, each of which has different load combinations. Partial factors may be thought of as analogous to the load and resistance factors in the AASHTO LRFD approach, albeit the specific values are different. The Eurocode sections include:

- EN 1990: (Eurocode 0) Basis of structural design
- EN 1991: (Eurocode 1) Actions on structures
- EN 1992: (Eurocode 2) Design of concrete structures
- EN 1993: (Eurocode 3) Design of steel structures
- EN 1994: (Eurocode 4) Design of composite steel and concrete structures
- EN 1995: (Eurocode 5) Design of timber structures
- EN 1996: (Eurocode 6) Design of masonry structures
- EN 1997: (Eurocode 7) Geotechnical design
- EN 1998: (Eurocode 8) Design of structures for earthquake resistance

- EN 1999: (Eurocode 9) Design of aluminum structures

In the UK, the British Standards Institution (BSI) publishes the Eurocodes, and EC7 is referred to as BS EN1997-1 and BS EN 1997-2, for parts 1 and 2 respectively. The UK's national annex was included within the 2013 version of the code, and is titled BS EN 1997-1: 2004+A1:2013.

For ultimate limits states, there are three design approaches with combinations of partial factors on actions (i.e., load factors) and partial factors on resistances (i.e., resistance factors) presented; the most applicable design approach is to be determined for the design case being assessed. All factors are 1.0 or greater, however, actions are multiplied by the factor whereas soil parameters and geotechnical resistance are divided by the factors.

Lateral loading of piled foundations is addressed in Section 7.7 of EC7 under the title "Transversely Loaded Piles." There is a requirement to demonstrate that a pile will support the design transverse load with adequate safety against failure, such that the following inequality is satisfied for all Ultimate Limit State (ULS) load cases and load combinations:

$$F_{\mathrm{tr;d}} \leq R_{\mathrm{tr;d}}$$

Where:

$F_{tr;d}$ = Factored transverse design loading.

$R_{tr;d}$ = Factored transverse resistance for design.

Failure mechanisms that are required to be considered include rotation for short piles acting as a rigid body and bending failure or local yielding/displacement of the soil near the top of the pile for long slender piles. Group effects are to be considered when assessing the resistance of transversely loaded piles, including effects of compression, tension and transverse forces in individual piles within the group.

The partial factors are included in the Annex for certain design cases or applications, but the case of transversely loaded piles is not specifically addressed. The standard indicates that if factors are not addressed in the Annex, then the factors in the Annex should be used as a guide. There are multiple design approaches, each with a different combination of partial factors, and therefore it appears that for the case of transversely loaded piles, the designer has some flexibility with regard to which approach and therefore which load factors are used as a guide from the Annex. The specific approach may be governed by local practice or the type of structure.

In general, the design approaches and factors include:

1. Design Approach 1: Two combinations:
    a. Combination 1: Load factors of 1.35 and 1.5 for permanent and variable loads (all loads factored up), partial factors for geotechnical parameters and resistances are 1.0.
    b. Combination 2: Load factors of 1.0 and 1.3 for permanent and variable loads, partial factors for geotechnical strength parameters are 1.25 to 1.40 (i.e., soil strengths are reduced), and factors for geotechnical resistance are 1.0.
        i. For axially loaded pile design:
            - Combination 1: same as above

- Combination 2: Load factors of 1.0 and 1.3 for permanent and variable loads, partial factors for geotechnical parameters are >1 only if the soil applies an unfavorable action, and factors for geotechnical resistance are 1.3 (resistance is reduced).

2. Design Approach 2: Load factors of 1.35 and 1.5 for permanent and variable loads (all loads factored up), partial factors for geotechnical parameters are 1.0, and partial factors for resistances are 1.1 (resistances are reduced).
3. Design Approach 3: Load factors of 1.3.5 to 1.5 for structural actions and 1.0 to 1.3 for geotechnical actions, partial factors for geotechnical strength parameters are 1.25 to 1.40 (i.e., soil strengths are reduced), and factors for geotechnical resistance are 1.0.

The British Standard indicates that for the Service Limit State (SLS) case, the partial factors on actions and resistances are normally set equal to 1.0. The standard indicates that limiting values for deformation should be set during the design, but it does not provide actual values for bridge foundation deflections.

The determination of transverse displacements is required to take account of non-linear ground stiffness and its variation with strain level, flexural stiffness of the individual piles, pile head fixity condition with the structure, the pile group effect, and effects of load reversals or cyclic loading. The specific method of lateral analysis is not detailed but references to the subgrade reaction model and p-y methodology are mentioned.

Based on this review and the information summarized above, the British Standard follows a similar approach to the AASHTO bridge specifications including load and resistance factors and indicates the same general design procedures as US-based publications, i.e., subgrade reaction model and p-y curves. Strain wedge model is notably not mentioned in the British Standard.

