Geotechnical Manual - LRFD

Bridge Division

April 2024

Manual Notice 2024-1

From: Graham Bettis, P.E., Director, Bridge Division

Manual: Geotechnical Manual - LRFD

Effective Date: April 26, 2024

Purpose

The purpose of this manual is to document policy on geotechnical characterization and design for bridges, retaining walls, slopes, and ancillary structures in Texas transportation projects. This manual replaces the July 2022 TxDOT Geotechnical Manual and must be used for all projects with designs beginning after July 31st, 2024.

Contents

The manual incorporates LRFD into TxDOT geotechnical evaluation and design. This manual includes chapters that address geotechnical evaluation, design, and quality controls.

Contact

For more information about any portion of this manual, please contact the TxDOT Bridge Division.

Archives

Past manual notices are available in a pdf archive.

Table of Contents

Chapter 1: About this Manual
Section 1: Introduction . . . 1-2
Purpose of the Manual . . . 1-2
Updates . . . 1-3
Organization . . . 1-3
Feedback . . . 1-4

Chapter 2: Investigations
Section 1: Subsurface Investigations . . . 2-2
Overview . . . 2-2
Review of Existing Data . . . 2-2
Boring Location . . . 2-2
Bridges . . . 2-3
Retaining Walls . . . 2-6
Other Structures . . . 2-7
Slopes and Embankments . . . 2-7

Chapter 3: Field Operations
Section 1: Drilling Preparation . . . 3-2
Drilling Considerations . . . 3-2
Access . . . 3-2
Utility Clearance . . . 3-2
Traffic Control . . . 3-2
Section 2: Drilling and Sampling Methods . . . 3-3
Drilling Overview . . . 3-3
Sampling Overview . . . 3-3
Section 3: Standard Field Testing and Sampling . . . 3-5
Standard Penetration Test (SPT) . . . 3-5
Thin-Walled Tube Sampling (Shelby Tube) . . . 3-5
Storing and Transport of Thin-Walled Tube Samples . . . 3-5
Bedrock Coring . . . 3-6
Caring, Handling, Storing, and Transport Rock Core Samples . . . 3-6
Pocket Penetrometer . . . 3-7
In-Place Vane Shear Test . . . 3-7
Torvane . . . 3-7
Section 4: Post-Drilling . . . 3-8
Borehole Backfilling . . . 3-8

Chapter 4: Subsurface Classification

Section 1: Soil and Bedrock Logging 4-2
Material Order of Description 4-2
Material 4-2
Unified Soil Classification System (ASTM D2487) 4-2
Relative Density or Consistency, Hardness 4-3
Relative Rock Strength 4-5
Moisture 4-6
Color 4-6
Cementation 4-6
Descriptive Adjectives 4-7
Weathered State of Rock 4-7
Rock Core Grain Size 4-8
Bedding and Discontinuity Spacing. 4-9
Discontinuity Condition. 4-9
Classification of Intact Rock and Rock Mass 4-10
Percent Recovery and Rock Quality Designation (RQD) 4-12
Fracture Frequency (FF) 4-14
Geological Strength Index (GSI) 4-14
Geotechnical Report. 4-14
Boring Log Format 4-15
Section 2: Laboratory Testing 4-16
Overview 4-16
Moisture Content Tests 4-16
Unit Weight 4-16
Grain Size Analysis and Passing No. 200 Sieve 4-16
Atterberg Limit Tests 4-16
Unconfined Compressive Strength Tests and Unconsolidated, Undrained Triaxial Compression Tests 4-16
Consolidated Undrained Triaxial Compression Tests (CU testing) 4-17
Direct Shear Tests 4-17
One Dimensional Consolidation Tests 4-17
One Dimensional Swell Tests 4-17
Resistivity, pH, Sulfates and Chloride Ion Content in Soils 4-17
Organic Content 4-17
Section 3: Quality Assurance and Quality Control 4-18
Overview 4-18
Sampling 4-18
Logger Qualification 4-18

Tester Qualification 4-18
Equipment 4-19
Laboratory Accreditation 4-19

Chapter 5: Foundation Design
Section 1: Design Methodology. 5-2
Overview 5-2
Loading and Resistance 5-2
Service Limit States 5-2
Strength Limit States 5-3
Extreme Event Limit States 5-3
Constructability 5-4
Design Process 5-4
Lateral 5-5
Group Effects 5-5
Section 2: Foundation Selection 5-6
Overview 5-6
Factors for Selection 5-6
Foundation Guidelines for Widening Structures 5-7
Section 3: Interpretation of Soil Data 5-8
Overview 5-8
In-situ vs. Lab Data 5-8
Disregard Depth 5-8
Drilling Data, Laboratory Data, and Subsurface Classification. 5-9
Capacity From Texas Cone Penetration Test 5-9
Section 4: Uplift and Downdrag 5-10
Overview 5-10
Uplift 5-10
Downdrag. 5-10
Section 5: Drilled Shafts 5-12
Overview 5-12
Resistance in Soils 5-12
Resistance in Rock and Intermediate Geomaterials. 5-12
Resistance Factors 5-13
Belled Shafts 5-13
Standing Water 5-13
Micropiles 5-13
Wing Wall Drilled Shafts 5-13
Strength Loads 5-13
Service Loads. 5-13

Drilled Shaft Reinforcement 5-14
Installation Nearby Other Structures 5-14
Drilled Shaft Integrity Testing 5-15
Layout Requirements and Notes 5-15
Drilled Shaft Foundation Design Reporting 5-16
Section 6: Driven Piling 5-18
Overview 5-18
LRFD General Design 5-19
Resistance Factors 5-19
Pile Static Design 5-19
Pile Dynamic Design 5-19
Dynamic Monitoring 5-20
Pile Tip Elevations 5-20
Difficult Driving and Drivability 5-20
Pile Setup and Restrike 5-21
Wing Wall Piling 5-21
Steel Piling Special Considerations 5-21
Service Loads 5-22
Pile Lateral Resistance 5-22
Pile Foundation Design Reporting 5-22
Section 7: Foundation Load Testing 5-24
Section 8: Scour 5-25
Overview 5-25
LRFD Design 5-25
Driven Piles and Scour 5-25
Scour Coding, Inspection, and Countermeasures 5-27
Stone Protection at Bridges 5-27

Chapter 6: Retaining Walls and Reinforced Soil Slopes
Section 1: Retaining Wall Selection 6-2
Overview 6-2
Section 2: Retaining Wall Layouts 6-3
General Content Layout 6-3
Plans for Specific Wall Types 6-4
Section 3: Design Considerations 6-8
General Design 6-8
Design Criteria for Specific Wall Types 6-8
Design Criteria for Reinforced Soil Slope 6-14
Section 4: Excavation Support 6-15
Overview 6-15

Trench Excavation Protection 6-15
Temporary Special Shoring 6-15

Chapter 7: Slope Stability
Section 1: Overview 7-2
Overview 7-2
Conditions 7-2
Section 2: Analysis and Design 7-3
Global Stability Analysis 7-3

Appendices
Appendix 1: Legacy Texas Cone Penetration (TCP) Evaluation A-2
Overview A-2
Legacy Procedure of Drilled Shafts from TCP A-3
Legacy Procedure of Driven Piles from TCP A-4
TCP Design Verification with Laboratory Test A-8
Appendix 2: Ancillary Structure Foundations A-10

Chapter 1: About this Manual

Contents:

[Section 1: Introduction]

Section 1: Introduction

Purpose of the Manual

The purpose of this manual is to document policy on geotechnical characterization and design for bridges, retaining walls, slopes, and ancillary structures in Texas transportation projects.

This manual is intended to assist Texas bridge and geotechnical designers in applying provisions documented in the American Association of State Highway and Transportation Officials *(AASHTO) LRFD Bridge Design Specifications, 2020, 9th Edition*, which designers must adhere to unless directed otherwise by this document. The following manuals and guides should be used in companion with this document for designing geotechnical components of bridges, slopes, and earth retaining structures in Texas.

- *Bridge Design Manual – LRFD*
  - https://txdot.gov/content/dam/txdotoms/brg/lrf/lrf.pdf
- *Bridge Design Guide*
  - https://ftp.txdot.gov/pub/txdot-info/brg/design/bridge-design-guide.pdf
- *Bridge Detailing Guide*
  - https://ftp.txdot.gov/pub/txdot-info/brg/design/bridge-detailing-guide.pdf
- *Bridge Railing Manual*
  - https://txdot.gov/content/dam/txdotoms/brg/rlg/rlg.pdf
- *Bent (Pier) Protection Guide*
  - https://ftp.txdot.gov/pub/txdot-info/brg/design/bent-pier-protection-guide.pdf
- *Bridge Project Development Manual*
  - https://txdot.gov/content/dam/txdotoms/brg/bpd/bpd.pdf
- *Bridge Inspection Manual*
  - https://txdot.gov/content/dam/txdotoms/brg/ins/ins.pdf
- *Scour Evaluation Guide*
  - https://ftp.txdot.gov/pub/txdot-info/library/pubs/bus/bridge/scour-guide.pdf
- *Scour Analysis Guide*
  - https://ftp.txdot.gov/pub/txdot-info/des/guides/scour-guide.pdf
- *Hydraulic Design Manual*
  - https://txdot.gov/content/dam/txdotoms/des/hyd/hyd.pdf
- *Quality Control and Quality Assurance Guide*

- https://ftp.txdot.gov/pub/txdot-info/library/pubs/bus/bridge/qa_qc_guide.pdf

All AASHTO LRFD Bridge Design Specifications Articles, Equations, and Tables referenced in this manual are from the 9th Edition of AASHTO LRFD Bridge Design Specifications, unless noted otherwise.

Updates

Updates to this manual are summarized in the following table.

Geotechnical Manual Revision History

Version	Publication Date	Summary of Changes
2024-1	April 2024	New Manual

Organization

Information in this manual is organized into the following chapters:

1. Manual Overview.

   Introductory information on the purpose and organization of the manual.

2. Investigations.

   Requirements for conducting soil surveys for projects with bridges, retaining walls, slopes and embankments, sign structures, illumination, sound walls, and radio towers.

3. Field Operations.

   Requirements for drilling, sampling, and field testing.

4. Subsurface Classification.

   Description of material order, level of description, and classification.

5. Foundation Design.

   Guidelines for selecting foundation types, drilled shafts, piling, and requirements for scour analysis.

6. Retaining Walls and Reinforced Soil Slopes.

   Requirements for retaining wall selection, layouts, design, and excavation support.

7. Slope Stability.

   Requirements for slope stability design and analysis.

Appendices:

Appendix 1 - Legacy Texas Cone Penetration (TCP) Evaluation

Appendix 2 – Ancillary Structure Foundations

Feedback

Direct any questions or comments on the content of the manual to the Geotechnical Branch of the Bridge Division.

Chapter 2: Investigations

Contents:

Section 1: Subsurface Investigations

Section 1: Subsurface Investigations

Overview

Conduct subsurface investigations for projects containing the following features:

- Bridges
- Retaining walls
- Slopes and embankments
- Sign structures
- Illumination
- Sound walls
- Radio towers

Perform minimum required testing as noted in the following section, including Standard Penetration Tests (SPT), Shelby Tube samples, Rock Quality Designation, and percent recovery.

Review of Existing Data

Review all existing data before determining new data requirements. Old bridge plans are a common source of this information. Old borings contain information that can be used to determine geotechnical testing approach.

Boring Location

The complexity and variability of geological conditions and the type, length, and width of a structure determine the number of borings required for foundation exploration. Except as noted here, follow AASHTO LRFD Bridge Design Specifications Article 10.4.2 as a starting point to determine the locations and depths of borings.

When developing boring location plan, consider structure type, estimated loading, and foundation geometry. Locate the borings in a feasible and accessible area. When determining the location of borings, always avoid overhead power lines and underground utilities. If possible, avoid steep slopes and standing or flowing water. Deviations within a 20-ft. radius of the staked location are not usually excessive but note these deviations on the logs and obtain the correct surface elevation.

When determining the location and depth of borings, carefully consider the following factors:

- Boring depth
- Lowering of gradeline

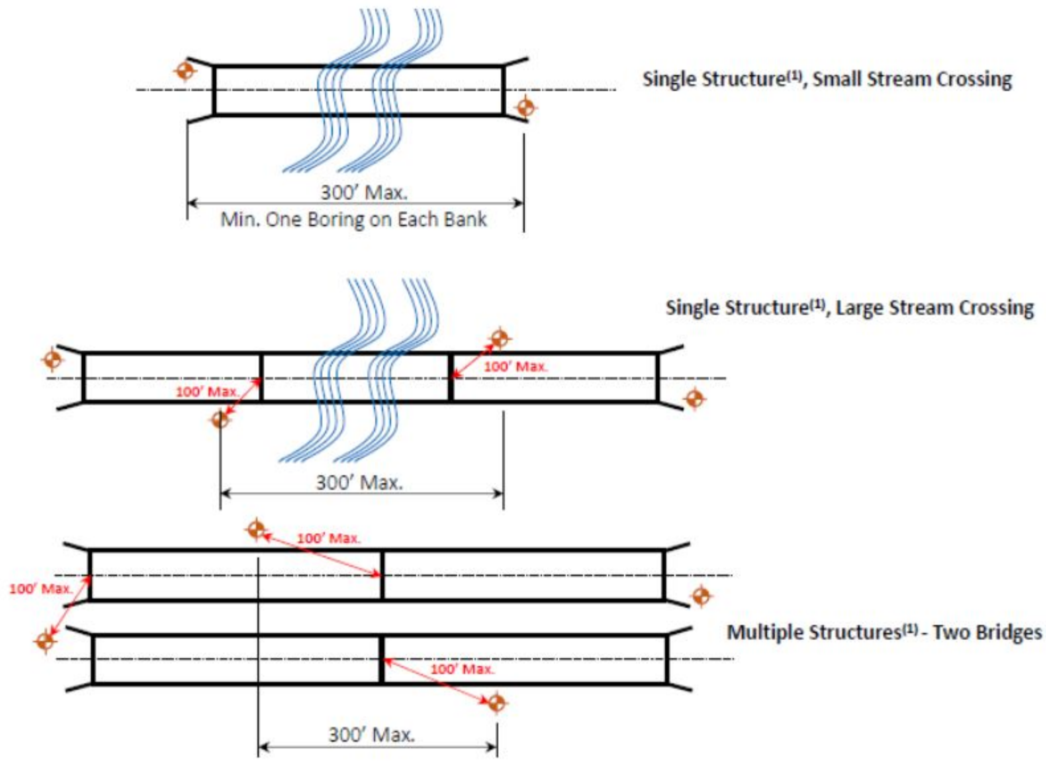
- Embankment
- Cuts over 5 feet
- Channel relocations and channel widenings
- Potential or observed environmental contamination of soil or groundwater
- Scour
- Foundation loads
- Foundation type

Bridges

Boring Locations and Spacing. Boring test holes at every new bridge must conform to the following criteria:

1. Two test holes minimum per bridge
2. Test holes must be located 100 ft or less from the anticipated center of each bent/substructure and each abutment
3. Test holes must be located 50 ft or less from the anticipated center of any monoshafts used on the bridge
4. Do not space test holes greater than 300 ft. apart along the bridge centerline

   Test holes from adjacent bridges can be used to fulfill the above criteria, granted they are deep enough and collect sufficient sampling information and results. Feature(s) being bridged or beneath the deck are inconsequential to location and spacing criteria. Access and right of way issues occur on any project and are reason to offset borings but contact Geotechnical Branch for major deviations to the above criteria or AASHTO LRFD Article 10.4.2. The following figures display example boring location plans for common types of bridge structures.



