Chapter 16

DEEP FOUNDATIONS

GEOTECHNICAL DESIGN MANUAL

January 2019

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CHAPTER 16

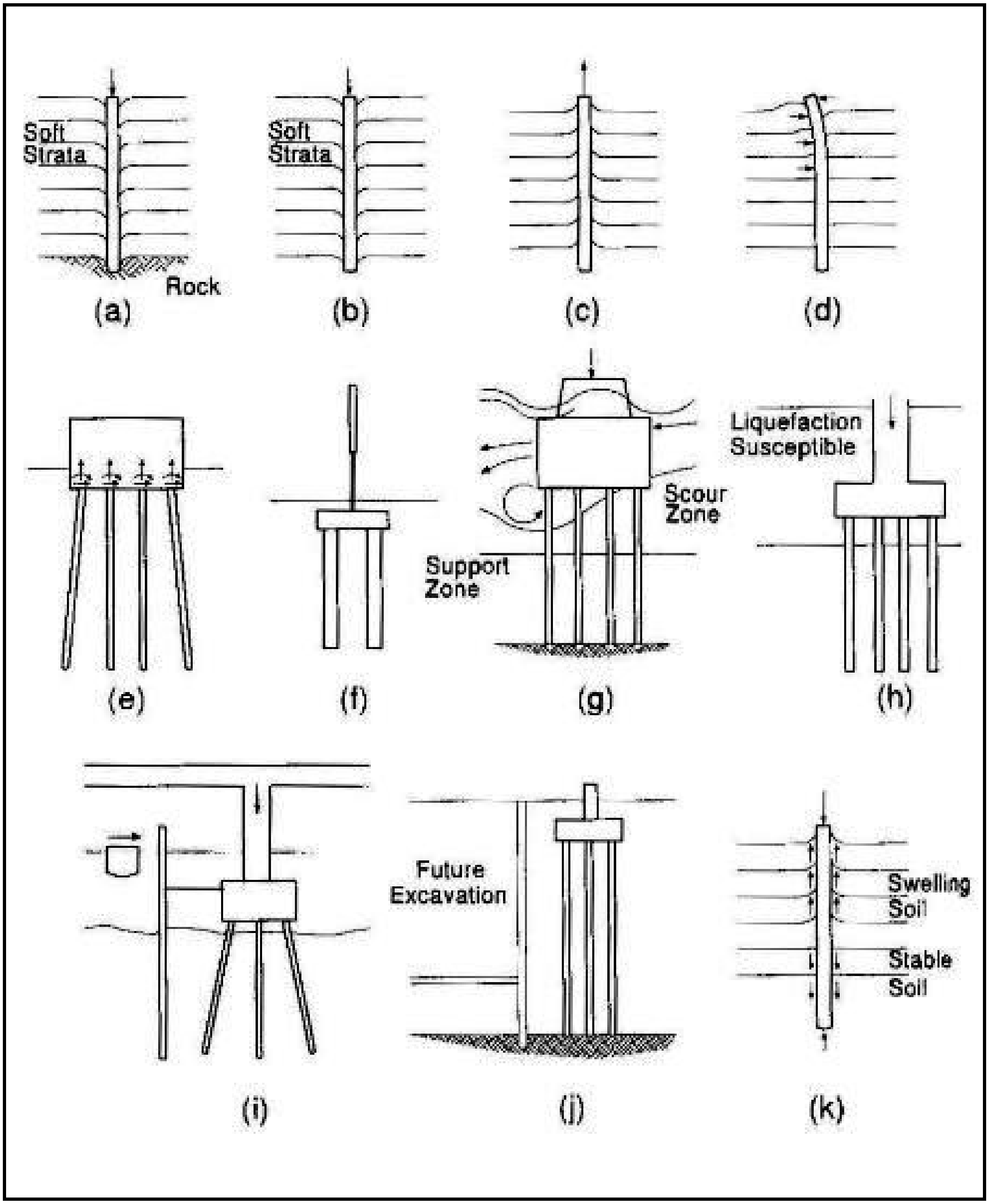
DEEP FOUNDATIONS

16.1 INTRODUCTION

This Chapter provides general guidance in the design and analysis of deep foundations used to support highway structures. Deep foundations consist of driven piles, drilled shafts or piers, drilled piles, auger cast-in-place (continuous flight auger, CFA) piles, micro-piles and pile or drilled shaft supported footings. Each foundation type has specific advantages and disadvantages that will be discussed in subsequent Sections. The design of deep foundations is comprised of 2 components, the axial and lateral capacity (resistance to shear) and settlement (performance); however, in the design of deep foundations axial resistance typically governs.

According to NAVFAC DM-7.2 (1986) deep foundations are defined as developing resistance at depths (D_f) greater than 5 times the size (diameter) (B_f) of the foundation (i.e., $D_f \geq 5B_f$). As indicated previously, axial resistance typically governs the design of deep foundations not settlement. The resistance of deep foundations is based on either the end resistance (R_t) or side resistance (R_s) along the shaft of the foundation acting independently of the other component or a combination of the 2 components acting together. Deep foundations need to be considered for several reasons:

- When the upper soil strata are too weak or compressible to support the required vertical loads (a), (b), (c) (letters refer to Figure 16-1);
- When shallow foundations cannot adequately support inclined, lateral, or uplift loads, and overturning moments (d), (e), (f);
- When scour around foundations could cause loss of bearing capacity at shallow depths (g);
- When soils around foundations are subjected to SSL during seismic events (h);
- When fender systems are required to protect bridge piers from vessel impact (i);
- When future excavations are planned which would require underpinning of shallow foundations (j), and;
- When expansive or collapsible soils are present, this could cause undesirable seasonal movements of the foundations (k).



Note: Illustrations (e) and (i) above shows battered piles, please note that SCODT prefers vertical piles.

Figure 16-1, Reasons for Deep Foundations (Hannigan, et al. (2006))

All deep foundation designs will be governed by the basic LRFD equation.

$$Q = \sum \eta_i \gamma_i Q_i \leq \varphi R_n = R_r = \varphi R_t + \varphi R_s$$ **Equation 16-1**

Where,

Q = Factored Load
Q_i = Force Effect
η_i = Load modifier
γ_i = Load factor
R_r = Factored Resistance (i.e., allowable capacity)
R_n = Nominal Resistance (i.e., ultimate capacity)
R_t = Nominal End or Tip Resistance
R_s = Nominal Side Resistance
φ = Resistance Factor

The selection of φ will be discussed in greater detail in the following Sections. Typically, φ is based on the method of construction control for piles and on the type of material and where (i.e., end or side) the capacity is developed. SCDOT does not use design method specific resistance factors (see Chapter 9) for the design of pile foundations but instead uses resistance factors based on load resistance construction verification. Drilled shafts are designed using either design method specific or construction verification resistance factors, with construction verification resistance factors taking precedence over the design method resistance factors if construction verification is used. The factored load is provided by the SEOR.

16.2 DESIGN CONSIDERATIONS

The design of deep foundations supporting bridge piers, abutments, or walls should consider all limit state loading conditions applicable to the structure being designed. A discussion of the load combination limit states that are used in deep foundation design is discussed in Chapter 8 and the deep foundation corresponding limit state is reproduced below in Table 16-1. Most substructure designs will require the evaluation of foundation and structure performance at the Strength and Service limit states; however there are instances where the EE limit state may control design. These limit states are generally similar to evaluations of ultimate capacity and deformation behavior in ASD, respectively.

Table 16-1, Deep Foundation Limit States

Performance Limit	Limit States		
	Strength	**Service**	**Extreme Event**
Axial Compression Load	√		√
Axial Uplift Load	√		√
Structural Capacity[1]	√		√
Lateral Displacements		√	√
Settlement		√	√

[1]Determined by SEOR

In addition, the environment (corrosive or non-corrosive) into which the deep foundations are installed should be evaluated as a part of design. As required in Chapters 4 and 7, the GEOR shall conduct electro-chemical tests and shall indicate whether the soils at the project site are Aggressive or Non-aggressive. This information shall be provided to the design team, specifically the SEOR. The SEOR and the design team shall evaluate the results of the electro-chemical tests and shall determine if the subsurface environment has the potential for foundation material deterioration and what measures need to be taken to avoid deterioration of the foundations.

16.2.1 Axial Load

Axial loadings should include both compressive and uplift forces in evaluation of deep foundations. Forces generated from the Strength limit state and EE limit state are used to determine nominal axial pile resistances from the axial design process. The Strength limit state is a design boundary condition considered to ensure that strength and stability are provided to resist specified load combinations, and avoid the total or partial collapse of the structure. The Service limit state is the design boundary condition for structure performance under the intended service loads. This boundary condition accounts for some acceptable level of deflection over the life of the structure. The Service limit state is checked to determine foundation movements. If the foundations excessively deflect, the performance of the structure could be compromised. All deflections determined by the GEOR shall be reported to the design team. The design team shall evaluate the foundation deflections and determine the impact of the deflections on the Performance Objective of the structure. The EE limit states are design boundary conditions considered to represent an excessive or improbable loading combination. Such conditions may include vessel or vehicular impacts, scour (check flood), and seismic events. Because the probability of these events occurring during the life of the structure is relatively small, a smaller safety margin (higher φ) is appropriate when evaluating this limit state.

The static resistance of a pile/shaft can be defined as the sum of soil/rock resistances along the pile/shaft surface and at the pile/shaft toe (tip) available to support the imposed loads on the pile. A static analysis is performed to determine the nominal bearing resistance (R_n) of an individual pile/shaft and of a pile/shaft group as well as the deformation response of a pile and/or group to the applied loads. The nominal bearing resistance (R_n) of an individual pile and of a pile group is the smaller of:

(1) The resistance of surrounding soil/rock medium to support the loads transferred from the pile/shaft or,
(2) The structural capacity of the pile/shaft.

The static pile/shaft resistance from the sum of the soil/rock resistances along the pile/shaft surface and at the pile/shaft toe can be estimated from geotechnical engineering analysis using:

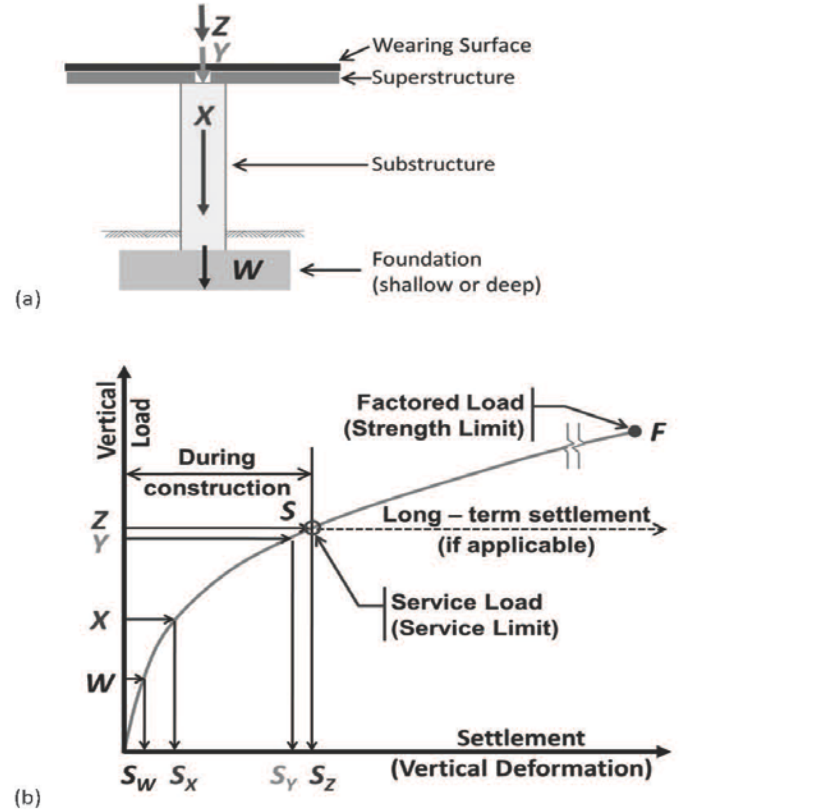
(1) Laboratory determined shear strength parameters of the soil and rock surrounding the pile;
(2) Standard Penetration Test (SPT) data;
(3) In-Situ Test data (i.e., CPT); or
(4) Full scale load test data.