8.2 AUSTRALIA

Foundation design guidelines and codes for Australia were obtained including the following:

- Austroads, Guide to Bridge Technology, Part 4: Design Procurement and Concept Design, published by Austroads Incorporated, Level 9, Robell House, 287 Elizabeth Street, Sydney NSW 2000 Australia.
- Australian Standard, Piling – Design and installation, AS 2159-2009, reissued incorporating Amendment No. 1 (October 2010), published by Standards Australia, GPO Box 476, Sydney, NSW 2001, Australia.
- Australian Standard, Bridge Design, Part 3: Foundations and soil-supporting structures, AS 5100.3-2004, AP-G15.3/04, published by Standards Australia, GPO Box 476, Sydney, NSW 2001, Australia, and
- AS 5100.3 Supplement 1-2008, Bridge Design-Foundations and soil supporting structures – Commentary, AS 5100.3 Supp 1-2008, AP-G15.3C/08, published by Standards Australia, GPO Box 476, Sydney, NSW 2001, Australia.

The Austroads Guide to Bridge Technology presents general design guidelines but no details regarding lateral pile analysis or design. The Standards, AS 2159 and AS 5100, provide the most detailed guidance for civil engineering works and bridges, respectively. These are summarized below, however, the detail regarding lateral pile analysis is limited.

From AS 5100 for bridge design, foundations must be designed for the ultimate geotechnical strength state and for serviceability state, similar to AASHTO requirements. For the ultimate geotechnical strength state, the maximum load factor combination should be used, geotechnical material strengths are unfactored, but the overall geotechnical resistance is factored. The design equation to be satisfied for design of foundations is:

$$\phi_g R_{ug} > S^*$$

Where:

S^* = Design action loads.

R_{ug} = Ultimate geotechnical strength.

ϕ_g = Geotechnical strength reduction factor.

The design action loads, S*, are to developed based on the combination of factored loads that produces the most adverse effect on the foundation in accordance with Australian Standard AS 5100.2.

The value of ϕ_g varies from 0.4 to 0.9 for design of piles depending on the amount of site investigation, the complexity of the calculations, the degree of construction testing, causes of failure, cyclic loading, and use of general or site-specific correlations. Without load testing, the values are in the range of 0.4 to 0.65; with load testing, values are in the range of 0.5 to 0.9 depending on the type of testing and whether the testing is carried to failure or not. The focus of the discussion on piles is based on axial load tests; there is no mention of lateral load tests or any differences in the design procedures for lateral pile design.

There is no indication or guidance regarding specific procedures for lateral piles analysis or allowable lateral deflections. The standard indicates that for cases or conditions not specifically addressed for the geotechnical strength reduction factor, to use the published factors and conditions within the standard as a guide.

For serviceability design, the standard states that foundations are to be designed by controlling or limiting settlement, horizontal displacement, and cracking. Deflections and horizontal displacements should be limited to ensure that the foundations and structure remain serviceable and that allowable displacements should be established and should consider the tolerance of the structure to deformation.

The serviceability state does not use a geotechnical strength reduction factor. Load factors for serviceability state are 1.0 to provide an accurate estimation of movements.

AS 2159 presents minimum requirements for design of piles for civil engineering and building structures on land. It states that AS 5100 series (summarized above) “should be considered” for design of foundations for bridges.

For the ultimate geotechnical strength, the geotechnical strength is factored similar to the AS 5100 Standard. However, there are additional steps in developing the geotechnical strength reduction factor, ϕ_g. These include Individual Risk Ratings (IRR) that serve to create a quantitative measure of the relative risk associated with the design, such as the level of investigation, the basis for the geotechnical strengths, the amount of construction testing, the level of construction control, etc. These IRR factors are then used in developing the value of ϕ_g, with lower ϕ_g values corresponding to higher relative risk. Basic geotechnical strength reduction factors range from 0.40 to 0.76, but can be increased if construction testing is used with higher values allowed for static testing compared to dynamic testing.

Lateral load tests are included in the discussion regarding pile testing, but only briefly. The majority of the discussion on testing pertains to axial testing of piles. The text indicates that for proof tests, including lateral load tests, the acceptance criteria has to be defined before the test, whereas for ultimate geotechnical strength tests, the test is conducted to geotechnical failure and no acceptance criteria is provided before the test.

For design of a pile subjected to lateral loads, the ultimate geotechnical design strength is determined as the lesser of:

- Short pile failure, which is the ultimate lateral resistance of the soil surrounding the pile fully mobilized along the entire pile length, or
- Long pile failure, in which the structural strength of the pile is fully mobilized before the ultimate soil resistance along the entire length of the pile.