Note (1): The transverse effect of test holes as shown may not be as critical as it for Multiple Structures with wide separation (e.g., >300 ft between bridges). The holes may be placed all along one side of the structures or directly along centerline of proposed roadway. Consult with Geotechnical Branch for more guidelines.

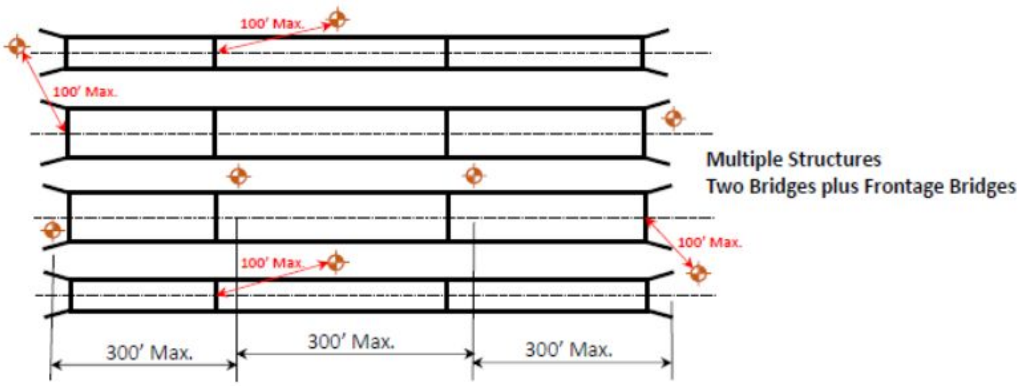


Figure 2–1. Minimum number of test holes for common types of structures

Boring Depth. In general, drill test holes 20 ft. deeper than the probable tip elevation of the pile or drilled shaft or a minimum of two times the minimum pile group dimension, whichever is greater. The probable founding depth or tip elevation can be estimated from experience with subsurface conditions in the area and **anticipated loading**, or the results of previous or legacy TCP tests and correlation graphs. When SPT data is used to judge probable founding or tip elevations, consider elevations where SPT refusal occurred in the legacy borings. Consider the need for borings with depths greater than 20 ft. below the anticipated tip elevations for foundations on major structures or other cases of high or unusual loading conditions.

Depending on the strength of the geomaterial estimated from the SPT blow counts and Rock Quality Designation (RQD) during the drilling operation, it may be necessary to adjust the termination depth of borings from the planned depths. Engineering decisions of early termination of borings and/or extending borings beyond the planned depths shall be made by a competent geotechnical engineer based on realtime SPT blow counts and RQD. Typically, drill deep enough until one of the following conditions are met:

- In non-water crossings and minor stream crossings, early termination of the boring depth at no shallower than 60 feet when drilling within bedrock when 25ft of continuous core within moderately weathered to nonweathered, hard to very hard rock had been recovered.
- In major stream crossings, rivers, and lake crossings, early termination of the boring depth at no shallower than 80 ft when drilling within bedrock when 40ft of continuous core within moderately weathered to nonweathered, hard to very hard rock had been recovered.

Stream Crossings. Structures over channels less than 200 ft. wide are classified as minor stream crossings. For these crossings, place a boring on each bank as close to the water's edge as possible. If boring information varies significantly from one side of the channel to the other, a boring in the channel may be necessary.

Major stream crossings require borings in the channel if no existing data is available. A site inspection by the driller or logger is necessary to evaluate site accessibility and special equipment needs.

Karst Features. Structures suspected to be in a karst formation may require more borings or geophysical survey.

Grade Separations. If the structure borings indicate soft cohesive surface soils, additional borings and testing may be required for the bridge approach embankments.

Bridge Field Exploration. The exploration should include the following:

- Boring spacing. Space borings near each abutment of the proposed structure plus enough intermediate borings to determine the depth and location of all significant soil and rock strata. If a reasonable correlation between boring and testing information (for example, SPT data, Pocket Penetrometer results, stratigraphy) cannot be made, consult with the design engineer to determine the need for additional borings.
- Soil sampling and testing. Conduct Standard Penetration tests (SPT) in accordance with AASHTO T 206 or ASTM D1586 every 5-ft. interval beginning at 5-ft depth from the surface. Where cohesive soils are observed collect thin-walled Shelby Tube samples in accordance with AASHTO T 207 or ASTM D1587 at the intermediate locations between SPT samples. Rock core drilling and sampling should be performed in accordance with AASHTO T 225.
- Near surface soil layer test. Test soft near surface soil layers (0 to 20 feet) as directed under the subsection in this chapter titled Slopes and Embankments.

- Soil and bedrock classification. Complete soil and bedrock classification and log record for each boring on the logs in accordance with Chapter 4, Soil and Bed Rock Logging.
- Ground water. Include ground water elevation measurements (including date of measurement) as part of the data acquisition. Obtain an additional groundwater elevation minimum 15 minutes after the initial encounter. Site conditions or the design objectives may require installation of piezometers to establish a long-term or steady state ground water conditions.

Retaining Walls

Obtain soil borings for walls carrying a traffic surcharge or any wall taller than 5 ft. Evaluate need for soil borings for walls shorter than 5 ft. on a case-by-case basis. For short-term conditions in cohesive soilsuse undrained shear strength determined using laboratory strength tests on undisturbed Shelby tube samples for design analysis. Strength measurement from pocket penetrometer tests and hand torvanes should not be solely used to evaluate undrained shear strength except as supplement to other laboratory strength tests on undisturbed tube samples. Within cohesionless material, SPT can be used to evaluate the internal friction angle but is not intended to supersede results from direct shear testing. A more rigorous sampling and testing program may be required for long-term evaluation of walls founded on cohesive soils.

Boring Locations and Spacing. Obtain borings at 200-ft. maximum spacing unless history or variability of site conditions warrant tighter spacing.

Boring Depth for Fill Walls. For Mechanically Stabilized Earth (MSE) walls, spread footing walls, temporary earth walls, and block walls, bore to a depth as deep as the height of the wall depending on wall type and existing and proposed ground lines. The minimum boring depth is 15 ft. below the bottom of the wall unless rock is encountered. Extending borings 5 ft. into rock for fill walls is usually adequate.

Boring Depth for Cut Walls. For tied-back walls, and soil and rock nail walls, always base the depth of boring on the final grade lines. Advance borings for soil nail and rock nail walls through the material that is to be nailed. Extend borings a minimum of 20-ft. below the bottom of the proposed wall. Borings for cut walls may need to penetrate rock significant distances depending on the depth of the cut and height of the wall.

Cantilever walls, including drilled shaft walls and sheet pile walls, require the depth of borings to extend beyond the anticipated depth of the shaft below the cut, which is typically between one and two times the height of the wall.

Soil Samples and Testing. Provide additional testing for taller walls, walls on slopes, or walls on soft foundations as necessary for complete evaluation of wall stability and settlement characteristics. Additional testing includes but is not limited to obtaining samples for applicable index testing, consolidation testing, triaxial testing, or in-place shear testing to determine soil strength. Consult with the wall designer for development of the complete soil exploration plan.

Ground Water. Include ground water elevation measurements (including date of measurement) as part of the data acquisition for retaining walls. Obtain an additional groundwater elevation at a minimum of 15 minutes after the initial measurement. Site conditions or the design objective may require the installation of piezometers to establish a long-term or steady state ground water conditions.

Other Structures

Conduct foundation investigations for high-mast illumination, radio towers, and overhead sign structures as close as feasibly possible when other borings are not located nearby.

The typical depth of the borings ranges from 30 to 70 ft. but depends on existing and proposed ground lines, soil strength, and structure loading.

Concrete sound (noise) walls require drilled shaft foundation design that results in shafts 20 to 30ft embedded and spaced 10 to 25ft center to center along the wall alignment. When adjacent structure borings are not present, place soundwall borings at maximum 200ft intervals and to a depth of 40 to 50ft depending on soil strength and anticipated planned height of soundwall.

Slopes and Embankments

Soil Borings. Obtain soil borings for cuts greater than 10 ft. or embankments taller than 15 ft. in areas with suspect foundation soils (less than or equal to 10 blows/ft.). Additional laboratory testing may be required to determine soil parameters for short-term and long-term stability analysis, and consolidation settlement analysis (for embankments). Consult with the wall or roadway designer for development of the complete soil exploration plan.

The exploration should include the following:

- The soil under future embankments. Advance borings to a minimum depth below existing grade equal to the height of the embankment or 20 ft., whichever is greater. If compressible soils exist extend the foundation soil boring considering height, width, and loading of embankment. Conduct Standard Penetration tests (SPT) in accordance with AASHTO T 206 or ASTM D1586 every 5-ft. interval beginning at 5-ft depth from the surface. Where cohesive soils are observed collect thin-walled Shelby Tube samples in accordance with AASHTO T 207 or ASTM D1587 at the intermediate locations between SPT samples.
- Soil in proposed cuts. Advance borings to a minimum depth of 15 ft. below the bottom of the proposed cut. Conduct Standard Penetration tests (SPT) in accordance with AASHTO T 206 or ASTM D1586 every 5-ft. interval beginning at 5-ft depth from the surface. Where cohesive soils are observed collect thin-walled Shelby Tube samples in accordance with AASHTO T 207 or ASTM D1587 at the intermediate locations between SPT samples.
- Ground water elevation measurements. Include ground water elevation measurements (including date of measurement) as part of the data acquisition for slopes and embankments. Obtain an additional groundwater elevation at a minimum of 15 minutes after the initial encounter. Site conditions or the design objective may require the installation of piezometers to establish a longterm or steady state ground water conditions.

Soil Testing. Perform the appropriate field and laboratory tests, in accordance with Chapter 3 field testing, and Chapter 4 Section 2 laboratory testing, necessary to determine the soil shear strength for proper soil evaluation of the structure being designed. Consider both the short-term and long-term conditions:

- Short-term conditions. In cohesive soils, use undrained shear strength determined using laboratory strength tests on undisturbed Shelby tube samples for design. Strength measurement from pocket penetrometer tests, hand torvanes, or field vane shear tests should not be solely used to evaluate undrained shear strength except as a supplement to laboratory strength tests on undisturbed tube samples. Avoid correlations of undrained shear strength based on SPT tests. Use unconfined compression tests, unconsolidated undrained (UU) triaxial tests, consolidated undrained (CU) and/or direct shear tests.
- Long-term conditions. Use consolidated undrained (CU) triaxial tests with pore pressure measurement and/or drained direct shear tests.

Estimation of long-term effective stress friction angle of clay soils based on published correlations with index properties of the soil is acceptable. Correlations between corrected SPT blow counts to drained angle of internal friction for granular soils presented in AASHTO LRFD Bridge Design Specifications Table 10.4.6.2.4-1 may be used, and a graphical representation is presented in Figure 2-2. However, the selection of specific values in the range may require experience and care. For cohesive soils, published PI correlations from Atterberg results proven to yield reasonable estimates of effective friction angle may be used.

For cuts with high plasticity clays exposed to weathering or cyclic wetting and drying, long-term shear strength reduction due to soil relaxation may be possible. For such instances a reduction in shear strength as appropriate to account for shear strength loss due to weathering and long-term relaxation should be considered in the evaluation of long-term stability of embankments or slopes.

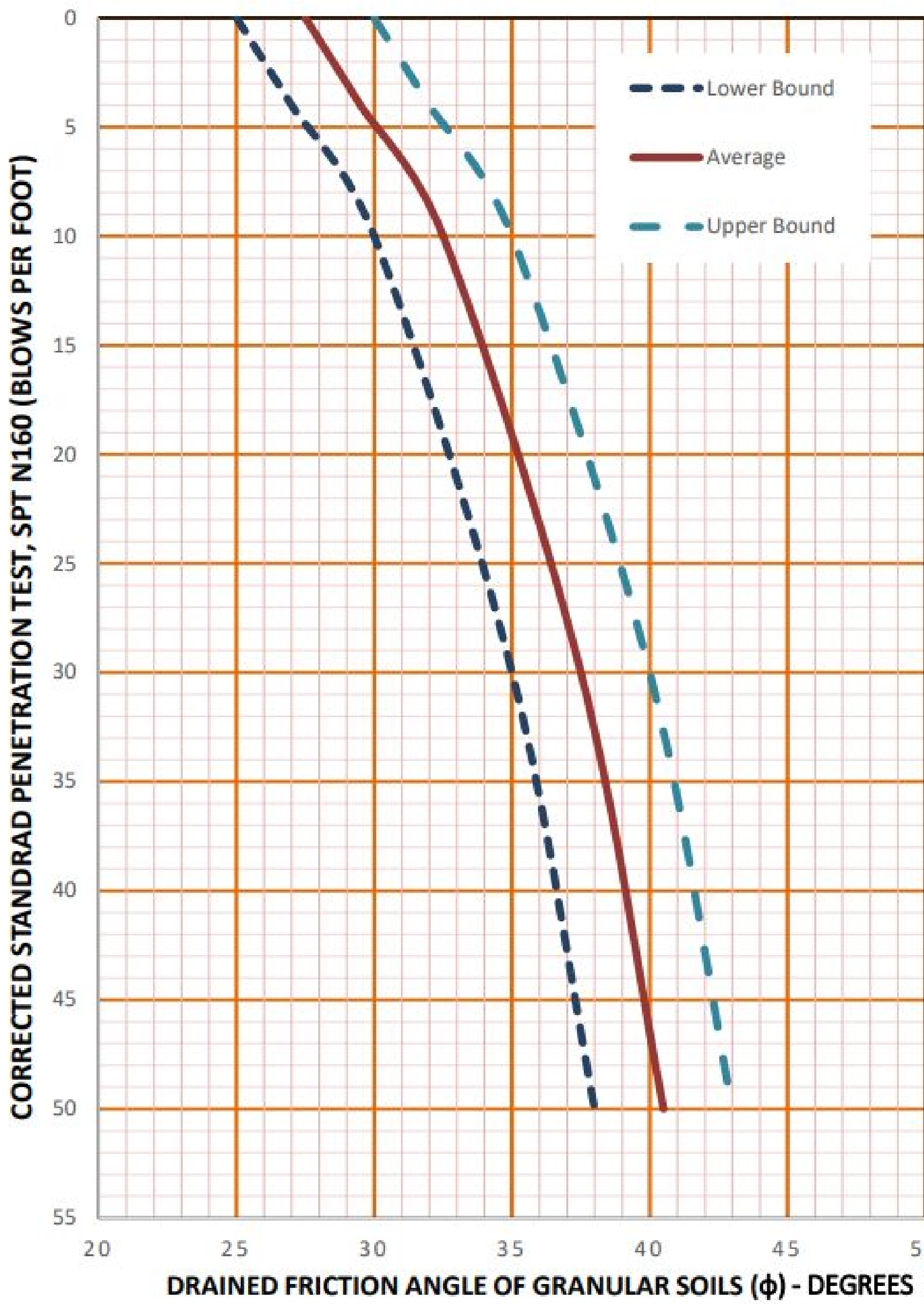


Figure 2–2. SPT vs. Angle of Internal Friction for Cohesionless Soils

Chapter 3: Field Operations

Contents:

Section 1: Drilling Preparation
Section 2: Drilling and Sampling Methods
Section 3: Standard Field Testing and Sampling
Section 4: Post-Drilling

Section 1: Drilling Preparation

Drilling Considerations

Consider the following items before starting core/borehole drilling operations:

- Access
- Utility clearance
- Traffic control
- Core/borehole drill equipment
- Drill rig
- Site preparation
- Barge work
- Borehole backfilling

Access

Coordinate drilling locations and work times with TxDOT. Ensure that operations are confined to Right of Way (ROW) and/or permission to enter private property has been secured before drilling.

