Chapter 17 - Foundation Design

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17.1 General, Standards And Policies

This chapter covers the geotechnical design of foundations. Which includes abutment resistance for bridges, shallow (spread footings) and deep (driven piles and drilled shaft) foundations, traffic structures, illumination, camera poles, sound walls and buildings. Foundation design requires performing an office study, obtaining an appropriate level of subsurface exploration information for design and construction, performing foundation analyses and providing written recommendations in a report for the designer, the project team and the contractor. See Chapter 3 for guidance on foundation information available through office studies and the procedures for conducting a thorough site reconnaissance. See Chapter 4 for requirements for exploration for foundation design. See Chapter 19 for foundation reporting requirements.

Unless otherwise stated in this manual, the Load and Resistance Factor Design approach (LRFD) shall be used for all foundation design projects, as prescribed in the most current version of the *AASHTO*. The ODOT foundation design policies and standards described in this chapter supersede those in the AASHTO LRFD specifications and FHWA design manuals. FHWA design manuals are encouraged for use in foundation design procedures and preferable in cases where foundation design procedures are not adequately provided in AASHTO. Structural design of bridge foundations, and other structure foundations, is addressed in the *ODOT Bridge Design Manual (BDM)*.

17.1.1 Definitions

Auger Cast Piles – also known as continuous flight auger (CFA) or drilled displacement pile "are a type of drilled foundation in the pile is drilled to the final depth using a continuous flight auger. As the auger is withdrawn from the hole concrete or grout is placed.

Cast-In-Place Piles – a predrilled excavation reinforced with a pile section that is concreted in-place. Sometimes referred to as a prebored pile.

Cyclic Direct Simple Shear (CDSS) Test – a shear strength test for evaluating the ability of soil to resist shear stresses induced in a soil mass during earthquake loading.

Driven Piles – a slender deep foundation, wholly or partly embedded in the ground, that is installed by driving, or otherwise and that derived its capacity from the surrounding soil and/or from the soil or rock strata below its tip. (AASHTO).

Drilled Shafts – a deep foundation unit, wholly or partly embedded in the ground, constructed by placing fresh concrete in a drilled hole with or without steel reinforcement. Drilled shafts derive their capacity from the surrounding soil and/or from the soil or rock strata below its tip. Drilled shafts are also commonly referred to as caissons, drilled caissons, bored piles, or drilled piers (AASHTO).

Footings – is an enlargement of the base of a column or wall for the purpose of transmitting the load to the subsoil" (Peck, Hanson, Thornburn, 1974).

Foundation – is part of a structure which has the primary function of transmitting loads from the structure to the natural ground. (Perloff and Baron, 1976).

Micropiles – a small-diameter drilled and grouted non-displacement pile (normally less than 12-in diameter) that is typically reinforced (AASHTO).

Spread Footing – also known as a shallow foundation it derives its support by transferring load directly to the soil or rock at a shallow depth (AASHTO).

17.1.2 Foundation Design Standards

The following items are highlights of items that need additional time and attention during development and are listed below. In-depth design procedures are outlined in each individual sub-section. These highlights are here to hopefully bring clarity and draw attention to anomalies in the design of these items.

17.1.2.1 Drilled Shafts Greater than 6' in Diameter

Based on the high risk exposure to the Agency of high load carrying foundations the geotechnical investigation, design, integrity and load testing require augmented review by the State Geotechnical Engineer. Drilled shaft design greater than 6' in diameter is required to be submitted at each phase gate, to State Geotechnical Engineer for review and concurrence. This provides time during project development to ensure appropriate subsurface investigation, design, and incorporate appropriate level of quality control during construction. Documentation expected for review at each phase gate includes: plans, loads at limit states, estimated resistance plots, calculation book documenting methods, calculations, assumptions, and resistance factors at each limit state, construction quality control measures, and how loads will be verified. Statewide reviews with comments will be documented in the quality folder of the project and plan to respond within two weeks of receipt.

17.1.2.2 Augercast Piles

Augercast piles can be very cost effective in certain situations. However, they present significant challenges with respect to verifying integrity and capacity. Therefore, it is ODOT current standard not to use augercast piles for bridge foundations.

17.1.2.3 Cast-In-Place Piles

Cast-in-place piles may appear to be cost effective and easy to construct. However, they present significant challenges with respect to design, and use of consistent design methodology between the Geotechnical Engineer and the Bridge Engineer. During construction, verification of integrity and capacity is not possible thus producing a foundation of unknown quality and unknown integrity with unknown capacity. Therefore, it is ODOT current standard not to use cast-in-place piles for bridge foundations.

17.1.2.4 CDSS Testing

Studies of Willamette Silt in Western Oregon were initiated in the mid-1990's and continue in an effort to determine the cyclic response of these unique soils which underlie the majority of Oregon's population. To better understand these soils specific sampling, and testing criteria is required to bolster the existing dataset of Willamette Silt data. If an ODOT STIP project can justify the cost of testing (~$20k) with savings in project costs. Until recently, CDSS testing availability for ODOT projects was limited to resources outside the Country. Currently, there are several consulting firms and two Universities in Oregon that are able to perform this testing.

A paired mud rotary and CPT are required for site investigation. Undisturbed sampling, storage, and transport to the laboratory require careful handling as these transitional soils are subject to easy disturbance.

Testing protocol requires the following tests to be performed for each sample: index tests, soil classification with particle size distribution, constant rate-of-strain consolidation test where $\sigma'_{vo} = \sigma'_{vc}$, a minimum of four constant-volume, monotonic direct simple shear tests over a range of OCRs from 1 to 8, and a minimum of four constant-volume, stress-controlled, Cyclic Direct Simple Shear (CDSS) tests. All test results in the raw data form, in excel format, are stored in ProjectWise with the associated project. Geotechnical Reporting Documents will include the laboratory test results, procedures, interpretation and application for each project.

If you have questions regarding the testing protocol requirements, data storage, interpretation, reporting requirements or application do not hesitate to contact the Senior Geotechnical Engineer at (503-428-1344). All paper and electronic files from these laboratory tests are retained in projectwise. Approach Fill Design And Use Of Passive Pressure

17.1.2.5 Drilled Shaft Base Tip Grouting

Shaft base grouting is a relatively new shaft construction technique in the U.S. and reliable consistent methods of performance, and construction are not vetted with standardized designs, guidelines, and practices. Therefore, it is ODOT's standard not to use base-tip grouting on ODOT projects.

17.1.2.6 Downdrag Loads

If a downdrag condition exists, follow the neutral plane design procedure outlined in GEC-10 (Brown and Castelli, 2010). The load factors for downdrag loads provided in Table 3.4.1-2 of the AASHTO shall be used for the strength limit state. However, this table does not address the situation in which the soil contributing to downdrag in the strength limit state consists of sandy soil, the situation in which a significant portion of the soil profile consists of sandy layers, nor the situation in which the CPT is used to estimate downdrag loads and the pile bearing resistance. Therefore, the portion of Table 3.4.1-2 in AASHTO that addresses downdrag loads has been augmented to address these situations as shown in Table 17-1).

Table 17-1 Type of Load, Foundation Type, and Method Used to Calculate Downdrag

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Piles, Nordlund Method, or Nordlund and λ Method	1.1	0.35
	Piles, CPT Method	1.1	0.40
	Drilled Shafts, O'Neill and Reese Method (WSDOT).	1.25	0.35

17.1.2.7 Timber Piles

Do not use timber piles.

17.1.2.8 Pre-stressed Concrete Piles

Do not use pre-stressed concrete piles.

17.1.3 Scour Design

Foundation design for the scour condition associated with the base flood (typ. 100-yr. event) is the same as the "no-scour" condition. Factored foundation resistances must be adequate to resist the factored loads associated with the strength and service limit states (AASHTO, Article 3.7.5). For the check flood condition the foundations must provide nominal bearing resistances (resistance factor equal to 1.0) sufficient to support the structure loads associated with the Extreme Limit State II (AASHTO, Article 10.5.5.3.2).

17.1.4 Traffic Structures

Various versions of the *"AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals"* are in effect and refer to *"AASHTO Standard Specifications for Highway Bridges"*. The design approach used for the foundation design must be consistent with the design approach used for the structure. At this time monotube VMS, sign bridges, and signal poles use three different standards. The table below provides the current standard in effect, associated standard drawings, standard foundation drawings, and special provisions.

Table 17-2 Traffic Structures Standards

Structure Type	Standard	Design Method	Standard Drawings	Standard Foundation Drawings	Special Provision
Monotube VMS	2017, "*AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.*"	LRFD	TM621 – TM628	TM627 and TM628	00921
Sign Bridges - Truss	1996, "*AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.*"	ASD	TM606 – TM620	TM611/TM619	00920
Sign Bridges - Monotube	2017, "AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals."	LRFD	TM627, TM628, TM693-TM697	TM627 and TM628	00921
Signal Poles SM1-SM5L	2003, "*AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.*"	ASD	TM650 – TM653	TM653	00963
Signal Poles SM6L-SM7L	2017, "*AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.*"	LRFD	TM655 – TM658	TM628	00921
Luminaires	1994, "*AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.*"	ASD	TM630	TM630	00962
Camera Poles	2009, "AASHTO LRFD Specifications	LRFD	DET4640	N/A	SPS 00965

Structure Type	Standard	Design Method	Standard Drawings	Standard Foundation Drawings	Special Provision
	for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.”				
High Mast Luminaires	2017, “AASHTO LRFD Standard Specifications for Bridge Design”	LRFD	N/A	N/A	00512 or 00921

17.1.4.1 Mast Arm Signal Poles

The Rutledge Method described in the AASHTO specifications is **not** an approved method for the design of signal pole drilled shaft foundations.