16.2.2 Lateral Load

Lateral loadings applied in foundation design shall consider foundation members placed through embankments, locations on, near or within a slope, loss of support due to erosion or scour, and the bearing strata significantly inclined. Forces generated from the Service and EE limit states are used to determine the horizontal and vertical movements of the foundation system. The Service limit state is a design boundary condition for structure performance under intended service loads, and accounts for some acceptable measure of structure movement throughout its performance life. The EE limit states are design boundary conditions considered to represent an excessive or improbable loading combination. Such conditions may include vessel or vehicular impacts, scour (check flood), and seismic events. Because the probability of these events occurring during the life of the structure is relatively small, a smaller safety margin is appropriate when evaluating this limit state.

16.2.3 Settlement

The amount of settlement is normally limited to the amount required to develop the resistance of the deep foundation element. Settlements are determined for the Service and Extreme Event limit states. The appropriate loads shall be used in the determination of settlement. The procedures discussed in the following Sections shall be used to determine the amount of settlement of the foundation elements. Typically settlement along shafts is limited to the amount required to develop side resistance which in turn limits the amount of displacement of the shaft tip thus reducing the amount of load carried by the tip. For deep foundations, the settlement of the group is normally determined. In addition, the elastic shortening of the deep foundation elements due to the load should be included in the overall settlement. The inclusion of elastic shortening is required, since the performance of the structure will be affected by this movement. Static analysis calculations of the deformation response to lateral loads and of pile/shaft groups' settlement are compared to the performance criteria established for the structure. All settlements shall be reported to the SEOR. It is the responsibility of the SEOR to determine if the settlement causes excessive deformation of the structure or induces additional stress on a particular element. Depending on the requirements of the particular project, the use of the Construction-Point Concept may be used. Unlike traditional settlement calculations which assume the bridge is instantaneously placed, the Construction-Point Concept determines the settlement at specific critical construction points (see Figure 16-2).



Legend

W	Load after foundation construction
X	Load after pier column/wall construction
Y	Load after superstructure construction
Z	Load after wearing surface construction
S	Service Load (SLS)
F	Factored Load (Strength Limit State)

S_W	Settlement under load *W*
S_X	Settlement under load *X*
S_Y	Settlement under load *Y*
S_Z	Settlement under load *Z*

(a) Identification of critical construction points
(b) conceptual load-displacement pattern for a given foundation

Figure 16-2, Construction-Point Concept
(DeMarco, Bush, Samtani, Kulicki and Severns (2015))

16.2.4 Scour

The design of deep foundations shall consider the effects of scour (design flood) on the capacity and length requirements of the foundation as part of the Strength and Service limit state design. The nominal resistance of deep foundations shall be determined for the soils beneath the scourable soils. The depth of scour shall be determined by the HEOR. The capacity of the scourable soils shall be added to the nominal capacity of driven piles when developing driving criteria, but no increase in capacity is required for the drilled shafts, CFAs and micro-piles because they are not driven. The following Sections will provide additional details for handling scour for each foundation type. In addition, the design of deep foundations shall include the effects of scour induced by the check flood. The check flood is part of the EE II limit state. All deep foundations shall be checked to ascertain that the soils beneath the check flood scour have sufficient capacity to resist the EE II loads. Similar to the Strength and Service limit state designs, all resistance shall be determined utilizing the soils beneath the check flood scour.

16.2.5 Downdrag

Downdrag on deep foundations is caused by 2 distinct phenomena, settlement of subgrade soils and seismically induced SSL, specifically liquefaction of Sand-Like soils. Settlement is normally anticipated to occur at the end bent of bridges where the bridge meets the embankment. As part of the settlement analysis the potential for lateral squeeze should be considered. Lateral squeeze of compressible soils may induce lateral loads on the deep foundations. Downdrag induced settlements are applied to the Strength limit state of the deep foundation. Downdrag loads will be discussed in the following Sections, while the settlement of the embankments is discussed in Chapter 17. The other phenomenon that may induce downdrag is seismically induced SSL. This downdrag load is applied to the EE I limit state and will be discussed in the following Sections in greater detail. The amount of seismically induced SSL settlement is determined using the procedures outlined in Chapter 13.

16.3 DRIVEN PILES

Driven piles typically used by SCDOT include prestressed concrete, steel H-piles, steel pipe piles and combination piles consisting of prestressed concrete and steel H-pile sections. In addition, SCDOT has used timber piles in the past; however, timber piles are typically only used for pedestrian bridge structures. The use of concrete cylinder piles shall be approved in writing by the PC/SDS and PC/GDS prior to commencing design. Piling is further categorized as either displacement or non-displacement. Displacement piles increase lateral ground stresses, densify Sand-Like soils, can weaken Clay-Like soils (temporarily), have large set up times for Clay-Like soils, and primarily get their capacity from skin resistance. Typically, prestressed concrete and closed-ended steel pipe piles are considered displacement piles. Non-displacement piles usually cause minimal disturbance to surrounding soil and primarily get their capacity from end bearing and are typically driven to dense/hard soils or rock. Steel H-piles and opened steel pipe piles are considered non-displacement piles. The BDM provides typical sizes for driven piles. Table 16-2 provides a summary of these pile types and sizes.

Table 16-2, Typical Pile Types and Sizes

Pile Type	Size
Steel H-piles	HP 12x53 HP 14x73 HP14x89 HP 14x117[1]
Steel Pipe Piles	16-inch[2] 18-inch[2] 20-inch[2] 24-inch[2]
Prestressed Concrete Piles (PSC)[3]	18-inch 20-inch 24-inch 30-inch[4] 36-inch[4]
Combination Piles	18-inch PSC[3] with W 8x58 stinger 20-inch PSC[3] with HP 10x57 stinger 24-inch PSC[3] with HP 12x53 stinger

[1]Used where penetration is minimal and nominal capacity is large
[2]Wall thickness is ½ inch, minimum, for all pipe pile sizes
[3]Prestressed concrete piles are solid and square in section
[4]These sizes are only allowed with the written approval of SCDOT

As required in the AASHTO LRFD Specifications, driven pile analyses and design should address the following:

- Nominal axial resistance, pile type, size of pile group, and how the nominal axial pile resistance will be determined in the field;
- Pile group interaction;
- Pile penetration required to meet nominal axial resistance and other design requirements;
- Minimum pile penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, lateral loads, SSL, and seismic conditions;
- Foundation deflection should meet the established movement and associated structure performance criteria;
- Pile foundation nominal structural resistance;
- Verification of pile driveability to confirm acceptable driving stresses and blow counts can be achieved, and;
- Long-term durability of the pile in service (i.e., corrosion and deterioration).

A thorough reference on pile foundations is presented in the FHWA publication Design and Construction of Driven Pile Foundations – Volume I and II (Hannigan, et al. (2006)).

16.3.1 Axial Compressive Resistance

There are numerous static analysis methods available for calculating the bearing resistance of a single pile. The axial compressive capacity for driven piles shall follow the procedures provided in the AASHTO LRFD Specifications (Section 10.7 - Driven Piles). The methods found in the AASHTO LRFD Specifications are used to satisfy the Strength, Service and EE limit states.

The basic LRFD equation presented previously and in Chapter 8 is expanded on the resistance side of the equation to account for the factored resistance of piles (R_r), and may be taken as:

$$\boldsymbol{Q \leq \varphi R_n = R_r = (R_t + R_s)\varphi} \qquad \textbf{Equation 16-2}$$

$$\boldsymbol{R_t = q_t * A_t} \qquad \textbf{Equation 16-3}$$

$$\boldsymbol{R_s = q_s * A_s} \qquad \textbf{Equation 16-4}$$

Where,

Q = Factored Load (demand)
R_r = Factored Resistance (i.e., allowable capacity)
R_n = Nominal Resistance (i.e., ultimate capacity)
R_t = Nominal End or Tip Resistance
q_t = Unit End or Tip Resistance of pile (force/area)
A_t = Area of pile tip (area)
R_s = Nominal Side Resistance
q_s = Unit Side Resistance of pile (force/area)
A_s = Surface area of pile side (area)
φ = Resistance Factor based on construction control (see Chapter 9)

The nominal resistance of driven pile at the Strength limit state shall include the effects of scour (design flood). The nominal resistance shall be developed beneath the scour elevation or depth; however, the resistance developed in the scourable soils shall be determined and added to the nominal resistance to obtain the required driving resistance (R_{nDR}) for use during pile installation.

The axial compressive design methodologies can be separated based on either total or effective stress methods or whether the soils are cohesionless (Sand-Like) or cohesive (Clay-Like) in nature. As indicated in the above equations the total axial compressive resistance of a deep foundation is based on the combination of unit side resistance and unit tip resistance values. Another factor that affects the axial compressive resistance of driven piles is the type of pile being installed (i.e., non-displacement vs. displacement). The followings methods shall be used to determine the resistance of driven piles:

(1) Nordlund Method: This method is an effective stress method and is used for sands and non-plastic silts (cohesionless soils (Sand-Like)). Further this method is based on field observations and considers the pile shape, and its soil displacement properties in calculating the shaft resistance. The unit side resistance is a function of: friction angle of the soil, the friction angle of the sliding soils, pile taper, the effective unit weight of the soil, pile length, the minimum pile perimeter, and the volume of soil displaced. The friction angle of the soil shall be determined in accordance with the procedures outlined in Chapter 7. While there is no limiting value for the side resistance, the effective overburden pressure shall be limited to 3 kips per square foot (ksf) for determining the tip resistance. For pile sizes greater than 24 inches, this method tends to overpredict the pile resistance.

(2) α-Method: A total stress analysis used where the ultimate resistance is calculated from the undrained shear strength of the soil and is applicable for clays and plastic silts (cohesive soils (Clay-Like)). The undrained shear strength shall be determined in accordance with the procedures provided in Chapter 7. This method assumes that side resistance is independent of the effective overburden pressure and that the unit side resistance can be expressed in terms of an empirical adhesion factor times the undrained shear strength. The coefficient α depends on the nature and strength of the clay, pile dimension, method of pile installation, and time effects. The unit tip resistance is expressed as a dimensionless bearing capacity factor times the undrained shear strength. The dimensionless bearing capacity factor (N_c) depends on the pile diameter and the depth of embedment, and is usually assumed to be 9.

(3) SPT 97 Method: A total stress method originally developed by the Florida Department of Transportation (FDOT). The method uses uncorrected N_{60}-values to determine the nominal resistance of driven piles. The method is based on the results of numerous load tests conducted by FDOT. The soils of the South Carolina Coastal Plain are similar to the soils in Florida. This is applicable to both cohesionless (Sand-Like) and cohesive (Clay-Like) soil.

(4) Historical Load Test Data: The nominal resistance for driven piles may be developed based on the results of historical load test data from the anticipated load bearing stratum (i.e., the same geologic formation). The use of this type of data for development of resistance shall be reviewed by the PC/GDS. The results of more than 5 load tests shall be used to develop the resistance. Load testing shall include static load tests, dynamic load tests and rapid load tests. A comparison to the soils at the load test site to the soils at the new location shall be performed.

For driven piles that will develop resistance in a layered subsurface profile consisting of both cohesionless (Sand-Like) and cohesive (Clay-Like) soils; the appropriate method will be used for each soil type and the nominal resistance determined by adding the results of the various layers together. For soil layers that are comprised of ϕ – c soils, the axial resistance for the layer should be determined using the Nordlund, SPT 97 and α methods with the actual resistance of the layer being the more conservative resistance.

The AASHTO LRFD Specifications provide additional methods for determining the axial compressive resistance of driven piles. These additional methods shall be used only as a check to the Norlund, SPT 97 and α methods discussed previously. These additional methods include:

(1) β-Method: An effective stress analysis used in cohesionless (Sand-Like), cohesive (Clay-Like), and layered soils.