Utility Clearance

Clear all locations proposed for drilling for utilities before the drilling team/crew mobilizes to the field. When utilities are present, ensure their exact locations are clearly marked by the utility company.

Call 1-800-545-6005 for utility clearance or utilize the Texas811 website to submit tickets. Obtain utility clearance at least 48 hours and no more than 14 days before starting the drilling and maintain current utility contact numbers during drilling operations.

Traffic Control

Provide traffic control in accordance with Texas Manual on Uniform Traffic Control Devices or Traffic Division Traffic Control Plan (TCP) standards for approval prior to starting the drilling operation.

Section 2: Drilling and Sampling Methods

Drilling Overview

Select drilling equipment that is properly suited to the site conditions prior to mobilization of equipment and crew to the field. Generally, truck-mounted drill rigs can access most sites. However, drilling operation in difficult site conditions such as soft ground, swampy areas, sloping/steep ground may require use of track-mounted, or all-terrain vehicle (ATV) mounted drill rigs. Drilling operation over large water bodies may require barge drilling. Barge drilling typically will require heavy planning and coordination for proper selection of barge capable of supporting the drilling rig, launch locations/sites, vessel for transporting the barge, potential permitting requirements, etc.

Select drilling methods that cause minimal disturbance to the samples obtained, yield good quality SPT results, and suitable for the subsurface conditions. Drilling methods such as wash and jet boring, airrotary drilling, etc., generally tend to cause disturbance and may yield unreliable samples and SPT results. Hence, take drilling methods into consideration if data is used for design applications on TxDOT projects. Dry and wet rotary drilling methods are common methods in TxDOT projects. In bedrock or rock masses, perform rock core drilling with diamond bits to obtain continuous rock core samples for evaluation of intact rock as well as rock mass properties. Selection of drill bits and core barrels should be such that the tools selected yield good recovery and daily production, cause minimal disturbance to the core samples recovered, and appropriate for the anticipated characteristics of the rock mass or deposit. The selection of proper drill bits and core barrels suitable for the anticipated characteristics of the rock mass or deposit is the responsibility of the driller. Hence, the driller must be experienced with the selection of proper equipment and tooling necessary to yield good recovery.

Sampling Overview

Sampling methods are generally governed by geological conditions and the geomaterials to be encountered in the field. Applicable sampling methods corresponding to soil types shall be employed to obtain appropriate samples for visual classification, and laboratory testing afterward. Perform Standard Penetration tests (SPT) in accordance with AASHTO T 206 or ASTM D1586 every 5-ft. interval beginning at 5-ft. depth from the surface. When a boring is proceeding in cohesionless materials, collect split-spoon samples along with the SPT every 5-ft interval. Where cohesive soils are encountered collect Thin-Walled (Shelby) Tube samples in accordance with AASHTO T 207 or ASTM D1587 at intermediate locations between the SPT. Continuous sampling within the top of 15 to 20 feet of borings may be necessary where soils are anticipated to vary at the top or when required for design for the proposed structure.

Borehole efforts should focus on SPT in overburden soils. Switch to coring when bedrock or a rock mass is identified. Where bedrock/rock are encountered, collect rock core samples in accordance with AASHTO T 225. Place the extracted rock cores into dedicated rock core boxes that are often constructed of wood or heavy-duty cardboard for laboratory testing. Determine the Rock Quality

Designation (RQD), percent recovery, and any other rock observations noted in Chapter 4 soon after as the core extraction is completed and record the values in the field logs.

Section 3: Standard Field Testing and Sampling

Standard Penetration Test (SPT)

Perform SPT in accordance with AASHTO T 206 or ASTM D1586, within all soil types. Perform SPT every 5-ft interval beginning at 5 ft. from the surface of the boring to the termination depth of the boring, or until the depth bedrock is encountered. Record the blow counts for each 6-inch SPT interval including the seating drive in the boring logs. The sum of the number of blows for the second and third 6-inch drives is termed as the “standard penetration resistance” or the uncorrected SPT “N-value.” Additionally, record the total length of sample recovered during the 18-inch drive within the split-spoon.

Driller shall develop calibration to determine specific hammer system efficiencies in general accordance with ASTM D4633 for dynamic analysis of driven piles. Drilling equipment or hammers shall be tested and calibrated annually and on a project specific basis as needed when questionable results are observed. Indicate calibrated, reported, or suspected efficiencies on the boring logs and geotechnical reports.

Design shall apply corrections to the standard penetration resistance or uncorrected SPT N-values from the geotechnical report boring log, for hammer efficiency and effects of overburden pressure, as applicable to the design method or correlation being used. Use Equations 10.4.6.2.4-1 through 10.4.6.2.4-3 in AASHTO LRFD Bridge Design Specifications for corrections of SPT blow counts.

Thin-Walled Tube Sampling (Shelby Tube)

Thin-Walled Tube sampling in accordance with AASHTO T207 or ASTM D1587 is critical to retrieving undisturbed, in-situ samples used for laboratory testing for obtaining useful soil parameters for use in AASHTO design methodologies. Use undisturbed in-situ samples retrieved in accordance with AASHTO T 207 or ASTM D1587 for lab testing. Where fine-grained, or cohesive soils are observed collect ThinWalled Tube samples at intermediate locations between the SPT. Use 3 inch diameter Thin-Walled Tubes unless deviations are requested in investigation and approved by the Bridge Division Geotechnical Branch. Indicate on logs length of sample collected (recovery) and if tube was unable to push the full 24” of undisturbed sample as specified by procedure.

Typically, samples can be extruded in the field and wrapped to prevent moisture loss or sealed in plastic bags immediately after measuring recovery length and obtaining pocket penetrometer and/ or torvane measurements. Project specific guidelines may dictate that tube samples be capped or waxed at the ends and cut open in the lab.

Storing and Transport of Thin-Walled Tube Samples

Careful handling, transportation and storage of all samples is required to minimize sample disturbance and ensure accurate results of lab work. Exercise the following general precautions, in handling, transportation, and storage of samples. Perform lab tests as soon as possible after drilling is completed.

- Do not allow samples to freeze or get too hot in a vehicle or be exposed to outdoors.
- Store samples in an upright position with same orientation it was collected
- Do not allow samples to bounce around in a vehicle.
- Do not stack samples on top of other samples or place samples below anything that may put pressure on the samples.

Bedrock Coring

The use of rock coring for foundation design is necessary to obtain useful design data for design methodologies in AASHTO. Observation of drill rig performance, identifying bedrock or rock mass layer, and switching to rock coring shall be at the driller and field logger discretion. Typically, this depth will be evident by drill rig performance, but also can be detected by SPT refusal or resistance when pushing for a Shelby tube sample. Cemented soil and shale can often be difficult to identify and collect core sample. Air, water, or water based, or combination of geotechnical coring media as appropriate to the geological conditions are acceptable for retrieving samples in 5-ft core runs through bit types of driller selection. Conduct coring operations in accordance with AASHTO T225.

Caring, Handling, Storing, and Transport Rock Core Samples

Exercise the following when caring, handing, storing, and transporting rock samples:

- Place samples into dedicated rock core boxes constructed of wood or heavy-duty cardboard.
- For quickly degradable rock types such as shales and mudstones: core recovery, *RRRRRR*, and fracture frequency (in accordance with Chapter 4) is to be measured and recorded in the field logs as soon after as the core extraction is completed.
- For intermediate materials that represent the boundary between soil and rock, or for rock that is sensitive to moisture content changes, care is highly critical. In such cases, rock core is to be wrapped in plastic wrap and/or waxed following extraction to prevent changes in moisture content prior to testing.
- Record presence and depths of any voids vuggy/porous texture.
- Include a photolog when reporting or along with bore logs of any core collected during field operations.

Pocket Penetrometer

The pocket penetrometer test is useful for estimating consistency and approximate measurements of unconfined compressive strength. It yields approximate information which is not suitable for foundation design. However, comparison of pocket penetrometer measurements at the time of sampling on the field and in the laboratory prior to laboratory strength testing may be useful for comparing consistency. Perform pocket penetrometer testing and record values on any and all cohesive or fine-grain samples collected from Thin-Walled /(Shelby) tube sampling and adhere to the following guidance:

- Take more than one (1) reading on a sample and average values.
- Cut off any observed fall-in or cuttings that were mixed in with the sample.
- Use firm, slow, and constant push on flat, flush surfaces cut perpendicular to the sample length.
- Keep in mind that the pocket penetrometer is at best a crude instrument. Soil around the tip or spring mechanisms may influence readings as would age of the spring. Comparison testing on almost identically dense clay material (side by side pushes) should result in a deviation of readings of no more than quarter ton.

In-Place Vane Shear Test

Use the in-place vane shear test to determine the in-place shearing strength of fine-grained soil, which does not lend itself to undisturbed sampling and triaxial testing. Use this test when encountering organic silty clay (muck) or very soft clay. Ensure these materials are free of gravel or large shell particles because pushing the vanes through these obstructions would disturb the sample and probably cause physical damage to the vanes. Use the test with extreme caution in soil that has Standard Penetration Test values harder than 15 blows/12 in. Correct the vane shear results to the soil index properties.

Torvane

This test is useful for index and classification purposes as it yields approximate information which is not suitable for foundation design. Adhere to the following when performing this test:

- Testing must be taken on a flat surface.
- Test slowly and with constant rate of shear.
- Fingers must not interfere with free rotation of measurement dial.

Section 4: Post-Drilling

Borehole Backfilling

Fill or plug drill holes using bentonite pellets or cement bentonite grout to prevent injury to livestock or people in the area and to minimize the entry of surface water into the bore hole. If surface contamination of lower aquifers or cross contamination is a concern, grout the hole with cement bentonite grout using tremie method. This is especially important in urban areas where ground contamination from leaking underground storage tanks is common. To avoid potential settlement or uplift of a pavement core, backfill all borings under existing pavement with bentonite pellets or cement bentonite grout to a minimum depth of 6 inches below the bottom of pavement structure. Then patch the hole with nonshrink grout (or materials matching existing pavement) to the top of pavement.

Chapter 4: Subsurface Classification

Contents:

Section 1: Soil and Bedrock Logging
Section 2: Laboratory Testing
Section 3: Quality Assurance and Quality Control

Section 1: Soil and Bedrock Logging

Material Order of Description

Keep soil and rock core descriptions simple, yet descriptive enough to be able to determine complete set of characteristics for each layer of strata. The order of description is as follows:

1. Material
2. Unified Soil Classification System
3. Density or consistency, hardness or strength
4. Moisture
5. Color
6. Cementation
7. Descriptive adjectives [minor features of soils, degree of weathering in rock cores, open or closed jointing, vuggy or karstic texture, assessment of discontinuities (joints, natural fractures, bedding, etc) such as spacing, size, etc]
8. Rock Quality Designation (RQD), percent recovery

Material

Use observations in the field in conjunction with results of lab testing to develop a soil and bedrock profile for use in the reporting. Keep the number of strata to a minimum. Remember that every small variation in a soil—such as a change in clay from “slightly sandy” to “sandy” does not necessarily warrant a stratigraphy change. The logger must define strata that have significance to designers and contractors who will use the core log information. Capture the primary and secondary soil or rock constituent and whether ground water is present.

Unified Soil Classification System (ASTM D2487)

This soil system is based on the recognition of the type and predominance of the constituents considering grain size, gradation, plasticity index, and liquid limit. General soil description is determined in the field based on visual observations and is confirmed or revised once laboratory testing data is available. USCS contains three major divisions of soil: coarse-grained, fine-grained, and highly organic. See ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), for the procedure for determining soil classification. TxDOT test procedures, Tex-141-E, Manual Procedure for Description and Identification of Soils and Tex-142-E, Laboratory Classification of Soil for Engineering Purposes may also prove useful in the determination of soil type.

Relative Density or Consistency, Hardness

Use the following charts to determine the density or consistency and hardness of material encountered.

Table 4-1: Soil Density, Cohesionless Soils

Relative Density/Density (Cohesionless)	Uncorrected SPT N-values	Legacy TCP Blowcounts	Field Identification
Very loose	Less than 4	0 to 8	Easily penetrated by rebar many inches (> 12) ½ inch rebar, pushed by hand
Loose	4 - 10	8 to 20	Easily penetrated with ½ inch rebar several inches, pushed by hand
Medium Dense	10 - 30	NA	Easily to moderately penetrated using ½ inch rebar driven by 5-pound hammer
Slightly compact	NA	20 to 40	Sample can be imprinted with considerable pressure
Compact	NA	40 to 80	Sample can be imprinted only slightly with fingers
Dense	~30 - 50	80 to 5in / 100	Penetrated 1 foot with difficulty using ½ inch rebar driven by 5-pound hammer Sample cannot be imprinted with fingers but can be penetrated with pencil
Very dense	> 50	5in / 100 to 0in / 100	Penetrated a few inches with ½ inch rebar driven by 5- pound hammer

Table 4-2: Soil Consistency, Cohesive / Clay Soils

Consistency (Cohesive)	Uncorrected SPT N-values	Legacy TCP Blowcounts	Approx. Undrained Shear Strength, Su (Tons per Square Foot)*	Field Identification
Very soft	< 2	0 to 8	< 0.125	Sample (height twice diameter) sags under own weight. Squeezes between fingers when fist is closed. Easily penetrated several inches by palm or fist.

Table 4-2: Soil Consistency, Cohesive / Clay Soils

Consistency (Cohesive)	Uncorrected SPT N-values	Legacy TCP Blowcounts	Approx. Undrained Shear Strength, Su (Tons per Square Foot)*	Field Identification
Soft	2 - 4	8 to 20	0.125 – 0.25	Sample can be pinched or imprinted easily with finger. Easily penetrated several inches by thumb.
Medium Stiff	4 - 8	NA	0.25 – 0.50	Molded by strong pressure of fingers. Can penetrated several inches by thumb with moderate effort.
Stiff	8 - 15	20 to 40	0.50 – 1.0	Sample can be imprinted with considerable pressure.
Very stiff	15-30	40 to 80	1.0 – 2.0	Sample can be imprinted only slightly with fingers.
Hard	30-60	80 to 5in/100	>2.0	Sample cannot be imprinted with fingers but can be penetrated with pencil.
Very hard	>60	5in/100 to 0in/100		Sample cannot be penetrated with pencil.

*Pocket Penetrometer and unconfined compression tests yield qu, within clay, Su = qu / 2

Table 4-3: Field Bedrock Hardness

Hardness (Relative Rock Hardness)	Mohs' Hardness Scale	Examples	Approx. SPT Values	Legacy TCP Blowcounts	Field Identification
Very hard	5.5 to 10	Sandstone, chert, schist, granite, gneiss, some limestone	SPT Refusal > 100 blows	0 in./100 to 2 in./100	Rock will scratch knife. Core requires many blow of hammer to fracture or chip. Hammer rebounds after impact
Hard	3 to 5.5	Siltstone, shale, iron deposits, most limestone	SPT Refusal > 100 blows	1 in./100 to 5 in./100	Rock can be scratched with knife blade or pick with difficulty
Moderate Hard	N/A	Shale, some limestone	SPT Refusal > 100 blows	2 in./100 to 5 in./100	Cannot scratch with fingernail but can be peeled with knife. Fracturing with single blow of hammer.

Table 4-3: Field Bedrock Hardness

Hardness (Relative Rock Hardness)	Mohs' Hardness Scale	Examples	Approx. SPT Values	Legacy TCP Blowcounts	Field Identification
Soft	1 to 3	Gypsum, calcite, evaporites, chalk, some shale	80 blows to 100 blows	4 in./100 to 6 in./100	Rock can be scratched with fingernail or knife. Crumbles under firm blow with hammer. Grains from sandstones and mudstones/ shales can be rubbed off with fingers.
Very Soft	~1	shale	< 80 blows	< 100 blows	Can be indented with fingers or crushed with fingers. Can be excavated easily with point of geologic hammer.