Mast arm signal pole foundations 60′ and greater are designed to the most recent edition of *AASHTO LRFD Bridge Design Specifications*. Section 17.9 of this chapter describes acceptable analysis methods to meet foundation design requirements.

17.1.5 End Bents

Historically a one-foot neat-line with drain material has been used. This detail allows for easy calculation of the excavation and drain material quantities. The detail does not provide limits for the backfill at the end bents and wing walls and while the specifications require granular structure backfill there is not consistent direction for the extent of the backfill. Thus, there are no assurances that the designed lateral earth pressures are achieved in construction.

For end bents, the lateral load of the bridge end fill must be considered in designing the end bent by both the Geotechnical Engineer and the Bridge Engineer. To more consistently model the behavior of the bridge and to ensure the design loads are constructed Standard Detail 3160 has been developed for use by the Geotechnical Engineer to provide relevant recommendations to the Bridge Engineer. The Geotechnical Engineer is responsible for providing the Bridge Engineer load diagrams and associated geotechnical notes.

Calculate and report active, at-rest, and passive lateral earth pressures in accordance with lateral earth pressure theory as provided in AASHTO 3.11.5.

Abutment type plays a large role in the Geotechnical Engineer’s recommendations. Both active and passive lateral earth pressures requires movement/mobilization minimum amount is specified AASHTO Table C.3.11.1-1. Generally, abutments that will meet this requirement are integral, semi-integral, stub, and single-row pile caps. These abutment types are allowed and designed to move longitudinally. Therefore, active earth pressure is appropriate for design.

Stiff abutment walls, such as those required for spread footings, drilled shafts greater than 3-ft in diameter, and piles with multiple rows of piles will not move based on the required structural stiffness. In this case recommendations using at-rest lateral earth pressures are appropriate for design.

Bridge designers are allowed to up to 70% of the passive earth pressure (Earthquake Restraining Systems and Earthquake Resisting Elements) as a method to dissipate energy during a seismic event if the horizontal seismic ground shaking can engage the passive pressure. It is the Geotechnical Engineer’s responsibility to determine and provide the passive lateral earth pressure and provide the values, minimum mobilization criteria, and earth pressure diagram to the bridge engineer.

17.2 Foundation Selection Criteria

The foundation type selected for a given structure should result in the design of a buildable, economical foundation, taking into account any constructability issues and project constraints. The Geotechnical Memo and Geotechnical Report documents the suitability of each foundation type to meet the performance criteria as well as project constraints. The selection of the most suitable foundation for the structure is based on the following considerations:

- The ability of the foundation type to meet performance requirements (e.g., deformation, bearing resistance, uplift resistance, lateral resistance/deformation) for all limit states including scour and seismic conditions.
- The constructability of the foundation type (taking into account issues like traffic staging requirements, construction access, shoring required, cofferdams).
- The cost of the foundation and cost of seismic mitigation for the foundation.
- Meeting the requirements of environmental permits (e.g. in-water work periods, confinement requirements, noise or vibration effects from pile driving or other operations, hazardous materials).
- Constraints that may impact the foundation installation (e.g., overhead clearance, access, surface obstructions, and utilities).
- The construction and post-construction impacts of foundation construction on adjacent structures, or utilities,
- The impact of the foundation installation (in terms of time and space required) on traffic and right-of-way.

This is the most important step in the foundation design process. These considerations should be discussed with the structural designer and documented in the Geotechnical Memo and Report. Bridge bent locations may need to be adjusted based on the foundation conditions, construction access or other factors described above to arrive at the most economical and appropriate design.

17.2.1 Spread Footings

Spread footings are typically very cost effective, given the right set of conditions. Spread footings work best in hard or dense soils or rock where there is adequate bearing resistance and provide tolerable settlement under load. Spread footings can get rather large depending on the structure loads and settlement requirements. Structures with tall columns or with high lateral loads which result in large eccentricities and spread footing uplift loads may not be suitable candidates for spread footing designs. Spread footings are not allowed where soil liquefaction can occur at or below the spread footing level. Other factors that affect the cost feasibility of spread spread footings include:

- The need for a cofferdam and seals when placed below the water table,
- The need for significant over-excavation and replacement of unsuitable soils,
- The need to place spread footings deep due to scour, liquefaction or other conditions,
- The need for significant shoring to protect adjacent existing facilities, and
- Inadequate overall stability when placed on slopes that have marginally adequate stability.

Settlement (service limit state criteria) often controls the feasibility of spread footings. The amount of spread footing settlement must be compatible with the overall bridge design. The superstructure type and span lengths usually dictate the amount of settlement the structure can tolerate and spread footings may still be feasible and cost effective if the structure can be designed to tolerate the estimated settlement (e.g., flat slab bridges, bridges with jackable abutments, etc.). Spread footings may not be feasible where expansive or collapsible soils are present near the bearing elevation. Refer to the FHWA Geotechnical Engineering Circular No. 6, *Shallow Foundations* (Kimmerling, 2002), and the FHWA publication, *Selection of Spread Footings on Soils to Support Highway Bridge Structures* (Samatini, 2010) for additional guidance on the selection and use of spread footings.

17.2.2 Deep Foundations

Deep foundations are the next choice when spread footings cannot be founded on competent soils or rock at a reasonable cost. Deep foundations are also required at locations where spread footings are unfeasible due to extensive scour depths, liquefaction or lateral spread problems. Deep foundations may be installed to depths below these susceptible soils to provide adequate foundation resistance and protection against these problems. Deep foundations should also be used where an unacceptable amount of spread footing settlement may occur. Deep foundations should be used where right-of-way, space limitations, or other constraints as discussed above would not allow the use of spread footings.

The two types of deep foundations most typically considered are: pile foundations, and drilled shaft foundations. The most economical deep foundation alternative should be selected unless there are other controlling factors. Shaft foundations are most advantageous where very dense intermediate strata must be penetrated to obtain the desired bearing, uplift, or lateral resistance, or where materials such as boulders or logs must be penetrated. Shafts are often cost effective where a single shaft per column can be used in lieu of a pile group with a pile cap, especially when a cofferdam, seal and/or shoring is required to construct the pile foundation and pile cap. Shafts are also sometimes used in lieu of piles where pile driving vibrations could cause damage to existing adjacent facilities or in situations where pile driving is restricted due to environmental regulations.

Shafts may not be desirable where contaminated soils are present, since the contaminated soil removed would require special handling and disposal. Constructability is also an important consideration in the selection of drilled shafts. For instance, artesian water pressure in subsurface soil layers could also preclude the use of drilled shafts due to the difficulty in maintaining stability of the shaft excavation.

When designing pile foundations keep in mind the potential cost impacts associated with the use of large pile hammers. Local pile driving contractors own hammers with rated energies typically ranging up to about 80,000 ft.-lbs. When larger hammers are required to drive piles to higher pile bearing resistance they have to rent the hammers and the mobilization cost associated with furnishing pile driving equipment may increase sharply. Larger hammers may also impact the design and cost work bridges due to higher hammer and crane loads.

For situations where existing substructures must be retrofitted to improve foundation resistance, where there is limited headroom available for pile driving or shaft construction, or where large amounts of boulders or obstructions must be penetrated, micropiles may be the best foundation alternative, and should be considered.

17.3 Seismic Design

Chapter 7 describes ODOT seismic foundation design practices regarding design criteria, performance requirements, ground motion characterization, liquefaction analysis, ground deformation and mitigation. The most current edition of the "AASHTO Guide Specifications for LRFD Seismic Bridge Design", including the latest interims, should be used for seismic foundation design. Once the seismic analysis is performed the results are applied to foundation design in the Extreme Event I limit state analysis as described in Section 10 of the AASHTO. Also refer to, and be familiar with,

Section 1.10.4; "Foundation Modeling", of the *ODOT Bridge Design Manual*. This section describes the various methods bridge designers use to model the response of bridge foundations to seismic loading and also the geotechnical information required to perform the analysis.

If the foundation soils are determined to be susceptible to liquefaction, then spread footings should not be recommended for foundation support of the structure unless proven ground improvement techniques are employed to stabilize the foundation soils and eliminate the liquefaction potential. Otherwise, a deep foundation should be recommended.

Deep foundations (piles and drilled shafts) supporting structures that are constructed on potentially liquefiable soils are normally structurally checked for two separate loading conditions; i.e. with and without liquefaction. Nominal resistances, factored resistances (as appropriate), downdrag loads (if applicable) and soil (p-y) interaction parameters should be provided for both non-liquefied and liquefied foundation conditions. Communication with the structural designer is necessary to insure that the proper foundation design information is provided.

17.4 Spread Footing Design

Refer to AASHTO LRFD Bridge Design Specification, Article 10.6 for spread footing design requirements and supporting FHWA documents by Kimmerling (2002) and Gifford, et al. (1987).

Once footings are selected as the preferred design alternative, the general spread footing foundation design process can be summarized as follows. Close communication and interaction is required between the structural and geotechnical designers throughout the footing design phase.

- Determine footing elevation based on location of suitable bearing stratum and footing dimensions (taking into account any scour requirements, if applicable)
- Determine foundation material design parameters and groundwater conditions
- Calculate the nominal bearing resistance for various footing dimensions (consult with structural designer for suitable dimensions)
- Select resistance factors depending on design method(s) used; apply them to calculated nominal resistances to determine factored resistances
- Determine nominal bearing resistance at the service limit state
- Check overall stability (determine max. bearing load that maintains adequate slope stability)

For footings located in waterways, the bottom of the footing should be below the estimated depth of scour for the check flood (typically the 500 year flood event or the overtopping flood). The top of the footing should be below the depth of scour estimated for the design flood (either the overtopping or 100-year event). As a minimum, the bottom of all spread footings should also be at least 6 feet below the lowest streambed elevation unless they are keyed full depth into bedrock that is judged not to erode over the life of the structure. Spread footings are not permitted on soils that are predicted to liquefy under the design seismic event.