For pile groups, the ultimate geotechnical strength, “in the absence of an alternative method”, is taken as the lesser of:

- The sum of the ultimate strength of individual piles, or
- The ultimate geotechnical strength of a block containing the piles and the soil between them.

For a piled foundation, in addition to the design of an individual pile, the geotechnical strength of the pile group must be analyzed for failure under the loading on the group.

Serviceability design is consistent with the AS 5100 standard. It indicates that the deflections for the serviceability state design should be limited to values appropriate for the intended design of the pile, but the methods for analyzing deflections and the actual allowable deflection values are not provided. No reduction factors are applied for serviceability Limit States.

The Standard indicates that for piles subjected to lateral ground movements, the bending moments, shear forces, and axial actions must be determined using an appropriate soil-structure interaction. Otherwise, there is no guidance with regard to the type of analyses to be performed, the use of p-y curves, etc.

8.3 HONG KONG

In a document prepared by the Geotechnical Engineering Office (2006) the following methods are presented:

- Brinch Hansen (1961) for short rigid piles
- Broms (1964) for fixed and free head piles in sand or clay
- Poulos (1985) for two-layer soil

As noted in the document, Kulhawy and Chen (1992) report Broms method tended to underestimate the ultimate lateral load by about 15 to 20 percent.

The document states the above design approaches are “simplified representations of pile behavior” useful for “obtaining a rough estimate of the likely capacity, and experience suggests that they are generally adequate for routine design.” It further suggests that if the design is likely to be governed by lateral load behavior, then load tests should be completed to verify design parameters.

A safety factor of 2.0 is allowed when lateral load tests are conducted and 3.0 when they are not. However, the design of a vertical pile to resist lateral loads is usually governed by limiting lateral deflection requirements.

Two references are mentioned when implementing the methods on sloping ground, Bhushan (1979) and Siu (1992).

The p-y method is mentioned in the document for assessment of non-linear response in layered soils, but no guidance or recommendations are presented.

8.4 CHINA

As reported in DFI (2012), the code in China is JGJ 94-4 and entitled “Technical Code for Building Foundations.” The summary contained herein is based on a description of the code presented by DFI (2012), and is assumed to be applicable to buildings, but not necessarily intended for transportation structures, based on the title. The actual Chinese code could not be obtained and reviewed for this study; the information presented herein is based on the summary in other sources (DFI 2012).

The code requires that for Class I buildings a static lateral load test is required to determine the allowable lateral pile capacity. Class I buildings are classified as “important residential and industrial structures.” The lateral capacity is taken as the load corresponding to a deflection of 10 mm (0.38 in) for precast concrete, steel, and bored piles with reinforcement ratios greater than 0.65 percent. For buildings sensitive to horizontal movement the limit is decreased to 6mm (0.24 in). It is not clear based on available information if the load testing is performed to failure or includes determination or verification of the ultimate lateral pile capacity; it is assumed that the tests are not carried to failure or ultimate capacity based on the indication that the testing is required for determining the allowable lateral pile capacity (DFI 2012), The safety factors used in conjunction with the load testing are not known based on available information at the time of this study.

When lateral load tests are not performed, for non-Class I buildings, lateral capacity is determined from empirical equations, which are not included in the referenced DFI document. There are separate formulas for elements controlled by deflection or structural capacity. Pile group effects are also accounted for in design.

8.5 INTERNATIONAL BUILDING CODE (IBC) (2012)

The subject of lateral loading for deep foundations is briefly covered in the IBC. A list of the specific topics mentioned is included below:

- Section 1810.2.1: buckling analysis is considered unnecessary for any soil, other than “fluid soil.”
- Section 1810.2.1: if the element is unsupported it can be considered “laterally supported at a point 5 feet into stiff soil or 10 feet into soft soil.”
- Section 1810.2.5: group effects should be included for lateral analysis
- Section 1810.3.3.2: allowable lateral load shall be from an “approved” method of analysis or from a lateral load test. From a lateral load test the allowable lateral load is to be less than or equal to 50 percent of the load that produces movement of 1 inch at the ground surface or top of pile.

8.6 ALP VERSION 19.1 (USER MANUAL)

ALP is a software package for the analysis of laterally loaded piles created and maintained by a British company. The program includes three analysis types: elastic-plastic, specified p-y curves, and generated p-y curves. The available p-y curves the program can generate include:

- Soft clay (Matlock 1970)
- Stiff clay (API 1989 and 2000)
- Sand (two choices: API 1989 [equivalent to Reese 1974] & API 2000)
- Weak rock (Reese 1997)
- Strong rock (Turner 2006)

9. SOFTWARE

Included in Table E-3 is a list of some of the current software for the assessment of lateral analysis of deep foundation elements.

