17.1 General

17.1.1 Overview

This chapter covers the geotechnical design of lightly loaded structures which include: noise barriers, sign bridges, cantilevered signs and signals, strain pole standards, luminaires, culverts not supported on foundation elements, and small buildings. Small buildings typically include single story structures such as structures in park and ride lots, rest areas, or WSDOT maintenance facilities. Standard Plan designs found in the *Standard Plans For Road, Bridge and Municipal Construction* M 21-01 have been developed for all of these structures except for small buildings and culverts. Both shallow (e.g. spread footings) and moderately deep foundations (trenches and shafts) have been designed to support these lightly loaded structures in a variety of soil and site conditions. The structural design of these facilities is addressed in the *Bridge Design Manual* and *Design Manual* M 22-01.

17.1.2 Site Reconnaissance

General procedures for site reconnaissance are presented in Chapter 2. Prior to the site reconnaissance, the location of the structures should be staked in the field, or an accurate and up-to-date set of site plans identifying the location of these structures should be available. An office review of all existing data pertinent to the site and the proposed foundations (see Chapter 2) should also be conducted prior to the site reconnaissance.

During the site reconnaissance, observations of the condition of existing slopes (natural and cut) in the immediate vicinity of the structures should be inspected for performance. It is especially important to establish the presence of high ground water and any areas of soft soil. Many of these structures have very shallow foundations and the investigation may only consist of general site reconnaissance with minimal subsurface investigation. The geotechnical designer should have access to detailed plan views showing existing site features, utilities, proposed construction and right-of-way limits. With this information, the geotechnical designer can review structure locations, making sure that survey information agrees reasonably well with observed topography. The geotechnical designer should look for indications of soft soil and unstable ground. Observation of existing slopes should include vegetation, in particular the types of vegetation that may indicate wet soil. Equisetum (horsetail), cattails, blackberry and alder can be used to identity wet or unstable soils. Potential geotechnical hazards such as landslides that could affect the structures should be identified. The identification and extent/condition (i.e., thickness) of existing man-made fills should be noted, because many of these structures may be located in engineered fills. Surface and subsurface conditions that could affect constructability of the foundations, such as the presence of shallow bedrock, or cobbles and boulders, should be identified.

17.1.3   Field Investigation

If the available geotechnical data and information gathered from the site review is not adequate to make a determination of subsurface conditions as required herein, then new subsurface data shall be obtained. Explorations consisting of geotechnical borings, test pits and hand holes or a combination thereof shall be performed to meet the investigation requirements provided herein. As a minimum, the subsurface exploration and laboratory test program should be developed to obtain information to analyze foundation stability, settlement, and constructability with respect to:

• Geological formation(s)

• Location and thickness of soil and rock units

• Engineering properties of soil and rock units such as unit weight, shear strength and compressibility

• Groundwater conditions (seasonal variations)

• Ground surface topography

• Local considerations, (e.g., liquefiable soils, expansive or dispersive soil deposits, underground voids from solution weathering or mining activity, or slope instability potential)

Standard foundations for sign bridges, cantilever signs, cantilever signals and strain pole standards are based on allowable lateral bearing pressure and angle of internal friction of the foundation soils. The determination of these values can be estimated by Standard Penetration Test (SPT). Portable Penetrometer Tests (PPT) may be used to obtain the soil data provided the blow count data is properly converted to an equivalent standard penetrometer "N" value. The designer should refer to Chapter 3 for details regarding the proper conversion factors of PPT to SPT. Every structure foundation location does not need to be drilled. Specific field investigation requirements for the structures addressed in this chapter are summarized in Table 17-1.

Structure Type	Field Investigation Requirements
Cantilever signals, strain poles, cantilever signs, sign bridges, and luminaires	Only a site review is required if the new structures are founded in new or existing embankments known to be constructed of gravel or select borrow and compacted in accordance with Method B or C of the WSDOT Standard Specifications. Otherwise, subsurface conditions should be verified using SPT, or PPT tests and hand augers for shallower foundations) should be performed. For foundations within approximately 75 feet of each other or less, such as at a small to moderate sized intersection, one exploration point for the foundation group is adequate if conditions are relatively uniform. For more widely spaced foundation locations, or for more variable site conditions, one boring near each foundation should be obtained. The depth of the exploration point should be equal to the maximum expected depth of the foundation plus 2 to 5 feet.
Noise barriers	For noise barriers less than 100 feet in length, the exploration should occur approximately midpoint along the alignment and should be completed on the alignment of the noise barrier face. For noise barriers more than 100 feet in length, exploration points should be spaced every 200 to 400 feet, depending on the uniformity of subsurface conditions. Locate at least one exploration point near the most critical location for stability. Exploration points should be completed as close to the alignment of the noise barrier face as possible. For noise barriers placed on slopes, an additional boring off the wall alignment to investigate overall stability of the wall-slope combination should be obtained.
Building foundations	The following minimum guidelines for frequency of explorations should be used. Borings should be located to allow the site subsurface stratigraphy to be adequately defined beneath the structure. Additional explorations may be required depending on the variability in site conditions, building geometry and expected loading conditions. | Building surface area (ft²)	Exploration points (minimum)
<200	1
200 - 1000	2
1000 – 3,000	3
>3,000	3 - 4 | The depth of the borings will vary depending on the expected loads being applied to the foundation and/or site soil conditions. The borings should be extended to a depth below the bottom elevation of the building foundation a minimum of 2.5 times the width of the spread footing foundation or 1.5 times the length of a deep foundation (i.e., piles or shafts). Exploration depth should be great enough to fully penetrate soft highly compressible soils (e.g., peat, organic silt, soft fine grained soils) into competent material suitable bearing capacity (e.g., stiff to hard cohesive soil, compact dense cohesionless soil or bedrock).
Culverts (without foundation elements)	If no new fill is being placed, the culvert diameter is 3 feet or less, soft soil is known to not be present immediately below the culvert, and the culvert is installed by excavating through the fill, only a site and office review conducted as described in Chapter 2 is required, plus hand holes to obtain samples for pH and resistivity sampling for corrosion assessment for the culvert. If new fill is being placed, the borings obtained for the design of the fill itself may suffice (see Chapter 9), provided the stratigraphy below the length of the culvert can be defined. Otherwise, a minimum of two borings should be obtained, one near the one-third or one-quarter points toward each end of the culvert. For culverts greater than 300 feet in length, an additional boring near the culvert midpoint should be obtained. Borings should be located to investigate both the subsurface conditions below the culvert, and the characteristics of the fill beside and above the culvert if some existing fill is present at the culvert site. If the culvert is to be jacked through existing fill, borings in the fill and at the jacking and receiving pit locations should be obtained, to a depth of 3 to 5 feet below the culvert for the boring(s) in the fill, and to the anticipated depth of the shoring/reaction frame foundations in the jacking and receiving pits. Hand holes and portable penetrometer measurements may be used for culverts with a diameter of 3 feet or less, if the depth of exploration required herein can be obtained. Otherwise, SPT and/or CPT borings must be obtained.