17.4.1 Nearby Structures

Refer to AASHTO, Article 10.6.1.8. Issues to be investigated include, but are not limit to, settlement of the existing structure due to the stress increase caused by the new footing, decreased overall stability due to the additional load created by the new footing, and the effect on the existing structure of excavation, shoring, and/or dewatering to construct the new foundation.

17.4.2 Service Limit State Design of Footings

Footing foundations shall be designed at the service limit state to meet the tolerable movements for the structure in accordance with AASHTO, Article 10.5.2. Consult with the bridge designer to obtain the maximum total and differential foundation settlements allowed for the proposed structure. The nominal unit bearing resistance at the service limit state shall be equal to or less than the maximum bearing stress that results in settlement that meets the tolerable movement criteria for the structure.

17.5 Driven Pile Foundation Design

Refer to AASHTO, Article 10.7 for pile design requirements. The FHWA publication "Design and Construction of Driven Pile Foundations" (Hannigan et al., 2016) may also be referenced for driven pile design guidance. Pile design should meet or exceed the requirements specified for each limit state.

The nominal bearing resistance of all driven piles shall be accepted based on either the FHWA Gates Equation, wave equation analysis, dynamic measurements with signal matching (PDA/CAPWAP) or full-scale load testing. Acceptance of driven piles shall not be accepted based solely on static analysis.

For piles requiring relatively low nominal resistances (<600 kips) and without concerns about high driving stresses, the FHWA Gates Equation is typically used for determining pile driving acceptance criteria. In cases where piles are driven to higher resistances or where high pile driving stresses are a concern, such as short, end bearing piles, the wave equation (GRLWEAP) is typically used for both drivability analysis and in determining the final driving acceptance criteria.

Pile acceptance based on the pile driving analyzer (PDA) is typically reserved for projects where it is economically advantageous to use, or for cases where high pile driving stresses are predicted and require monitoring. The PDA (with signal matching) method can be most cost effective on projects that have a large number of long, high capacity, friction piles.

Full-scale static pile load tests are less common in practice due to their inherent expense. However, they may be economically justified in cases where higher bearing resistances can be verified through load testing and applied in design to reduce the cost of the pile foundation. If static load testing is considered for a project it should be conducted early on in the design stage so the results may be utilized in the design of the structure. Also, the pile load test should be taken to complete failure if at all possible. Refer to AASHTO, Section 10 for descriptions on how to use the results of the static load tests results to determine driving criteria. Static load test results should be used in combination with either PDA/CAPWAP testing or wave equation analysis to develop final driving criteria for the production piles.

Once the pile (bent) locations and foundation materials and properties are defined, the pile foundation design process for normal bridge projects typically consists of the following:

- Determine scour depths (if applicable)
- Determine liquefaction potential and depths; estimate seismic induced settlement (if applicable)
- Evaluate long-term embankment settlement and downdrag potential
- Select most appropriate pile type
- Select pile dimension (size) based on discussions with structural designer regarding preliminary pile loading requirements (axial and lateral)
- Establish structural nominal resistance of the selected pile(s)

- Conduct static analysis to calculate nominal single pile resistance as a function of depth for the strength and extreme limit states (or a pile length for a specified resistance)
- Select resistance factors based on the field method to be used for pile acceptance (e.g. dynamic formula (FHWA Gates Equation), wave equation, PDA/CAPWAP, etc.)
- Calculate single pile factored resistance as a function of depth
- Estimate downdrag loads; consolidation and/or seismic-induced (if applicable)
- Calculate pile/pile group settlement or pile lengths required to preclude excessive settlement
- Determine nominal (and factored) uplift resistance as a function of depth
- Determine p-y curve parameters for lateral load analysis
- Modify parameters for liquefied soils (if applicable)
- Provide P-multipliers as appropriate for pile groups. P-multipliers are not required for pile groups installed in rock sockets where calculated lateral displacements are minimal (i.e., <0.50”).
- Determine required pile tip elevation(s) based on structural and geotechnical design requirements including the effects of scour, downdrag, or liquefaction
- Obtain and verify final pile tip elevations and required resistances (to resist factored and unfactored loads) from the structural designer; finalize required pile tip elevations and assess the following:
    - Determine the need to perform a pile drivability analysis to obtain required tip elevation
    - Evaluate pile group settlement (if applicable). If settlement exceeds allowable criteria, adjust pile lengths or the size of the pile layout and/or lengths
- Determine the need for pile tip reinforcement

17.5.1 Required Pile Tip Elevation

Required pile tip elevations should typically be provided for all pile foundation design projects. The required pile tip elevation is provided to ensure the constructed foundation meets the design requirements of the project, which may include any or all of the following conditions and criteria:

- Pile tip reaches the designated bearing layer
- Scour
- Downdrag
- Uplift
- Lateral loads

A general note is included on the bridge plans designating the “Pile Tip Elevation for Minimum Penetration” for each bent.

The required tip elevation may require driving into, or through, very dense soil layers resulting in potentially high driving stresses. Under these conditions a wave equation drivability analysis is necessary to make sure the piles can be driven to the required embedment depth (tip elevation). Higher grade steel (ASTM A252, Grade 3 or A572, Grade 50) are sometimes specified if needed to meet drivability criteria. If during the structural design process, adjustments in the required tip elevations are necessary, or if changes in the pile size or section are necessary, the geotechnical designer should be informed so that pile drivability can be re-evaluated.

17.5.2 Pile Drivability Analysis and Wave Equation Usage

High pile stresses often occur during pile driving operations and, depending on subsurface and loading conditions, a Wave Equation analysis should always be considered to evaluate driving stresses and the possibility of pile damage. A pile drivability analysis is typically used in most pile foundation designs to determine the nominal geotechnical resistance that a pile can be driven to without damage. Foundation piles should typically be driven to the highest geotechnical axial resistance feasible based on wave equation analysis so the maximum structural resistance of the pile is utilized, resulting in the most cost-effective pile design.

All piles driven to nominal resistances greater than 600 kips should be driven based on wave equation criteria. Piles driven to nominal resistance less than or equal to 600 kips may also require a wave equation analysis depending on the subsurface conditions (such as very short end bearing piles) and the pile loads. Engineering judgment is required in this determination. It is also advantageous to use the wave equation method to verify pile resistance because of the higher resistance factor (0.50) that can be used versus the FHWA Gates Equation factor of 0.40. Pile driving stresses should be limited to those described in AASHTO, Article 10.7.8.

17.5.3 Pile Setup and Restrike

Using a waiting period and restrike after initial pile driving may be advantageous in certain soil conditions to optimize pile foundation design. After initially driving the piles to a specified tip elevation, the piles are allowed to “set up” for a specified waiting period, which allows pore water pressures to dissipate and soil strength to increase. The piles are then re-struck to confirm the required nominal resistance.

The length of the waiting period depends primarily on the strength and drainage characteristics of the subsurface soils (how quickly the soil can drain) and the required nominal resistance. The minimum waiting period specified in the Standard Specifications is 24 hours. If needed, this waiting period may be extended in the contract special provisions to provide additional time for the soils to gain strength and the piles to gain resistance. However, consideration should be given to increased contractor standby costs that may be incurred by extended waiting periods. The pile design should compare the cost and risk of extending the standard waiting period to gain sufficient strength versus designing and driving the piles deeper to achieve the required bearing.

For projects with piles that require restrike, at least 2 piles per bent or 1 in 10 piles in a group (whichever is more) should typically be re-struck for pile acceptance. Additional restrike verification testing should be conducted on any piles that indicate lower resistance at the end of initial driving or if subsurface conditions vary substantially within a pile group. Restrike should be performed using a warm pile hammer, which has been warmed up with at least 20 blows on another pile.

Restrike resistance (blows per inch) should be determined by measuring the total pile set in the first 5 blows of driving and in successive 5 blow increments thereafter up to a total of at least 20 blows or until refusal driving conditions are reached (>20 blows per inch). The driving resistance reported (in blows per inch) is then determined by taking the inverse of the set (inches/blow) per each 5 blow increment. The hammer stroke during the restrike should also be carefully measured and recorded since this is use in combination with the driving resistance (bpi) to determine the nominal pile resistance when using either the FHWA Gates formula or from wave equation criteria. For more sensitive soils (clays and some silts), it may be advantageous to use a pile driving analyzer for initial driving and restrike.

17.5.4 Driven Pile Types, and Sizes

The pile types generally used on most permanent structures are steel pipe piles (driven either open or closed-end) and steel H-piles. Either H-pile or open-end steel pipe pile can be used for end bearing conditions. For friction piles, steel pipe piles are often preferred because they can be driven closed-end (as full displacement piles) and because of their uniform cross section properties, which provides the same structural bending resistance in any direction of loading. This is especially helpful under seismic loading conditions where the actual direction of lateral loading is not precisely known. Uniform section properties of steel pipe piles also aid in pile driving. Closed-end steel pipe piles are typically not filled with concrete after driving.

Potential corrosion of steel piles must be taken into account during design according to AASHTO design procedures and as described in ODOT BDM Section 1.26.5.

Pipe piles are available in a variety of diameters and wall thickness; however there are some sizes that are much more common than others and therefore usually less expensive. The most common pipe pile sizes used on ODOT projects are:

- PP 16 x 0.5
- PP 18 x 0.5
- PP 20 x 0.5
- PP 24 x 0.5

The most common steel H-pile sizes used on ODOT projects are:

- HP10x42
- HP10x57
- HP 12x53
- HP 12x74
- HP 14x73
- HP 14x89
- HP 14x117

Do not use timber piles.

Do not use prestressed concrete piles.