(2) λ-Method: An effective stress method that relates undrained shear strength and effective overburden to the shaft resistance.

(3) Meyerhof SPT data Method: This method was derived by empirical correlations between Standard Penetration Test (SPT) results and static pile load tests for cohesionless (Sand-Like) soils.

(4) Nottingham and Schmertmann CPT Methods: The method uses Cone Penetrometer Test data relating pile shaft resistance to CPT sleeve friction.

In addition, Hannigan, et al. (2006) provides additional procedures for determining the axial compressive resistance of driven piles.

(1) Brown Method: An empirical method using SPT data for cohesionless (Sand-Like) materials.

(2) Elsami and Fellenius Method: A CPT based method that correlates the effective tip resistance to the unit shaft resistance.

(3) Laboratoire des Ponts et Chaussess (LPC) Method: A CPT based method that correlates the tip resistance, soil type, pile type and installation method to the unit shaft resistance.

As with the other methods listed in the AASHTO LRFD Specifications, these methods shall only be used to check the resistances determined by the Norlund, SPT 97 and α methods.

For driven piles bearing in rock with an RQD greater than 10 percent (see Chapter 6), the nominal resistance of the pile is typically limited by the structural capacity of the foundation element itself. This is especially true with prestressed concrete piles driven into rock, and why prestressed concrete piles typically have pile points when driven to bearing in rock. In many cases steel piles are fitted with “reinforced tips” to avoid damage to the foundation element. If the depth to rock with RQD greater than 10 percent is less than 10 feet, then the pile should be installed as a drilled pile. Therefore piles should be driven to rock when the depth to top of rock is greater than 10 feet. For rock with RQD less than 10 percent and soils with 100 or more blows per foot of penetration, it has been the experience of SCDOT that piles can be driven into these materials. Penetrations typically range from 5 to 10 feet.

There are numerous computer software packages available for performing the axial compressive resistance of driven pile foundations. The preferred software packages are APILE v2014® as produced by ENSOFT, Inc. or SPT 97 as developed by the University of Florida for

FDOT. The latest version of SPT 97 is contained within FB-Deep© as developed by the University of Florida, Bridge Software Institute (http://bsi-web.ce.ufl.edu/). APILE v2015® uses the Norlund method for determining axial resistance for cohesionless (Sand-Like) soils (tip and skin friction). While for cohesive (Clay-Like) soils the α method is used for determining the tip and side resistance. In APILE v2015® these methods are collectively called the "FHWA Method". FB-Deep© can be applied to both cohesionless (Sand-Like) and cohesive (Clay-Like) soils. Other computer software packages may be used to determine axial compressive resistance of driven piles; however, prior to being used, the designer must submit copies of the output, the method used for design, a set of hand calculations performed using the procedure and evidence of applicability and acceptability using load testing information. This information shall be submitted to the PC/GDS for technical review prior to being approved and is in addition to the requirements of Chapter 26. It is incumbent upon the GEOR, that prior to using any software, that the methodologies used by the software are fully understood.

16.3.2 Axial Uplift Resistance

The axial uplift resistance should be evaluated when tensile forces may be present. The side resistance of the driven pile shall be determined using either the Norlund or α methods. All resistance losses due to scour shall not be included in the determination of the axial uplift resistance. In addition, static settlement induced downdrag loads shall not be included, since it is anticipated that at some point in time settlement will cease. The factored uplift resistance (R_r) may be evaluated by:

$$\boldsymbol{Q \leq \varphi R_n = \varphi_{up} R_s = R_r}$$ **Equation 16-5**

Where,

Q = Factored Load (demand)
R_r = Factored Resistance (i.e., allowable capacity)
R_n = Nominal Resistance (i.e., ultimate capacity)
R_s = Nominal Side Resistance
φ and φ_{up} = Uplift Resistance Factors (see Chapter 9)

16.3.3 Group Effects

The analysis procedures discussed in the preceding paragraphs are for single driven piles. For most structures, driven piles are installed in groups. Typically SCDOT uses trestle bents (i.e., a single row of piles). Trestle bents shall be considered to be groups for the purpose of determining group efficiency. The nominal axial (compressive or tensile) resistance of a pile group is the lesser of:

- The sum of individual nominal pile resistances, or
- The nominal resistance of the pile group considered as a block.

The minimum center-to-center spacing in a trestle bent is 2-1/2 times the nominal pile size; therefore, the group efficiency shall be taken as 1.0. For pile groups having 2 or more rows of piles, the group efficiency shall be determined following the procedures outlined in Section 10.7 of the AASHTO LRFD Specifications. The spacing between piles shall not be less than a center-to-center spacing of 2-1/2 times the nominal pile size in either the longitudinal or transverse directions. The procedures for determining the dimensions of the block are presented in the following section.

16.3.4 Settlement

Typically, the settlement of deep foundations is comprised of immediate and primary consolidation settlement and elastic compression (shortening). Secondary compression is not normally considered as part of the settlement of deep foundations. In many cases primary consolidation settlement is not a concern, since most deep foundations are founded in cohesionless soils (Sand-Like), overconsolidated (OCR $\geq$ 4) (Clay-Like) soils, or rock. Elastic compression is included since the deep foundation will elastically deform when a load is applied. Pile groups are used in determining the amount of settlement instead of single piles, since very rarely are single piles used to support a structure. The total settlement is defined by the following equation.

$$\Delta_v = S_t = S_i + S_c + S_s + \Delta_E \qquad \textbf{Equation 16-6}$$

Where,

S_t = Δ_v = Total Settlement
S_i = Immediate Settlement
S_c = Primary Consolidation Settlement
S_s = Secondary Compression Settlement
Δ_E = Elastic Compression

Elastic compression is the compression (deflection or shortening) of a single pile caused by the application of load at the top of the pile. The elastic compression of combination piles is complex and difficult to determine. Therefore, engineering judgment should be used in determining if the concrete or steel portion of the combination pile contributes more to the settlement of the pile group. Elastic compression should be determined using the following equation.

$$\Delta_E = \frac{Q_a * L}{A * E} \qquad \textbf{Equation 16-7}$$

Where,

Q_a = Applied load
L = Pile length (embedment)
A = Cross sectional area of pile
E = Elastic modulus of pile material

For piles founded in cohesionless (Sand-Like) soils and in overconsolidated (OCR $\geq$ 4) cohesive (Clay-Like) soils, the settlement shall be determined using elastic theory as presented in Chapter 17. An equivalent foundation is used to determine the dimensions required. The width of the foundation (B_f) is either the pile diameter or face dimension for pile bents or the center to center dimension of the outside piles along the shortest side of a pile footing (group). The length (L_f) is the center to center dimension of the outside piles along the length of the pile bent or pile footing. The depth of the equivalent foundation shall be 2/3 of the pile embedment depth into the primary bearing resistance layer. The applied bearing pressure (q_o) shall be taken as the sum of the pile loads at the limit state being checked divided by the area of the equivalent footing. For each subsequent layer, the equivalent foundation is enlarged 1 horizontal to 2 vertical (1H:2V) proportion until the settlement for all subsequent layers is determined.

The settlements for pile foundations placed in NC to slightly OC (1 < OCR <4) plastic cohesive (Clay-Like) soils shall be determined using consolidation theory as presented in Chapter 17. Similar to the elastic settlement determination an equivalent foundation shall be placed 2/3 of the pile embedment depth into the primary bearing resistance layer and the applied bearing pressures and changes in stress determined according. The applied bearing pressure (q_o) shall be taken as the sum of the pile loads at the limit state being checked divided by the area of the equivalent footing. For each subsequent layer, the equivalent foundation is enlarged 1 horizontal to 2 vertical (1H:2V) proportion until the settlement for all subsequent layers is determined.

As indicated previously, settlement is determined at the Service and Extreme Event limit state loading. All settlements will be reported to the SEOR to determine if the structure can tolerate the displacement (Service and Extreme Event).

16.3.5 Pile Driveability

Pile driveability refers to the ability of a pile to be driven to a desired penetration depth and/or resistance. Pile driveability shall be performed as part of the design process. When evaluating driveability, the soil disturbance during installation and the time dependent soil strength changes should be considered.

There are 3 methods available for predicting and/or checking pile driveability.

- Wave Equation Analysis
- Dynamic Testing and Analysis
- Static Load Tests

Geotechnical Resistance factors for each of these 3 methods for analysis and level of resistance determination are provided in Chapter 9.

Wave equation analysis is required during design and again during construction. The following graphic illustrates some of the variables involved with the model. The most widely accepted program is GRLWEAP, and is available at http://pile.com/pdi/. It is incumbent upon the GEOR, that prior to using any software, that the methodologies used by the software are fully understood.

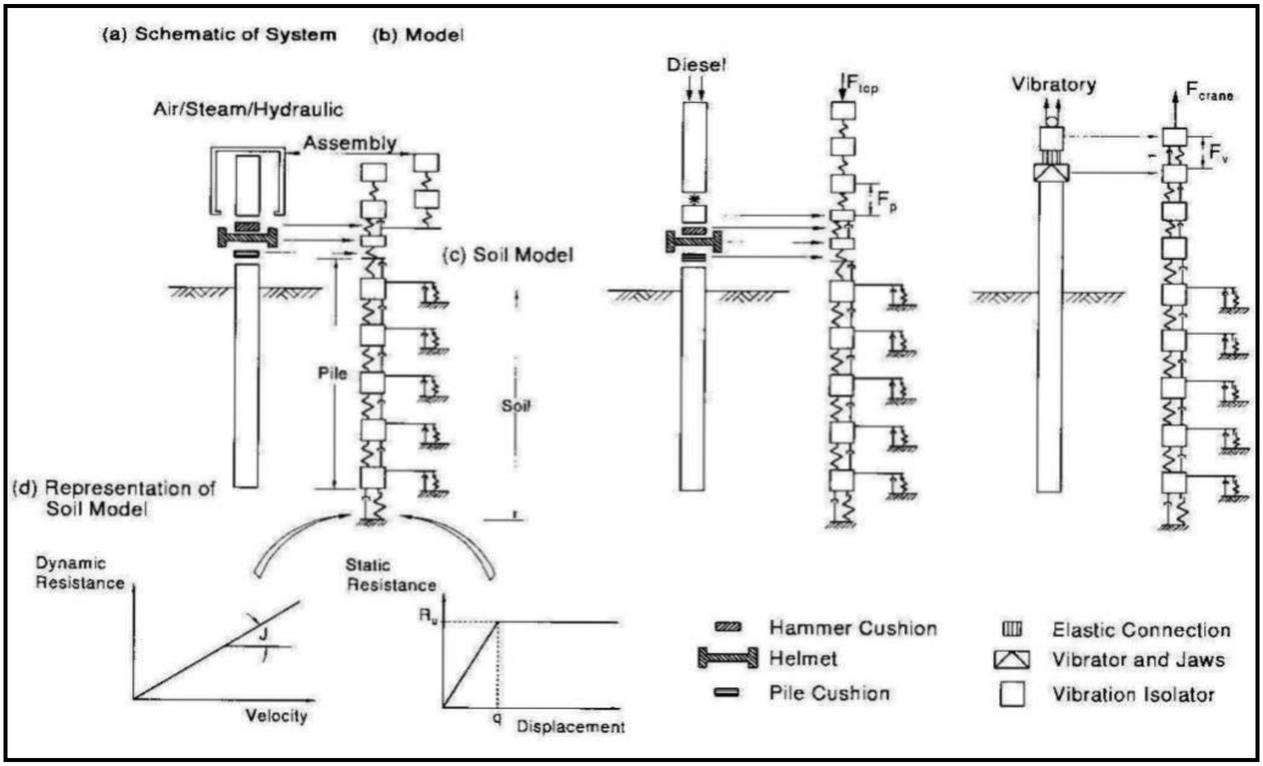


Figure 16-3, Typical Wave Equation Model (Hannigan, et al. (2006))

Some of the parameters that must be considered are hammer type, cushion material, pile properties and sizes, soil resistance distributions, soil quake and damping parameters. Some of these parameters are placed on the drawings (see Table 16-3). The wave equation is a computer simulation of the pile driving process that models wave propagation through the hammer-pile-soil system. The wave equation analysis should be used to establish the range of hammer energies, based on achieving a penetration between 36 and 180 blows per foot. The R_{nDR} (see Foundation Length) shall be used in wave equation analyses.