In addition to the exploration requirements in Table 17-1, groundwater measurements conducted in accordance with Chapter 2 should be obtained if groundwater is anticipated within the minimum required depths of the borings as described herein.

Field Investigation Requirements for Cantilever Signals, Strain Poles, Cantilever Signs, Sign Bridges, Luminaires, Noise Barriers, and Buildings
Table 17-1

17.2   Foundation Design Requirements for Cantilever Signals, Strain Poles, Cantilever Signs, Sign Bridges, and Luminaires - General

The standard foundation designs provided in the Standard Plans for cantilever signals, strain poles, cantilever signs, sign bridges, and luminaires should be used if the applicable soil and slope conditions as described herein for each of these structures are present. If soil or rock conditions not suitable for standard foundations are present, if conditions are marginal, or if nonstandard loadings are applied, a detailed foundation analysis should be conducted. Design for cantilever signals, strain poles, cantilever signs, sign bridges, and luminaires shall be performed in accordance with the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (AASHTO, 2001).

17.2.1   Design by Correlation for Cantilever Signals, Strain Poles, Cantilever Signs, Sign Bridges, and Luminaires

WSDOT standard foundation designs for cantilever signals, strain poles, cantilever signs, sign bridges, and luminaires are based on allowable lateral bearing pressures and soil friction angles developed from correlation (Patterson, 1962) and many years of WSDOT experience for the design of these types of small foundations. The original correlation was based on the measured resistance to pull out a 1.5 inch diameter auger through the foundation soil. The correlation reported by Patterson (1962) ranged from a 200 lbs pullout force in "very soft soil" that was equated to an allowable lateral bearing of 1,000 psf, to a 750 to 1,000 lbs pullout force in "average soil" equated to an allowable lateral bearing of 2,500 psf, and to a pullout force of 2,000 to 2,500 lbs in "very hard soil" equated to an allowable lateral bearing of 4,500 psf. For WSDOT use, this correlation was conservatively related to SPT N values (uncorrected for overburden pressure) using approximate correlations between soil shear strength and SPT N values such as provided in AASHTO (1988). The allowable lateral bearing pressures that resulted from this correlation is presented in Table 17-2. This correlation is based on uncorrected N values (not corrected for overburden pressure).

A friction angle for the soil is also needed for the foundation design for these structures, typically to evaluate torsional stability. See Chapter 5 for the determination of soil friction angles, either from correlation to SPT N values, or from laboratory testing.

Table 17-2 should be used to check if standard foundation designs are applicable for the specific site. The values in Table 17-2 may also be used for special site specific foundation design to adjust depths or dimensions of standard foundations (except noise barriers) to address soil conditions that are marginal or poorer than the conditions assumed by the standard foundation design, or to address nonstandard loadings. In such cases, the values from Table 17-2 should be used as the allowable soil pressure S_1 in Article 13.10 of the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (AASHTO, 2001).

Soil Consistency as Identified in Patterson (1962)	Standard Penetration Test Resistance, N (blows/ft)	Allowable Lateral Bearing Pressure (psf)
Very Soft Soil	2	750
	3	800
	4	900
	5	1000
	6	1100
	7	1200
Poor Soil	8	1300
	9	1400
	10	1500
	11	1700
	12	1900
Average Soil	13	2100
	14	2300
	15	2500
	16	2700
	17	2900
Good Soil	18	3100
	19	3300
	20	3500
Very Hard Soil	25	4200
	30	>4500
	35	>4500

Design Parameter Correlations for the Design of Signal, Signs, Sign Bridge, and Luminaire Foundations
Table 17-2

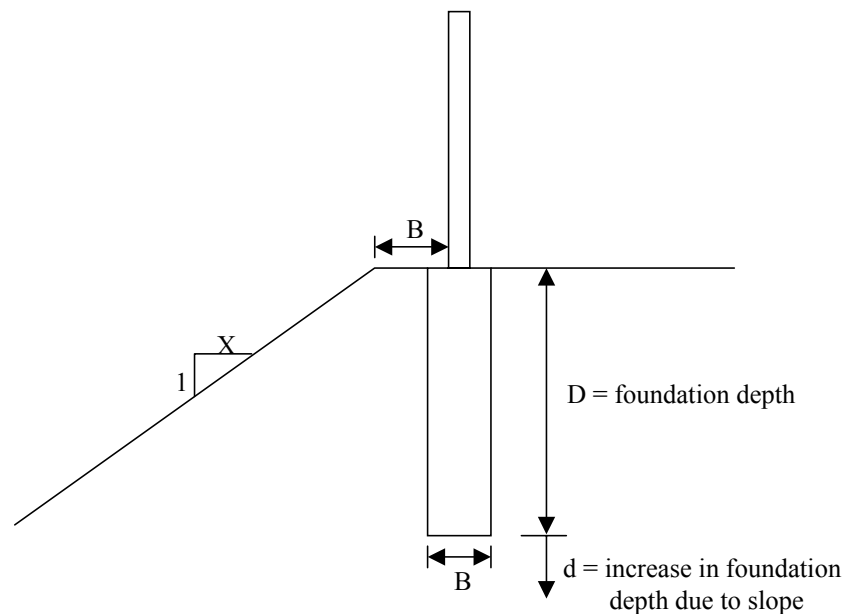Some additional requirements regarding characterization of marginal soil conditions are as follows:

- Consider the soil throughout the entire depth of the proposed foundation. Where the foundation soil is stratified, a weighted average N value should be used to design the foundation. An exception would be where soft soils are encountered at the ground surface, in which case the use of a weighted average is not appropriate.

- For foundations installed in embankments constructed from select or gravel borrow compacted using Method B or C in the WSDOT Standard Specifications, it can generally be assumed that standard foundations can be used, as such embankments will generally have "N" values of 25 or more, which are more than adequate for standard foundations. A standard foundation may also be used where 75% or more of the foundation is to be placed in new fill, provided that the foundation soil below the fill has a SPT of 8 or more. For Common Borrow compacted using Method B or C in the WSDOT Standard Specifications, standard foundations designed allowable lateral bearing pressures of 2,000 psf or less may be used.

- In general, vertical loads for sign, signal, and luminaire structure foundations are very low (i.e., 2 ksf or less) and usually do not control design. However, if it is discovered that very soft silts, clays, or peat (say, N = 4 or less) is present within the bottom 1 to 2 feet or more of the foundation, consideration should also be given to a special foundation design in this case to avoid direct bearing on these very soft soils.