The ASTM steel specifications and grades in the ODOT Standard Specifications are as follows:

- Steel Pipe Piles: ASTM A 252, Grade 2 or 3, or API 5L X42 or X52
- Steel H-piles: ASTM A 36

The higher grade steel such as ASTM A252 Grade 3 (for steel pipe piles) and A572 Grade 50 (for steel H-piles) are often specified for various reasons, including higher nominal resistances, high lateral bending stresses or less potential for pile damage during installation. These higher grades are also often available at a nominal cost over the cost of the standard steel grades.

Reinforced pile tips may be warranted in some cases where piles may encounter, or are required to penetrate through, very dense cobbles and/or boulders. Pile tips are useful in protecting the tip of the pile from damage. However, installing a reinforced pile tip does not eliminate all potential for pile damage. High driving stresses may occur at these locations and still result in pile damage located just above the reinforce pile tip. A drivability analysis should be performed in these cases where high tip resistance is anticipated. All reinforced tips are manufactured from high strength (A27) steel.

Tip reinforcement for H-piles are typically called pile points. These come in a variety of shapes and designs. H-pile tips are listed on the ODOT QPL. For pipe piles tip reinforcement are typically termed “shoes”, although close-end “points”, like conical points, are also available. Pipe pile shoes may be either inside or outside-fit. Besides protecting the pile tip, inside-fit shoes are sometimes specified to help in delaying the formation of a pile “plug” inside the pipe pile so the pile may penetrate further into, or even through, a relatively thin dense soil layer. If outside-fit shoes are specified, the outside lip of the shoe may affect (reduce) the pile skin friction and this effect should be taken into account in the pile design.

17.5.5 Extreme Event Limit State Design

For the applicable factored loads for each extreme event limit state, the pile foundations shall be designed to have adequate factored axial and lateral resistance.

17.5.6 Scour Effects on Pile Design

The effects of scour, where scour can occur, shall be evaluated in determining the required pile penetration depth. The pile foundation shall be designed so that the pile penetration after the design scour events satisfies the required nominal axial and lateral resistance. The pile foundation shall also be designed to resist debris loads occurring during the flood event in addition to the loads applied from the structure. At pile locations where scour is predicted, the nominal axial resistance of the material lost due to scour should be determined using a static analysis. The piles will need to be driven to the required nominal axial resistance plus this nominal skin friction resistance that will be lost due to scour.

Equation 17-1

$\Sigma\eta_i\gamma_i Q_i \leq \varphi R_n$

The summation of the factored loads ($\Sigma\gamma iQi$) must be less than or equal to the factored resistance (φR_n). Therefore, the nominal resistance needed, R_n, must be greater than or equal to the sum of the factored loads divided by the resistance factor φ:

Equation 17-2

$Rn \geq (\Sigma\gamma iQi)/\phi dyn$

For scour conditions, the total pile resistance needs to account for the resistance in the scour zone that will not be available to contribute to the resistance required under the extreme event (scour) limit state. The total driving resistance, R_{ndr}, needed to obtain Rn, is therefore:

Equation 17-3

$R_{ndr} = R_n + R_{scour}$

Note that R_{scour} remains unfactored in this analysis to determine R_{ndr}.

Pile design for scour is illustrated further in Figure 16.1, where,

R_{scour} = skin friction which must be overcome during driving through scour zone (KIPS)

$Q_p = (\Sigma\gamma iQi)$ = factored load per pile (KIPS)

$D_{est.}$= estimated pile length needed to obtain desired nominal resistance per pile (FT)

φ_{dyn} = resistance factor

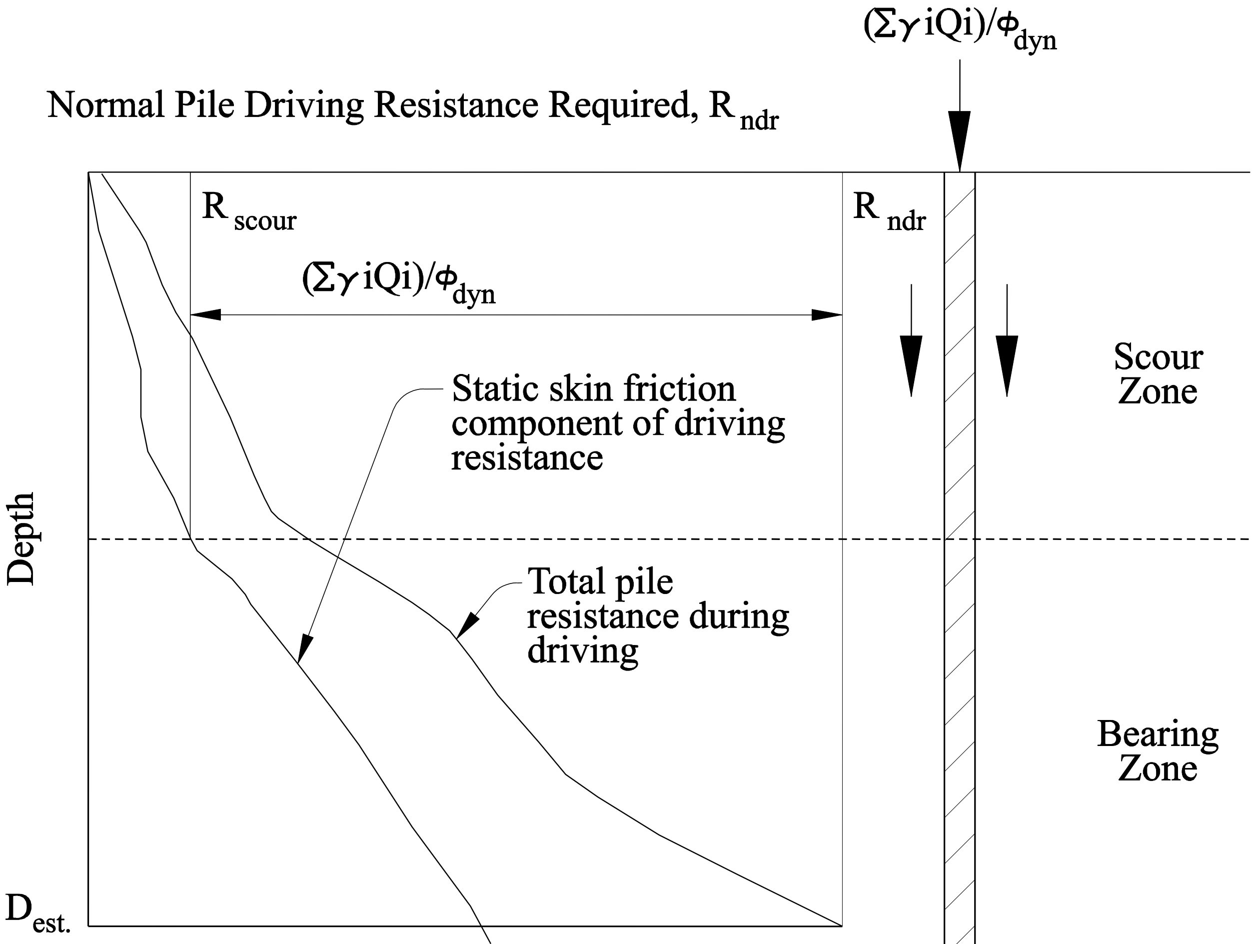

Figure 17-1 Design of pile foundations for scour

17.5.7 Seismic Design for Pile Foundations

For seismic design, all soil within and above liquefiable zones, shall not be considered to contribute axial compressive resistance. Downdrag resulting from liquefaction induced settlement shall be determined as specified in AASHTO and included in the loads applied to the foundation. Static downdrag loads should not be combined with seismic downdrag loads due to liquefaction.

The available factored geotechnical resistance should be greater than the factored loads applied to the pile, including the downdrag, at the extreme event limit state. The pile foundation shall be designed to structurally resist the downdrag plus structure loads. Pile design for liquefaction downdrag is illustrated in Figure 17-2, where,

R_{Sdd} = skin friction which must be overcome during driving through downdrag zone

Q_P = $(\Sigma\gamma iQi)$ = factored load per pile, excluding downdrag load

DD = downdrag load per pile

$D_{est.}$ = estimated pile length needed to obtain desired nominal resistance per pile

φ_{seis} = resistance factor for seismic conditions

γ_P = load factor for downdrag

The nominal bearing resistance of the pile needed to resist the factored loads, including downdrag, is therefore,

Equation 17-4

$$R_n = (\Sigma\gamma iQi)/\varphi_{seis} + \gamma_P DD/\varphi_{seis}$$

The total driving resistance, R_{ndr}, needed to obtain R_n, must account for the skin friction that has to be overcome during pile driving that does not contribute to the design resistance of the pile. Therefore:

Equation 17-5

$$R_{ndr} = R_n + R_{Sdd}$$

Note that R_{Sdd} remains unfactored in this analysis to determine R_{ndr}.