Table E-3: List of lateral analysis software.

Software	Software Developer	Analysis Method	Single or Group Capable	Comments
ALP	Oasys Software	p-y	Single	Commercial
COM624P	FHWA	p-y	Single	Freeware, DOS platform
CGI-DFSAP	Computers and GeoEngineering, Inc.	Strain wedge model	Single	Commercial
DFSAP	Washington State Department of Transportation	Strain Wedge model	Single and Group	No longer available or supported
FB-MultiPier	Bridge Software Institute (BSI), University of Florida	p-y	Group	Commercial
GROUP	Ensoft, Inc.	p-y	Group	Commercial, group version of Lpile
Lpile	Ensoft, Inc.	p-y	Single	Commercial
SWM	Geopile, LLC	Strain wedge model	Single and Group	Commercial

10. CASE HISTORY SUMMARY

Two types of case histories are described in this section. The first includes published studies summarizing databases of available load test information and performing statistical comparisons of design methods with those load test results. These consist of studies where insights or conclusions are drawn from review and analysis of large data sets. The second type of case history discussed involves research projects performed by DOT agencies using full scale lateral load tests. These case histories were specifically performed to investigate or develop p-y parameters for lateral load analyses to improve the state of the local practice.

Additional case histories, are in many cases are either similar to the studies presented herein or are isolated case histories involving design development and/or validation on one particular project. Due to limitations in scope for this literature review, such individual case histories were not reviewed in detail or summarized here.

10.1 DATABASE SUMMARIES

Paikowsky (2007), Yang, et al. (2007), and the Washington State DOT (1988) published summarized databases related to the accuracy and effectiveness of design methodologies as briefly summarized below.

Paikowsky (2007) analyzed over 100 case histories comparing the accuracy of deflection estimates for various methodologies including p-y curves, Broms method, and SWM for open and closed-end pipe piles. The document states that there is good agreement between measured and calculated deflections for both the p-y and SWM methods, with measured values about 10 percent greater than predicted values for most cases, and a coefficient of variation of about 35 percent. Broms method is recommended for initial estimates of lateral force for a given displacement and large diameter piles. Other related statements from the document included:

- For SWM there is a small sensitivity to independent reasonable parameter selection
- Uncertainty of predictions is greater in sand than in clay

Yang et al. (2007) assessed 24 to 48-inch diameter drilled shafts and provided recommendations related to the design of noise wall foundations. Broms and Brinch Hansen estimation methods were compared for assessment of the lateral capacity. The paper concludes that Broms method is preferred because, in about 30 percent of the cases, using the Brinch Hansen method resulted in "unsafe" estimations, such that the predicted ultimate capacity was lower than the measured ultimate capacity. For comparison, this over-prediction occurred in less than 10 percent of the cases using the Broms method. Additionally, the p-y methodology (using COM624P) and NAVFAC DM7 estimation method were compared for estimating shaft head deflection. The paper indicates that at lower loads the NAVFAC method over-predicts deflection while estimates using p-y methodology are in good agreement. At higher loads both methods over-predicted movement. As such, the manual recommends using p-y methodology.

In preparation of WSDOT (1988), over 100 lateral load tests were reviewed for assessment of p-y curve development in western Washington. A list of the conclusions and recommendations from that document is included below:

- P-y methodology should continue to be used. It has improved safety and economy of pile foundations.
- Clay curves tend to over-predict deflection of large diameter piles.
- The Integrated Clay Criteria is recommended for modeling clay (due to more supporting data).
- The Extended Hyperbolic Criteria is recommended for sands (more accurate for tapered piles and H-piles).
- More research into p-y curves for other soil types (gravel and silt) is needed

10.2 DOT LATERAL LOAD TEST STUDIES

A number of DOTs have performed lateral load test research programs, often with FHWA as a sponsoring organization. Most of these test programs have been performed in the timeframe from 2000 to 2014 with a focus on developing or improving p-y parameters for various geomaterials, such as rock or weathered rock, or investigating group multipliers. In some cases, the research programs also include assessments of other simplified or empirical methods, such as Broms method, or development of other correlations or simplified approaches for ultimate lateral foundation capacity. However, there seems to be very little attention or focus paid to the SWM in these research programs. The case histories presented herein are considered representative of the efforts in the industry; other research projects may exist but are anticipated to be similar to these.

Missouri

A research study was performed by the University of Missouri for the Missouri Department of Transportation (MoDOT) for LRFD design of laterally loaded drilled shafts (Boeckmann et al. 2014). The study was undertaken to develop LRFD procedures for lateral analysis of drilled shafts for MoDOT as well as to improve knowledge of and reduce uncertainty regarding models for lateral resistance in shale. The study notes that AASHTO design specifications stipulate a resistance factor of 1.0 be used, and the FHWA drilled shaft manual (2010) recommends a resistance factor of 0.67 based on the author's judgment (not based a reliability study or data).