The allowable lateral soil bearing values in Table 17-2 apply only to relatively flat conditions. If sloping ground is present, some special considerations in determining the foundation depth are needed. Always evaluate whether or not the local geometry will affect the foundation design. For all foundations placed in a slope or where the centerline of the foundation is less than 1B for the shoulder of the slope (B = width or diameter of the Standard Foundation), the Standard Plan foundation depths should be increased as follows, and as illustrated in Figure 17-1:

- For slopes 3H:1V or flatter, no additional depth is required.
- For 2H:1V or flatter, add 0.5B to the depth.
- For 1.5H:1V slopes, add 1.0B to the depth.

Interpolation between the values is acceptable. These types of foundations should not be placed on slopes steeper than 1.5H:1V. If the foundation is located on a slope that is part of a drainage ditch, the top of the standard foundation can simply be located at or below the bottom of the drainage ditch.

Foundation Design Detail for Sloping Ground
Figure 17-1

Note that these sloping ground recommendations do not apply to luminaire foundations.

When a nonstandard foundation design using Table 17-2 is required, the geotechnical designer must develop a table identifying the soil units, soil unit boundary elevations, allowable lateral bearing pressure, and soil friction angle for each soil unit. The structural designer will use these data to prepare the nonstandard foundation design.

17.2.2   Special Design for Cantilever Signals, Strain Poles, Cantilever Signs, Sign Bridges, and Luminaires

For foundations in rock, a special design is always required, and Table 17-2 is not applicable. Fracturing and jointing in the rock, and its effect on the foundation resistance, must be evaluated. In general, a drilled shaft or anchored footing foundation will be required. Foundation designs based on Table 17-2 are also not applicable if the foundation soil consists of very soft clays, silts, organic silts, or peat. In such cases, a footing designed to "float" above the very soft compressible soils, over-excavation and replacement with higher quality material, or very deep foundations are typically required.

For shaft type foundations in soil, the Broms Method as specified in the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (AASHTO, 2001) or the procedures specified in Chapter 8 for lateral load analysis of deep foundations (e.g., P-y analysis) should be used for conditions where Table 17-2 is not applicable, or as an alternative to Table 17-2 based design. For shafts in rock, nominal lateral resistance should be estimated based on the procedures provided in Chapter 8. This means that for special lateral load design of shaft foundations, the geotechnical designer will need to provide P-y curve data to the structural designer to complete the soil-structure interaction analysis. For spread footing design, the design methods provided in Chapter 8 to estimate nominal bearing resistance and settlement should be used, but instead of the referenced load groups and resistance factors, the AASHTO Standard Specifications for Highway Bridges (2002) combined with a minimum bearing capacity safety factor of 2.3 for Load Factor Design (LFD), or 3.0 for allowable stress or service load design (ASD) should be used for static conditions, and a safety factor of 1.1 should be used for seismic conditions, if seismic conditions are applicable. Note that in general, the foundations for the types of structures addressed in this chapter are not mitigated for liquefaction (see Chapter 6). For anchored footing foundations over bedrock, anchor depth, spacing, and nominal resistance shall be assessed considering the degree of fracturing and jointing in the rock (see Chapters 5, 8, and 12 for design requirements).

17.2.3   Cantilever Signals and Strain Pole Standards

17.2.3.1   Overview

There are eight types of cantilever signal and strain poles standards that are covered in Section J-7 of the WSDOT Standard Plans. Type PPB (pedestrian push bottom pole), PS (pedestrian head standard), Type I/RM (vertical head and ramp meter), Type FB (flashing beacon standard) and Type IV (strain pole standard) are structures that generally consist of a single vertical metal pole member. Type II (mast arm standard), Type III (lighting and mast arm standard) and Type V (lighting and strain pole standard) have a vertical metal pole member with a horizontal mast arm. Lights and/or signals will be suspended from the mast arm. The standard signal foundations designs assume that the foundation soil is capable of withstanding the design lateral soil bearing pressure created by wind and dead loads. The details on the foundation designs can be found in Section J-7 of the *Standard Plans*, in the <u>Signing Foundations Chapter 1020 and Signal Foundations Chapter 1330</u> of the *Design Manual* M 22-01.

17.2.3.2   Standard Foundation Designs

The standard foundations for these structures consist of square or round shafts that vary in diameter from 1.5 feet to 3.0 feet for square and 2.0 feet to 4.0 feet for round shaft foundations. The standard designs assume a concrete to soil contact. For structure types PPB, PS and I/RM, the foundation depths are quite shallow and vary between 1.5 feet and 3.0 feet in depth. Foundation depths vary from 6 feet to 15 feet for signal structure Types II, III, IV and V. Standard foundations for signal structures Types PPB, PS and I are designed for 1500 psf (N ≥ 10 bpf) average allowable lateral bearing pressure. Standard foundations for signal structures Types II, III, IV and V have been designed for 1000 psf (N ≥ 5 bpf), 1500 psf (N ≥ 10 bpf), and 2500 psf (N ≥ 15 bpf) average allowable lateral bearing pressure. If the foundation is placed in new compacted fill – standard foundations may be used as specified in Section 17.2.1.

For round shafts, the standard foundation designs assume for torsional stability that the soil to foundation contact friction angle is 30º, which is typical for concrete cast against soil for moderate strength soils.

17.2.3.3   Construction Considerations

Structures that require short round or square foundations (i.e. < than 9 feet) could be easily formed in an open excavation. The backfill placed around the foundation in the excavation must be compacted in accordance with the WSDOT Standard Specifications M41-10, Section 2-09.3(1)E and using high quality soil backfill. Foundation construction shall be in accordance with the WSDOT Standard Specifications M41-10, Sections 8-20.3(2) and 8-20.3(4). Following the removal of the concrete forms (the forms can be left in place if corrugated metal pipe is used), compacted backfill shall be placed around the shaft to provide containment. If the backfill cannot be properly compacted, then controlled density fill could be used instead.

Deep shaft foundations greater than 9 feet may require the use of temporary casing, slurries or both. Generally in most cases, the temporary casing can be removed. Special foundations designs may be required if the geotechnical designer determines that permanent casing is necessary. In this situation, the structural designer must be informed of this condition. These structures are under lateral and rotational loads. The shear capacity of the foundation under a rotational force is reduced if steel casing remains in the ground. It is important to note here that if the foundation design assumes that the soil around the shaft, assuming the contractor makes an open excavation and then backfills the excavation cavity around the formed foundation, is properly compacted, the degree of compaction is somehow verified in the field. The geotechnical designer needs to make sure that the construction specifications are clear in this regard, and that the project inspectors know what needs to be done to enforce the specifications. If the degree of compaction cannot be verified in the field due to the depth of the open excavation and safety regulations, this needs to be taken into consideration in the selection of soil design parameters. The specifications also need to be clear regarding the removal of temporary forms (e.g., sonotubes) for the foundations. If for some reason they cannot be removed due to the depth of the hole or other reasons, sonotubes should not be used. Instead, corrugated metal pipe should be used so that torsional resistance of the foundation is maintained.