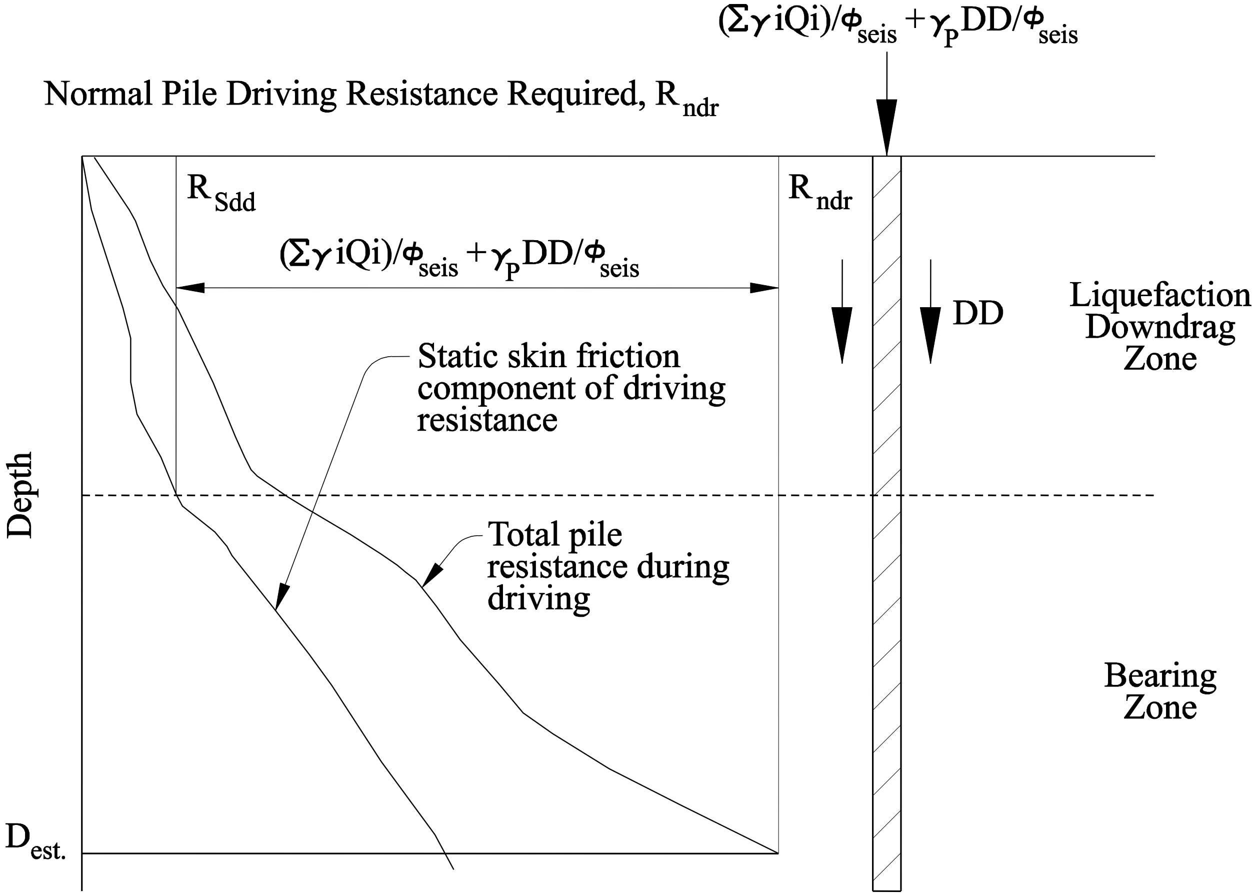

Figure 17-2 Design of pile foundations for liquefaction downdrag (WSDOT, 2006)

The static analysis procedures in the AASHTO should be used to estimate the skin friction within, above and below, the downdrag zone and to estimate pile lengths required to achieve the required bearing resistance. For this calculation, it should be assumed that the soil subject to downdrag still contributes overburden stress to the soil below the downdrag zone.

The pile foundation shall also be designed to resist the horizontal force resulting from lateral spreading, if applicable, or the liquefiable soil shall be improved to prevent liquefaction and lateral spreading. For lateral soil resistance of the pile foundation, the P-y curve soil parameters should be reduced to account for liquefaction. To determine the amount of reduction, the duration of strong shaking and the ability of the soil to fully develop a liquefied condition during the period of strong shaking should be considered.

The force resulting from flow failure/lateral spreading should be calculated as described in Chapter 7. In general, the lateral spreading force should not be combined with the seismic forces. See Chapter 7, "Seismic Design" for additional guidance regarding this issue.

17.6 Drilled Shaft Foundation Design

Refer to AASHTO, Article 10.8 for drilled shaft design requirements. Also reference the FHWA design manual "Drilled Shafts: Construction Procedures and LRFD Design Methods" (Brown, et al., 2010) for additional design guidance. Drilled shaft design should meet or exceed the requirements specified for each limit state provided by the bridge engineer.

Common shaft sizes range from 3 feet to 8 feet in diameter in 6 inch increments. Larger shaft diameters are also possible. Based on recent experience with the design and construction of drilled shafts any drilled shaft that may be design greater than 6' in diameter is required to be submitted no later than DDAP to State Foundation Engineer for review and concurrence. This provides time during project development to investigate, design, and incorporate appropriate level of quality control during construction.

Once the shaft locations and foundation materials and properties are known, the drilled shaft design process for normal bridge projects typically consists of the following:

- Determine scour depths (if applicable),
- Determine liquefaction potential and depths; estimate seismic induced settlement (if applicable),
- Evaluate long-term embankment settlement and downdrag potential,
- Select most appropriate shaft diameter(s) in consultation with structure designer,
- Determine (in consult with the structure designer) whether or not permanent casing will be used,
- Calculate nominal single shaft resistance as a function of depth,
- Select and apply resistance factors to nominal resistance,
- Estimate downdrag loads (if applicable),
- Estimate shaft or shaft group settlement and adjust shaft diameter or lengths if necessary to limit settlement to service state limits,

- Determine p-y curve parameters for lateral load analysis; modify parameters for liquefied soils (if applicable),

The diameter of shafts will usually be controlled by the superstructure design loads and the configuration of the structure but consideration should also be given to the foundation materials to be excavated. If boulders or large cobbles are anticipated, attempt to size the shafts large enough so the boulders or cobbles can be more easily removed if possible. Shaft diameters may also need to be increased to withstand seismic loading conditions. The geotechnical engineer and the bridge designer should confer and decide early on in the design process the most appropriate shaft diameter(s) to use for the bridge, given the loading conditions, subsurface conditions at the site and other factors. Also decide early on with the bridge designer if permanent casing is desired since this will affect both structural and geotechnical designs. Specify each shaft as either a “friction” or “end bearing” shaft since this dictates the final cleanout requirements in the specifications.

When the drilled shaft design calls for a specified length of shaft embedment into a bearing layer (rock socket) and the top of the bearing layer is not well defined, consideration should be given to adding an additional length of shaft reinforcement to the length required to reach the estimated tip elevation. This extra length is to account for the uncertainty and variability in the final shaft length. This practice is much preferred instead of having to splice on additional reinforcement in the field during which time the shaft excavation remains open. Any extra reinforcement length that is not needed can be easily cut off prior to steel placement once the final shaft tip elevation is known. CSL tubes would also need to be either cut off and recapped or otherwise adjusted. This additional reinforcement length should be determined by the geotechnical engineer based on an evaluation of the site geology, location of borehole information and the potential variability of the bearing layer surface at the plan location off the shaft. The additional recommended length should be provided in the Geotechnical Report and included in the project Special Provisions. Refer to the *Standard Special Provisions for Section 00512* for further guidance and details of this application. If a minimum rock embedment (socket) depth is required, specify the reason for the rock embedment.

Settlement may control the design of drilled shafts in cases where side resistance (friction) is minimal, loads are high and the shafts are primarily end bearing on compressible soil. The shaft settlement necessary to mobilize end bearing resistance may exceed that allowed by the bridge designer. Confer with the bridge designer to determine shaft service loads and allowable amounts of shaft settlement. Refer to the AASHTO methods to calculate the settlement of individual shafts or shaft groups. Compare this settlement to the maximum allowable settlement and modify the shaft design if necessary to reduce the estimated settlement to acceptable levels.

17.6.1 Drilled Shaft Base Grouting

Drilled shaft base grouting (or post-grouting) is a process that generally involves pumping cement grout under pressure beneath the base of the shaft to increase the tip resistance. This technique is mostly effectively used for sandy soils with very little fines content. The grout is pumped through pipes into a grout-distribution system attached to the base of the drilled shaft reinforcement. After the shaft is constructed and the concrete has gained adequate strength, grout is pumped through the grout system until grout is returned to the surface. The return valves are then closed and pressure is applied to the system to force grout out of tubes at the base of the shaft into the soil or to inflate a rubber membrane. Grout is pumped under pressure until a specified pressure criteria is achieved.

Shaft base grouting is a relatively new shaft construction technique in the U.S. and currently not addressed in AASHTO. As such, the use of shaft post grouting on ODOT projects must be approved with a design deviation prior to use.

17.6.2 Nearby Structures

Where shaft foundations are placed adjacent to existing structures, the influence of the existing structure(s) on the behavior of the foundation, and the effect of the foundation on the existing structures, including vibration effects due to casing installation, should be investigated. In addition, the impact of caving soils during shaft excavation on the stability of foundations supporting adjacent structures should be evaluated. At locations where existing structure foundations are adjacent to the proposed shaft foundation, or where a shaft excavation cave-in could adversely affect an existing foundation, the design should require that casing be advanced as the shaft excavation proceeds.

17.6.3 Scour

The effect of scour shall be considered in the determination of the shaft penetration. The shaft foundation shall be designed so that the shaft penetration and resistance remaining after the design scour events satisfies the required nominal axial and lateral resistance. For this calculation, it shall be assumed that the soil lost due to scour does not contribute to the overburden stress in the soil below the scour zone. The shaft foundation shall be designed to resist debris loads occurring during the flood event in addition to the loads applied from the structure.

Resistance factors for use with scour at the strength limit state are the same as those used without scour. The axial resistance of the material lost due to scour shall not be included in the shaft resistance.

17.6.4 Extreme Event Limit State Design of Drilled Shafts

For downdrag due to liquefaction, the nominal shaft resistance available to support structure loads plus downdrag shall be estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. For this calculation, it shall be assumed that the soil contributing to downdrag does contribute to the overburden stress in the soil below the downdrag zone. The available factored geotechnical resistance should be greater than the factored loads applied to the shaft, including the downdrag loads, at the strength limit state. The shaft foundation shall be designed to structurally resist the downdrag plus structure loads.