Thirty-two drilled shafts were constructed at two MoDOT sites. All shafts were instrumented and founded in shale. The shafts were laterally load tested in accordance with ASTM D3966 and the results were analyzed finite element methods (FEM) to match the measured deflections. The results of 25 tests were used to develop p-y curves for drilled shafts founded in shale with the intent that these curves could be used for projects in similar conditions. The curves are generally stiffer than those based on "stiff clay" in LPILE.

Resistance factors were developed for use in future analyses as well. The resistance factors developed are to be applied to the p-y curves by factoring the p values, which differs from the approach in the FHWA manual (2010) in which the resistance factor is to be applied as an additional load factor. Also, the resistance factors are to be applied to both the Service and Strength Limit States.

The resistance factors range from 0.2 to 0.6 for the Service Limit State and from 0.10 to 0.6 for the Strength Limit State. The resistance factors vary as a function of the coefficient of variation (COV) of the mean UCS value of the rock, as well as the probability of failure (P_f). Lower values of COV correspond to higher resistance values, and higher COV values correspond to lower resistance factors. In other words, the more variable the UCS data (and therefore the more variable the foundation conditions, as indicated by a higher COV), the lower the resistance factor is, and vice versa. The resistance factors are also lower for lower probability of failure.

The report notes that the resistance factors are significantly lower than the 1.0 factor indicated in AASHTO specifications. However, it is noted that these lower resistance factors should be used with the stiffer p-y curves derived from the load tests, and that the effects of this combination are expected to offset. No actual comparison of analyses using the proposed curves and resistance factors to an analysis based on AASHTO guidance is performed or presented. However, the report states that approach of using these resistance factors and p-y curves is considered more rational than the guidance in AASHTO or FHWA (2010) and is expected to improve the state of the practice.

North Carolina

A study was performed to improve the state of the practice with regard to lateral design of drilled shafts in areas of weathered rock (Gabr et al. 2002). Existing p-y curves do not accurately model weathered rock materials. Six full scale lateral load tests were performed on instrumented drilled shafts founded in weathered rock. Finite element modeling (FEM) and laboratory work were used to model and study p-y curves. P-y curves for weathered rock were developed and validated using the 3-D FEM modeling, laboratory work, and load tests. Methods to evaluate in-situ stiffness of weathered rock materials are presented as well, including the use of a rock dilatometer as well as RQD, joint conditions, and material strengths. The report indicates that the resulting weathered rock p-y models can be used to develop much more cost-efficient designs than if the “stiff clay” LPILE model were used.

There is no discussion on resistance factors in the report. However, the study was performed and the findings issued before widespread adoption of the LRFD design approach.

Ohio

A research study was performed for the Ohio Department of Transportation (ODOT) for laterally loaded drilled shafts socketed into rock (Nusairat et al. 2006). The study report indicates that there is no a well-developed rational approach for design of laterally loaded drilled shafts in rock. The design of such elements often follows guidance for piles and/or analyses based on soils. Because of the increasing use of drilled shafts and the fact that the rock socket is the most expensive part of a drilled shaft, improvements in the design methodology for such elements has the potential for cost savings for ODOT.

Five drilled shafts were constructed and load tested. In addition, in-situ testing of the rock using dilatometer and pressuremeter was performed. Rock types at the test sites included mudstone, shale, and sandstone, with some sites having interbedded rock types. Results of other tests performed in other states were used to supplement the data. A 3-D analysis of a drilled shaft socketed in rock was performed to investigate and analyze the data.

The overall results of the research program include a validated method for developing p-y curves from load test data using polynomial curve fitting techniques, an improved method for estimating p-y curves from in-situ testing using dilatometers, and an elastic solution for estimating lateral deflections of drilled shafts, an empirical solution for estimating ultimate capacity of drilled shafts in rock, and a criterion for developing p-y curves for drilled shafts in rock. P-y curves were developed and validated using the criterion and the load test results.

There is no discussion regarding the use of resistance factors aside from noting that the use of load tests allows a higher resistance factor to be used. Actual values of resistance factors for use with either Service or Strength Limit States are not discussed or presented. This study does not appear to be referenced on the Ohio DOT website or Bridge Design Manual.

Colorado

CDOT commissioned a research study to develop uniform and improved design methods for drilled shafts for noisewalls, signs, and signals (Nusairat et al. 2004). Two lateral load tests from the Denver, Colorado area were considered in the study, one in sand and one in clay, as well as load tests performed in Ohio. Comparison of the practice between CDOT staff and consulting engineers was also performed.