17.2.4   Cantilever and Sign Bridges

17.2.4.1   Overview

Sign bridge foundation details are shown in the WSDOT Standard Plan G-2a. There are three foundations types and they are identified as Type 1, 2 and 3. Type 1 sign bridge foundations consist of a single 3 feet diameter drilled shaft with a shaft length that can vary between 11.5 and 16.5 feet. The shaft length is a function of the sign bridge span length which can vary less than 60 feet to a maximum of 150 feet. Type 2 and 3 foundations consist of massive concrete trench foundations that are 3 feet × 10 feet in plan area with a embedment that can vary between 5.5 feet to 11.5 feet depending on span length. All designs assume a concrete to soil contact.

There are three cantilever sign foundation types in the WSDOT Standard Plans. The structural details are shown in Standard Plan G-3a. These foundations are similar to the sign bridge foundations. Type 1 cantilever sign foundations consist of two 10 feet long drilled shafts. The Type 2 and 3 foundations are a massive concrete trench foundation that is 3 feet × 10 feet in plan area with an embedment that can vary between 8 feet and 12.5 feet. Embedment depth of the foundation is controlled by the total square feet of exposed sign area. All designs assume a concrete to soil contact.

17.2.4.2   Standard Foundation Designs

Standard foundation for cantilevered and sign bridges Types 1 and 2 have been prepared assuming the site soils meet a minimum 2,500 psf allowable lateral bearing pressure. Using the Table 17-2, a soil with a penetration resistance N ≥ 15 would provide adequate support for these structures. A Type 3 foundation was designed for slightly poorer soils using a lateral bearing pressure of 1,500 psf for structural design. Using Table 17-2, a soil with a penetration resistance of ≥10 bpf would provide adequate lateral resistance for a Type 3 foundation.

17.2.4.3   Construction Considerations

The construction of the trench footings may be performed as a cast-in-place foundation that is poured directly against the soils, or they could be constructed in a large open excavation using wide trench boxes and concrete forms. If a standard foundation design is to be used, but is installed in an open excavation, the backfill placed around the foundation in the excavation must be compacted in accordance with Method C of the WSDOT Standard Specifications and using high quality soil backfill.

The geotechnical designer must evaluate the stability of open excavations. Obviously, high groundwater could affect the stability of the side slopes of the excavation. Casing for drilled shafts or shoring boxes for the trench footing would be required under these conditions. All of these foundations have been designed assuming a concrete to soil contact. Generally in most cases, the temporary casing for drilled shafts can be removed. Special foundations designs may be required if the geotechnical designer determines that permanent casing is necessary. In this situation, the structural engineer must be informed of this condition. These structures are under lateral and rotational loads. The shear capacity of the foundation under a rotational force is reduced if steel casing remains in the ground.

It is important to note here that if the foundation design assumes that the soil around the shaft, assuming the contractor makes an open excavation and then backfills the excavation cavity around the formed foundation, is properly compacted, the degree of compaction is somehow verified in the field. The geotechnical designer needs to make sure that the construction specifications are clear in this regard, and that the project inspectors know what needs to be done to enforce the specifications. If the degree of compaction cannot be verified in the field due to the depth of the open excavation and safety regulations, this needs to be taken into consideration in the selection of soil design parameters. The specifications also need to be clear regarding the removal of temporary forms (e.g., sonotubes) for the foundations. If for some reason they cannot be removed due to the depth of the hole or other reasons, sonotubes should not be used. Instead, corrugated metal pipe should be used so that torsional resistance of the foundation is maintained.
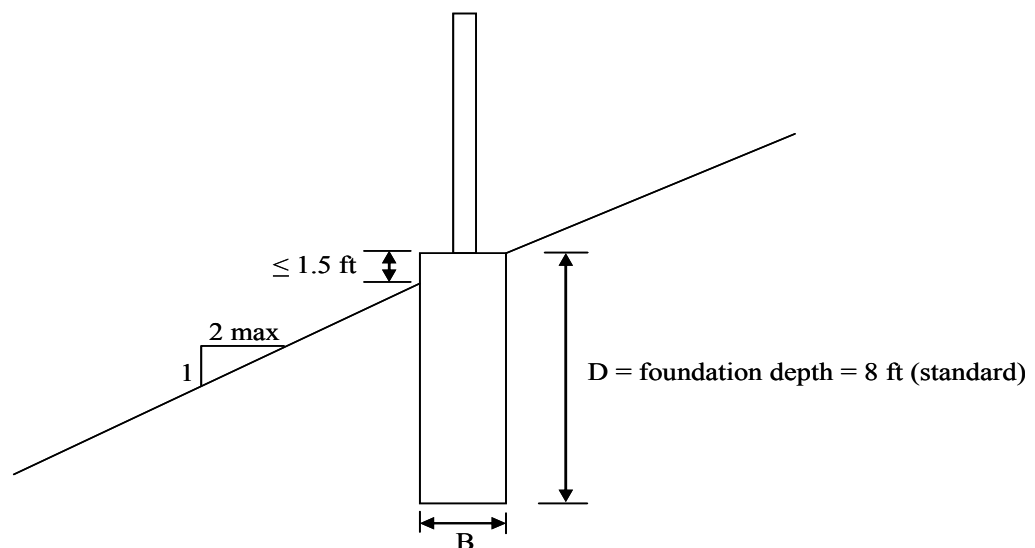
17.2.5   Luminaires (Light Standards)

17.2.5.1   Overview

Standard luminaire (light standard) foundations consist of 3 feet diameter round shafts. The foundation details are shown in WSDOT Standard Plan J-1b. The standard foundation depth is 8 feet.

17.2.5.2   Standard Foundation Design

Standard foundations for luminaires (light standards) have been prepared assuming the site soils meet a minimum 1,500 psf allowable lateral bearing pressure. Using the Table 17-2, a soil with a penetration resistance $N \geq 10$ would provide adequate support for these structures. The standard foundation design is applicable for foundations on slopes of 2H:1V or flatter as shown in Figure 17-2.

The standard foundation designs assume for torsional stability that the soil to foundation contact friction angle is 30°, which is typical for concrete cast against soil for moderate strength soils.