17.7 Micropiles

Micropiles shall be designed in accordance with Article 10.9 of the *AASHTO*. Additional information on micropile design may be found in the FHWA Reference Manual; *Micropile Design and Construction*, Publication No. FHWA NHI-05-039 (Sabatini, et. al., 2005). While micropiles are great for resisting high axial loads lateral resistance is small and should be a consideration during design. Because of the low lateral resistance micropiles should not be used for new bridge construction with seismic or other lateral loads.

17.8 Traffic Structures

As Previously Stated, various versions of the "AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals" are in effect and refer to "AASHTO Standard Specifications for Highway Bridges". The design approach used for the foundation design must be consistent with the design approach used for the structure. At this time monotube VMS, sign bridges, and signal poles use three different standards. The table below provides the current standard in effect, associated standard drawings, standard foundation drawing, and special provision.

Table 17-3 Traffic Structures Standards

Structure Type	Standard	Design Method	Standard Drawings	Standard Foundation Drawing	Special Provision
Monotube VMS	2017, "*AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.*"	LRFD	TM621 – TM628	TM627 and TM628	00921
Sign Bridges - Truss	1996, "*AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.*"	ASD	TM606 – TM620	TM611/TM619	00920
Sign Bridges - Monotube	2017, "AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals."	LRFD	TM627, TM628, TM693-TM697	TM627 and TM628	00921
Signal Poles SM1-SM5L	2003, "*AASHTO LRFD Specifications for Structural Supports for Highway Signs,*	ASD	TM650 – TM653	TM653	00963

Structure Type	Standard	Design Method	Standard Drawings	Standard Foundation Drawing	Special Provision
	Luminaires, and Traffic Signals."				
Signal Poles SM6L-SM7L	2017, "*AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals."*	LRFD	TM655 – TM658	TM628	00921
Luminaires	2015, "*AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals."*	LRFD	TM630	TM630	00962
High Mast Luminaires	2017, "AASHTO LRFD Standard Specifications for Bridge Design"	LRFD	N/A	N/A	00512 or 00921

16.9.1 Mast Arm Signal Pole Foundations

The standard drawings for Mast Arm Signal Poles are TM 650 through TM 658. These structures consist of a single vertical metal pole member of various heights and a horizontal signal (or mast) arm of various lengths. Lights, signals, and/or cameras will be suspended or supported from the mast arm. Currently there are two foundation design methodologies in place. Those less than 60' in length and those mast arm lengths 60' and greater. Regardless of size, the Rutledge Method described in the AASHTO specifications is **not** an approved method for the design of signal pole drilled shaft foundations.

17.8.1.1 Mast arm signal poles less than 60' in length

Standard drawings TM650-TM653 are used for the design of the foundations for these structures and are the most common signal pole foundations. The standard foundation lengths provided in Table 17-4 and Table 17-5 are for signal poles supported in cohesionless soil. These depths may be used when the conditions listed for each table can be met.

Table 17-4 Minimum Lateral Embedment Depths for Standard Foundation of SM1 – SM5L Signal Poles in Cohesionless Soil when Groundwater is at Least 9 ft Below the Tip of the Foundation where γ = 100 pcf and ϕ = 26 degrees and *k* = 25 pci

SM1 ft.	SM2 ft.	SM3 ft.	SM4 ft.	SM5 ft.	SM1L ft.	SM2L ft.	SM3L ft.	SM4L ft.	SM5L ft.
12	14	15	16	17	14	15	16	17	18

Table 17-5 Minimum Lateral Embedment Depth for Standard Foundation of SM1 – SM5L Signal Poles in Cohesionless Soil and with groundwater at the ground surface where γ = 38 pcf, ϕ = 26 degrees, and k = 20 pci

SM1 ft.	SM2 ft.	SM3 ft.	SM4 ft.	SM5 ft.	SM1L ft.	SM2L ft.	SM3L ft.	SM4L ft.	SM5L ft.
17	18	22	21	21	18	21	21	22	25

If any of the above assumptions cannot be met then complete a project specific design using LPile, as specified in AASHTO LRFD, to determine the length to fixity and the maximum lateral deflection of 0.50 inch is allowed at the top of the shaft (bottom of the cap). Factor of Safety to be used is 2.5 for side friction or a φ = 0.40.

Resistance to torsion is not included in the design for signal pole foundations governed by standard drawings TM650-TM653. Mast arm signal poles are not designed for seismic loads, nor mitigated for liquefaction effects.

Report the foundation conditions at the signal pole site characterized in terms of soil type, soil unit weight, and soil friction angle or undrained shear strength and recommended foundation depth.

Where solid bedrock is confirmed to be within the depth of the shaft foundation, then the rock should be characterized in terms of its unconfined compressive strength (q_u) and overall rock mass quality. In general, if the bedrock can be classified with a hardness of at least R1 (100 psi) and is unfractured or with tight, moderately close to very wide-spaced joints then a minimum shaft embedment depth of 5 feet can be used, as shown in Figure 17-.3, for mast arm pole types SM1-SM5L as specified on TM653.

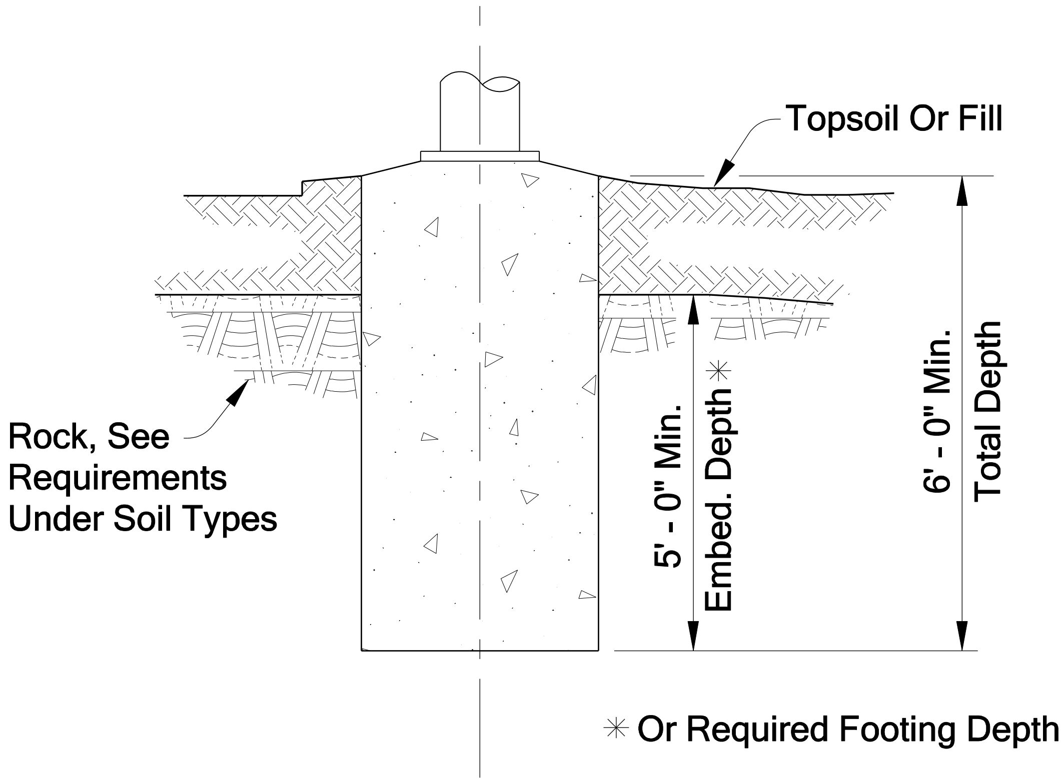

Figure 17-3 Rock Installation Requirements

If the rock is weaker than R1, moderately weathered or contains open fractures, then the properties of the rock mass should be more thoroughly investigated and a design should be performed based on the procedures previously specified in this chapter. For allowable stress design of drilled shafts in rock use a minimum factor of safety of 2.5 (for both side shear and end bearing) in determining allowable axial capacity. Use the soil-structure interaction (P-y) methods described in *AASHTO "LRFD Bridge Design Specifications,"* for lateral load analysis of drilled shafts in rock.

17.8.1.2 Mast arm signal poles 60' and greater than in length

Standard drawings TM655-TM658 are used for the design of the foundations for these structures and are not common signal pole foundations. Broms' Method and Rutledge are ***not*** an approved methods for the design of signal pole drilled shaft foundations with mast arms 60' and greater. Use LPile, as specified in AASHTO LRFD, to determine the length to fixity and the maximum lateral deflection of 0.50 inch at the top of the shaft (ground line).

Signal pole foundations governed by standard drawings TM655-TM658 are designed to resist torsion. Recent research studies have concluded and verified that existing methods produce acceptable results, for cohesionless and cohesive, soils (Hou, Kuang-Yuan, et al., 2019, Li et al., 2017, Stuedlein, 2016). Methods vary from state-to-state ranging from only using shaft friction to performing finite element analysis for each design. In an effort to standardize, and use a common method the following narrative outlines the ODOT's procedures for calculating torsion. Whether cohesionless or cohesive soils AASHTO methods to calculate nominal shaft side resistance (R_s) are used excluding the top five feet of the drilled shaft. Torsion resistance is comprised of friction along the shaft where the total resistance to torsion is:

$$R_{Tor} = R_s = r \sum_{L=5\,ft}^{L=tip\ of\ shaft} A_s\, q_s \qquad (16\text{-}6)$$

Where R_{Tor} is the torsional resistance due to skin friction along the shaft and r is the radius of the drilled shaft.

Mast arm signal poles are not designed for seismic loads, nor mitigated for liquefaction effects.Report the foundation conditions at the signal pole site characterized in terms of soil type, soil unit weight, and soil friction angle or un-drained shear strength and recommended foundation depth.