Table 16-3, Driveability Analysis

Skin Quake (QS)	0.10 in
Toe Quake (QT)	0.08 in
Skin Damping (SD)	0.20 s/ft
Toe Damping (TD)	0.15 s/ft
% Skin Friction	80%
Distribution Shape No.[1]	1
Resistance Distribution Model	Proportional[2]
Toe No. 2 Quake	0.15 in
Toe No. 2 Damping	0.15 s/ft
End Bearing Fraction (Toe No. 2)	0.95
Pile Penetration	80%
Hammer Energy Range[3]	25 – 60 ft-kips

[1]Distribution Shape No. varies with depth: 0 at the ground surface (creek bottom); 1 at a depth of 5ft; and 1 to a depth beyond driving depth below the ground surface.

[2]Bearing Graph options – proportional, constant skin friction, constant end bearing
Note: GRLWEAP (XXXX) was used to perform the wave equation analysis.

[3]Based on achieving a penetration rate ranging from 80 to 120 bpf; however, the SCDOT Standard Specifications for Highway Construction requires a penetration rate ranging from 36 to 180 bpf

During construction, additional wave equation analysis should be performed on the actual driving system and cushions to be used. The model should be used for checking for adequate hammer energy, establishing fuel settings, for checking compressive and tensile stresses, and to see if the penetration rate falls within a certain range. The required number of blows shall range from 36 to 180 blows per foot for the driving system to be acceptable. Practical refusal is defined as 5 blows per quarter (1/4) inch or 20 blows per inch.

Dynamic Testing and Analysis shall be in accordance with ASTM D4945. This test consists of measuring strain and acceleration near the pile top during driving, or restrike using a Pile Driving Analyzer (PDA). The PDA is used to calculate valuable information such as pile driving stresses, energy transfer, damping and quake values, and the nominal pile resistance. Additional analysis of the data collected in the field can be performed by using signal matching methods such as CAPWAP. Unlike static load testing which typically requires the cessation of pile driving, PDA testing is performed during initial pile installation as well as at some point later in time (i.e., restrike) to determine pile setup. During initial pile installation PDA testing only requires time to install the monitoring equipment. Restrikes will require some additional time to perform, but are anticipated to require less than a day for testing. PDA testing further allows for the capacity of the pile to be determined relatively quickly and allows for a determination of the stresses induced on the pile by the pile driving equipment. Additional information on the dynamic testing is provided in Chapter 24.

Static load tests are the most accurate method of determining the nominal resistance of a pile (if carried to failure). While this method accurately determines the obtained resistance and the penetration required to achieve the nominal resistance, it does not determine if there is any damage to the pile during installation. If static load testing is recommended for a project with driven piles, then dynamic testing and Wave Equation analysis will also be required. This procedure has limited applicability since static load testing requires several days to setup and perform the testing. Static load testing can add several weeks to a construction project. Optimally, static load testing should be performed as a part of the design phase of a project, when the results can more readily be used to affect the design. A comprehensive report by the FHWA on this topic is Static Testing of Deep Foundations (Kyfor, Schnore, Carlo, and Bailey (1992)).

16.4 DRILLED SHAFTS

A drilled shaft (also called drilled caisson or caisson) is a deep foundation element that is constructed by excavating a hole with power auger equipment. Reinforcing steel and concrete are then placed within the excavation. In unstable soils, casing and drilling slurry is used to maintain the stability of the hole. Drilling slurry typically consists of natural materials (i.e., bentonite); the use of polymer materials is not allowed. For certain geologic conditions (i.e., sound rock) the use of plain water (potable) as a drilling fluid is allowed; however, permission to use plain water must be obtained from SCDOT. Drilled shafts should be considered when large loads are anticipated (compressive, uplift or lateral) and where the amount of allowable deformation is small. Additionally, drilled shafts should be considered in locations where the losses due to scour are large, seismically induced downdrag loads are large or where the instability of slope cannot be maintained using conventional methods. Further drilled shafts should be considered when there is a limitation on water crossing work.

Drilled shaft sizes (diameters) can typically range from 30 inches (2-1/2 feet) to 144 inches (12 feet). Drilled shaft sizes typically used by SCDOT range from 42 inches (3-1/2 feet) to 84 inches (7 feet) in diameter. Drilled shaft diameters should be a minimum of 6 inches larger than the column above the shaft. Unless approved otherwise by the PC/GDS, all shafts shall be detailed with construction casing. The portion of the shaft below the bottom of the casing, in rock, shall be detailed with a diameter that is 6 inches smaller than the diameter of construction casing. According to the BDM drilled shafts are typically used when the span length of a bridge is greater than 50 feet.

As required by the AASHTO LRFD Specifications, the drilled shaft analyses and design should address the following:

- Nominal axial resistance of a single shaft and of a group of shafts.
- The resistance of the underlying strata to support the load of the shaft group.
- The effects of constructing the shaft(s) on adjacent structures.
- Minimum shaft penetration necessary to satisfy the requirements caused by uplift, scour, downdrag, settlement, lateral loads, SSL, and seismic conditions.
- Drilled shaft nominal structural resistance
- Satisfactory behavior under service loads
- Long-term durability of the shaft in service (i.e., corrosion and deterioration)

A thorough reference on shaft foundations is presented in the FHWA publication Drilled Shafts: Construction Procedures and LRFD Design Methods (Brown, Turner, Castelli, and Loehr (2018)).

16.4.1 Axial Compressive Resistance

There are numerous static analysis methods available for calculating the bearing resistance of a single drilled shaft. The axial compressive resistance for drilled shafts shall follow the procedures provided in the AASHTO LRFD Specifications (Section 10.8 – Drilled Shafts). The methods found in the AASHTO LRFD Specifications are used to satisfy the Strength, Service and EE limit states.

The basic LRFD equation presented previously and in Chapter 8 is expanded on the resistance side of the equation to account for the factored resistance of drilled shafts (R_r), and may be taken as:

$$\boldsymbol{Q \leq \varphi R_n = R_r = \varphi_t R_t + \varphi_s R_s} \qquad \textbf{Equation 16-8}$$

$$\boldsymbol{R_t = q_t * A_t} \qquad \textbf{Equation 16-9}$$

$$\boldsymbol{R_s = q_s * A_s} \qquad \textbf{Equation 16-10}$$

Where,

Q = Factored Load (demand)
R_r = Factored Resistance (i.e., allowable capacity)
R_n = Nominal Resistance (i.e., ultimate capacity)
R_t = Nominal End or Tip Resistance
q_t = Unit tip resistance of drilled shaft (force/area)
A_t = Area of drilled shaft tip (area)
R_s = Nominal Side Resistance
q_s = Unit side resistance of drilled shaft (force/area)
A_s = Surface area of drilled shaft side (area)
φ, φ_t and φ_s = Resistance Factors (see Chapter 9)

Where construction (permanent) casing is used, the side resistance along the length of the casing shall not be included in the nominal or factored resistances for axial compression or tension. However, any downdrag developed along the length of the cased section shall be added to the Strength, Service and EE limit state axial loads. Construction casing should normally be used on all drilled shafts in order to facilitate column construction above the shaft. If the nominal loads provided by the SEOR are located at the top of the column, then the GEOR shall add the weight of the column to the axial compressive load in order to develop the appropriate nominal resistance. However, if the SEOR provides the nominal loads at the top of the drilled shaft, then the GEOR shall not include the weight of the column.

The axial compressive design methodologies can be separated based on either total or effective stress methods or whether the soils are cohesionless (Sand-Like) or cohesive (Clay-Like) in nature. As indicated in the above equations the total axial compressive resistance of a deep foundation is based on the combination of unit side resistance and unit tip resistance values. The combination of side and tip resistance shall be settlement compatible (i.e., the settlement required to achieve side friction shall be used to develop tip resistance). The factored tip resistance shall be reduced to limit the amount of settlement of the drilled shaft; therefore, satisfying the Service limit state for the drilled shaft. See Chapter 17 for settlement analysis methods.

The following methods shall be used to determine unit side resistances in soils:

(1) α-Method: A total stress analysis used where ultimate capacity is calculated from the undrained shear strength of the soil (clay or plastic silt (Clay-Like)). This approach assumes that side resistance is independent of the effective overburden pressure and that the unit shaft resistance can be expressed in terms of an empirical adhesion factor times the undrained shear strength. The coefficient α is related to the undrained shear strength and is derived from the results of full-scale pile and drilled shaft load tests. The top 5 feet should be ignored in estimating the nominal shaft side resistance. If a construction casing is used, the shaft resistance shall be determined from the bottom of the casing to the bottom of the shaft. The maximum unit shaft resistance shall not exceed 5 ksf unless supported by load test data.

(2) β-Method: An effective stress analysis used in cohesionless (Sand-Like) soils. The unit shaft resistance is expressed as the average effective overburden pressure along the shaft times the β coefficient. This load transfer coefficient (β) is based on the effective preconsolidation pressure as determined using N_{60}-values in the design zones under consideration.

(3) Shafts in Rock: The side-wall resistance of drilled shafts in rock is based upon the uniaxial compressive strength of rock and “normal” rock sockets. “Normal” rock sockets are defined as sockets constructed using conventional equipment resulting in clean, smooth side-walls where the rock does not decompose nor is artificial roughing required. If the side-wall is roughened, then the side-wall shear will be greater. This increased side-wall resistance shall be confirmed by load testing. If the uniaxial compressive strength of the rock is greater than the concrete strength, the concrete strength shall be used in design. Factors that should be considered when applying an engineering judgement to neglect either side or tip resistance component from the total shaft resistance include but are not limited to: the presence of a rock socket, is the shaft bearing a karstic formation or if the rock strength is greater than the shaft concrete strength.

(4) Shafts in IGM: IGM is material that is transitional between soil and rock in terms of strength and compressibility and is defined in Chapter 6. Drilled shafts bearing in cohesive IGM should follow the modified α-Method contained in Brown, et al. (2018). Drilled shafts bearing in cohesionless IGM shall use the β-method.

The following methods shall be used to determine unit tip resistances in soils:

(1) α-Method: A total stress analysis method is used to determine the ultimate unit tip resistance capacity and is calculated from the undrained shear strength of a cohesive soil (clay or plastic silt (Clay-Like)). The unit tip resistance is expressed as a dimensionless bearing capacity factor times the undrained shear strength. The dimensionless bearing capacity factor (N_c) depends on the shaft diameter and the depth of embedment, and is usually assumed to be less than 9. This method limits the unit tip resistance to 80 ksf and is based on the undrained shear strength of the soil located within 2 diameters of the tip of the shaft.

(2) β-Method: The unit tip resistance is based on the average SPT N_{60} blow counts being less than or equal to 50 blows per foot (bpf). The ultimate unit tip resistance of cohesionless soils is determined using a total stress analysis method. The method is based on the N_{60} and is limited to 60 ksf.

(3) Shafts in Rock: The ultimate unit tip resistance for rock is based on the quality and strength of the rock within 2 diameters of the tip.

(4) Shafts in IGM: IGM is material that is transitional between soil and rock in terms of strength and compressibility and is defined in Chapter 6. Drilled shafts bearing in cohesive IGM should follow the modified α-Method contained in Brown, et al. (2018). Drilled shafts bearing in cohesionless IGM shall use the β-method.

As an alternate to the procedures provided for development of side and tip resistances of drilled shafts, the GEOR may elect to use historical load test data. The nominal resistance for drilled shafts may be developed based on the results of historical load test data from the anticipated load bearing stratum (i.e., the same geologic formation). The use of this type of data for development of nominal resistance shall be reviewed by the PC/GDS and the PCS/GDS. The results of more than 5 load tests shall be used to develop the resistance. Load testing shall include static, rapid and dynamic load tests. A comparison to the soils at the load test site to the soils at the new location shall be performed.