Luminaire Foundation Design Detail for Sloping Ground
Figure 17-2

17.2.5.3   Construction Considerations

Luminaire foundations could be easily formed in an open excavation. The backfill placed around the foundation in the excavation must be compacted in accordance with the WSDOT Standard Specifications M41-10, Section 2-09.3(1)E and using high quality soil backfill. Foundation construction shall be in accordance with the WSDOT *Standard Specifications* M41-10, Sections 8-20.3(2) and 8-20.3(4). Following the removal of the concrete forms (the forms can be left in place if corrugated metal pipe is used), compacted backfill shall be placed around the shaft to provide containment. If the backfill cannot be properly compacted, then controlled density fill could be used instead.

Deep shaft foundations (i.e., special designs) greater than 9 feet may require the use of temporary casing, slurries or both. Generally, in most cases, the temporary casing can be removed. Special foundations designs may be required if the geotechnical designer determines that permanent casing is necessary. In this situation, the structural designer must be informed of this condition. These structures are under lateral and rotational loads. The shear capacity of the foundation under a rotational force is reduced if steel casing remains in the ground.

It is important to note here that if the foundation design assumes that the soil around the shaft, assuming the contractor makes an open excavation and then backfills the excavation cavity around the formed foundation, is properly compacted, the degree of compaction is somehow verified in the field. The geotechnical designer needs to make sure that the construction specifications are clear in this regard, and that the project inspectors know what needs to be done to enforce the specifications. If the degree of compaction cannot be verified in the field due to the depth of the open excavation and safety regulations, this needs to be taken into consideration in the selection of soil design parameters. The specifications also need to be clear regarding the removal of temporary forms (e.g., sonotubes) for the foundations. If for some reason they cannot be removed due to the depth of the hole or other reasons, sonotubes should not be used. Instead, corrugated metal pipe should be used so that torsional resistance of the foundation is maintained.

17.3   Noise Barriers

17.3.1   Overview

There are 20 standard designs for noise barriers that are covered in WSDOT Standard Plans D-2a through D-2t. The Standard Plans contains detailed designs of seven cast-in-place concrete, seven pre-cast concrete, and five masonry block noise barriers.

Three foundation options are available for the cast-in-place and pre-cast concrete barriers. They include round shafts and spread footings. The spread footing foundation option has two designs. One design consists of an offset panel and a second design consists of a uniform panel where the panel wall bears in the middle of the footing. The following is a summary of the critical design elements of noise barrier walls:

• All noise barrier spread footing standard foundations have been designed assuming an allowable bearing pressure of 2 kips per square foot (ksf).

• The diameter and length of the standard shaft foundations can also vary with soil condition, exposed panel height and loading condition. The lengths vary from 4.75 feet to 13.25 feet, and shaft diameters vary between 1.0 to 2.5 feet.

17.3.2   *Foundation Design Requirements for Noise Barriers*

Foundation design for noise barrier shall be conducted in accordance with the most current AASHTO Guide Specifications for Structural Design of Sound Barriers, including interims (AASHTO 1989). Currently, design of noise barriers is based on Load Factor Design (LFD). Therefore, the load factors and safety factors specified in the AASHTO manual for sound barrier foundation design, except as specifically required in this chapter of the GDM, should be used.

In addition, the geotechnical designer shall perform a global stability analysis of the noise barrier when the barrier 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 1.3 or greater for static conditions and 1.1 for seismic conditions. Note that in general, the foundations for noise barriers are not mitigated for liquefaction (see Chapter 6).

All Standard Plan noise barrier structures have been designed to retain a minimal amount of soil that must be no more than 4 feet in height with a level backslope. The retained soil above the noise barrier 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 noise barrier foundation designs shall include the effects of any differential fill height between the front and back of the wall.

17.3.2.1   Spread Footings

For spread footing design, the design methods provided in Chapter 8 to estimate nominal bearing resistance and settlement should be used, but instead of the referenced load groups and resistance factors, the AASHTO Guide Specifications for Structural Design of Sound Barriers (1989) and AASHTO Standard Specifications for Highway Bridges (2002) combined with a minimum bearing capacity safety factor of 2.3 for Load Factor Design (LFD), or 3.0 for allowable stress or service load design (ASD) should be used for static conditions, and a safety factor of 1.1 should be used for seismic conditions, if seismic conditions are applicable. Note that in general, the foundations for noise barriers are not mitigated for liquefaction (see Chapter 6).

The noise barrier footing shall be designed to be stable for overturning and sliding. The methodology and safety factors provided in the AASHTO Standard Specifications for Highway Bridges (2002) applicable to gravity walls in general for overturning and sliding (FS of 2.0 and 1.5, respectively for static conditions, and 1.5 and 1.1 for seismic conditions), shall be used to assess noise barrier stability for these two limit states, using service loads.

The geotechnical designer will also be responsible to estimate foundation settlement using the appropriate settlement theories and methods as outlined in Chapter 8. The geotechnical designer will report the estimated total and differential settlement.

The soil properties (unit weight, friction and cohesion) shall be determined using the procedures described in Chapter 5.

Noise barrier footings shall be located relative to the final grade to have a minimum soil cover over the top of the footing of 2 feet.

For the Standard Plan noise barrier footing foundation, the geotechnical designer shall use the procedures described above to estimate the allowable bearing resistance for the foundation with consideration to the actual site and subsurface conditions for the wall, and to verify that the allowable bearing resistance is greater than the standard foundation design bearing stress of 2.0 ksf. Note that the standard noise barrier foundations have been designed to resist a PGA of 0.35g. This corresponds to a peak bedrock acceleration (PBA) from Figure 6-6 in Chapter 6 of 0.3g and an amplification factor of 1.18, corresponding to stiff soil.