For sound barrier walls, the study recommends that the Broms method with a factor of safety of 2 be used for the Strength Limit State (previously CDOT was using 2.5 to 3.0). The serviceability Limit State should be analyzed using LPILE with a maximum deflection of one inch, or limiting the deflection to the soil’s elastic limit under repetitive loading estimates from LPILE. Recommendations for appropriate and geotechnical investigation and testing methods are made in order to have accurate data for proper subsurface material characterization. Guidelines for performing lateral load tests are included, and FEM modeling is recommended for large or critical projects with unusual subsurface conditions. The report states that the Broms method and p-y method are preferred by FHWA (at the time of the study) and that other methods considered are not recommended either due to inaccuracies or not being applicable based on the foundation type and/or ground conditions.

For overhead signs and signals, the report recommends CDOT continue its existing practice of using standard details and designs. The CDOT practice is for lateral analysis to be performed using Broms method with a factor of safety of 2.5 to 3. The CDOT design practice limits deflections to the elastic response and therefore prevents the buildup of irrecoverable deformations. The report notes that No failures have been observed with this approach.

There is no discussion of reliability factors, although the factors of safety discussed for the Broms method can be used to develop reliability factors. No factor of safety is indicated for use in the Service Limit State LPILE analysis.

Joint program by Utah, Arizona, California, New York, and Washington:

A pooled-fund research program was led by Utah DOT with support from California, New York, Arizona, and Washington DOTs. A test program of laterally load testing of individual pipe piles and pipe pile groups was performed on the reconstruction of I-15 in Salt Lake City, Utah. This testing included static and dynamic testing and cyclic testing. Results were analyzed with LPILE for individual piles and GROUP and FLPIER for pile groups. Results have been published by Rollins et al. (2003) for Utah DOT, although the report notes that the views or interpretations in that report do not necessarily reflect the views or interpretations of other agencies involved in the study.

The results include findings related to single piles and pile groups. For single piles, it was found that gaps form around the pile near the ground surface due to cyclical loading. Analysis results using LPILE were found to accurately model the virgin condition for loading, but the condition where a gap forms under reloading and cyclic loading conditions does not match well. LPILE and FLPIER do not have options for inclusion of a gap due to cyclic loading. This is identified as an area for further research and improvement for p-y curve development. A similar observation of a reduction of lateral resistance due to cyclic loading was observed for pile groups.

For pile groups, it was found that the lateral resistance of the piles a group is a function of the location of the row within the group and was not dependent on the location of the piles within a row; the piles on the edge of a row did not carry more load than those on the interior of a row. The front row of piles carry the greatest load, with the second and third rows carrying successively lesser load. Beyond the third row, there is little change in the pile load, except that the back row of piles carries slightly more load. Average lateral load resistance is also a function of pile spacing. The deflection necessary to fully develop group effects increases as the pile spacing within the group increases. The stiffness of a fixed pile group is significantly more than the same pile group under free-head conditions even with the formation of gaps around the piles due to cyclic loading.

P-multipliers were developed based on the results of the testing. P-multipliers available in GROUP and FLPIER at the time of the study were found to be inaccurate, with GROUP under-predicting deflections and FLPIER over-predicting. The recommended p-multipliers from the study were found to accurately predict the deflections, including the cyclic loading case when the soil profile was softened.

The pile groups were also tested with statnamic (dynamic) testing. It was found that the virgin loading resistance for statnamic testing is significantly higher than for static testing, but for reloading conditions the resistances were comparable. Lateral resistance is also a function of row location under statnamic testing, although group reduction effects were less than for static loading. Group effects were much less for reloading under statnamic testing than for static testing. No consistent pattern of load distribution within a row of piles was observed for statnamic testing, similar to static testing. The depths to maximum bending moment and zero moment were the same during statnamic testing and static testing. The analyses of statnamic and static loading conditions indicates that the difference in responses is primarily due to damping resistance, which is more significant for virgin loading than for reloading. Soil damping at large displacements has a significant effect on the lateral load response of pile groups and is identified as an area for additional research.

Caltrans, one of the agencies involved with the research sponsorship, also issued recommendations based on the study results (Caltrans 2003). Caltrans presents recommendations for p-multipliers modified from the results in the Rollins et al. (2003) report and indicates that designs should consider these p-multipliers instead of using automatically generated p-multipliers from software.

Other Testing Programs or Case Histories

Other lateral load test programs have been found for individual projects or for research into other topics, such as the behavior of composite piles under lateral loads. An exhaustive review of these case histories is beyond the scope of this literature review. However, a common element of published load test case histories or studies is a focus on or use of the p-y method for analysis, either to develop better or site-specific p-y parameters, group parameters such as p-multipliers, or to develop pile properties for use in p-y analyses. These case histories and the example research programs discussed above imply that the p-y method is the most widely used and accepted method for analysis in current practice. There appears to be much less research or use of the SWM method, Broms, or other methods for actual load test case histories or research.