17.8.2 Cantilever Sign Foundations

Cantilever signs consist of large metal posts supporting a cantilevered metal arm, which carries various types and sizes of signs and luminaires. Standard Drawings TM621 – TM628 cover the entire standard for this type of traffic structure. There are currently nine standard spread footing designs and three drilled shaft designs. Foundation design is based on the reactions at the base plate. There are two standard foundation drawings that can be used in VMS Monotube Cantilever Sign Design the spread footing shown TM627 or the drilled shaft TM 628.

The spread footing foundation is a rectangular spread footing, as shown on Drawing TM627. The dimensions of the spread footings range from 9′ by 16′ to 13.5′ by 28′. All footings are 2′- 3″ thick with a minimum 3′-0″ of cover over the top of the footing. Footing dimensions are based on the Structure Design Numbers (1 – 9) and whether the footing is constructed on non-buoyant or buoyant soil conditions. Drawing TM627 contains soil properties, nominal bearing resistance, factored bearing resistance and resistance factors for each soil condition.

The difference between non-buoyant and buoyant soils is buoyant soils assume the groundwater table can rise up above the top of the footing and fully saturate the minimum 3 foot soil cover depth overlying the footing. If so, this reduces the effective unit weight of the overlying soils and the uplift resistance of the footing. The footing dimensions then have to be increased to compensate for this effect.

For spread footing recommendations, the Engineer of Record must report buoyant or non-buoyant condition, how the engineering soil properties are verified, minimum size spread footing which will meet the loading criteria with associated resistance factors.

Drilled shaft standard drawing is shown on TM628. Drilled shaft diameters range from 4.5′ to 5′ in diameter. As with the spread footing these are based on the Cantilever Structure Design Numbers 1-9. Broms' and Rutledge is **not** an approved methods for the geotechnical design of monotube sign and cantilever VMS drilled shaft foundations. Use LPile, as specified in AASHTO LRFD, to determine the length to fixity and the maximum lateral deflection of 0.5 inch at the top of the shaft (ground line). Resistance to torsion is calculated for cantilever sign and VMS foundations governed by standard drawings TM628. Torsion resistance will be determined using ignoring the top 5 ft of the friction, following AASHTO friction resistance methods for cohesive and cohesionless soils and total torsion computed using equation 16-6 . Cantilever sign/VMS drilled shaft foundations are not designed for seismic loads, nor mitigated for liquefaction effects.

Report the foundation conditions at the site characterized in terms of soil type, soil unit weight, and soil friction angle or un-drained shear strength, recommended foundation depth, controlling load (moment, torsion, lateral, axial) and whether this is a side-friction or end-bearing drilled shaft.

17.8.3 Sign And VMS Bridge Foundations

Currently there are two sets of standard sign/VMS bridges drawings. Standard drawings TM606-TM620 are for the truss style bridge. The second is the Monotube sign/vms bridge with standard drawings TM627, TM628 and TM693-TM697. Regardless of the style, the sign/vms bridge spans the roadway and lengths range from 50 feet to 167 feet.

Spread footings for sign/VMS bridges range in size from 12’ by 24’ to 20.5’ by 41’, depending on soil type (buoyant or non-buoyant) and truss span length. Minimum embedment over the top of the footing is 3’. All footings are 2.5’ thick. Additional differential settlement criteria apply to these structures as noted on the drawings. Differential and uniform settlement should not exceed 2 inches. Footings are to be constructed on undisturbed soil or compacted granular structure backfill.

The difference between non-buoyant and buoyant soils is buoyant soils assume the groundwater table can rise up above the top of the footing and fully saturate the minimum 3 foot soil cover depth overlying the footing. If so, this reduces the effective unit weight of the overlying soils and the uplift resistance of the footing. The footing dimensions then have to be increased to compensate for this effect.

For spread footing recommendations, the Engineer of Record must report buoyant or non-buoyant condition, loading criteria, document how the engineering soil properties are verified, and recommended spread footing size.

17.8.4 Luminaire Supports

Standard luminaire poles consist of metal poles typically 30’ to 70’ high with a luminaire mast arm attached at the top. Standard foundations for luminaire supports are shaft foundations. Shafts may be either drilled shafts or constructed with concrete forms, backfilled, and compacted. These footings are either 30” or 36” in diameter or width and range from 6.5 feet to 9.0 feet in depth. The standard foundation design shown on Drawing TM631 is based on a soil parameter c = 600 psf for cohesive soil and φ=25° and γ=100 pcf and fully saturated.

If bedrock is expected to be encountered at shallow depths then a special design should be considered. If the bedrock is relatively hard, difficult to excavate or drill through, and would greatly impact the time required to construct the foundation excavation then develop a special foundation design, taking into account the higher foundation material strengths.

Report how the engineering soil properties for luminaires were verified and if bedrock is expected to be encountered.

17.8.5 High Mast Luminaire Support

High Mast Luminaire Supports are multi-arm illumination generally over 55 ft in height. Standard drawings do not exist for high mast illumination. If high mast illumination is required on a project, the foundations for these structures shall be drilled shafts. The foundation design and report should be developed based on site-specific soils investigation and a full soil-structure interaction analysis as described in this chaper for bridges. The traffic structures designer should be consulted for design loads and other design requirements.

17.8.6 Sound Walls

ODOT currently has three standard designs for sound walls which are designed in accordance with AASHTO Guide Specifications for Structural Design of Sound Barriers, 1989. The three standard sound wall designs are:

- Standard Reinforced Concrete Masonry Sound Wall; Drawing No. BR730
  - Foundation Type: Continuous Spread Footing
- Standard Precast Concrete Panel Sound Wall; Drawing No. BR740
  - Foundation Type: 3-ft- diameter drilled shafts
- Standard Masonry Sound Wall on Pile Footing; Drawings No. BR750 & BR751
  - Foundation Type: 2- to 3-ft-diameter drilled shafts

Standard foundation designs for these structures typically consist of spread footings (continuous or individual) or drilled shafts (with or without pilasters). These standard drawings are typically used at sites where the soil conditions are relatively uniform with depth. Lateral loads such as wind and seismic usually govern the foundation designs for these structures. The foundation designs provided on the Standard Drawings have been developed over many years, using a variety of foundation design methods.

Therefore, the foundation design method used for each of the standard drawings is discussed separately in the following sections.

Seismic Design

Sound walls are also designed for seismic loading conditions as described in the "*AASHTO Guide Specifications for Structural Design of Sound Barriers.*" No liquefaction analysis or mitigation of ground instability is required for sound walls.

Backfill Retention

All Standard Drawings for sound wall structures have been designed to retain a minimal amount of soil that must be no more than 2 ft. in height with a level back slope. The retained soil above the sound wall foundation is assumed to have a friction angle of 34° and a wall interface friction of 0.67φ, resulting in a K_a of 0.26 for the retained soil, and a unit weight of 125 pcf. All standard and non-standard sound wall foundation designs shall include the effects of any differential fill height between the front and back of the wall.

17.8.6.1 BR730 Spread Footings

Continuous spread footings are required for the Standard Reinforced Concrete Masonry Sound wall (Drawing No. BR730). The footing dimensions shown on this drawing are all based on the "Average" soil conditions even though a description of "Good" soil is provided. Sound wall footings shall be located relative to the final grade to have a minimum soil cover over the top of the footing of 1 ft.

Sloping Ground Conditions

The standard foundation designs used for the Standard Plan sound walls are based on level ground conditions. Level ground conditions are defined as follows:

- **Good Soils:** 10H:1V max.
- **Average Soils:** 14H:1V max.

Sound walls are often constructed on sloping ground or near the edge of a steep break in slope. When the ground slope exceeds the above limits, the foundation design must be modified to account for slope effects. For the continuous spread footing design (BR730), a special design is necessary since there is no standardized method of modifying the standard footing widths or depths shown on the standard drawing.

Perform settlement calculations to confirm the required noise barrier height is maintained for the design life of the wall. The geotechnical designer will be responsible for estimating foundation settlement using the appropriate settlement theories and methods as outlined earlier in this chapter. The estimated total and differential settlement should be provided in the Geotechnical Report. In these cases, the total allowable settlement and differential settlement of the sound wall should follow retaining wall standards in AASHTO.

In addition to foundation design, an overall stability analysis of the sound wall should be performed when the wall is located on or at the crest of a cut or fill slope. The design slope model must include a surcharge load equal to the footing bearing stress. The minimum slope stability factor of safety of the structure and slope shall be 1.5 or greater for static conditions and 1.1 for seismic conditions.

17.8.6.2 BR740, BR750 and BR751 Drilled Shafts

The footings for Drawings BR 740 and BR 750 (drilled shafts) are designed by Load Factor design. The footing (shaft) embedment lengths for these walls were design by the Rutledge Equation using S1 = RL/3, where "S1" is the Allowable Ultimate Lateral Soil Capacity. "R" equals the Ultimate Lateral Soil capacity obtained by the log-spiral method increased by a 1.5 isolation factor and includes a 0.90 soil strength reduction factor.

All of the standard drawings for sound walls are based on the same set of foundation soil descriptions and designations. These are described as follows:

- **Good soil:** Compact, well graded sand or sand and gravel. Design $\phi = 35°$, density 120 pcf, well drained and not located where water will stand.
- **Average soil:** Compact fine sand, well drained sandy loam, loose coarse sand and gravel, hard or medium clay. Design $\phi = 25$, density = 100 pcf. Soil should drain sufficiently so that water will not stand on the surface.
- **Poor soil:** (Soil investigation required) Soft clay, loams, poorly compacted sands. Contains large amounts of silt or organic material. Usually found in low lying areas that are subject to standing water.

For special designs, such as for "poor" soil conditions, buoyant conditions, or hard rock the geotechnical designer needs to provide the soil properties necessary to perform the foundation design. Foundation designs for these conditions should be performed using the Broms' method as described in "*AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*".