For nonstandard noise barrier designs, use Mononabe-Okabe analysis in accordance with Chapter 15 to determine the seismic earth pressure if the noise barrier retains soil.
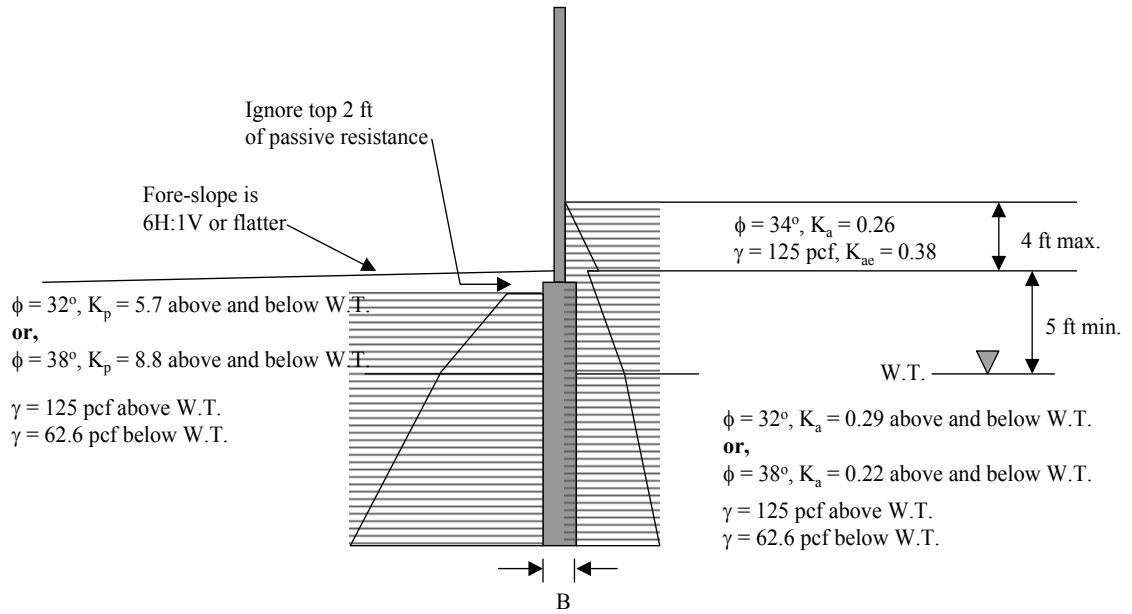
17.3.2.2 Shaft Foundations

In general, shaft supported noise barriers are treated as non-gravity cantilever walls for foundation design. Shaft foundations have been designed for Standard Plan noise barriers using two soil strength conditions. D1 and D2 trench and shaft foundations have been designed assuming a soil friction of 32 and 38 degrees respectively. The geotechnical designer is responsible to determine the in-situ soil strength parameters using the appropriate field correlations and/or laboratory tests as described in Chapter 5. The geotechnical designer provides recommendations as to which deep foundation(s) is appropriate for inclusion in the contract plans. If the soil strength parameters lie between 32 and 38 degrees, the foundation design based on 32 degrees shall be used if a Standard Plan wall is to be used. If multiple soil layers of varying strength have been identified within the depth of the trench or shaft foundation, soil strength averaging may be used to select the appropriate standard foundation type and depth. For example, if the average soil strength along the length of the shaft is 38º or more, the 38º standard foundation may be used.

The standard foundation designs used for the Standard Plan noise barriers are based on the following assumptions:

- Noise barrier standard foundation designs assume one of the following:
  - The wall is founded at the crest of a 2H:1V slope with a minimum of 3 feet of horizontal distance between the panel face and the slope break. The top 2 feet of passive resistance below the assumed ground surface at the noise barrier face is ignored in the development of the wall pressure diagram. For this case, groundwater must be at or below the bottom of the noise barrier foundation.
  - The wall is founded on a near horizontal slope (i.e., 6H:1V or flatter) with a minimum of 3 feet of horizontal distance between the panel face and the slope break. The top 2 feet of passive resistance below the assumed ground surface at the noise barrier face is ignored in the development of the wall pressure diagram. For this case, groundwater must be at or below 5 feet below the top of the noise barrier foundation.

- The standard shaft foundation designs have been designed for two different soil conditions, assuming the slope conditions in front of the wall as indicated above. One design assumes an average soil friction angle of 32 degrees (D1), resulting in a design K_p of 1.45 (2H:1V slope) or 5.7 (near horizontal slope) and K_a of 0.29, and the second design assumes an average soil friction angle of 38 degrees (D2), resulting in a design K_p of 2.2 (2H:1V slope) or 8.8 (near horizontal slope) and K_a of 0.22. All values of K_a and K_p reported above have been corrected to account for the angular deviation of the active or passive force from the horizontal (in these design cases, the correction factor, Cos (δ), where δ is the interface friction angle, is approximately equal to 0.9 to 0.93). The standard shaft foundation designs are based on standard earth pressure theory derived using logarithmic spiral method for K_p and the Coulomb method for K_a, assuming the interface friction between the foundation and the soil to be 0.67φ. A unit weight of 125 pcf was also assumed in the design. This unit weight assumes that the ground water level at the site is below the bottom of the noise barrier foundation. For the case where groundwater is considered, the effective unit weight of the soil is used below the water table (i.e., 62.6 pcf). For the shaft foundation design, it is assumed that the passive earth pressure is applied over a lateral distance along the wall of 3B, where B is the shaft diameter and 3.0 is the magnitude of the isolation factor for discrete shafts, or the center-to-center spacing of the shafts, whichever is less. A factor of safety of 1.5 should also applied to the passive resistance.

- The PGA for seismic design is assumed to be 0.35g. This corresponds to a peak bedrock acceleration (PBA) from Figure 6-6 in Chapter 6 of 0.3g and an amplification factor of 1.18, corresponding to stiff soil. K_{ae}, the seismic lateral earth pressure coefficient, was developed assuming that the acceleration A = 0.5PGA.

- All standard foundation designs assume a concrete to soil contact.

- Figures 17-3 and 17-4 illustrate the assumptions used for the standard trench or shaft foundation designs.
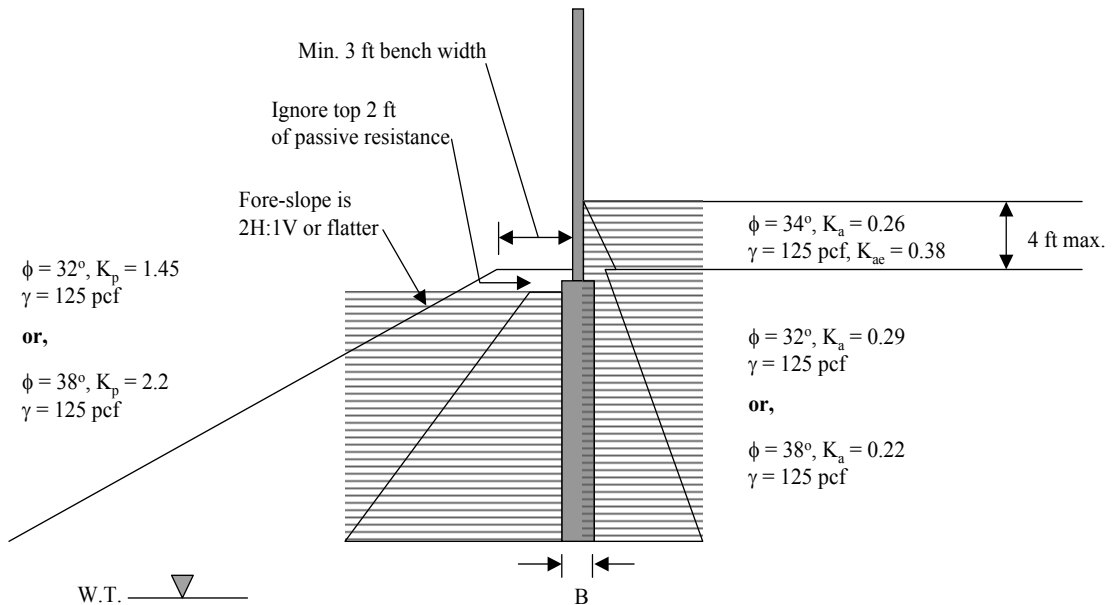
Special designs will be required if the site and soil conditions differ from those conditions assumed for design.