Relative Rock Strength

Estimate the relative strength of intact rock in the field with a use of a geological hammer or pocket-knife and record in the boring logs. Field identification methods should be confirmed by laboratory uniaxial compressive strength tests performed on representative rock core sample(s) within the stratum and presented in the boring logs. Perform uniaxial compressive strength tests in accordance with ASTM D7012, Method C.

Table 4-4 provides relative strength descriptions of intact rock based on field identification methods **and laboratory uniaxial compressive strength tests of rock**. Use Table 4-4 in combination with the field observations from Table 4-3 for boring log strength classification of rock.

Table 4-4: Criteria and Description for Relative Rock Strength

Grade Designation	Strength Description	Field Identification	Approximate Compressive Strength (psi)
R0	Extremely weak rock	Specimen can be indented by thumbnail	35 – 150
R1	Very weak rock	Specimen crumbles under sharp blow with point of geological hammer and can be peeled by a pocketknife	150 – 725

Table 4-4: Criteria and Description for Relative Rock Strength

Grade Designation	Strength Description	Field Identification	Approximate Compressive Strength (psi)
R2	Weak rock	Shallow cuts or scrapes can be made in a specimen with a pocketknife. A firm blow with a geological hammer creates shallow dents.	725 – 3,500
R3	Medium strong rock	Specimen cannot be scraped or cut with a pocketknife. Specimen can be fractured with a single firm blow with a geological hammer point.	3,500 – 7,250
R4	Strong rock	Specimen requires more than one firm blow of the point of a geological hammer to fracture.	7250 – 14,500
R5	Very strong rock	Specimen requires many blows of geological hammer to cause fracture	14,500 – 36,250
R6	Extremely strong rock	Specimen can only be chipped with firm blows from the hammer end of a geological hammer.	> 36,250

Moisture

If any moisture exists, note the extent present. The samples will be assumed dry if the degree of moisture is not indicated. If free water is present, describe the soil as wet or water-bearing.

Color

Describe the primary color and restrict description to one color. If one main color does not exist in a sample, call it multicolored.

Cementation

Identify the degree of cementation if any is present. Use Table 4-5 for the classification:

Table 4-5: Cementation Status

Description	Field Identification	Approximate SPT
Cemented Sand / Soil	Difficult drilling or SPT layer(s) comprised of sand and fines.	30 to 99
Highly cemented sand / soil, weathered sandstone / bedrock	1” or longer in-situ specimen can still be collected in split-spoon often designated as top of bedrock layer if no weaker strata is encountered below.	SPT Refusal, > 100 blows
Bedrock	1” or shorter in-situ specimen collected in split-spoon and coring efforts should commence. Use weathering, strength and rock descriptions located in this section.	SPT Refusal, > 100 blows

Descriptive Adjectives

Use any descriptive adjectives that might further aid in the description. This is especially important for core material recovered.

Weathered State of Rock

Weathering is the process of chemical and/or mechanical degradation of the rock mass over the course of time through exposure to the elements such as rain, wind, ground water, ice, changing temperature, etc. In general, the strength, stiffness, and general quality of intact rock tends to decrease with increase in the degree of weathering. As weathering advances significant changes occur in the physical properties and general quality of the intact rock, until ultimately the rock is decomposed to soil. Therefore, weathering is an important component of classification for engineering purposes.

Identify and record the weathering grades of the rock mass in accordance with the weathering grade shown in Table 4-6 below.

Table 4-6: Descriptive Terms for Weathering State of Rock

Term	Description	Grade
Fresh (F)	No visible sign of rock material weathering; slight discoloration on major discontinuity surfaces is possible.	I
Slightly weathered (WS)	Discoloration indicates weathering of rock material and discontinuity surfaces. All rock material may be discolored by weathering and the external surface may be somewhat weaker than in its fresh condition.	II

Table 4-6: Descriptive Terms for Weathering State of Rock

Term	Description	Grade
Moderately weathered (WM)	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones. A minimum 2 in. diameter sample cannot be broken readily by hand across the rock fabric.	III
Highly weathered (WH)	More than half of the rock is decomposed and/or disintegrated to soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones. A minimum 2 in. diameter sample can be broken readily by hand across the rock fabric.	IV
Completely weathered (WC)	All rock material is decomposed and/or disintegrated to soil. The original mass structure is largely still intact. Material can by granulated by hand.	V
Residual soil (RS)	All rock material is converted to soil. Material can be easily broken apart by hand.	VI

Rock Core Grain Size

Depending on if evident in visual observations of the core sample (intact rock), indicate grain sizing according to Table 4-7. Grain size refers to the sizes of individual particles or mineral crystals that comprise the intact rock. Unlike soils, where grain size is generally characterized based on sieve or hydrometer tests, the grain size for intact rock is generally characterized from visual observation.

Table 4-7: Criteria for Defining Rock Grain Size

Grain Size	Description	Criteria
< 0.003 in. (< 0.075 mm)	Very Fine-Grained	Cannot be distinguished by unaided eye. Few to no mineral grains are visible with a hand lens
0.003 – 0.02 in. (0.075 – 0.425 mm)	Fine-Grained	Few crystal boundaries are visible; grains can be distinguished with difficulty by the unaided eye but can be somewhat distinguished by hand lens
0.02 – 0.8 in. (0.425 – 2 mm)	Medium-Grained	Most crystal boundaries are visible; grains distinguishable by eye and with hand lens
0.8 – 2 in. (2 – 4.75 mm)	Coarse-Grained	Crystal boundaries are visible; grains distinguishable with naked eye
2 in. (> 4.75 mm)	Very Coarse-Grained	Crystal boundaries are clearly visible; grains are distinguishable with the naked eye

Bedding and Discontinuity Spacing

Spacing refers to the distance between fractures or thickness of beds visible in the core. In the case of fractures, spacing does not represent the thickness of the open space produced by a fracture, but rather the amount of rock material between two distinct fractures. For bedding thickness, this represents the amount of rock material between two distinct bedding planes. Discontinuities, such as joints and fractures, are often found in crystalline rock that has undergone deformations. Whereas bedding terms are typically used for sedimentary rocks such as sandstones and limestones.

Table 4-8: Joint and Bedding Terms

Joint Term	Bedding Term	Spacing (inch)
Very Close	Laminated	< 0.5
Close	Very Thin	0.5 – 2
Moderately Close	Thin	2 – 12
Wide	Medium	12 - 36
Very Wide	Thick	> 36

Discontinuity spacing is the distance between natural discontinuities as measured along the borehole core. Evaluate the discontinuity spacing within each core run, and report on the boring logs in accordance with the criteria provided in Table 4-9 below. Do not include mechanical breaks due handling or drilling in the measurement of discontinuity spacing.

Table 4-9: Discontinuity Spacing

Description	Discontinuity Spacing
Very widely spaced	>10 feet
Widely spaced	3 feet to 10 feet
Moderately Spaced	1 feet to 3 feet
Closely Spaced	2 inches to 12 inches
Very Closely Spaced	Less than 2 inches

Discontinuity Condition

The surface properties of discontinuities, in terms of roughness, wall hardness, and/or gouge thickness, affects the shear strength of the discontinuity. As the discontinuities within each core run, and report in the boring logs in accordance with the descriptions and conditions provided in Table 4-10 below.

Table 4-10: Discontinuity Condition

Condition	Discontinuity Spacing (feet)
Excellent Condition	Very rough surfaces, no separation, hard discontinuity wall
Good Condition	Slightly rough surfaces, separation less than 0.05 inches, hard discontinuity wall
Fair Condition	Slightly rough surfaces, separation greater than 0.05 inches, soft discontinuity wall
Poor Condition	Slickensided surfaces, or soft gouge less than 0.2 inches thick, or open discontinuities 0.05 to 0.2 inches
Very Poor Condition	Soft gouge greater than 0.2 inches thick, or open discontinuities greater than 0.2 inches

Classification of Intact Rock and Rock Mass

Design and construction of engineering structures on rock or rock deposits heavily depend on proper characterization of both the “intact rock” as well as the “rock mass” with discontinuities. For the purposes of this manual “intact rock” is defined as an intact piece of rock containing no discontinuities. “Rock mass” is defined as rock as it occurs in-situ, including its system of discontinuities, and weathering profile.

The extent of characterization of intact rock properties and rock mass properties shall be determined in accordance with data needs for the design and construction of the proposed structure, the type of proposed structure, and criticality of the proposed structures

Establish and report the properties of both the intact rock as well as the rock masses in the boring logs and the geotechnical report.

Intact rock is generally classified based on qualitative observations and simple measurements as described in the sections in this chapter. Laboratory tests using uniaxial compressive strength tests (Table 4-4) shall also be used to supplement qualitative observations and classify the relative strength of intact rock.

The primary basis for classification of intact rock is rock type. Establish rock type by first identifying the origin, whether the intact rock is igneous, sedimentary, or metamorphic in origin. Establish the specific rock type from consideration of additional characteristics such as mineralogy, texture, and experience with local geology. Tables 4-10 to 4-12 show the three rock origins, and rock types found depending on their origin. Texas Geology contains mostly sedimentary rocks and a few exposures of Precambrian igneous and metamorphic that are less common. The Geologic Atlas of Texas is primary resource that investigation should use ahead of drilling to anticipate rock type:

USGS - Pocket Texas Geology

Texas geology contains a variety of rock types and investigation should be aware of rock type to expect in any unique region or project location. Should anticipated bedrock not be observed during the drilling, indicate what rock type and characteristics are present in the investigation.

Table 4-11: Common igneous rocks

Intrusive	Extrusive	Primary Minerals	Common Secondary Minerals
Granite	Rhyolite	Quartz, K-Feldspar	Plagioclase, Mica, Amphibole, Pyroxene
Quartz Diorite	Dacite	Quartz, Plagioclase	Hornblende, Pyroxene, Mica
Diorite	Andesite	Plagioclase	Mica, Amphibole, Pyroxene
Gabbro	Basalt	Plagioclase, Pyroxene	Amphibole Olivine

Table 4-12: Common Sedimentary Rocks

Clastic		Non-Clastic		
Rock Type	**Original Sediment**	**Rock Type**	**Primary Mineral**	**HCl Reaction**
Conglomerate	Sand, gravel, cobbles	Limestone	Calcite	Strong
Sandstone	Sand	Dolomite	Dolomite	Weak
Siltstone	Silt	Chert	Quartz	None
Claystone	Clay			
Shale	Laminated clay & silt			

Table 4-13: Common Metamorphic Rocks

Foliation	Rock Type	Texture	Formed From	Primary Minerals
Foliated	Slate	Platy, fine-grained	Shale, Claystone	Quartz, Mica
	Phyllite	Platy, fine-grained with silky sheen	Shale, Claystone, Fine-grained Pyroclastic	Quartz, Mica
	Schist	Medium grained with irregular layers	Sedimentary & Igneous Rocks	Mica, Quartz, Feldspar, Amphibole
	Gneiss	Layered, medium to coarse grained	Sedimentary & Igneous Rocks	Mica, Quartz, Feldspar, Amphibole
Non-Foliated	Greenstone	Crystalline	Intermediate Volcanics & Mafic Igneous	Mica, Hornblende, Epidote
	Marble	Crystalline	Limestone & Dolomite	Calcite & Dolomite
	Quartzite	Crystalline	Sandstone & Chert	Quartz
	Amphibole	Crystalline	Mafic Igneous & Calcium-Iron Bearing Sediments	Hornblende & Plagioclase

In addition to rock type, classify intact rock according to relative strength or hardness, degree of weathering, grain size or texture. Color and grain size are often key characteristics that facilitate identification of rock type.

In ASTM D5878 several systems of rock mass classifications are described. Certain design methodologies in AASHTO require rock mass classification using Geological Strength Index (GSI). Classify the strength of a jointed rock mass using GSI in accordance with AASHTO LRFD Bridge Design Specifications Article 10.4.6.4.

Percent Recovery and Rock Quality Designation (RQD)

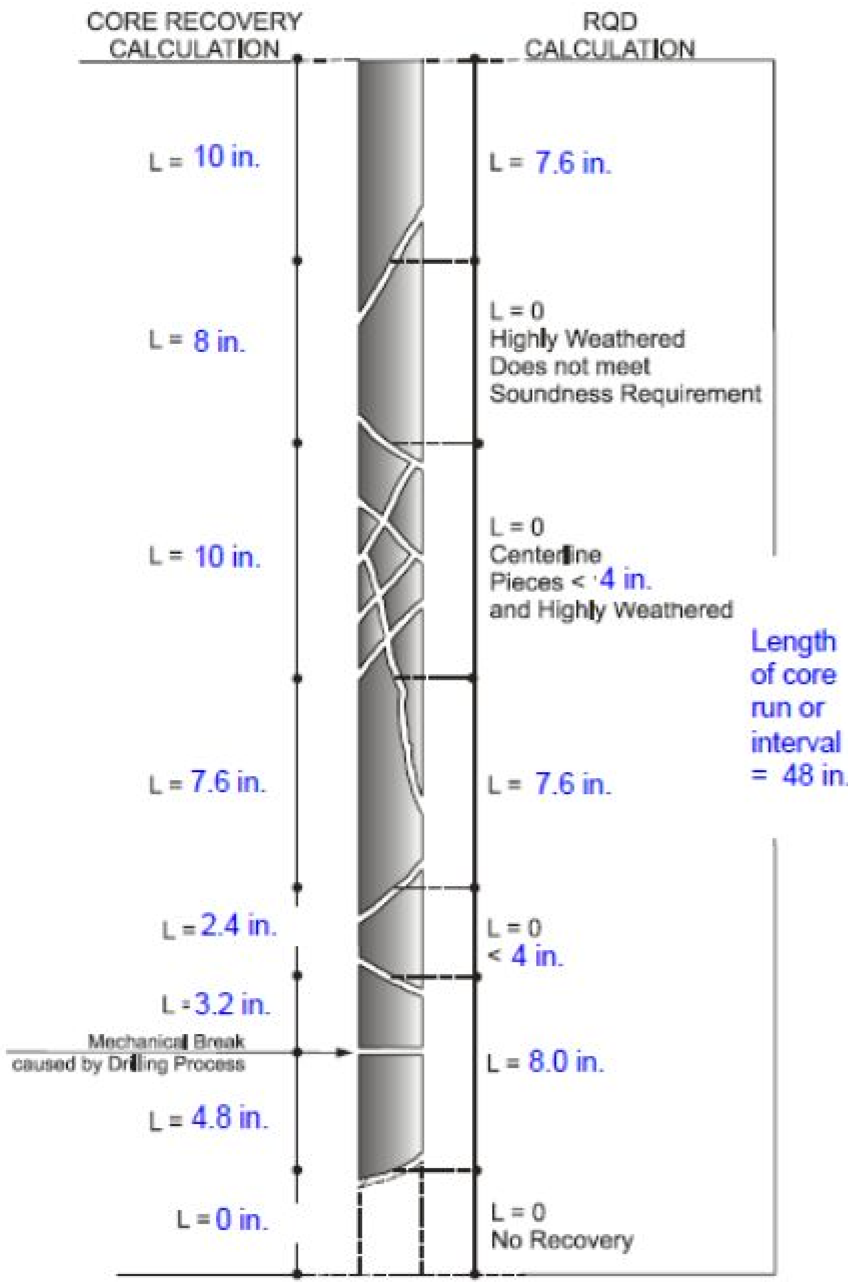
Percent Recovery is defined as the ratio of core recovered to the run length expressed as a percentage:

$$\text{Percent Recovery}(\%) = \frac{\text{Length of core recovered* 100}}{\textit{Length of core run or interval}}$$

Determine the RQD for rock core samples following ASTM Test Procedure D6032, Standard Test Method for Determining Rock Quality Designation (RQD) of Rock Core.

$$\text{RQD}(\%) = \frac{(\sum \text{Length of sound core segements} > \text{or} = 4\text{inch}es) \text{ *100}}{\textit{Lenght of core run or interval}}$$

As illustrated by the example:



$$\text{Percent Recovery} = \frac{(10\text{ in.} + 8\text{ in.} + 10\text{in.} + 7.6\text{ in.} + 3.2\text{in.} + 4.8\text{ in})}{48} = 96\%$$

$$\text{RQD }(\%) = \frac{(10\text{inches}+7.5\text{inches}+8.0\text{inches}*\ 100)}{48} = 53\%\text{Fair}$$

Mechanical breaks in the core (perpendicular to the length of the core) should not be counted towards RQD reduction. Use segments of 4” or above only by breaks identified as natural fractures or joints within the rock mass. Record the rock type (limestone, shale, sandstone, etc.), degree of weathering (highly, moderate, minimal, unweathered), natural fracture frequency (number of visible joints or natural discontinuities within a typical 12” segment of recovered core) and jointing condition (closed or open), and size of the jointing or discontinuities to use classification criteria as specified in this chapter.