The analysis procedures discussed in the preceding paragraphs are for single drilled shafts. For some structures, drilled shafts are sometimes installed in groups. Drilled shaft groups installed in cohesive (Clay-Like) and cohesionless (Sand-Like) soils will typically have group efficiencies less than 1 with spacing's less than 6 and 4 diameters, respectively. The efficiencies of shaft groups are typically less than 1 due to overlapping zones of shear deformation and because the construction process tends to relax the effective stresses.

SCDOT recommends the φ provided in Chapter 9 for analysis for drilled shaft group capacity in Clay-Like soils. This φ is based on block failure of the Clay-Like soils, which is more due to settlement of the group. There is no group resistance factor for Sand-Like soils other than reduction required for group spacing. For additional information on the analysis of drilled shaft groups please refer to Section 10.8 in the AASHTO LRFD Specifications or the FHWA publication Drilled Shafts: Construction Procedures and LRFD Design Methods (Brown, et al. (2018)).

SHAFT v2012 (Ensoft, Inc. at http://www.ensoftinc.com/) is a windows-based program used to compute the axial resistance and the short-term, load versus settlement curves of drilled shafts in various types of soils. SHAFT v2012 can analyze drilled shaft response in 9 types of strata:

i) Sand (FHWA)
ii) Clay (FHWA)
iii) Shale (Aurora and Reese (1976))
iv) Strong Rock (FHWA, q_u > 1,000 psi)
v) Decomposed Rock/Gravel (FHWA)
vi) Weak Rock (FHWA)
vii) Strong Rock (Side friction and Tip resistance)
viii) Gravelly Sand (Rollins, Clayton, Mikesell and Bradford (2005))
ix) Gravel (Rollins, Clayton, Mikesell and Bradford (2005))

The program allows for any combination of soil layers to be placed in a layered profile. Most of the analytical methods used by SHAFT v2012 are based on suggestions from the FHWA manual Drilled Shafts: Construction Procedures and LRFD Design Methods (Brown, et al. (2010)). It is incumbent upon the GEOR, that prior to using any software, that the methodologies used by the software are fully understood.

16.4.2 Uplift Resistance

The uplift resistance should be evaluated when there are chances that upward forces may be present. The shaft side resistance should be determined from 1 of the methods presented above. The factored uplift resistance (R_r) may be evaluated by:

$$\boldsymbol{Q \leq \varphi R_n = R_r = \varphi_{up} R_s = \varphi_{up} q_s A_s} \qquad \textbf{Equation 16-11}$$

Where,

Q = Factored Load (demand)
R_r = Factored Resistance (i.e., allowable capacity)
R_n = Nominal Resistance (i.e., ultimate capacity)
R_s = Nominal Side Resistance
q_s = Unit side resistance of drilled shaft (force/area)
A_s = Surface area of drilled shaft side (area)
φ and φ_{up} = Resistance Factors (see Chapter 9)

Shaft group uplift resistance is the lesser of:

- The sum of the individual shaft uplift resistance, or
- The uplift resistance of the shaft group considered as a block.

16.4.3 Group Effects

The analysis procedures discussed in the preceding paragraphs are for single drilled shafts. For most structures, drilled shafts are installed in groups. Typically SCDOT uses frame bents (i.e., a single row of drilled shafts with a column on top of each shaft); these types of bents shall be considered to be groups for the purpose of determining group efficiency. Group effects are affected by the soil the drilled shaft is founded in; therefore, discussed below are the group effects for cohesive and cohesionless soils.

According to Brown, et al. (2018):

> For cohesive *(Clay-Like)* geomaterials in which installation of the foundations is not considered to have a significant effect on the in-situ soil and state of stress, the resistance for the geotechnical strength limit state should be determined from

the lesser of a block failure mode or the sum of the individual shaft resistances. That is, the efficiency cannot exceed 1.0 as shown in *Equation 16-12*. The nominal resistance of the block (R_{Block}) is estimated as described *below*, while the individual drilled shaft nominal resistance ($R_{n,i}$) is estimated as *discussed previously*.

$$\boldsymbol{\eta_g = \frac{R_{Block}}{\sum_{i=1}^{n} R_{n,i}} \leq 1}$$ **Equation 16-12**

Where,

R_{Block} = Nominal resistance of block (see Figure 16-4) formed by drilled shafts
$R_{n,i}$ = Nominal resistance of individual drilled shafts

R_{Block} can be estimated as the sum of the side shear resistance determined from the surface area of the block and the bearing capacity resistance determined from the block footprint area.

$$\boldsymbol{R_{Block} = f_{max} * [2D * (Z + B)] + q_{max} * (Z * B)}$$ **Equation 16-13**

Where,

f_{max} = Nominal unit side resistance of the block
q_{max} = Nominal base resistance of the block
D = Depth of the block (see Figure 16-4)
Z = Length of the block (see Figure 16-4)
B = Width of the block (see Figure 16-4)

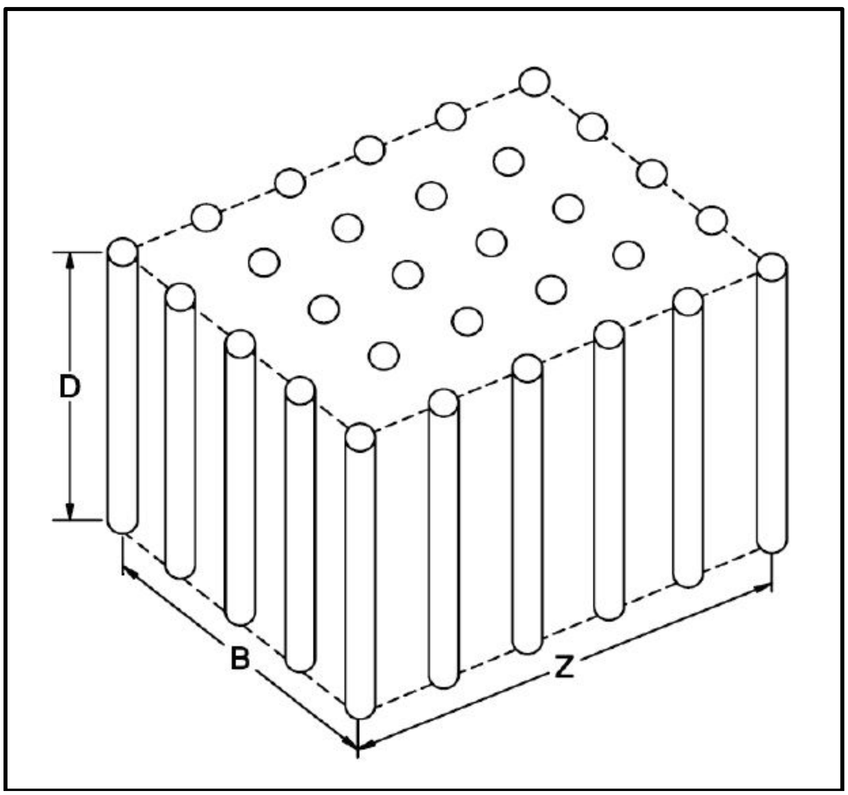


Figure 16-4, Block Failure Model
(Brown, et al. (2018))

The nominal base resistance, q_{max}, must take into account a zone of influence deeper for the block failure model than for a single drilled shaft. The DOSI from conventional shallow foundation design (see Chapter 15) will be used to determine the zone of influence of the block as well as the q_{max}.

For drilled shafts founded in Sand-Like soils the individual nominal shaft resistances of each shaft in the group is reduced by a group efficiency factor, η (i.e., the group reduction factor). Provided in the following table are the η-values to be used.

Table 16-4, Group Reduction Factor Values, η

Shaft Group Configuration	Drilled Shaft Center-to-Center Spacing	Group Reduction Factor, η
Single Row	2D	0.90
	3D or more	1.00
Multiple Row	2.5D	0.67
	3D	0.80
	4D or more	1.00

For drilled shafts founded in rock use an η of 1.0 regardless of the spacing.

16.4.4 Settlement

Settlements of single drilled shafts under axial compression loadings (Service limit state) shall be determined. Settlements determine the distribution of load caring capacity between side and tip resistances. Determine the distribution of load between the side and tip using the procedures outlined in Section 10.8 of the AASHTO LRFD Specifications.

Typically, the settlement of deep foundations is comprised of immediate and primary consolidation settlement and elastic compression (shortening). Secondary compression is not normally considered as part of the settlement of deep foundation. In many cases primary consolidation settlement is not a concern, since most deep foundations are founded in Sand-Like, overconsolidated (OCR ≥ 4) Clay-Like soils, or rock. Elastic compression is included since the deep foundation will elastically deform when a load is applied. The settlement of drilled shaft groups shall be used instead of the using the settlement for single drilled shafts. However, in some cases (i.e., hammer heads) single drilled shafts are used to support a structure. The total settlement is defined by the following equation.

$$\boldsymbol{\Delta_v = S_t = S_i + S_c + S_s + \Delta_E} \qquad \textbf{Equation 16-14}$$

Where,

S_t = Δ_V = Total Settlement
S_i = Immediate Settlement
S_c = Primary Consolidation Settlement
S_s = Secondary Compression Settlement
Δ_E = Elastic Compression

Elastic compression is the compression (deflection or shortening) of a drilled shaft caused by the application of load at the top of the drilled shaft. The elastic compression of drilled shafts is complex or difficult to determine; therefore, engineering judgment should be used in determining the elastic properties of a drilled shaft. Elastic compression should be determined using the following equation.

$$\Delta_E = k * \left(\frac{Q_a L}{AE}\right)$$ **Equation 16-15**

Where,

Q_a = Applied load
L = Drilled shaft length (embedment)
A = Cross sectional area of drilled shaft
E = Elastic modulus of drilled shaft material
k = Factor that accounts for load distribution along drilled shaft (see Table 16-5)

Table 16-5, k Factor

Loading Condition	k Factor
All End Bearing[1]	1.00
All Side Resistance	0.50
Combination of End and Side	0.67

[1]Drilled shafts founded in rock are included in this category

For drilled shafts founded in Sand-Like soils and in overconsolidated (OCR $\geq$ 4) Clay-Like soils, the settlement shall be determined using elastic theory as presented in Chapter 17. An equivalent foundation is used to determine the dimensions required. The width of the foundation (B in Figure 16-4) is either the drilled shaft diameter or the center to center of the outside shafts along the shortest side of a shaft footing (group). The length (Z in Figure 16-4) is measured from the center to center of the outside shafts along the length of the shaft frame or shaft footing. The depth of the equivalent foundation shall be 2/3 of the drilled shaft embedment depth into the primary bearing resistance layer. The applied bearing pressure (q_o) shall be taken as the sum of the drilled shaft service loads divided by the area of the equivalent footing. For each subsequent layer, the equivalent foundation is enlarged 1 horizontal to 2 vertical (1H:2V) portion until the settlement for all subsequent layers is determined.

The settlements for drilled shaft foundations placed in NC to slightly OC (1 < OCR < 4) plastic Clay-Like soils shall be determined using consolidation theory as presented in Chapter 17. Similar to the elastic settlement determination an equivalent foundation shall be placed 2/3 of the drilled shaft embedment depth into the primary bearing resistance layer and the applied bearing pressures and changes in stress are determined accordingly. The applied bearing pressure (q_o) shall be taken as the sum of the drilled shaft service loads divided by the area of the equivalent footing. For each subsequent layer, the equivalent foundation is enlarged 1 horizontal to 2 vertical (1H:2V) portion until the settlement for all subsequent layers is determined.

Once the total settlement (S_t or Δ_V) is determined, then the distribution of the load between side and end should be determined as indicated previously.

16.4.5 Constructability

The constructability of drilled shafts consists of estimating the soil and rock excavation quantities as well as estimation of the elevation of the top and bottom of the construction casing. The quantity for soil excavation should be estimated to include all materials that have an N_{meas} less than 50 blows for 6 inches of penetration (50/6") (N_{meas} < 50/6"). Note that N_{meas} is being used as opposed to N_{60} or $N_{1,60}$, since N_{meas} is the value that the contractor will have access to on the Soil Test Logs. Materials with an N_{meas} greater than or equal to 50/6" ($N_{meas} \geq$ 50/6") should be for the purposes of estimating drilled shaft rock excavation quantities. Report estimated quantities for soil and rock excavation as required in Chapter 22.