Ignore top 2 ft
of passive resistance

Fore-slope is
6H:1V or flatter

$\phi = 34^o$, $K_a = 0.26$
$\gamma = 125$ pcf, $K_{ae} = 0.38$

4 ft max.

5 ft min.

$\phi = 32^o$, $K_p = 5.7$ above and below W.T.
or,
$\phi = 38^o$, $K_p = 8.8$ above and below W.T.

$\gamma = 125$ pcf above W.T.
$\gamma = 62.6$ pcf below W.T.

W.T.

$\phi = 32^o$, $K_a = 0.29$ above and below W.T.
or,
$\phi = 38^o$, $K_a = 0.22$ above and below W.T.

$\gamma = 125$ pcf above W.T.
$\gamma = 62.6$ pcf below W.T.

B

Use $3K_p$ applied to foundation width, B, for discrete foundation units (shafts), and $1.0K_p$ for trench foundation.
Use FS = 1.5 applied to K_p (K_p values shown above have not been factored).
K_a is applied over foundation width, B.

Standard Foundation Design Assumptions for Shaft or Trench
Foundations, Assuming Near Level Ground Conditions and Ground
Water Above Bottom of Foundation
Figure 17-3

Min. 3 ft bench width

Ignore top 2 ft
of passive resistance

Fore-slope is
2H:1V or flatter

$\phi = 34^o$, $K_a = 0.26$
$\gamma = 125$ pcf, $K_{ae} = 0.38$

4 ft max.

$\phi = 32^o$, $K_p = 1.45$
$\gamma = 125$ pcf

or,

$\phi = 38^o$, $K_p = 2.2$
$\gamma = 125$ pcf

$\phi = 32^o$, $K_a = 0.29$
$\gamma = 125$ pcf

or,

$\phi = 38^o$, $K_a = 0.22$
$\gamma = 125$ pcf

W.T.

B

Use $3K_p$ applied to foundation width, B, for discrete foundation units (shafts), and $1.0K_p$ for trench foundation.
Use FS = 1.5 applied to K_p (K_p values shown above have not been factored).
K_a is applied over foundation width, B.

Standard Foundation Design Assumptions for Shaft or Trench Foundations,
Assuming 2H:1V Slope in Front of Wall and Ground Water Below Foundation
Figure 17-4

17.3.2.3   Non-Standard Foundation Design

A non-standard foundation design will be required if the site or soil conditions are not consistent with the conditions assumed for the standard foundation designs as described in Section 17.3.4.2. For example, if slopes steeper than 2H:1V are present below the wall, if the soil is weaker than 32°, or if the ground water level is above the bottom of the foundation (Figure 17-4), a non-standard foundation design will be needed. If the foundation must be installed in rock, a non-standard foundation may also be required.

If non-standard foundation designs are required, the geotechnical designer should provide the following information to the structural designer:

- Description of the soil units using Unified Soil Classification System (Chapters 4 and 5).
- Ground elevation and elevation of soil/rock unit boundaries.
- Depth to the water table along the length of the wall.
- Earth pressure diagrams and design parameters developed in accordance with Chapter 15 and this section. Soil unit strength parameters that include effective unit weight, cohesion, φ, K_a, K_p, and K_{ae}. For shaft foundations, passive pressures are assumed to act over 3 shaft diameters, and a factor of safety of 1.5 should be applied to the passive resistance.
- The allowable bearing resistance for spread footings and estimated wall settlement.
- Overall wall stability.
- Any foundation constructability issues resulting from the soil/rock conditions.

The structural designer will use this information to develop a special foundation design for the noise barrier.

17.3.3   Construction Considerations

The presence of a high groundwater table could affect the construction of shaft foundations. The construction of noise barriers with shaft foundations would be especially vulnerable to caving if groundwater is present, or if have lose clean sands or gravels. The concrete in all shaft foundations have been designed to bear directly against the soils. Generally, temporary casing for drilled shafts should be removed. Special foundations designs may be required if the geotechnical designer determines that permanent casing is necessary. In this situation, the structural engineer must be informed of this condition.

17.4   Culverts

17.4.1   Overview

This section only addresses culverts, either flexible or rigid, that do not require foundation elements such as footing or piles. Culverts that require foundation elements are addressed in Chapter 8.

17.4.2   Culvert Design and Construction Considerations

Culvert design shall utilize the LRFD approach. For culverts, the soil loads and design procedures to be used for design shall be as specified in Sections 3 and 12 of the AASHTO LRFD Bridge Design Specifications. The following design situations are typically encountered regarding culverts:

1. The culvert simply needs to be replaced because of performance problems (e.g., leaking, partial collapse, or undersized), or a new culvert is needed, and open excavation is used to remove and replace the culvert, or to install the new culvert, and the excavation is simply backfilled.

2. The culvert simply needs to be replaced because of performance problems (e.g., leaking, partial collapse, or undersized), or a new culvert is needed, and the culvert is installed by "jacking" it through the existing embankment.

3. An existing culvert is extended and new fill is placed over the culvert.

For case 1, little geotechnical design is needed. The soil conditions in the fill and just below the culvert should be investigated, primarily to assess constructability issues such as excavation slopes and shoring design (usually done by the contractor). If soft soils are present near the bottom of the culvert, the feasibility of obtaining stable excavation slopes of reasonable steepness should be assessed. The presence of boulders in the fill or below the fill, depending on the shoring type anticipated, could influence feasibility. However, settlement and bearing issues for the new or replaced culvert should not be significant, since no new load is being placed on the soil below the culvert.

For case 2, the effect of the soil conditions in the fill on the ability to jack the culvert through the fill should be evaluated. Very dense conditions or the presence of obstructions in the fill such as boulders could make jacking infeasible. Ground water within the fill or the presence of clean sands or gravels that could "run" could again make jacking problematic, unless special measures are taken by the contractor to prevent caving. Since a stable jacking platform must be established, along with the shoring required to form the jacking and receiving pits, deeper test hole data adequate for shoring design must be obtained and analyzed to assess earth pressure parameters for shoring design, and to design the reaction frame for the jacking operation.