For the standard drilled shaft foundations (BR740 and BR751), methods are shown on the drawings for adjusting the length of the shafts to account for slope effects. The maximum slope angle that shafts may be constructed on, using the standard drawings, are:

- **Good Soils:** 1½H:1V max.
- **Average Soils:** 2H:1V max.

For drilled shafts, the minimum horizontal setback distance is 3.0 ft. from the panel face to the slope break.

17.8.7 Buildings

Foundations shall be designed in accordance with the provisions outlined in the most recent version of the Oregon Structural Specialty Code (OSSC).

17.9 Construction Considerations

There are construction consideration for all foundation types. Each foundation type has construction considerations which must be in the plans, and special provisions. Construction considerations include but are not limited to access, construction platform and groundwater. Each of these elements will affect construction of spread footings, driven piles and drilled shafts differently. The discussion that follows is provides insight for spread footing, driven pile and drilled shaft construction their access, platform, and groundwater consideration.

Regardless of foundation type the contractor needs to be able to gain access to the foundation location with equipment. Spread footings require large areas to be excavated frequently in rock and/or on steep slopes. Excavation equipment needs a safe approach and the ability to work below the slopes they are excavating, with overhead room and enough reach to perform the work. This may require work platforms, and/or shoring. Additionally, pile driving will require a crane for moving piles, and overhead room to drive piles. Like driven piles, drilled shafts need a crane to move rebar cages, casing as well as space for a concrete truck and most likely a concrete pump truck also. Access needs of the contractor need to be accounted for during project delivery.

Limited right-of-way or constrained access may lead to the need of a “construction platform”. Construction platforms to provide access range from rock subgrade improvement to the use of temporary work access structures. Regardless of the type, construction platforms need to be considered and provided in the plans, specifications and cost estimates.

Other construction consideration, if not accounted for, that can become expensive in construction is groundwater control. Dewatering of excavations provide for safety and allow for a construction of spread footing. While deep foundations (driven piles and drilled shafts) do not typically need the excavation of a spread footing they do need to connect/integrate with abutments which can be quite tall and have been known to intercept the groundwater table. Groundwater issues need to be identified early in the project, included in project plans, specs and estimates to avoid claims and contract change orders.

Structures that require short round or square foundations could be easily formed in an open excavation. Following the removal of the concrete forms, backfill should be placed and compacted around the footing to provide containment and lateral support. Footings constructed using forms and backfill should be backfilled using Granular Structure backfill material compacted to the requirements specified in Section 00510 of the *ODOT Standard Specifications*. The geotechnical designer should make sure the contract specifications clearly state the backfill and compaction requirements for the backfill material placed around the formed foundation and that the degree of compaction is verified in the field.

Shaft foundations may require the use of temporary casing, drilling slurries or both. Most shaft foundations are designed with the concrete in direct contact with the soil. Special foundation designs may require the use of permanent casing if recommended by the geotechnical designer, in which case, the concrete will not be in direct contact with the soils.

An example of this is where the foundation soils may be too soft and weak to allow for the removal of temporary casing. In this situation, the structural designer must be informed of this condition. The use of permanent casing alters the stiffness and strength of the shaft as well as the soil-shaft friction and torsional shaft capacity.

The presence of a high groundwater table could affect the construction of shaft foundations. Shaft foundations are especially vulnerable to caving if groundwater is encountered and there are loose clean sands or gravels present.

17.10 References

- AASHTO, 2010, *AASHTO Guide Specifications for Structural Design of Sound Barriers*.
- AASHTO, 2002, "*AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.*"
- AASHTO, 2003, "*AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.*"
- AASHTO, 2017, "*AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals.*"
- AASHTO, 2002, "*Standard Specifications for Highway Bridges,*" American Association of State Highway and Transportation Officials, Seventeenth Edition, Washington, D.C., USA, p. 686.
- AASHTO, "*LRFD Bridge Design Specifications*", American Association of State Highway and Transportation Officials, most current edition, Washington, D.C., USA.
- AASHTO, 2010, "Standard of Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications", AASHTO Report R 27-01 (2010), American Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO, 2011. "AASHTO Guide Specifications for LRFD Seismic Bridge Design", 2nd Edition and Interim Revisions, American Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO, 2017, *"LRFD Bridge Design Specifications"*, American Association of State Highway and Transportation Officials, Eighth Edition, Washington, D.C. (Note: most current edition shall be used).
- Adama Engineering, Inc., 2003, "*Foundation Stress and Settlement Analysis Program (FoSSA)*", Newark, N.J., http://www.geoprograms.com/
- Arneson, L.A., Zevenbergen, L. W., Lagasse, P.F., Clopper, P.E., 2012. *"Evaluating Scour at Bridges"*, FHWA-HIF-12-003 HEC-18, 5th Edition, Federal Highway Administration, Washington, D.C.
- Beavers, J.A. and Durr, C.L., 1998, "Corrosion of Steel Piling in Non-Marine Applications", NCHRP Project 10-46, Report 408, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C
- Bowles, J., 1988. FOUNDATION ANALYSIS AND DESIGN 4th Edition, McGraw-Hill Book Company.

- Briaud, J., and Tucker, L., 1997. "*Design and Construction Guidelines for Downdrag on Uncoated and Bitumen-Coated Piles", NCHRP Report 393*, Transportation Research Board, National Research Council, Washington, D.C.
- Brown, D.A., Turner, J.P., Castelli, R. J., 2010. *"Geotechnical Engineering Circular No. 10, Drilled Shafts: Construction Procedures and LRFD Design Methods"*, Report No.FHWA-NHI-10-016, Federal Highway Administration, Washington, D.C.
- Elias, V., Christopher, B.R., and Berg, R.R., 2001, "*Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines*", Federal Highway Administration, FHWA-NHI-00-043.
- Gifford, D. G., J. R. Kraemer, J. R. Wheeler, and A. F. McKown. 1987. "*Spread Footings for Highway Bridges." FHWA/RD-86/185*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, p. 229.
- Hannigan, P.J., Rausche, F., Likins, G.E., Robinson, B.R. and Becker, M.L., 2016. "*Design and Construction of Driven Pile Foundations, Vol 1 & Vol 2",*. FHWA-NHI-16-009 and FHWA-NHI-16-010, Federal Highway Administration, Washington, D.C.
- Kavazanjian, E., Jr., Wang, J-N., Martin, G.R., Shamsabadi, A., Lam, I.P., Dickenson, S.E., and Hung, C.J., 2011. "*Geotechnical Engineering Circular No. 3, LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations*", Report No. FHWA-NHI-11-032, Federal Highway Administration, Washington, D.C.
- Kimmerling, R. E. 2002. " *https://www.fhwa.dot.gov/engineering/geotech/pubs/010943.pdf ", Report No. FHWA-SA-02-054*, Federal Highway Administration, Washington, D.C.
- Moulton, L. K., H. V. S. GangaRao, and G. T. Halverson. 1985. "*Tolerable Movement Criteria for Highway Bridges", FHWA/RD-85/107*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, p. 118.
- Oregon Department of Transportation, *Bridge Design Manual*, Bridge Engineering Section.
- Oregon Department of Transportation, *Hydraulics Manual*, Geo-Environmental Section.
- Oregon Department of Transportation, *Soil and Rock Classification Manual*, Geotechnical Engineering and Engineering Geology Section, 1987.
- Oregon Department of Transportation, "*Standard Specifications for Highway Construction*", 2018 Edition and related Standard Special Provisions.
- Reese, L. C. 1984. "*Handbook on Design of Piles and Drilled Shafts Under Lateral Load". FHWA-IP-84/11*, Federal Highway Administration, U.S. Department of Transportation, Washington, DC.
- Reese, L. C., 1986. "*Behavior of Piles and Pile Groups Under Lateral Load",Report No. FHWA/RD-85/106*, U. S. Department of Transportation, Federal Highway Administration, Office of Engineering and Highway Operations Research and Development, Washington D. C., 311
- Sabatini, P.J., Bachus, R.C., Mayne, P.W., Schneider, J.A., and Zettler, T.E., 2002, *Geotechnical Engineering Circular No. 5*, *Geotechnical Engineering Circular No. 5, Evaluation of Soil and Rock Properties*, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.
- Sabatini, P.J., Tanyu, B., Armour, T., Groneck, P., Keeley, J., 2005. "*Micropile Design and Construction" (Reference Manual for NHI Course 132078), Federal Highway Administration Report No. FHWA NHI-05-039*, Washington, D.C.

- Samtani, N.C., Nowatzki, E.A., Mertz, D.R., 2010. "*Selection of Spread Footings on Soil to Support Highway Bridge Structures*", Federal Highway Administration Report No. FHWA-RC/TD-10-001, Washington, D.C.
- Samtani, N. C. and Nowatzki, E., 2006. *"Soils and Foundations Reference Manual, Volumes I and II"*, FHWA-NHI-06-088 and FHWA-NHI-06-089, Federal Highway Administration, Washington, D.C.
- Washington State Department of Transportation, 2013, *https://www.wsdot.wa.gov/Publications/Manuals/M46-03.htm*.
- Kimmerling, R. E. 2002. " *https://www.fhwa.dot.gov/engineering/geotech/pubs/010943.pdf* ", *Report*
- Washington State Department of Transportation, 2013, *https://www.wsdot.wa.gov/Publications/Manuals/M46-03.htm*.
- Ivey, D. L. and L. Hawkins, "*Signboard Footings to Resist Wind Loads," Civil Engineering*, December 1966.
- Oregon Department of Transportation, *ODOT Traffic Structures Design Manual*, Roadway Traffic Section, 2021.
- Patterson, D., 1962, "*How to Design Pole-Type Buildings,"* American Wood Preservers Institute, Chicago, Third Edition.

Peck, R. B., W. E. Hanson, and T. H. Thornburn, 1974. "*Foundation Engineering,"* Second Edition, John Wiley and Sons, Inc., New York, NY, p. 514