Typically the top elevation of the construction casing is estimated by the SEOR in consultation with the GEOR and is typically indicated on the construction plans. In dry environments, the top of casing elevation should be set at the ground line. In wet or fluctuating water environments, the top of casing elevation should be set 5 feet above the water elevation expected during construction. If the column supported on a drilled shaft would be less than 5 feet tall, the Contractor should be given the option, at no additional cost to SCDOT, of extending the shaft to the bottom of the bent cap. It should be noted that the estimated quantity for soil (wet and dry) excavation includes the length from the groundline to the top of the casing for this case.

The GEOR typically estimates the bottom elevation of the casing. The bottom elevation of the casing is governed by several factors including the soils encountered at the site, the anticipated loading conditions (i.e., lateral loads, scour, downdrag, etc.) and other factors determined by the project team. All construction casings should extend approximately a minimum of 20 feet beneath the original ground surface or 20 feet beneath any cut excavations required to achieve the proposed finished grade of the project, whichever is deeper. In Clay-Like soils the construction casing should extend to an $N_{1,60}$ of approximately 20 blows per foot ($N_{1,60} \sim 20$ bpf) or 20 feet as previously described, whichever is deeper. In Sand-Like soils the construction casing should extend to an $N_{1,60}$ of approximately 35 blows per foot ($N_{1,60} \sim 35$ bpf) or 20 feet as previously described, whichever is deeper. If materials with N_{meas} greater than 50/6" ($N_{meas} >$ 50/6") occur within the top 20 feet, then the casing tip can be estimated to extend 1 foot into this material.

16.5 DRILLED PILES

Drilled piles are constructed normally at end bents where the depth to rock is less than 10 to 15 feet. Drilled piles can be a subset of drilled shafts or driven piles depending on the strength of the rock. An RQD of less than 10 percent indicates that the pile may be driven; however, refusal criteria still apply (i.e., 5 blows in 1/4 inch). The capacity of the drilled pile is determined based on whether the pile is driven or not after being placed in the bore hole. Piles placed in the bore hole and not driven shall be designed using drilled shaft design procedures. This design methodology requires coordination with the SEOR to ensure that adequate load transfer from the steel to the concrete occurs. Drilled piles typically consist of steel H-piles having sizes of HP12x53 and HP14x73. The borehole should have a diameter that measures the diagonal dimension of the pile plus 6 inches to allow for the insertion of the pile and the placement of concrete. The use of concrete and combination piles is allowed only with the prior written permission of the PC/GDS. The GEOR should be prepared to adequately explain how the resistance of the pile will be evaluated and how the pile will be constructed. Drilled piles are typically used only at end bents. Prior approval of both the PC/GDS and PC/SDS shall be required prior to using drilled piles at interior bents.

16.6 CONTINUOUS FLIGHT AUGER PILES

Continuous flight auger piles (CFAs) also known as Auger Cast Piles are a new technology being considered by FHWA for transportation projects. CFAs may be used on SCDOT projects; however, CFAs should not be used to support bridges without prior approval. The use of CFAs on any SCDOT project must be approved prior to completion of preliminary design. Approval shall be in writing from either the RPE or the Preconstruction Support Engineer (PSE). In addition, the designer shall contact the PC/GDS for instructions on analytical methods for determining capacity. CFAs will range in size from 18 to 30 inches (1-1/2 to 2-1/2 feet, respectively) in diameter for SCDOT projects.

16.7 MICROPILES

The AASHTO LRFD Specifications allows for the use of micropiles to support structures. Section 10.9 of the AASHTO LRFD Specifications provides a list of when micropiles would be acceptable; however, approval by both the PC/GDS and PC/SDS shall be obtained prior to designing micropiles. The design of micropiles when allowed shall follow Section 10.9 of the AASHTO LRFD Specifications.

16.8 LATERAL RESISTANCE

Deep foundations are subjected to lateral loads from wind, traffic loading, bridge curvature, vessel or vehicular impact or seismic loadings. The lateral capacity for deep foundations may be designed using either lateral load tests or analytical methods. Full scale load tests are typically not performed and will therefore not be discussed in this Chapter. Analytical methods will be presented only as an overview. More detailed information and the theory can be found in the FHWA publication <u>Handbook on Design of Piles and Drilled Shafts Under Lateral Load</u> (Reese (1984)). According to Hannigan, Goble, Likins, and Raushce (2006),

> The design of laterally loaded piles requires the combined skills of the geotechnical (*GEOR*) and structural (*SEOR*) engineer. It is inappropriate for the geotechnical engineer to analyze a laterally loaded pile without a full understanding of pile-structure interaction. Likewise it is inappropriate for the structural engineer to complete a laterally loaded pile design without a full understanding of how pile section or spacing changes may alter soil response. <u>Because of the interaction of pile structural and geotechnical considerations, the economical solution of lateral pile loading problems requires communication between the structural and geotechnical engineer.</u> (Underline added for emphasis.)

It is therefore anticipated by SCDOT that the proper development of lateral loads and resistances will require an iterative process between the GEOR and the SEOR.

The movements or deflections associated with lateral loadings should be within Performance Limits from Service and EE limit state loadings established by the design team. These movements should account for soil parameters, pile parameters and lateral load parameters. The soil parameters consist of soil type, appropriate shear strength parameters, moisture-density relationship (unit weight), moisture content, moisture-plasticity relationship, groundwater level and the Coefficient of Horizontal Subgrade Reaction (k_s). k_s is the ratio of horizontal pressure (σ_h) per unit of vertical surface area and the corresponding horizontal displacement (Δ_h). The pile parameters consist of the physical properties of the pile (shape, material and dimensions), pile head condition (fixed or free), method of pile placement and any group action. The lateral load parameters consist of whether the load is applied statically or seismically and if the loads are applied eccentrically (i.e., moment coupled with shear forces).

Methods of analysis that use manual computation include Broms' Method which should only be used for preliminary analysis only. Reese (1984) developed a nonlinear response analysis method that models the horizontal soil resistance using P-y curves. The soil is represented as nonlinear springs distributed over the length of the pile (see Figure 16-3).

The horizontal movements determined during the foundation design stage may be analyzed using computer applications that consider soil-structure interactions. Computer programs are available for analyzing single piles and pile groups. The computer program LPILE (Ensoft, Inc.

at http://www.ensoftinc.com/) is typically used by SCDOT to determine the lateral capacity of deep foundations.

The design team performs the lateral soil-structure interaction analysis with computer programs such as LPILE or FB-Pier. The design team uses this information to compute lateral displacements and to analyze the structural adequacy of the columns and foundations. The lateral soil-structure interaction analysis is also used to select the appropriate method (point-of-fixity, stiffness matrix, linear stiffness springs, or P-y nonlinear springs) to model the bridge foundation in the structural design software. If lateral design controls the minimum point of penetration for a deep foundation, the BGER should indicate this fact. In addition, for driven piles, the nominal capacity should be increased to account for the additional installation depth required to achieve the tip elevation governed by lateral design.

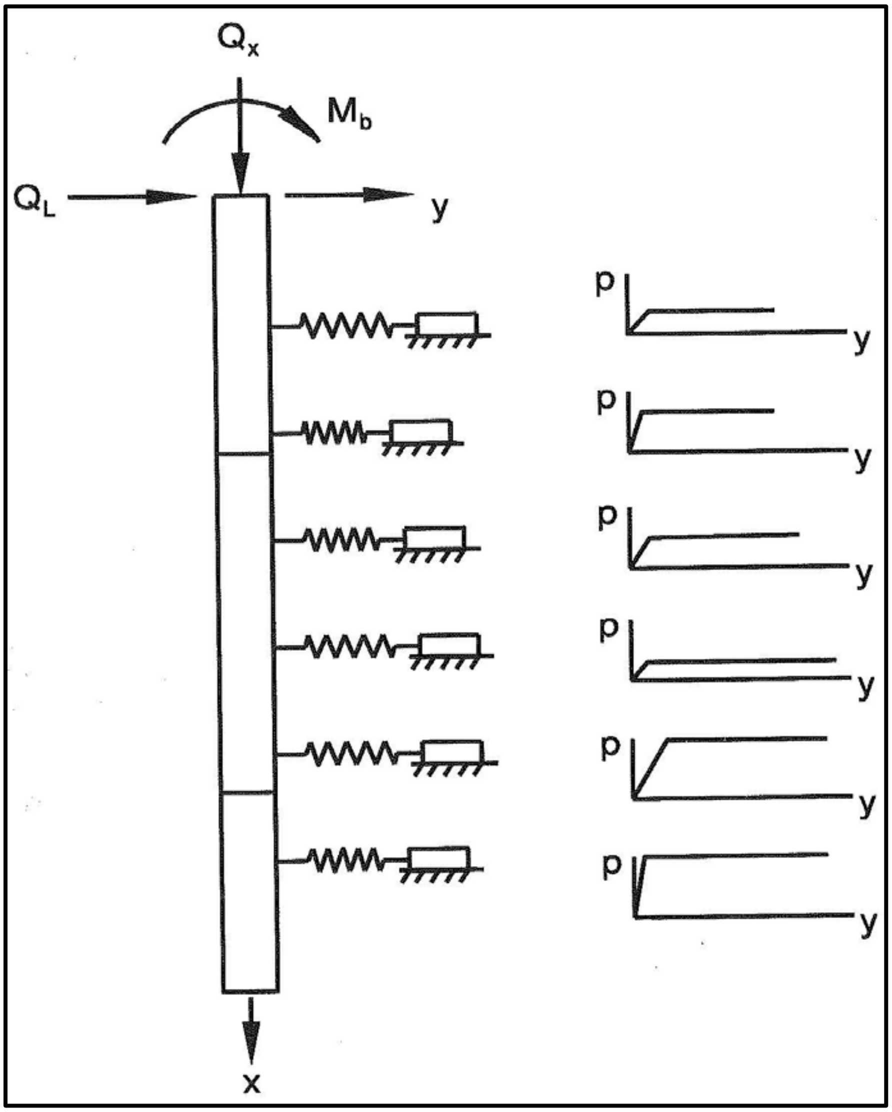


Figure 16-5, Typical LPILE Pile-Soil Model (Hannigan, et al. (2006))

According to Brown, et al. (2018) lateral designs are controlled by either geotechnical or structural strength requirements or by serviceability requirements. Each of these controlling limit states are discussed in greater detail in the following Sections.

16.8.1 Lateral Stability – Geotechnical Check

The deep foundation must be of sufficient size and depth to support the nominal design loads for each limit state (Strength and EE) checked without the potential for geotechnical failure (i.e., lateral stability is maintained). It is anticipated that the GEOR will perform this lateral stability check. For these geotechnical limit state checks deflections are not the controlling consideration. The geotechnical limit state checks shall be determined using a P-y analysis method as described by Brown, et al. (2018). The modified steps recommended by Brown, et al. (2018) are presented below:

1. Model the deep foundations as a simple linear elastic beam with the elastic modulus of concrete (E_c) (use E_s for steel deep foundations) determined as indicated in the following equation and the moment of inertia (I) equal to the uncracked cross section;

$$\boldsymbol{E_c = 33,000 * K_1 * w_c^{1.5} * \sqrt{f'_c}}$$ **Equation 16-16**

$$\boldsymbol{0.090 \leq w_c \leq 0.155}$$ **Equation 16-17**

$$\boldsymbol{f'_c \leq 15.0}$$ **Equation 16-18**

   Where,

   E_c = Elastic modulus of concrete, ksi
   E_s = Elastic modulus of steel, 29,000 ksi
   K_1 = Correction factor for aggregate source, use 1.0 unless determine by physical tests and value is provided
   w_c = Unit weight of concrete (see Chapter 8), kcf
   f'_c = 28-day cylinder strength of concrete, ksi

2. The soil is modeled using the appropriate soil parameters of each limit state (use the procedures in Chapter 7 to develop a composite profile of the site);
3. Apply various lateral loads up to and exceeding the nominal design load for the appropriate limit state thus performing a “pushover” type of analysis. For the Strength limit state exceed the nominal by at least 20 percent; no increase is required for the EE I limit state;
4. “Although deflection is not the controlling consideration for stability, the computed deflection must be a reasonable value (*e.g., 10 percent of the nominal foundation size*) at and slightly larger than the factored design loads…” (Brown, et al. (2018)), the reasonable value shall be determined by the design team;
5. The use of larger than nominal design loads at the appropriate limit state is necessary to ensure that a ductile load response exists and there is adequate reserve to account for site variability and variation in construction methods.