11. GAPS IN THE STATE OF THE PRACTICE

The most significant gap in the state of the practice is the lack of a single consistent analysis method for the various Limit States with specific guidance for the various related topics, e.g., seismic events, head fixity, group multipliers. As the goal of GEC 9 is to provide guidance on these subjects, discussion follows in Section 12. Also lacking is a single consistent design approach considering the available methods; for example, some publications allow any method for any foundation or design stage, others suggest simplified analyses (Broms) for preliminary design or simple structures and more detailed analyses (p-y) for final design or complicated structures, and others provide no information or guidance at all. Included below are the related topics that due to the lack of information, consensus among professionals, or other reasons could be considered for further research.

As the source material for the majority of this document is from DOTs, AASHTO, and FHWA, it is noted that it is preferential to bridge design. However, in the available manuals and publications that are more generally oriented, the discussion on laterally loaded deep foundations does not distinguish in detail differences between the structure type or application. For example, the publications generally do not distinguish different procedures or requirements based on whether the foundations are for a retaining wall, sound wall, excavation support, bridge abutment, bridge pier, or other structure. Loading conditions, such as seismic loads and vessel impact, are commonly addressed as extreme event conditions.

11.1 STRAIN WEDGE MODEL

Based on our current literature review, there do not appear to be any fundamental problems or documented objections with use of the strain wedge model. The reason for its lack of predominance in current practice is unclear; it may simply be that it was developed after the p-y method, and given the wide acceptability, usage, and familiarity with the p-y method, practitioners have been reluctant to change to a less known or less widely used method without being required by code or design standard. Most references that include a mention of the model recognize it may be more relevant than the p-y methodology for short and/or large diameter elements, but only New York makes a formal recommendation for it as a preference. Washington DOT recently removed their recommendation, albeit with the stated reasons including a lack of general acceptance by the professional community, similar to the presumption noted above. It is our opinion that further literature review or research with the goal of preparing a formal comparison between the two methods for short and/or large diameter piles could be worthwhile to advance this state of the practice.

11.2 SLOPE REINFORCEMENT OR STABILIZATION

The method for analyzing soil slopes in global shear failure is not a focus of this document, but load application is a consideration for which very little specific guidance has been found in the published manuals or codes. Nonetheless, a number of case histories have been presented in journal articles and conferences, possibly due to the lack of clear guidance in agency publications.

The Ohio Department of Transportation (ODOT) has issued a Geotechnical Bulletin (GB 7) on “Drilled Shaft Landslide Stabilization Design” (ODOT 2014). The bulletin provides detailed guidance on the use of drilled shafts for landslide stabilization including site reconnaissance and investigation, stability analyses, and design of drilled shafts for stabilizing landslides. Once the landslide configuration and subsurface parameters have been established, the slope is analyzed using the computer program UA Slope 2.1, which was developed as part of a similar research study. The analysis is UA Slope 2.1 is used to determine the loads on the drilled shafts for landslide stabilization.

The loads from the UA Slope analyses are used in LPILE for the design of the laterally loaded drilled shafts. The bulletin provides guidance on how to perform the LPILE analysis, including reducing the UA Slope loads for input into LPILE, the use of p-multipliers, soil layering, shaft length, reinforcement, pile head loadings, etc. The geotechnical resistance is also discussed with options on how to evaluate this for the Strength Limit State. The bulletin notes that ODOT does not advocate using the Geotechnical Strength Limit State check in the FHWA drilled shaft manual (2010) because ODOT considers that procedure to produce overly conservative results. The bulletin indicates load and resistance factors consistent with AASHTO LRFD guidelines.

Although it is only applicable for drilled shafts and specifically uses one program and shaft configuration (single rows of drilled shafts) for analysis and stabilization, this publication by ODOT appears to be one of the most comprehensive publications by a transportation agency on the use of deep foundations for landslide stabilization.

11.3 ROCK SOCKET DESIGN

Much of the design guidance and available information for lateral load analysis of deep foundations focuses on soil. Specific p-y curves for rock is a significant gap in the available industry information, especially considering the potential variability in strength, fracturing, elasticity, and other properties of various rock types and masses.

In addition, design approaches to deal with high shear loads at the rock surface in short piles is an issue that lacks guidance in the available publications researched for this report.