For case 3, differential and total settlement along the culvert is the key issue that must be evaluated, in addition to the case 1 issue identified above. See Chapter 9 for the estimation of settlement due to new fill.

17.5   Buildings

17.5.1   Overview

The provisions of this section cover the design requirements for small building structures typical of WSDOT rest areas, maintenance and ferry facilities. It is assumed these buildings are not subject to scour or water pressure by wind or wave action. Typically, buildings may be supported on shallow spread footings, or on pile or shaft foundations for conditions where soft compressible soils are present.

17.5.2   Design Requirement for Buildings

Foundations shall be designed in accordance with the provisions outlined in Chapter 18 of the 2003 International Building Code (IBC, 2002). This design code specifies that all foundations be designed using allowable stress design methodology. Table 1804.2 from the IBC provides presumptive values for allowable foundation bearing pressure, lateral pressure for stem walls and earth pressure parameters to assess lateral sliding. Note that these presumptive values account for both shear failure of the soil and settlement or deformation, which has been limited to 1 inch.

Materials	Allowable Foundation Pressure (psf)[d]	Lateral Bearing (psf/ft below natural grade)[d]	Coefficient of friction[a]	Resistance (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	-----
2. Sedimentary and foliated rock	4,000	400	0.35	-----
3. Sandy gravel and/or gravel (GW & GP)	3,000	200	0.35	-----
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM, and GC)	2,000	150	0.25	-----
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH	1,500[c]	100	------	130

a. Coefficient to be multiplied by the dead load.

b. Lateral sliding resistance value to be multiplied by the contact area, as limited by Section 1804.3 of the 2003 IBC.

c. Where the building official determines that in-place soils with an allowable bearing capacity of less than 1,500 psf are likely to be present at the site, the allowable bearing capacity shall be determined by a soils investigation.

d. An increase on one-third is permitted when using the alternate load combinations in Section 16.3.2 of the 2003 IBC that include wind or earthquake loads.

**Allowable Foundation and Lateral Pressure
(as Provided in 2003 IBC, in Table 1804.2)**
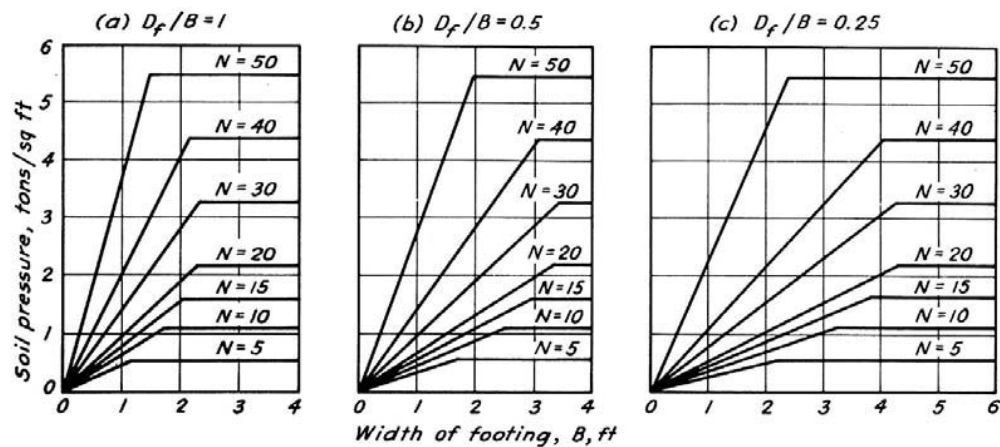*Table 17-3*

In addition to using the 2003 IBC design code, the geotechnical designer should perform a foundation bearing capacity analyses (including settlement) using the methods outlined in Chapter 8 to obtain nominal resistance values. These design methods will result in ultimate (nominal) capacities. Normally, allowable stress design is conducted for foundations that support buildings and similar structures. Appropriate safety factors must be applied to determine allowable load transfer. Factors of safety to be used for allowable stress design of foundations shall be as follows:

Load Group	Method	*Minimum Geotechnical Factor of Safety, FS		
		Spread Footings	Shafts	Piles
ASD (unfactored DL + LL, or service load level)	Static shear strength analysis from soil/rock properties, compression	3.0	2.5	2.5
	Static analysis from soil/rock properties, uplift		3.0	3.0
	Load test conducted (number of tests depends on uniformity of conditions)		2.0	2.0
	WSDOT driving formula			2.5
	Wave equation with PDA (min. one per pier and 2 to 5% of the piles			2.5
	PDA with CAPWAP (min. one per pier and 2 to 5% of the piles			2.25

Minimum Factors of Safety for ASD Foundation Design
Table 17-4

The results of the ASD foundation bearing capacity analyses, after reducing the foundation bearing capacity by the specified FS from Table 17-4, and further reduced to meet settlement criteria for the foundation (normally, no FS is applied for settlement analysis results), should be checked against the IBC design code, and the most conservative results used.

For allowable stress design, spread footings on sandy soils may alternatively be designed for bearing and settlement by using Figure 17-5. When using Figure 17-5, a FS from Table 17-4 does not need to be applied, as the bearing stresses in the figure represent allowable bearing resistances. The design bearing resistance in Figure 17-5 has been developed assuming footing settlement will be limited to no more than 1 inch. The N-values needed to estimate bearing resistance in the figure should be determined from SPT blow counts that have been corrected for both overburden pressure and hammer efficiency, and hence represent $N1_{60}$ values (see Chapter 5).

Design Chart for Proportioning Shallow Footings on Sand
(After Peck, et al., 1974)
Figure 17-5

Note that other issues may need to be addressed regarding the design of buildings and associated structures. For example, significant earthwork may be required. For cut and fill design, see Chapters 9 and 10. For the stabilization of unstable ground, see Chapter 13. If ground improvement is required, see Chapter 11. If retaining walls are required, see Chapter 15.

If septic drain field(s) are needed, local regulations will govern the geotechnical design, including who is qualified to perform the design (i.e., a special license may be required). In general, the permeability of the soil and the maximum seasonal ground water level will need to be assessed for septic system designs.

Note that in general, the foundations for the types of structures addressed in this chapter are not mitigated for liquefaction (see Chapter 6). However, for building foundations, liquefaction and other seismic hazards are at least assessed in terms of the potential impact to the proposed structures. Liquefaction and other seismic hazards are mitigated for building and other structures for which the International Building Code (IBC) governs and mitigation is required by the IBC.

17.6   References

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