Always note the percent recovery and RQD on boring logs where rock is encountered.

Fracture Frequency (FF)

Fracture frequency is defined as the number of natural fractures per unit length of core recovered.

$$FF = \frac{\text{number of natural fractures}}{\text{total length of core recovered}}$$

The fracture frequency can be determined for the entire length of a core run, or for a smaller segment of core. As is the case with *RQD*, artificial fractures or mechanical breaks created during drilling or core handling should be neglected when calculating fracture frequency.

Geological Strength Index (GSI)

Classify the strength of a jointed rock mass using GSI in accordance with AASHTO LRFD Bridge Design Specifications Article 10.4.6.4. Present GSI values in the geotechnical data or design report to aid in foundation design or when used as a basis for foundation recommendations.

Geotechnical Report

Geotechnical Data Reports are required to contain the following minimum information:

- Project information
- Dates
- Site map with boring locations
- Site geolog
- Drilling and sampling methods
- Boring location table with coordinates and depths
- Boring logs with in-situ results and depths and sample types of soil for lab testing
- Photolog of any rock core recovered from borings
- Lab testing summary and individual test results
- Groundwater measurements during drilling or from piezometer installation
- Signed and sealed by engineer responsible for the investigation

See Chapter 5 for Geotechnical Design Report requirements for both drilled shaft and driven pile foundation design. See Chapter 7 for Geotechnical Design Report requirements for retaining wall design.

Boring Log Format

Standard log forms are available in various software packages to display all required information and description within each borehole. TxDOT Wincore was developed for and can only be used with legacy TCP data and is not sufficient for use in LRFD geotechnical design. Group the materials encountered into strata consisting of the same or similar constituents. Pay close attention to the classification descriptions within this Chapter.

Currently PDF export of logs, or inclusion of logs within a PDF of the geotechnical data report is acceptable in contact plans. See the TxDOT Bridge Detailing Guide for boring display requirements.

Section 2: Laboratory Testing

Overview

To supplement field testing results, laboratory testing is required on samples properly acquired and retained from the drilling operations. This lab testing on soil and rock samples is used to correctly classify the material type and ascertain the nature, strength, and consolidation characteristics of the subsurface layers. Additional lab testing may be required at specific sites to determine swell potential, corrosion potential, permeability, and durability. Perform all laboratory testing in accordance with the relevant TxDOT, ASTM or AASHTO procedures.

Moisture Content Tests

Perform laboratory determination of moisture content in accordance with Tex-103-E, AASHTO T265 or ASTM D2216. Determine the moisture content at each geologic unit or stratum identified to establish the moisture profile at each boring.

Unit Weight

Perform unit weight of soils in accordance with ASTM D7263.

Grain Size Analysis and Passing No. 200 Sieve

Perform Particle Size Analysis of Soils in accordance with Tex-110-E, and Amount of Material in Soils Finer than the 75 micrometer (No. 200) Sieve in accordance with Tex-111-E.

Atterberg Limit Tests

Perform liquid limit, plastic limit, and plasticity index of soils in accordance with Tex-104-E, Tex-105-E, and Tex-106-E, or equivalent AASHTO or ASTM methods.

Unconfined Compressive Strength Tests and Unconsolidated, Undrained Triaxial Compression Tests

Perform unconfined compressive strength test in accordance with AASHTO T 208 or ASTM D2166.

Perform Unconsolidated, Undrained Triaxial compression test on cohesive soils in accordance with Tex118-E, or AASHTO T 296 or ASTM D2850.

Perform Unconfined Compressive Strength Tests and Unconsolidated, Undrained Triaxial Compression Tests on undisturbed 3-inch diameter specimens obtained in accordance with AASHTO T 207 or ASTM D1587.

Specimens shall be free from tailings, cuttings, seams, cracks, and/or other disturbance that may affect the strength result obtained. Do not use specimens with noticeable disturbance for testing. Do not use specimens obtained within the upper 6 inches of the thin-walled tube sampler for testing

Transport specimens for strength test to the laboratory and test as soon as practicable (after sampling) to prevent loss of moisture.

Consolidated Undrained Triaxial Compression Tests (CU testing)

Perform Consolidated Undrained (CU) triaxial tests accordance with Tex-131-E, or ASTM D4767. Perform CU on undisturbed 3-inch diameter specimens obtained in accordance with AASHTO T 207 or ASTM D1587. Specimens shall be free from tailings, cuttings, seams, cracks, and/or other disturbance that may affect the strength result obtained. Do not use specimens with noticeable disturbance for testing. Do not use specimens obtained within the upper 6 inches of the thin-walled tube sampler for testing.

Transport specimens for strength test to the laboratory and test as soon as practicable (after sampling) to prevent loss of moisture.

Direct Shear Tests

Perform Direct Shear testing under consolidated drained conditions in accordance with AASHTO T236 or ASTM D3080.

One Dimensional Consolidation Tests

Perform one-dimensional consolidation tests in accordance with AASHTO T216 or ASTM D2435.

One Dimensional Swell Tests

Perform one-dimensional swell tests in accordance with ASTM D4546.

Resistivity, pH, Sulfates and Chloride Ion Content in Soils

Perform soil resistivity tests in accordance with Tex-129-E, or AASHTO T288. Perform pH test of soils in accordance with Tex-128-E, or AASHTO T289. Perform water-soluble Sulfate Ion or Chloride Ion content in soils in accordance with Tex-620-J, or AASHTO T290 and AASHTO T291, respectively.

Organic Content

Perform soil organic content tests in accordance with AASHTO T267.

Section 3: Quality Assurance and Quality Control

Overview

Perform Quality Assurance and Quality Control (QA-QC) at all levels to ensure quality of data produced and deliverables. QA-QC shall be conducted in accordance with drilling or investigation contract documentation, but generally include the protocol outlined in this section.

Sampling

A minimum of three-man crew consisting of a driller, driller's helper, and a logger shall perform the sampling and logging on the field. To ensure consistency, the same crew shall complete the drilling, sampling, and logging on a project. Photo log of all samples shall be provided to the client along with the field logs for verification of consistency of the logging.

Each crew shall maintain a copy of the TxDOT Geotechnical Manual and relevant Test Methods (TxDOT, AASHTO, or ASTM) required for drilling, sampling, and classification of soil and bedrock materials.

Logger Qualification

A qualified logger shall perform all sampling, identification of the drilled material, and logging the soil profile. The following minimum requirement shall be used for a logger:

1. A geologist or Engineer-in-training with at least two years of related experience in local soils and bedrock identification, testing, and data collection techniques, or

2. An engineering technician with at least five years of verifiable experience in local soils, bedrock description, testing, and data collection techniques.

Drilling Logs and test data presented must be reviewed and evaluated by a registered professional engineer.

Tester Qualification

Engineering technicians performing laboratory tests shall be qualified in accordance with the TxDOT Quality Assurance Program or other TxDOT approved programs. Approval of the engineering technician by TxDOT or other TxDOT approved programs does not relieve the professional/geotechnical engineer of the responsibility of ensuring the engineering technician is fully qualified to correctly perform the laboratory testing.

Equipment

Laboratory equipment used for testing shall be calibrated in accordance with TxDOT, National Institute of Standards (NIST), AASHTO, and ASTM requirements and the Geotechnical Engineer shall ensure the equipment meets these requirements.

Laboratory Accreditation

Laboratory testing shall be performed at TxDOT or AASHTO certified/accredited laboratories.

Chapter 5: Foundation Design

Contents:

Section 1: Design Methodology
Section 2: Foundation Selection
Section 3: Interpretation of Soil Data
Section 4: Uplift and Downdrag
Section 5: Drilled Shafts
Section 6: Driven Piling
Section 7: Foundation Load Testing
Section 8: Scour

Section 1: Design Methodology

Overview

Current TxDOT practice is to use the Load and Resistance Factor Design (LRFD) methodology for foundation design whenever practical and in accordance with AASHTO LRFD Bridge Design Specifications (current edition) and applicable Federal Highway Administration (FHWA) reference materials. This reliability-based design methodology (compared to the former TCP driven design correlations) creates greater utility for the state by accounting for a uniform level of reliability due to multiple factors and allowing for local calibration of factors depending on level of confidence through research. The basic equation for this method is:

$$\sum \eta i \gamma i Q i \leq \phi R_n = R_r$$

Where: η_i = a factor that includes the effects of ductility, redundancy, and importance

γ_i = the load factor for a particular load

Q_i = a service level load

ϕ = the resistance factor

R_n = the nominal (i.e., ultimate) resistance

R_r = the factored resistance

Proper foundation design requires communication between the geotechnical engineer and the structural engineer with consideration of data collected to address what information is needed along with when and how information will be exchanged.

Loading and Resistance

Substructure elements are designed to carry all the loads specified in AASHTO LRFD Bridge Design Specifications and the TxDOT Bridge Design Manual-LRFD. Selecting the controlling load conditions requires good judgment and coordination with the bridge engineer.

In accordance with AASHTO LRFD Bridge Design Specifications 3.6.2, neglect the dynamic load allowance on foundation components completely buried.

Evaluate structural resistance in accordance with the TxDOT Bridge Design Manual-LRFD. Evaluate geotechnical resistance according to criteria in this chapter.

Service Limit States

Include the following service limit states for foundation design:

- Settlement,

- Horizontal movements,
- Overall stability, and
- Scour at the design flood.

Foundation movement criteria shall be consistent with the function and type of structure, anticipated service life, and consequences of unacceptable movements on structure performance. Consideration of foundation movements shall be based upon structure tolerance to total and differential movements, rideability and economy. Foundation movements shall include all movement from settlement, horizontal movement, and rotation. The tolerable movement criteria shall be established by empirical procedures and/or structural analyses.

Evaluate foundation settlement, horizontal movement, and rotation using applicable loads in the Service I Load Combination specified in AASHTO LRFD Bridge Design Specifications Table 3.4.1-1. Transient loads may be omitted from settlement analyses for foundations bearing on or in cohesive soils that are subject to time-dependent consolidation settlement.

Strength Limit States

Evaluate the nominal foundation geotechnical resistance at the strength limit state considering the following:

- Axial compression resistance,
- Axial uplift resistance,
- Punching of shafts or piles through strong soil into a weak layer,
- Lateral geotechnical resistance of soil and rock strata,
- Resistance when scour occurs,
- Axial resistance of the structural element when downdrag may occur, and
- Pile drivability and driving stresses (for driven piles only).

Extreme Event Limit States

Structures must remain stable for an Extreme Event II limit state that considers scour due to the check flood required by the TxDOT Hydraulic Design Manual. This limit state need not include ice loads, vehicle collision loads, and vessel collision loads simultaneously. See Section 8 of this Chapter for additional information regarding scour analysis.

See the TxDOT Bridge Design Manual-LRFD for structures requiring consideration of earthquake effects.

Constructability

Design of foundations must consider the effects of the anticipated method of construction, including the construction sequencing. Such considerations shall consist of, but not be limited to: the need for shoring, the use of cofferdams, tremie seals, dewatering, excavation stability, downdrag considerations, and the need for permanent or temporary casing for drilled shafts or micropiles.

Design Process

Typical design steps are as follows:

1. . Establish design requirements for layout/geometry, loading, scour depths, tolerance to settlement (see recommendations above) and other service deformation/deflection
2. Determine depth of scour and hydraulic requirements of the structure in coordination with the hydraulic engineer
3. . Conduct geotechnical investigation (see Chapters 2, 3, and 4)
4. Select most appropriate foundation type and shaft/pile diameter(s) in coordination with structure designer
5. Evaluate need for permanent casing at individual foundations
6. Calculate nominal (unfactored) resistance of single drilled shafts or static compressive resistance (for piles) as a function of depth
7. Apply resistance factors to nominal axial resistance for strength and extreme limit states. Driven piles require additional resistance factors to be used during dynamic analysis based on field method to be used for pile acceptance (e.g., Hammer Formulas, wave equation, high strain dynamic load testing, etc.)
8. Conduct more extensive, nonstandard design required if deemed from subsurface conditions, bridge geometry, lateral loading, or service level criteria:
    1. Estimate downdrag potential and downdrag loads
    2. Check service level loads for shaft/pile single vs. group settlement as a function of depth (to maximum permissible settlement criteria)
    3. Check for uplift resistance as a function of depth
    4. Use P-Y curve parameters and horizontal movements in strength/extreme limit states to check for pushover/global/fixity. P-multipliers are not required for shaft/pile groups installed in rock sockets and lateral displacements are minimal (i.e., < 0.5 inches, or < 10% of shaft diameter)

5. Structural engineer evaluates applied lateral loads at the strength limit state using soil parameters determined by the geotechnical engineer.

6. Service level checks using unfactored service loading for top of shaft/pile deflection, including influence or downdrag loads if present, and effect of lateral squeeze and lateral deformations

9. Enter final parameters coordinated with structural analysis into plans and contract (with construction notes).

    1. Pile driving foundations can contain notes to perform pile drivability analysis and testing to obtain final required tip elevation or details for pile tip reinforcement

    2. Field control methods (such as integrity testing) can be included in notes and quantities

Lateral

Lateral depth checks and resistance should be considered depending on the height of the column, proposed substructure elements, and span configuration. Typical first checks on section involve the maximum moment and shear at top of shaft to determine the depth to the 2nd zero in the service load case, or the depth at which lateral defection at the top of shaft or pile is not affected by increased foundation depth in the strength load case.

Group Effects

Design close drilled shaft or closely spaced driven piles using group effect factors per AASHTO LRFD Bridge Design Specifications Article 10.8.3.6.

Section 2: Foundation Selection

Overview

Design foundations of new bridges as either drilled shafts or piling. Study all the available soil data and choose the type of foundation most suitable to the existing soil conditions and the particular structure.