The deflection determined in Step 4 above is determined at the design ground line (i.e., not the top of the deep foundation and shall include the scour caused by the design flood) and is anticipated to prevent the collapse of other portions of the structure. Should this limit, as determined by the design team, be exceeded, the design team shall be informed and the design team shall decide if the deflections are tolerable. If the deflections are intolerable then the size and/or the embedment depth of the foundation should be increased and the analysis performed again. This methodology assumes that the deep foundation is free to rotate at the head. This geotechnical check may be used to determine the critical penetration depth. This is the depth at which the soil has sufficient strength to resist overturning of the foundation element.

16.8.2 Lateral Stability – Structural Check

The structural check is used to determine the resistance of the foundation element to axial, flexure (bending) and shear for all appropriate limit states. It is anticipated that the lateral capacity structural checks will be conducted by the SEOR. As with LRFD, the resistances should be greater than the nominal loads. If the resistances aren't then redesign may be required.

16.8.3 Lateral Stability – Serviceability Check

According to Brown, et al. (2018), "Deformation limits should be chosen based upon actual serviceability requirements for the structure rather than "rule of thumb" criteria." Therefore, acceptable deflections shall be determined by the design team, based on the anticipated Performance Objectives (Service and EE) of the structure. The serviceability check is conducted at the Service and EE limit state conditions and limits the deflections of the foundation element under Service and EE loads to an acceptable deflection. The deflections should be determined at the top of the column or bent cap, since deflections at this location typically exceed the deflections at the ground line. It will take the combined effort of both the SEOR and the GEOR to determine the deflections.

16.8.4 Lateral Resistance – Groups

The design method presented in the preceding Sections is for a single pile or drilled shafts. Pile bents and drilled shaft frames are typical SCDOT practice and both bents and frames are considered to be groups. Group loadings used in the P-y method of analysis require reduction to account for the "shadowing effect" of adjacent piles or shafts. Therefore, P-multipliers shall be used and determined in accordance with AASHTO LRFD Specifications Section 10.7 – Driven Piles.

16.9 DOWNDRAG

Downdrag loads (also known as Negative Skin Resistance) can be imposed on piles and shafts where:

- Sites are underlain by compressible material such as clays, silts, or organic soils,
- Fill will be or has recently been placed adjacent to the piles or shafts, such as is frequently the case for bridge approach fills,
- The groundwater is substantially lowered, or,
- Liquefaction of loose sandy soils can occur.

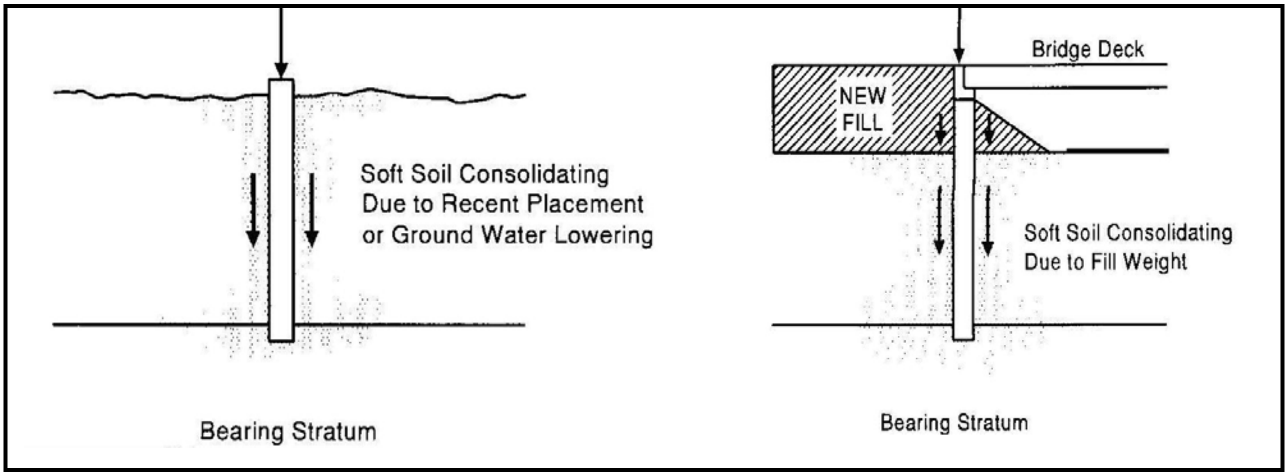


Figure 16-6, Downdrag Scenarios due to Compressible Soils (Hannigan, et al. (2006))

According to Briaud and Tucker (1997) if any of the following criteria are met, then downdrag on the deep foundation should be considered:

- Total settlement of ground surface (Δ_V) is more than 4 inches
- Settlement of ground surface after installation of foundation is more than 0.4 inches
- Embankment height exceeds 6-1/2 feet (assumed to be additional embankment height)
- Thickness of compressible layer is more than 10 feet
- The groundwater table will be permanently lowered more than 13 feet

Downdrag is typically caused by static movements (i.e., settlements) and is termed DD in the GDM. Further, downdrag may also be caused by seismic settlement resulting from SSL, specifically liquefaction of Sand-Like soils, and is termed DD_{SL} in the GDM. The AASHTO LRFD Specifications indicate that DD and DD_{SL} are not to be combined. According to the AASHTO LRFD Specifications static downdrag is considered to be a permanent load and therefore has a permanent load factor, γ_p, applied to the downdrag load. While this is appropriate for static DD, SCDOT has determined that DD_{SL} is more closely related to live loads than dead loads will therefore, apply a seismic load factor γ_{EQ} of 1.0 to DD_{SL}. It is noted that Chapter 8 indicates that γ_{EQ} is typically 0.0; however, SSL induced downdrag is the exception. Deep foundations that are anticipated having uplift loads and that experience downdrag shall use the minimum γ_p indicated in Chapter 8 for the static design method selected. There are 2 methods for determining downdrag that can be applied to both static and seismic conditions. The first method is the Traditional Approach and the second method is the Alternative Approach. Each approach is discussed in more detail in Hannigan, et al. (2006).

16.9.1 Traditional Approach

The Traditional Approach assumes that the deep foundation does not move relative to the soil column (i.e., Δ_V of the deep foundation is equal to or less than 0.4 inches). Therefore all settlement is used to develop drag loads on the deep foundation. The appropriate static method and γ_p corresponding to the individual soil layers are used to develop the downdrag. DD is added to both the Strength and Service limit state loads as indicated in the following equations:

$$\boldsymbol{R_{nST}} = \frac{\sum_1^i \gamma_{pi} * Q_{STi}}{\varphi_{dyn}} + \frac{\gamma_p * DD}{\varphi_{dyn}}$$ **Equation 16-19**

$$R_{nSV} = \frac{\sum_1^i \gamma_{pi} * Q_{SVi}}{\varphi_{dyn}} + \frac{\gamma_p * DD}{\varphi_{dyn}} \qquad \textbf{Equation 16-20}$$

Where,

R_{nST} = Nominal resistance at the Strength limit state
γ_{pi} = Permanent load factor for each force effect
Q_{STi} = Force effect at the Strength limit state
γ_p = Permanent load factor applied to downdrag load, DD
DD = Downdrag load
φ_{dyn} = Resistance factor based on the use of dynamic construction control
R_{nSV} = Nominal resistance of the Service limit state
Q_{SVi} = Force effect at the Service limit state

Typically R_{nST} and R_{nSV} are provided by the SEOR after the GEOR has provided the factored downdrag load (γ_p * DD). It is noted that R_{nSV} is used to determine if the deep foundation settles.

Similarly to the statically induced DD loads, DD_{SL} is caused by settlement induced by liquefaction of Sand-Like soils. It is not anticipated that DD_{SL} will be caused by the loss of shear strength in Clay-Like soils. As with the DD, it is assumed that the deep foundation does not settle. For those soil layers that undergo SSL, residual shear strengths shall be used to determine DD_{SL} while for those soils not affected by SSL peak shear strength shall be used in the determination DD_{SL}. The nominal resistance for the EE I limit state is determined using the following equation.

$$R_{nEEI} = \frac{\sum_1^i \gamma_{pi} * Q_{EEIi}}{\varphi_{EQ}} + \frac{\gamma_{EQ} * DD_{SL}}{\varphi_{EQ}} \qquad \textbf{Equation 16-21}$$

Where,

R_{nEEI} = Nominal resistance at the EE I limit state
γ_{pi} = Permanent load factor for each force effect
Q_{EEIi} = Force effect at the EE I limit state
γ_{EQ} = Seismic load factor applied to downdrag load, DD_{SL}
DD_{SL} = Downdrag load induced by SSL
φ_{EQ} = Seismic resistance factor

Typically the SEOR will provide the summation of γ_{pi} and Q_{EEIi} as the EE I load and the GEOR will add the factored DD_{SL} to determine the EE I limit state nominal resistance (R_{nEEI}).

16.9.2 Alternative Approach

As indicated previously, Hannigan, et al. (2006) provides an Alternative Approach to developing downdrag loads on deep foundations. Briaud and Tucker (1997) presented the Alternative Approach in NCHRP Report 393 – Design and Construction Guidelines for Downdrag on Uncoated and Bitumen-Coated Piles. The basic concept is that since both the soil and deep foundation are moving in the same direction no downdrag loads are developed along the shaft of the deep foundation. This approach is also called the Neutral Point or Neutral Plane method, since a neutral plane is developed where the settlement of the deep foundation exceeds the settlement of the soil (see Figure 16-5). For a detailed procedure of how to use the Alternative Approach see Hannigan, et al. (2006). Please note that in order to use the Alternative

Approach, the deep foundation must settle into the subsurface soils. It is anticipated that piles with appreciable end bearing will not settle sufficiently; therefore, the Alternative Approach should only be used for friction piles. The amount of settlement required to develop tip resistance in drilled shafts may allow for the use of the Alternative Approach; however, the Service limit state check of the structure may not allow this approach.

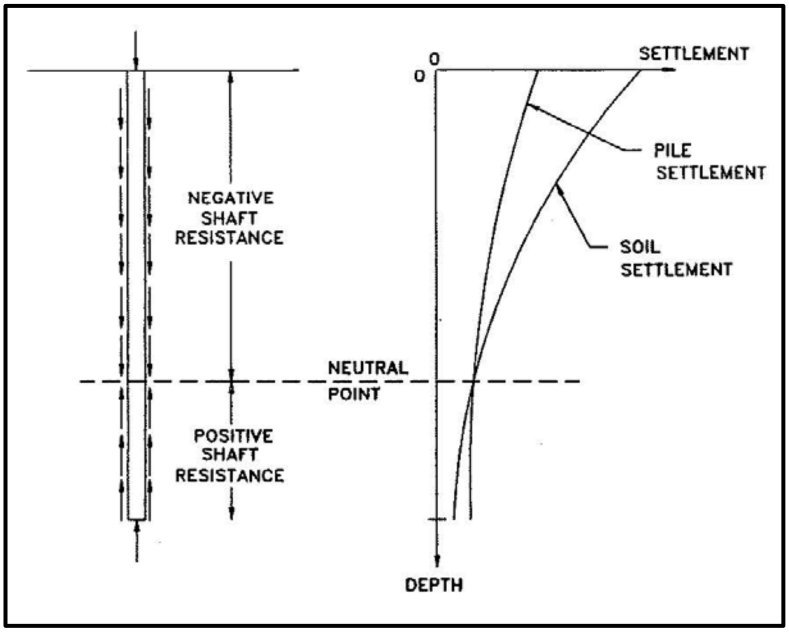


Figure 16-7, Neutral Point Determination (Briaud and Tucker (1997))

If during the EE I event SSL occurs in soil layers above the location of the Neutral Point before the EE I event, then SSL will have limited effect on the deep foundation. If SSL occurs in soil layers below the pre-EE I event Neutral Point, it will increase the axial compression load (i.e., downdrag will occur) in the deep foundation as well as result in additional deep foundation settlement.