11.4 INTERMEDIATE GEOMATERIALS (IGM)

There is a growing awareness within the geotechnical design community that intermediate geomaterials (IGMs), such as decomposed or highly weathered rock, partially cemented sands, and some marl formations, are not adequately addressed in available publications and resources. This is a gap in the state of the practice for lateral design of deep foundations as well. Most available guidance or information, such as p-y curves, is for either sand or clay soils. Therefore, current practice requires making simplifying assumptions or characterizations when dealing with design in IGMs.

11.5 SEISMIC DESIGN CONSIDERATIONS

Several of the available publications discuss seismic design approach and criteria. However, a noticeable gap in the available information is p-y curves for liquefied soils.

11.6 BATTER PILES

In general, the literature reviewed either did not address batter piles specifically or only made passing mention of them. The literature that includes discussion on analyses and approaches for analyzing piles for lateral load applications only addressed vertical piles. Where batter piles are mentioned, it is indicated that batter piles can be used to resist lateral loads through axial and end bearing capacity.

12. PRELIMINARY ASSESSMENT OF METHOD FOR GEC 9

Based on this literature review, it is evident that there is not a consistent or unified approach among various agencies and practitioners for the current state of the practice for lateral load design of deep foundations. The main standard between agencies and countries is the concept that either the Broms method or a SSI method (typically p-y method) should be used to analyze deep foundations which are controlled by lateral design. Other topics related to the selected design depth, head fixity, deflection limits, Limit States, and engineering responsibilities are not consistent, but generalities do tend to exist. Below is a summary of the anticipated direction for the design procedure that will be presented within GEC 9.

12.1 PRELIMINARY DESIGN

The Broms method is widely used and accepted by the majority of the entities investigated. Some accept the method for final design, while others limit it to preliminary design or for structures with relatively low lateral loads. As such it is recommended that for the purposes of preliminary design or for special design cases (simple structures or structures with low lateral loads), Broms method should be included in GEC 9. However, it is not recommended that Broms method be recommended for structures regardless of size where lateral load controls either the structural design section or embedment depth.

Additionally, it is recognized that establishment of a preliminary point of fixity for structural engineers to perform an equivalent cantilever beam analysis is important in the early stages of a project for the purposes of estimating anticipated element sizes and spans. The method proposed by Davisson (1965), which is recommended in AASHTO LRFD, is considered an acceptable approach which has been used successfully in practice for many years.

12.2 FINAL DESIGN

The procedure for final design is anticipated to be modeled after that in the FHWA manual for drilled shafts (GEC 010), with modifications to account for long and/or slender elements. The basic outline presented for discussion includes these general steps:

1. Determine whether the deep foundation element(s) classify as short or long and determine the point of fixity.
    a. Guidance for procedures for determining short versus long, as well as point of fixity, will be provided in the GEC; however, at the time of this Literature Review Report, the available methods are currently under review and a final approach has not yet been determined.
2. If the element classifies as short, then analyze:
    a. Geotechnical Strength Limit State—pushover analysis for uncracked section with factored loads and factored resistance
    b. Structural Strength Limit State—check cracked structural section for factored loads and nominal resistance
    c. Service Limit State—check cracked structural section deflections are within acceptable limits for service loads and nominal resistance
3. If the element classifies as long, then analyze:
    a. Structural Strength Limit State—check cracked structural section for factored loads and nominal resistance
    b. Service Limit State
        i. Check cracked structural section deflections are within acceptable limits for service loads and nominal resistance
        ii. Determine embedment required (with criteria according to "critical depth" or the second point of zero deflection) for service loads and nominal resistance

Guidance within GEC 009 is expected to be supplied for:

- Use of P-multipliers related to group effects
- General procedure for the development of liquefaction loads
- Selection of p-y curves and soil properties for liquefied strata
- Head fixity
    - Note that current literature review suggests that fixed head condition results in more accurate deflection estimates
- Use of Strain Wedge Model (SWM) method
    - This literature review has found that the p-y method is more widely used and accepted than the SWM, and there appears to be a larger body of testing and case histories as well as region- or state-specific parameters applicable to the p-y method. Therefore, the p-y method will be presented as the preferred method for analysis. However, the PWM method will be considered to be a valid alternative method and can be used in cases where the designer is comfortable with the method and software, where there is adequate precedence or site/region specific data, and/or where an additional assessment is desired, such as cases where lateral capacity is critical, where subsurface conditions are unusual, and/or where the results of the p-y method appear to be

unusual or questionable. However, the SWM method should not be used in lieu of the p-y method where there is wide acceptance in local practice and/or site or region specific p-y parameters based on DOT guidelines or testing.

- Responsibilities of structural and geotechnical engineers

Based on general practice from State DOTs it is anticipated that it will be recommended that geotechnical engineers supply soil properties or p-y curves to structural engineers who perform the lateral analysis. Additionally, it will be suggested that a final review be conducted of the results by the geotechnical engineer.