Factors for Selection

Ultimately, the designer is responsible for selecting the appropriate bridge foundation. Consider the following factors in that selection:

- Nonstandard Bridge Geometry. Size and weight of proposed substructure and superstructure elements can vary greatly between projects and geotechnical engineer must work with planning and bridge engineer to determine the locations of foundations and conditions that would trigger additional design checks.
- Design load. The magnitude and type of design loading dictates the required size of the foundations from a structural standpoint. The foundation engineer must work in collaboration with the structural engineer to adjust sizes, quantities, and material types of foundation elements to meet or exceed resistances needed for the loading. Design loads typically will be provided and should be calculated for the strength, service, and extreme event limit states. This is consistent with structural considerations.
- Subsurface stratigraphy. The depth and strength of subsurface stratigraphy determine the type of foundation chosen. In general, drilled shafts are well suited to areas with competent soil and rock. While drilled shafts have been successfully installed in soft soil, they may be less efficient than piling. Very hard material at or near the surface makes driven pile installation impractical.
- Corrosive conditions. Salts, chlorides, and sulfates are detrimental to foundations. Where these conditions exist, take preventive measures. Use sulfate-resistant concrete as defined in Standard Specification Item 421 for construction in seawater or soils with high sulfate content. Consult the list of recommended corrosion protection areas for specific areas of Texas that may have structures with possible corrosion due to sulfate soil or salt water. Do not use steel piling in corrosive environments without an appropriate protective coating and/or providing additional steel section to ensure proper performance of the foundation elements over the design service life.
- Economic considerations. Consider economics in the final selection. Compare the foundation types. The cost of a drilled shaft foundation, for instance, may be less than piling. It may be feasible to use fewer piles at higher design loads, or fewer drilled shafts with larger diameters to maximize economy. If no clear economic difference exists between piling and drilled shafts, consider including both and offer the contractor alternate designs in the contract plans.

- Superstructure type. The type of superstructure chosen for the bridges may dictate or eliminate certain foundation types. For instance, short-span structures over streams may work well with trestle piling, but tall, single column flyovers justify footings with multiple shafts or piling.
- Special design requirements. Special designs are sometimes necessary to straddle another structure or utilities and may require a different type of foundation than the rest of the structure.

Foundation Guidelines for Widening Structures

Study test-boring data along with any available information regarding the existing foundation, including but not limited to drilled shaft or pile driving records. Though often collected with historic TCP drilling methods, usually, old test-boring data is adequate for widening the structure. In widening structures, consider special designs to prevent differential movement between the new and the old foundations. This is normally accomplished by founding the new foundations at approximately the same elevation as the existing foundations, if applicable. Do not use piling in widening structures founded on spread footings.

Widening Structures on Piling. Widen structures on piling with piling tipped in the same stratum, when possible. If loads for piling supporting the widened portion of the structure are the same or lower than loads for the original construction, tip the new piling at approximately the same elevation as the existing piling. If new loads are higher, longer or larger piling may be required. Avoid extreme variations between the new and existing tip elevations to minimize differential movement. Foundation design for new widening structures must consider the historic TCP methods for determining capacity in the original structure.

Widening Structures on Drilled Shafts. Widen structures on shafts with shafts at approximately the same tip elevations. Often existing structures with belled shafts may be widened with straight shafts tipped at the same elevation due to current higher allowable soil design loads and use of skin friction in drilled shaft design. Foundation design for new widening structures must consider the historic TCP methods for determining capacity in the original structure.

Widening Structures on Spread Footings. The most critical situation occurs when widening a structure founded on spread footings. If the existing footings are less than 6 ft. below natural ground and on rock, widen with spread footings at the same elevation. For abutment and interior bents on deep spread footings, widening with drilled shafts is usually more economical with the shafts founded near the existing footing elevation. This is not always practical, as in the case of widening a structure on spread footings with drilled shafts. In a case like this, evaluate the soil for shrink/swell potential.

Section 3: Interpretation of Soil Data

Overview

A critical step in foundation design is determining strata and reasonable strengths to be assigned to each stratum. Selecting design parameters that exceed those existing at the project site will result in increased risk of unacceptable performance. Selecting design parameters that are substantially less (or overly conservative) than those that exist will lead to increased costs from excessively conservative design or construction issues.

Divide the subsurface materials into strata based on material description and test values. Review all tests within each stratum to evaluate the variability of the data. If a single, unusually high strength test is present among a group of distinctly lower test values, disregard the anomalous test value. An average strength may be assigned for an entire layer(s) broken down into Engineering Stratigraphic Units (ESUs) when the test values are reasonably similar.

Avoid defining very thick strata with widely variable test values. Subdivide thick strata with test values varying from soft near the top to distinctly harder toward the bottom into two or more strata with compatible values. Failure to subdivide may result in an unconservative average strength being applied to foundations that terminate in the upper zone of that stratum.

In-situ vs. Lab Data

Geotechnical data reports including boring logs and in-situ and lab testing will be used in collaboration to assign design parameters to stratigraphic units. As previously mentioned, SPT provides an acceptable means to gage strength of sands, but hits refusal in bedrock and Intermediate Geomaterial (IGM) and results are highly variable in cohesive material. Lab results (in accordance with Chapter 4) are essential to isolate compressive and shear strength parameters when in clay or rock.

Disregard Depth

Disregard surface soil in the design of deep foundations, i.e., drilled shafts and driven piles. The disregarded depth is the amount of surface soil that is not included in the design of the foundation due to potential erosion from design flood or check flood, future excavation, seasonal soil moisture variation (shrinkage and swelling), lateral migration of waterways, and other factors. Amount of disregard may be different based on the design check being performed. For axial and compressive loading, disregard a minimum amount of 5 ft. over non-water crossings and 10 ft. over stream crossings. For abutments, disregard the portion of foundation passing through embankment fills. Note that the length of disregard may differ depending on the design check being performed.

When permanent casing is used for deep foundation installation, disregard side resistance with respect to axial design checks.

For projects where the existing ground line is at an elevation considerably higher than the proposed grade line (roadway is to be depressed) soil softening, swelling or heave must be accounted for in design of embankment slopes, roadways, retaining walls and foundation elements. Soils in these conditions respond to the removal of overburden (unloading). This response could have a dramatic impact on the design approach taken.

Additional considerations for disregard depth are required when encountering downdrag or scour. See Section 4 of this Chapter for downdrag. Information regarding disregarded depth and scour methodology at bridge foundations can be found in Section 6 and within the TxDOT Scour Analysis Guide.

Drilling Data, Laboratory Data, and Subsurface Classification

Acquire geotechnical borings in accordance with Chapter 2 and 3. Perform laboratory testing, borehole logging, soil/rock identification, classification and reporting in accordance with Chapter 4.

Identify rock type and characteristics in accordance with Chapter 4. Perform core recovery such that at least one (1) unconfined compressive test can be performed per bedrock unit or group with similar characteristics and per each boring.

Capacity From Texas Cone Penetration Test

Do not use Texas Cone Penetration Test for new designs. Refer to Appendix A for procedure to evaluate existing structures using TCP data.

Section 4: Uplift and Downdrag

Overview

Soil conditions and bridge geometry may control design and facilitate the need for additional design checks for uplift and downdrag.

Uplift

Substructure configuration typically results in a compressive load being applied to all of the shafts or piles for the service and strength limit states. However, some load combinations may require some of the members within a foundation system to resist an uplift (tension) force. Although this can occur when checking strength or service (due to wind or centrifugal force, etc.), this most commonly occurs in the extreme event case designing for impact loading.

The second source of uplift in deep foundations is often referred to as uplift pressure from swelling clays within the subgrade. This uplift force on the shafts and piles due to swelling of any active clays can be approximated by assuming a uniform swell pressure (from swell testing) acting over the perimeter of the shaft or pile to an ‘Active Zone’ depth of 10 to 20 feet, depending on professional judgement of engineer. Provide reinforcing in shafts to resist the uplift forces.

Downdrag

Downdrag (or negative skin friction) is an additional force acting on an installed driven pile or drilled shaft foundation which tends to drag or pull the foundation downward. Conditions where it should be evaluated are outlined in AASHTO LRFD Bridge Design Specifications Article 3.11.8. In-situ behavior depends on top of pile/shaft loading, soil stiffness and interface friction properties, lateral stress, and time-rate effects such as consolidation and soil set-up. The downdrag force, or dragload, typically develops by consolidation of soft soils underneath embankments. As soil consolidation progresses, shear stresses (“drag” forces) are induced between the relatively fixed pile or shaft and the adjacent, downward moving embankment soil. Site conditions, which might promote a modest to large dragload effect include:

- Changes in overburden weight/geometry at, or adjacent to, foundations with compressible soil strata. Including embankment widening, excavation removal and replacements, and other general construction earth moving operations.
- Deep foundations installed through compressible soil with ongoing processes of slowly consolidating soils from previous fill placement.
- Dewatering or changes in native groundwater or soil moisture.

Sufficient penetration into natural soil is required to counteract all the anticipated negative friction plus dead and live load forces. Disregard side resistance in all fill material when designing in strength limit state during conditions of potential downdrag. Also disregard an additional 5-10 feet of the natural soil under fill to compensate for the weight of the fill imposed on the load carrying stratigraphy.

At the geotechnical strength limit state, the entire shaft or pile is moving downward relative to the soil and therefore negative skin friction is not present. Foundation settlement is concern especially for friction piles and drilled shafts. Downdrag or dragload should not influence the geotechnical strength limit state analysis, rather the concern is at the structural strength limit state and geotechnical service limit state. AASHTO LRFD Bridge Design Specifications Articles 10.7.1.6.2 (for driven piles) and 10.8.3.4 (for drilled shafts) provide guidance on downdrag assumptions and the “neutral plane method” to calculate dragload for use in service limit state analysis as described in FHWA-NHI-16-009.

Methods to consider for addressing potential downdrag include:

- Preloading or surcharging an embankment with waiting period
- Removal and replacement with material less prone to consolidate
- Increase the pile or shaft size and length of embedment
- Drilling shafts with casing or using sleeves or a bitumen coating on driven piling

Section 5: Drilled Shafts

Overview

Comply with the TxDOT Bridge Design Manual-LRFD and the AASHTO LRFD Bridge Design Specifications (current edition) for all aspects of foundation design, unless otherwise specified by TxDOT Bridge Division and Geotech Manual.

Drilled shafts are the most common foundation type selected for TxDOT bridges. Deep foundations could be designed considering exclusively side resistance (skin friction) or a combination of side and end resistance at the discretion of foundation geotechnical engineer.

Resistance in Soils

As specified in AASHTO LRFD Bridge Design Specifications, designers are directed to α-method in clays and cohesive soil layers and β-method within sands and non-cohesive material. For the later, note that engineering judgement is necessary when determining drained friction angle for an individual layer. AASHTO LRFD Bridge Design Specifications Table 10.4.6.2.4-1 presents friction angle φf ranges according to measured (N1)60 values. When selecting an effective soil friction angle according to AASHTO LRFD Bridge Design Specifications Equation 10.8.3.5.2b-3, use an additional reduction factor of 0.9 to account for the lower end of the range of friction angles in granular material with significant fraction of fines, such that:

$$\varphi'f = 0.9 * (27.5 + 9.2 \log [(N_1)_{60}])$$

Resistance in Rock and Intermediate Geomaterials

Rock-socketing into competent foundation layers is a common practice throughout Texas. Throughout the state competent foundation layer will vary from very hard, intact, non-weathered bedrock; to very soft, friable with poor jointing conditions, and/or extremely fractured “bedrock-like” conditions; to Intermediate Geomaterials (IGMs, as defined in AASHTO Article 10.8.2.2.3) displaying characteristics of both rock and soil. Foundation designer is responsible for determining if (within socket) only side or end resistance can be considered in their determination of total resistance; or in cases of softer competent foundation, they can incorporate part or all of both. When encountering fractured strong rock, or softer cohesive IGMs such as shale and/or severely weathered limestone; note that alternative methods are specified by AASHTO and GEC-10 (2010 and 2018). In stratified or visibly jointed rock bearing layers, it’s difficult to determine how much of actual load will be transferred to base of the drilled shaft and in lieu of load testing at locations, practical design should assume that the axial load will be resisted entirely by side resistance.

When relying on a rock layer for capacity, minimum rock socket to use on any project is 1 diameter length into such rock. Should rock be located near the ground surface, shafts should be drilled at minimum 10 feet or 3 diameters in length (whichever greater).

Resistance Factors

Use Resistance Factors for Drilled Shafts per AASHTO LRFD Bridge Design Specifications Table 10.5.5.2.4-1 for Strength and Extreme limit states, unless otherwise specified in other approved methods outlined in FHWA-NHI-18-024, GEC-10 Appendix B.

Belled Shafts

Do not use belled shafts for bridge foundations.

Standing Water

Drilled shafts installed in lakes or rivers require use of a casing placed from above the water surface to a minimum embedment into the river or lake bottom. Define the top of the drilled shaft in water as 2 ft. typical above the normal water elevation. If the water level is variable, add a provision allowing the top of the drilled shaft to be adjusted vertically based on water level at the time of construction. If casing is to be left in place, disregard side resistance along the length of the casing. If permanent casing is used in standing water, consideration should be given to painting the portion of casing extending above the mud line.

Micropiles

Design micropiles in accordance with AASHTO LRFD Bridge Design Specifications Article 10.5 and 10.9. Additional background information on micropile design may be found in the FHWA Micropile Design and Construction Guidelines Implementation Manual, Publication No. FHWA-SA-97-070 (Armour. et al., 2000) or FHWA-NHI-05-039.

Wing Wall Drilled Shafts

Found wing shafts in similar founding material as abutment cap shafts to minimize the potential for differential settlement. Maximum length of wing wall drilled shafts is limited to 30 x D.

Strength Loads

Design foundations to resist factored maximum strength loading case and extreme event loading case. Coordination with the structural engineer is required to ensure clarity on derivation and location of loading to be designed for.

Service Loads

See the following Table 5-1 for maximum drilled shaft service loads recommended without conducting a detailed structural analysis. Foundation design outlined in this chapter must be followed to ensure the proper sizes are selected and embedment criteria is specified in the contract plans.

Table 5-1: Maximum Allowable Drilled Shaft Service Loads

Size	Load
24 in.	175 tons
30 in.	275 tons
36 in.	400 tons
42 in.	525 tons
48 in.	700 tons
54 in.	900 tons
60 in.	1,100 tons
66 in.	1,350 tons
72 in.	1,600 tons
84 in.	2,175 tons
96 in.	2,850 tons
108 in.	3,625 tons
120 in.	4,475 tons

Drilled Shaft Reinforcement

Drilled shaft reinforcement is to be designed for axial, lateral, and uplift load (included within nonstandard design checks). The reinforcement will follow the Common Foundation Details (FD) Standard, unless site specific designs are required which require alternate reinforcement. The longitudinal reinforcement for the drilled shaft will extend the full length of the shaft.

Installation Nearby Other 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.

Drilled Shaft Integrity Testing

Various testing methods are available to determine the integrity of drilled shafts, which are Crosshole Sonic Logging (CSL), Gamma-Gamma testing, and Thermal Integrity Profiling (TIP). TIP is the preferred testing method, as it is done during the curing of the concrete and does not delay construction. Other methods are approved based on the priorities of the project. Bridge Division has developed a Special Specification for TIP testing titled “Thermal Integrity Profiler (TIP) Testing of Drilled Shafts.”

TIP or other integrity testing should be considered for use under one or more of the following conditions:

- Mono-shafts;
- Large diameter shafts (60” diameter, or greater);
- Drilled shafts with a diameter > 24 inches encountering water bearing sands in the soil profile and on critical roadways, such as interstate systems, high ADT roadways, emergency routes, evacuation routes, etc.

Number and frequency of tests is at discretion of foundation engineer and dependent on site specific conditions and redundancy designed into the foundation system.

Consult with the TxDOT Bridge Division Geotechnical Branch to determine if a specific project might be considered a candidate for TIP or other integrity testing.