Each of the downdrag loads discussed previously are used to determine the length of deep foundation required to resist the respective nominal load (i.e., R_{nST}, R_{nSV} or R_{nEQ}).

16.9.3 Downdrag Mitigation

The effect of DD and DD_{SL} can be mitigated through the use of embankment surcharge loads, ground improvement techniques, and/or vertical drainage and settlement monitoring measurements. In addition, either coatings or sleeves/jackets may be applied to the piles allowing the soil to slide adjacent to the piles.

16.10 FOUNDATION LENGTH

The BGER is used to report the geotechnical resistances that should be used in the design of foundations for bridges and bridge related structures. For drilled shaft/pile bents and drilled shaft/pile group footings, the BGER provides estimated pile/shaft tip elevations, the minimum pile/shaft tip elevations required to maintain lateral stability (critical penetration), and the necessary soil parameters to develop a P-y soil model of the subsurface that is used in performing foundation lateral soil-structure interaction analyses. The estimated tip elevations shall be established using $R_{nDR(ST)}$, $R_{nDR(SV)}$ (not anticipated to control), $R_{nDR(EEI)}$ or $R_{nDR(EEII)}$.

$R_{nDR(ST)}$ and $R_{nDR(SV)}$ shall account for the effects of scour caused by the design flood, while $R_{nDR(EEI)}$ shall account for any losses due to liquefaction of Sand-Like soils and $R_{nDR(EEII)}$ shall account for the effects caused by the check flood or impact (vehicular or vessel) loadings. Soil layers that are anticipated to scour or undergo SSL shall not be included in the determination of resistance; however, these soils shall be included in the determination of nominal required driving resistance, R_{nDR} as indicated in the following equations:

$$\boldsymbol{R_{nDR(ST)}} = \frac{\sum_1^i \gamma_{pi} * Q_{STi}}{\varphi_{dyn}} + \frac{\gamma_p * DD}{\varphi_{dyn}} + R_{designfldscr} + R_{DD}$$ **Equation 16-22**

$$\boldsymbol{R_{nDR(SV)}} = \frac{\sum_1^i \gamma_{pi} * Q_{SVi}}{\varphi_{dyn}} + \frac{\gamma_p * DD}{\varphi_{dyn}} + R_{designfldscr} + R_{DD}$$ **Equation 16-23**

$$\boldsymbol{R_{nDR(EEI)}} = \frac{\sum_1^i \gamma_{pi} * Q_{EEIi}}{\varphi_{dyn}} + \frac{\gamma_{EQ} * DD_{SL}}{\varphi_{dyn}} + R_{DD(SL)}$$ **Equation 16-24**

$$\boldsymbol{R_{nDR(EEII)}} = \frac{\sum_1^i \gamma_{pi} * Q_{EEIIi}}{\varphi_{dyn}} + R_{checkfldscr}$$ **Equation 16-25**

Where,

$R_{nDR(ST)}$	=	Required driving resistance at the Strength limit state
$R_{nDR(SV)}$	=	Required driving resistance of the Service limit state
$R_{nDR(EEI)}$	=	Required driving resistance at the EE I limit state
$R_{nDR(EEII)}$	=	Required driving resistance at the EE II limit state
γ_{pi}	=	Permanent load factor for each force effect
γ_p	=	Permanent load factor applied to downdrag load, DD
γ_{EQ}	=	Seismic load factor applied to downdrag load, DD_{SL}
Q_{STi}	=	Force effect at the Strength limit state
Q_{SVi}	=	Force effect at the Service limit state
Q_{EEIi}	=	Force effect at the EE I limit state
Q_{EEIIi}	=	Force effect at the EE II limit state
DD	=	Downdrag load
DD_{SL}	=	Downdrag load induced by SSL
φ_{dyn}	=	Resistance factor based on the use of dynamic construction control
$R_{designfldscr}$	=	Unfactored soil resistance from soils scoured by the design flood
$R_{checkfldscr}$	=	Unfactored soil resistance from soils scoured by the check flood
R_{DD}	=	Unfactored soil resistance from soils that undergo static settlements
$R_{DD(SL)}$	=	Unfactored soil resistance from soils that undergo SSL induced settlement at the EE I limit state

As part of the design process the GEOR shall determine the anticipated minimum tip elevation required to achieve the required driving capacity (i.e., $R_{nDR(ST)}$, $R_{nDR(SV)}$, $R_{nDR(EEI)}$ or $R_{nDR(EEII)}$). The report shall clearly indicate the governing conditions for development of the tip elevation using the words depicted in Table 16-6.

Table 16-6, Governing Conditions

Limit State	Loading Direction
Strength or Service	Axial (Compression or Tensile)
Extreme Event I or II	Lateral

Each governing condition shall consist of a loading type and a loading direction (i.e., Extreme Event I Lateral or Strength Axial). In addition to indicating which governing condition was used to develop the minimum tip elevation, the report shall also include a loading table that will provide the information depicted in Table 16-7, Pile Resistance or Table 16-8, Drilled Shaft Resistance.

Table 16-7, Pile Resistance

	Strength or Service Limit State[1,2]	EE I or EE II Limit State[1,3]
Factored Design Load	112 kips[4]	152 kips[4]
Geotechnical Resistance Factor[5]	0.40	1.00
Nominal Resistance	280 kips	152 kips
Resistance from: Design Flood Scourable Soils[6] Soils undergoing static downdrag[6]	 40 kips 0 kips	 NA
Resistance from Liquefiable Soils[7]	NA	220 kips
Required Driving Resistance	320 kips	372 kips

[1]Use only 1 column; middle column represents static resistance while last column represents Extreme Event resistance. Use the column that governs driving resistance.

[2]Indicate whether Strength or Service limit state controls resistance

[3]Indicate whether EE I or EE II limit state controls resistance

[4]Factored design loads include DD or DD_{SL}. Note that in this example the Strength limit state DD = 0.0 kips

[5]Use appropriate construction control resistance factor

[6]Design flood scour and static downdrag are not included with Extreme Event limit state loading conditions

[7]Full resistance that is developed by soils within the liquefiable zone during pile installation

The R_{nDR} is used to determine the driving resistance (see Pile Driveability above) and acceptability of the driving equipment. Depending on the controlling condition, the piles will be driven to a higher capacity than required to achieve the Nominal Resistance and the Pile Driveability analysis shall account for this higher required resistance. Alternatively the driving resistance could be the Resistance required to achieve a minimum tip elevation. The minimum tip elevation is typically governed by the geotechnical lateral stability of the pile, but may also be the tip required to limit the amount of settlement of the pile. If settlement controls the minimum tip elevation, contact the design team to discuss the effects of the settlement. In addition, this may affect the pile driving equipment that a contractor selects.

Table 16-8, Drilled Shaft Resistance

	Strength or Service Limit State[1,2]	EE I or EE II Limit State[1,3]
Factored Design Load	1400 kips[4]	1400 kips[4]
Factored Resistance – Side	1130 kips	1130 kips
Factored Resistance – End	270 kips	270 kips
Geotechnical Resistance Factor – Side[5]	0.50	1.0
Geotechnical Resistance Factor – End[5]	0.50	1.0
Total Nominal Resistance	2800 kips	1400 kips

[1]Use only 1 column; middle column represents static resistance while last column represents Extreme Event resistance, use the column that governs resistance

[2]Indicate whether Strength or Service limit state controls resistance

[3]Indicate whether EE I or EE II limit state controls resistance

[4]Factored design loads include DD or DD_{SL}. Note that in this example the Strength limit state DD = 0.0 kips

[5]Use appropriate construction control resistance factor for static and φ_{EQ} equal to 1.0 for seismic

Please note that the weight of a drilled shaft is not subtracted from the nominal capacity, since the geotechnical resistance factors were obtained from static load tests. Therefore the resistance factors already account for the weight of the shaft in both compression and tension. However, depending on where the loads are applied, the weight of the column above the drilled shaft shall be added to the axial load. The column weight is added if the loads are applied at the top of the column, however, if the loads are applied at the top of the shaft, the column weight is not added. The factored column weight shall be determined by the SEOR and provided to the GEOR. In addition, the SEOR shall indicate where the loads are applied on the load data sheet.

If the Downdrag loads exceed the Nominal Resistance of the deep foundation, then additional length will be required. For driven piles this additional length shall be accounted for in the R_{nDR}. For drilled shafts the tip elevation shall be changed to reflect this increase and a Total Nominal Resistance shall be indicated on the plans.

16.11 REFERENCES

American Association of State Highway and Transportation Officials, (2017), AASHTO LRFD Bridge Design Specifications Customary U.S. Units, 8th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

Aurora, R. P., and Reese, L. C., (1976), Behavior of Axially Loaded Drilled Shafts in Clay-Shales, CFHR 3-5-72-176-4, Texas Department of Highways and Public Transportation, Austin, Texas.

Briaud, J.-L., and Tucker, L, (1997), “Design and Construction Guidelines for Downdrag on Uncoated and Bitumen-Coated Piles”, NCHRP Report 393, Transportation Research Board.

Brown, D. A., Turner, J. P., and Castelli, R. J., (2010), Drilled Shafts: Construction Procedures and LRFD Design Methods, (Publication No. FHWA NHI-10-016), National Highway Institute, U.S. Department of Transportation, Federal Highway Administration, Washington D.C.

Brown, D. A., Turner, J. P., Castelli, R. J., and Loehr, E. J., (90% DRAFT 2018), Drilled Shafts: Construction Procedures and LRFD Design Methods, (Publication No. FHWA NHI-18-XX), National Highway Institute, U.S. Department of Transportation, Federal Highway Administration, Washington D.C.

DeMarco, M., Bush, P., Samtani, N. C., Kulicki, J. M., and Severns, K., (2015), Draft - Incorporation of Foundation Deformations in *AASHTO LRFD* Bridge Design Process, 2nd Strategic Highway Research Program (SHRP2), Transportation Research Board, The National Academies of Sciences, Engineering, and Medicine, Washington, D.C.

Department of Defense, Department of the Navy, Naval Facilities Engineering Command, (1986), Foundations & Earth Structures – Design Manual 7.2, (Publication No. NAVFAC DM-7.02), Alexandria, Virginia.

Hannigan, P. J., Goble, G. G., Likins, G. E., and Rausche, F., (2006), Design and Construction of Driven Pile Foundations – Reference Manual - Volume I and II, (FHWA Publication Nos. FHWA-NHI-05-042 and FHWA-NHI-05-043). U.S. Department of Transportation, National Highway Institute, Federal Highway Administration, Washington D.C.

Kyfor, Z. G., Schnore, A. R., Carlo, T. A. and Bailey, P. F., (1992), Static Testing of Deep Foundations, (Publication No. FHWA-SA-091-042), U.S. Department of Transportation, Office of Technology Applications, Federal Highway Administration, Washington D.C.

Reese, L. C., (1984), *Handbook on Design of Piles and Drilled Shafts Under Lateral Loads*, (FHWA Publication No. FHWA-IP-84-11). U.S. Department of Transportation, Federal Highway Administration, Washington D.C.

Rollins, K. M., Clayton, R. J., Mikesell, R. C., and Bradford, B. C., (2005), "Drilled Shaft Side Friction in Gravelly Soils." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Volume 131, Issue 8, pp. 987-1003.

South Carolina Department of Transportation, (2006), Bridge Design Manual, South Carolina Department of Transportation, https://www.scdot.org/business/structural-design.aspx.