Layout Requirements and Notes

Label foundations on plan set bridge layouts with the following:

- Number and size of drilled shaft at each bent and abutment
- Anticipated shaft length and tip elevation
- Location of geotechnical borings used for design of the foundations and note referencing boring logs within plan set
- Maximum tipping elevation of permanent casing (should designer deem that permanent casing is warranted at any or all of the bents/abutments)
- Note to advise contractor that subsurface conditions may require the use of temporary casing and/or slurry in accordance with Item 416 (should designer deem that high groundwater or flooding conditions are present)
- When drilled shaft capacity depends heavily on penetrating a specific hard layer, add a plan note instructing the contractor and field personnel of the penetration requirement. If no specific penetration into a hard layer is required, no plan note is necessary

Typical notes on bridge layouts:

- *"Found drilled shafts a minimum of two shaft diameters into hard rock"*, or
- *"Found drilled shafts at the elevations (lengths) shown or deeper (longer) to obtain a minimum XX drilled shaft diameter penetration into hard rock"*, where XX is determined by the design.

The designer can use the control of elevation or length if elevations are not called out on the layout. Expand the words "hard rock" to distinguish the type of material anticipated. Although not a common practice, the first note allows a drilled shaft to be shortened if rock is encountered at higher than anticipated elevations, and it requires the shaft to be lengthened if rock is not encountered where expected.

Rock at surface. When rock is present at or near the surface, consider load-carrying capacity along with the stability of the superstructure on the foundation. For these shafts, a minimum shaft length of three shaft diameters is recommended. That is, a minimum three-diameter shaft length, not a three-diameter penetration into rock. The final length of the drilled shafts should be based on both axial and lateral loading (if required). If the potential scour extends down to the top of rock, then the minimum embedment of the drilled shaft should be three shaft diameters or deeper to obtain the required axial and lateral capacity.

Plan notes should be specific as to the type of material to be penetrated. If more than one material is likely to be encountered, it is acceptable to have multiple descriptions, such as “into sandstone, and/or shale.” Avoid using vague terms such as “hard strata” or “founding material.” In stream or river environments, the channel flow line and estimated depth of scour should be considered in determining the final shaft length and necessary penetration.

Drilled Shaft Foundation Design Reporting

Include the following information on geotechnical design reports for drilled shaft foundation design:

- Geotechnical Data Report and Borings (see Chapter 4)
- Summary of proposed construction, factored foundation loads, applicable limit states, performance criteria (settlements, lateral deformation)
- Scour and hydraulic assumptions
- Applicable site constraints such as any suspected environmental restrictions, utility conflicts, adjacent structures, or limitations on construction (ROW, headroom, etc.)
- Summary of soil and bedrock and IGM parameters and design analysis
- Recommendations for ground improvement to increase bearing resistance and reduce settlement (if needed)
- Description of design procedures with summary of results and explanation of interpretation, particularly:

  - Shaft tip elevations or estimated lengths

- Assumptions on casing
- Nominal geotechnical resistances and resistance determination method
- Corrosion effects or chemical/biological attack susceptibility
- Specified integrity and/or load testing requirements
- Expanded analysis preformed such as drawdown with neutral plane axis, settlement estimates, and lateral load resistance and deformation.

- Construction recommendations and recommendations for notes required on contract drawings/ plans
- Signed and sealed by the Engineer

Section 6: Driven Piling

Overview

Conduct geotechnical design of driven pile foundations, and all related considerations, per AASHTO LRFD Bridge Design Specifications Article 10.7. Piling design should consider skin friction and may consider point bearing as well. Because piling has small tip areas and is generally placed in softer soil, the point bearing contribution is modest and is often disregarded in static design. Exception should be made when tipping a driven pile into bedrock or very competent, hard founding material. In these cases, end bearing is crucial, and piles could be designed exclusively from tip resistance or using a combination of tip and side (depending on pile type used).

Technical specifics of many common driven pile types can be found in FHWA GEC-12 Design and Construction of Driven Pile Foundations – Volume I (FHWA-NHI-16-009), Chapter 6, Section 6.1. These can be used when making preliminary pile type selection in design.

Driven piles are not designed nor accepted based solely on static analysis. The nominal bearing resistance of all driven piles must be calculated and accepted based on Hammer Formula (Item 404, Section 3.5, for dynamic bearing resistance), wave equation analysis (e.g., drivability analysis and final driving acceptance criteria by GRLWEAP), dynamic measurements with signal matching (PDA/CAPWAP), or full-scale load testing results.

On refusal, assume that the piling has developed the maximum allowable service load for the pile. If required, perform a drivability study to establish a hammer & driving system that can install the pile without overstressing. Include this study in Geotechnical Design Report.

Design based on process outlined in Section 1 with additional steps:

- Specify resistance factors to use based on field methods to use dynamic formulas for pile acceptance (or refer to Item 404 Section 3.5 specification for Hammer Formula Method of Bearing Evaluation to determine the allowable dynamic bearing resistance).
- Perform a pile drivability analysis to obtain required tip elevations (*if required*). Pile acceptance based on the pile driving analyzer (PDA) is for projects where it is cost effective on large number of friction piles or where high pile driving stresses are predicted and require monitoring

Pile design resistance should meet or exceed the requirements specified for each limit state, both in static analysis and dynamically.

Design with caution when designing piling in areas with shallow hard or dense soils. If piling cannot be driven through these areas, the contractor will need to pilot hole or jet the piling to achieve the desired penetration. Jetting should avoid an area with existing foundations and utilities. Excessive pilot holes and jetting may affect the foundation capacity.

LRFD General Design

Use Resistance Factors for Driven Pile found in AASHTO 9th Ed. Table 10.5.5.2.3-1. For soils side resistance in static design, use the Norland/Thurman method in cohesionless soils in accordance with AASHTO LRFD Bridge Design Specifications Article 10.7.3.8.6f, and the a-method in cohesive soils in accordance with AASHTO LRFD Bridge Design Specifications Article 10.7.3.8.6b.

TxDOT utilizes static analysis for design and dynamic analysis for acceptance of driven pile foundations.

Resistance Factors

Use Resistance Factors for driven piles found in AASHTO 9th Ed. Table 10.5.5.2.3-1 for Strength and Extreme limit states.

Load testing and dynamic testing can be used to increase a resistance factor more than using Hammer Equations alone.

Pile Static Design

General components of static analyses to consider are:

1. Nominal resistance in **axial compression** of a single pile or pile group. These calculation methods are used to determine the long-term resistance of the foundation and soil resistance subject to scour, downdrag, or events in the long term. Static analyses are used to establish minimum pile penetration requirements, lengths for bid quantities, and estimates of soil resistance at the time of driving (SRD) and the required nominal driving resistance (R_{ndr}).

2. Nominal resistance in **axial tension** of a single pile or pile group. These calculations are performed to determine the soil resistance to uplift or tension loading.

3. Nominal **lateral** resistance and lateral deformation of a single pile or pile group. These soilstructure interaction analysis methods consider the soil strength and deformation behavior as well as pile structural properties and are used in pile type selection.

4. **Settlement** of a pile group. These calculations are performed to determine the vertical foundation deformation under the structure service loads.

Pile Dynamic Design

High pile stresses occur during pile driving operations. A pile drivability analysis is typically used to determine the nominal geotechnical resistance a pile can be driven to without pile structural damage.

The Hammer Equation found in Item 404 is used to determine acceptance criteria (for final embedment and length). Where piles are driven to higher resistances or where high pile driving stress is a concern (i.e., short, end bearing piles), the wave equation analysis (through GRLWEAP) should be used for drivability and final driving acceptance. In cases where high pile driving stress is predicted and require monitoring, consider using pile driving analyzer (PDA) with wave analysis (through program such as CAPWAP).

Dynamic Monitoring

Dynamic monitoring of a pile during driving can be accomplished using a Pile Driving Analyzer (PDA) testing system. PDA testing measures the strain and acceleration in the pile as a result of the impact of the hammer. PDA testing of a pile can help to determine the stresses in the pile during driving and monitor the pile for damage or integrity. The capacity of the pile and time dependent changes in capacity (if a restrike is undertaken) can be obtained when the PDA testing data is used with the Case Pile Wave Analysis Program (CAPWAP).

For critical structures, projects with a large number of piling, or in difficult soil conditions PDA testing should be considered for use. Consult with the Geotechnical Branch to determine if a specific project might be considered as a candidate for PDA testing.

Pile Tip Elevations

To ensure constructed foundation meets the design requirements, pile tip elevations or pile lengths are required on the contract plans. As noted in section 1: Design Process, the final length and tip elevation may be controlled by any or all of the following criteria:

- Pile tip to reach designated bearing layer
- Scour
- Downdrag
- Uplift
- Lateral Loads

Difficult Driving and Drivability

If it is necessary to advance the piling through a strong or stiff layer where refusal is possible, an additional pile penetration note as follows may be required, "The contractor’s attention is drawn to the hard material in the soil profile, jetting and/or pilot holes may be necessary to advance the piling to the required penetration depth.”

Be aware that under these conditions of potentially high driving stresses, a wave equation drivability analysis is necessary to ensure piles can be driven to required embedment depth. Higher grade steel can be specified if needed to meet drivability criteria. Coordinate any changes in the pile size, section, or tip elevations with the structural engineer. The geotechnical foundation engineer is responsible for reevaluating pile drivability during this iterative process.

Candidate pile types that cannot be driven to the required nominal resistance and/or minimum pile penetration without exceeding material stress limits and within a reasonable blow count of 30 to 120 blows per foot with appropriately sized driving systems should be eliminated from consideration. 120 blows per foot or 10 blows per inch is often considered refusal driving conditions by many hammer manufacturers.

Pile Setup and Restrike

Using a waiting period and restrike after initial pile diving may be advantageous in certain soil conditions to optimize pile foundation design. Setup for a specified waiting period allows pore water pressures to dissipate and soil strength to increase. Restriking then confirms if higher nominal resistance is achieved. The length of the waiting period depends on the strength and drainage characteristics of the subsurface soils, and the required nominal resistance. Refer to Standard Specification 404 for additional pile driving construction criteria.

Wing Wall Piling

Found and tip wing wall piling in similar founding material as abutment cap piles to minimize the potential for differential settlement.

Steel Piling Special Considerations

- Corrosion:

  Steel piles driven through contaminated soil and groundwater conditions may be subject to high corrosion rates and should be designed appropriately through the use of larger section, galvanization or concrete cover. Corrosion may occur if piles are driven into disturbed ground, landfills or cinder fills, or low pH soils. Corrosion should also be evaluated for piles located in marine environment, or if piles are subject to alternate wetting and drying from tidal action. Rates are a function of the ambient temperature, pH, access to oxygen, and chemistry of the aqueous environment in contact with the steel member(s).

- Grade Separations:

  Foundation elements for grade separations are subject to potential vehicular impact. The use of steel sections in a trestle configuration in those potential impact zones is highly discouraged. Instead, steel H piling can potentially be used under pile footings for interior bents or abutments at grade separations.

- Water Crossings:

  Foundation elements for crossings over waterways are subject to scour, drift impact and have a higher propensity for corrosion. Steel piling needs to be analyzed for potential corrosion over the life span of the structure and need to be evaluated for both axial and lateral loadings under the scoured condition. Steel piling that have been evaluated for the above conditions and found to be acceptable could be used for trestle bents. However, the steel piling must be coated to a minimum depth of 15 feet below the maximum predicted scour elevation. Steel piling can be used to support pile footings as long as the footing is embedded at a depth below the maximum predicted scour depth thus minimizing the risk of exposure. Piling used in a footing configuration must be coated a minimum distance of 15' below the bottom of footing. Piling can be used for foundation elements for abutments.

Service Loads

See the following table for maximum piling length and structural loads recommended without conducting a detailed structural analysis. Many soils are not capable of developing these maximum loads. Before final structural design, conduct foundation design using site specific soil information to verify the ability of subsurface to provide resistance to the loading.

Table 5-2: Maximum Allowable Precast Concrete Pile Service Loads

Size	Maximum Length	Abutments and Trestle Bents	Footings (per Pile)
16 in. Square	85 ft.	75 ton	125 tons
18 in. Square	95 ft.	90 tons	175 tons
20 in. Square	105 ft.	110 tons	225 tons
24 in. Square	125 ft.	140 tons	300 tons

Pile Lateral Resistance

Pile foundations are subjected to horizontal loads due to wind, traffic loads, bridge curvature, and vessel or traffic impact. Evaluate the nominal resistance of pile foundation to horizontal loads based on both subsurface strata and structural properties.

Refer to AASHTO LRFD Bridge Design Specifications Article 10.7.2.4 for detailed requirements regarding determination of lateral resistance. Use a minimum spacing of 3 pile diameters (3D) to the extent possible. Should closer spacing be required due to geometric constraints, the following P_m values may be used at spacing 3D to 2D in accordance with Article 10.7.2.4:

- For Row 1, $P_m = 0.45$
- For Row 2, $P_m = 0.33$
- For Row 3 and higher, $P_m = 0.25$

Pile Foundation Design Reporting

Include the following in Geotechnical Design Reports for driven pile foundation design:

- Geotechnical Data Report and Borings (see Chapter 4)
- Summary of proposed construction, factored foundation loads, applicable limit states, performance criteria (settlements, lateral deformation)
- Scour and hydraulic assumptions
- Applicable site constraints including any suspected environmental restrictions, utility conflicts, adjacent structures, or limitations on construction (ROW, headroom, etc.)
- Summary of soil and bedrock and intermediate geomaterial parameters and design analysis
- Recommendations for ground improvement to increase bearing resistance and reduce settlement
- Description of design procedures with summary of results and explanation of interpretation, particularly:
    - Pile tip elevations or estimated pile lengths
    - Minimum pile penetration (see AASHTO LRFD Bridge Design Specifications Article 10.7.7)
    - Pile driving requirements (hammer size, sequence, etc)
    - Nominal driving resistance and resistance determination method (driving criteria) (see AASHTO LRFD Bridge Design Specifications Table 10.5.5.2.3-1)
    - Corrosion effects or chemical/biological attack susceptibility
    - Specified load testing requirements or test piles
    - Expanded analysis preformed such as drawdown with neutral plane axis, settlement estimates, and lateral load resistance and deformation.
- Construction recommendations and recommendations for notes required on contract drawings/plans
- Signed and sealed by the Engineer

Section 7: Foundation Load Testing

Foundation load testing is a reliable means of determining the capacity of the foundation elements. Foundation load testing is governed by Standard Specification Item 405. The various testing methods that can be used are:

- Static load testing,
- Bi-directional Osterberg Cell Load Testing,
- High strain dynamic testing, and
- Statnamic testing.

Not all foundations will require foundation load testing. Typically, load testing of a drilled shaft foundation is used in conjunction with Thermal Integrity Profiling (TIP) or other integrity testing. Consult with the Geotechnical Branch prior to using foundation load testing on a project.

Driven piles use PDA while installing foundation which can be considered another form of load test. Drivability analysis is required for cases of driven piles and clear termination criteria should be established based on equipment used by contractor, as described in Section 4.

Section 8: Scour

Overview

Incorporate the effects of scour in the determination of shaft and pile penetration. Design the foundations so that the penetration and resistance remaining after the design scour events satisfies the required nominal axial and lateral resistances. Both reduced geotechnical resistances and increased unsupported length in the columns must be accommodated for in design.

Scour at the foundations is not a force effect. However, scour can change the substructure conditions and topography and alter the consequences of force effects acting on the structure and foundations. AASHTO LRFD Bridge Design Specifications Article 2.6.4.4.2 requires changes in foundation conditions resulting from the design flood be evaluated at the strength and service limit states. Foundation condition changes from the check flood are to be considered and evaluated at the extreme event limit state.

Refer to the TxDOT Scour Analysis Guide for background to aid when determining total scour on any specific design flood and check flood for design purposes.

Design the foundations to resist debris loads occurring during flood events in addition to the loads applied from the structure.

LRFD Design

Use the same resistance factors when evaluating conditions with scour at the strength limit state as those used without scour. Do not include the axial resistance of the material lost due to scour in the shaft resistance.

Include on the plans the unfactored resistance to be achieved during construction for the unscoured bridge condition. This resistance will be the minimum target to achieve during dynamic analysis of pile installation.

Driven Piles and Scour

Design pile foundations such that the pile penetration after the design scour events satisfies the required nominal axial and lateral resistance. 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.

$$\sum \eta i \gamma i Q_i \leq \phi R_n$$

Nominal resistance needed (R_n) in the static final condition after compensating for design scour must be greater than the factored loads ($\sum \eta i \gamma i Qi$). Include an additional resistance factor (ϕdyn) for the driving resistance checks during construction based on the dynamic method used:

$$R_n \geq (\sum \eta i \gamma i Q_i) / \phi dyn$$

Normal pile driving resistance achieved during construction (R_{ndr}) includes the skin friction (side resistance) contribution that would be lost in the scour zone:

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

R_{ndr} = Nominal (ultimate) resistance during pile driving, dynamically evaluated

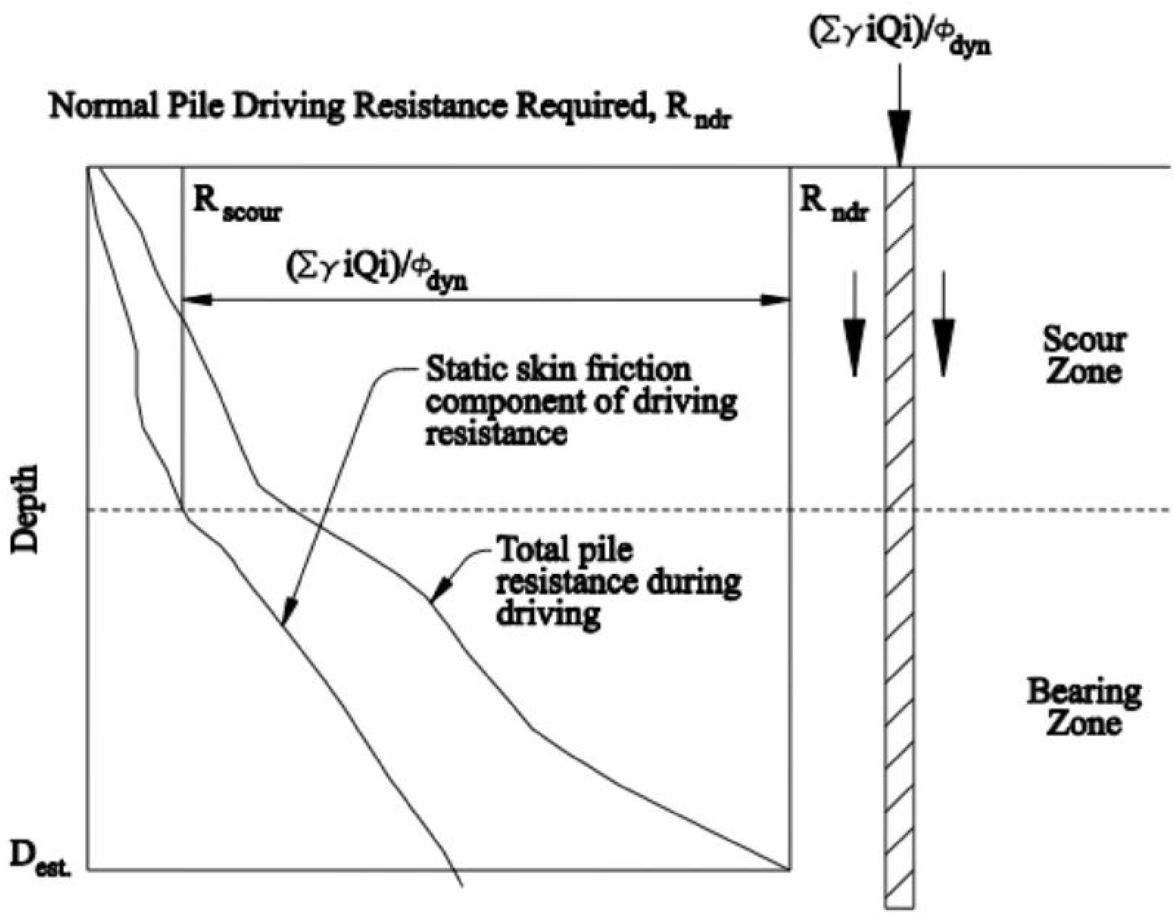
R_n = Nominal (ultimate) resistance needed in the final static condition

R_{scour} = Unfactored skin friction which must be overcome during driving in scour zone (kips)

Q_p = ($\sum \eta i \gamma i Qi$) = factored load per pile (kips)

ϕdyn = resistance factor

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



Scour Coding, Inspection, and Countermeasures

The Bridge Division establishes program requirements and provides geotechnical subject matter expertise for the determination of soil characteristics to be used for scour analyses and for the phases of scour evaluation that occur after a scour analysis: bridge inspection documentation, screenings, assessments, and scour countermeasures. This guidance can be found in the TxDOT Scour Evaluation Guide.

Stone Protection at Bridges

Protecting abutments and piers at bridges is beneficial in limiting the effects of scour. Use flexible armoring (i.e. stone protection) for wet crossing structures. Concrete riprap, due to its rigidity, masks problems. Consequently, voids can form under them and eventually undermine the pavement or approach slab. Guidance on the use of stone and sizing and thickness to specify can be found in Chapter 11 of the TxDOT Scour Evaluation Guide.

In the plans Stone Protection should be specified and called out as follows (on each abutment side or location of placement):

Riprap (Stone Protection) XX in. (where XX is the size in inches)

Thickness = YY in. (where YY is the appropriate thickness)

Chapter 6: Retaining Walls and Reinforced Soil Slopes

Contents:

Section 1: Retaining Wall Selection
Section 2: Retaining Wall Layouts
Section 3: Design Considerations
Section 4: Excavation Support

Section 1: Retaining Wall Selection

Overview

The project engineer who seals the plans is responsible for ensuring that the retaining wall selected for a given location is appropriate. Use the following criteria to choose a retaining wall:

- Geometry. Determine applicability of wall type—cut, cut/fill, or fill—based on geometry, site constraints, and wall alignment and location. Identify available right of way. Identify location and type of existing and proposed utilities. Identify location and type of existing and proposed drainage structures.
- Economics. Evaluate the total cost of wall, durability, maintenance, and life cycle cost including needed excavation shoring. Identify required utility adjustments and costs. Identify project schedule, speed of construction, phasing requirements, and effect on wall construction and design.
- Stability. Evaluate all walls to ensure that minimum factors of safety are met for global stability. When possible, avoid placing walls on slopes. A slope in front of the wall dramatically reduces passive earth pressure (resistance) and can compromise global stability, increasing the probability of wall failure. For situations where walls above a slope cannot be avoided, conduct a rigorous stability analysis following conditions identified in the Design Considerations section of this chapter.
- Constructability. Determine whether walls are near water or subject to inundation. Identify access limitations for equipment. Ensure adequate horizontal and vertical clearances are provided for installation of retaining wall types, particularly tied-back, nailed, and drilled shaft walls. Existing and proposed utilities should be considered for the constructability of the retaining wall.
- Aesthetics. Ensure that the aesthetic treatment of the wall complements the retaining wall and does not disrupt the functionality or selection of wall type. Be careful with aesthetic treatments that involve landscaping: design additional drainage measures if extensive watering is anticipated to prevent excessive hydrostatic pressures from building up behind the wall.

Section 2: Retaining Wall Layouts

General Content Layout

In general, retaining wall layouts include the following information.

- Plan View. Include the following in the plan view:
  - Beginning and ending wall points by station, offset, and roadway alignment
  - Additional points as necessary to describe the relationship of wall alignment to roadway alignment(s)
  - Indication of which side is the face of the wall
  - Horizontal curve information if applicable for wall alignment
  - Location of soil borings (Include boring name, station, offset, and top-of-hole elevation.)
  - Signing, lighting, etc., mounted on or passing through wall (Designate and locate the sheets that contain information for these elements.)
  - Surface and subsurface drainage structures or utilities that could affect or be affected by wall construction (Designate and locate the sheets that contain information on the structure or utilities.)
  - Limits of temporary special shoring
- Elevation view. Include the following in the elevation view:
  - Existing ground line along wall alignment
  - Proposed finished grade line at face of wall
  - Bottom of wall for payment
  - Top of retaining wall grade line (Does not include the top of rail.)
  - Soil boring information where possible, shown at the correct elevation and scale
  - Designation for “Back Face of Wall” when back of wall is shown
  - Panel numbers when applicable
  - Drainage structures and features including slope at flowline, sizing, and maintenance recommendations within contract notes
  - Utilities, signing, lighting, etc., as noted above
- Estimated quantity table. Include the estimated quantity table for each retaining wall type. Refer to a specific wall type for list of bid codes. Include the following in the estimated quantity table:
  - Area of retaining wall

  - Linear footage of railing on wall
  - Miscellaneous quantities associated with wall (riprap, etc.)
- Typical section. A typical section should contain the following information:
  - Cross section showing the relationship of the wall to the roadway
  - Control point for horizontal and vertical alignment, typically shown at the top outermost corner of the wall
  - Indication of maximum slope on top of and in front of wall
  - Location of proposed finished grade
  - Railing type, flume, mow strip, etc., if applicable
  - Distance from back of wall panel to face of abutment cap, if applicable
- General notes. Include the following in the general notes:
  - A note stating the required wall embedment depth if the specified embedment is greater than 1 ft. for slopes up to 4:1 in front of wall or 2 ft. for slopes in front of wall that are steeper than 4:1, as well as a note stating that the wall is measured between top of wall and "X" ft. below finished grade
  - Reference to all applicable standard sheets for pertinent information
  - Other pertinent information regarding wall design and construction
  - Foundation design criteria (nominal bearing resistance, resistance factor and others)

Plans for Specific Wall Types

For specific retaining wall types, include the following additional information on the layout and in the plan set.

Spread Footing Walls. For spread footing walls, include the following additional information:

- Panel design designation (for example, LC-10-32) for each panel corresponding to the appropriate cast-in-place spread footing wall standard sheet. The designation includes a reference to the controlling standard drawing, design height, and panel width information.
- Location of expansion and construction joints (Assuming 32-ft. panels, every third joint is typically designated as an expansion joint.)
- Set bottom of wall (top of footing) horizontal and stepped to meet minimum embedment criteria. (Distance from one step to the next is typically greater than 6 in. Provide bottom of wall elevations for all panels.)
- Appropriate standard sheets pertaining to cast-in-place spread footing walls. Select appropriate standard design case based on available surcharge and slope condition. Limit wall height to

maximum of 20 feet as per standard. Wall height more than this limit will require custom design and approval from the Department.

Designate all information necessary for the contractor to construct the wall on retaining wall layouts for spread footing walls. This type of wall does not have a proprietary vendor to provide shop drawings, so the plan set must be complete with details.

Mechanically Stabilized Earth (MSE) Walls. For MSE walls, include the following additional information:

- Bottom of wall shown following the proposed finished grade offset at the minimum embedment depth specified
- Appropriate standard sheets pertaining to MSE walls (e.g., RW(MSE)DD – Mechanically Stabilized Earth Retaining Wall Design Data)

Concrete Block Walls. For concrete block walls, include the following additional information:

- Bottom of wall shown following the proposed finished grade offset at the minimum embedment depth specified
- Appropriate standard sheets pertaining to concrete block walls (e.g., RW(CB)DD sheet)

Tied-Back Walls. For tied-back walls, include the following additional information:

- Type, location and spacing of tied back anchors
- Bar dimension, incline angle, and material grade
- Drill hole diameter, minimum bond length requirements, and corrosion protection measures
- Panel and closure-pour width dimensions
- Bottom of wall shown with a level footing elevation, also referred to as having steps. (Distance from one step to the next is typically greater than 6 in.)
- Performance load test requirement

Designate all information necessary for the contractor to construct the wall on retaining wall layouts for tied-back walls. This type of wall does not have a proprietary vendor; however, shop drawings are required to fully detail the panel schedule to be used on the project and information regarding proposed anchor length.

Soil/Rock Nailed Walls. For soil or rock nailed walls, include the following additional information:

- Location of expansion and construction joints spaced at intervals not to exceed 90 ft.
- Set bottom of wall horizontal and stepped to meet minimum embedment criteria. (Distance from one step to the next is typically greater than 6 in. Provide bottom of wall elevations for all panels.)
- Estimated quantity for “Soil/Rock Nail Anchors”

- Typical section showing existing or proposed foundations or other obstructions that may interfere with wall construction
- Test nail lengths, loads, and quantities
- Nail incline angle, bar grade and size, and corrosion protection measures
- Wall facing and nail head connection
- Reinforcement details and shotcrete thickness
- Proof and verification load test requirements

Designate all information necessary for the contractor to construct the wall on retaining wall layouts for nailed walls. This type of wall does not have a proprietary vendor to provide shop drawings, so the plan set must be complete with details.

Drilled Shaft Walls. For drilled shaft walls, include the following additional information:

- Set bottom of wall horizontal and stepped to meet minimum embedment criteria
- Panel width dimensions and concrete closure pour connection
- Provide cap, stud anchor connection detail sheets including closure pilaster if any
- Bottom of wall shown with a level footing elevation, also referred to as having steps. (Distance from one step to the next is typically greater than 6 in. Provide bottom of wall elevations for all panels.)
- Estimated quantity for “Drilled Shaft” used on wall (This quantity is broken into specified shaft diameters.)

Designate all information necessary for the contractor to construct the wall on retaining wall layouts for drilled shaft walls. This type of wall does not have a proprietary vendor to provide shop drawings, so the plan set must be complete with details.

Temporary MSE Walls. For temporary MSE walls, include the following additional information:

- Bottom of wall shown following the proposed finished grade offset at the minimum embedment depth specified
- Appropriate standard sheets pertaining to temporary MSE walls (including RW(TEW)DD sheet)

Reinforced Soil Slope. For reinforced slopes, include the following information in additional to applicable retaining wall general content layout:

- Bottom and top of existing slope following the proposed finished grade offset
- Existing and proposed slope grade
- Geogrid reinforcement length, details and backfill requirements
- Temporary special shoring if needed

- Limits of excavation and benching for the placement of backfill

Designate all information necessary for the contractor to construct RSS. This type of system does not have a proprietary vendor to provide shop drawings, so the plan set must be complete with details.

Section 3: Design Considerations

General Design

Design and analyze walls following accepted geotechnical engineering industry standards. In analyses, use earth pressure loads that follow governing sections of the current edition of the AASHTO LRFD Bridge Design Specifications. Consider strength, service, and extreme limit states per AASHTO LRFD Bridge Design Specifications requirements. Use resistance factors in accordance with the AASHTO LRFD Bridge Design Specifications.

The project engineer must ensure that the retaining wall system is appropriate for its location. Check walls to ensure all potential modes of failure are met at specified limit states. These include sliding, overturning (eccentricity), bearing resistance, and overall (global) stability. Perform external and overall stability check at the Strength limit state using appropriate load combinations in accordance with AASHTO LRFD Bridge Design Specifications 11.6.3. Check vertical and lateral movements of the retaining walls for applicable load combinations at Service limit states. Consult governing wall standard sheets and AASHTO LRFD Bridge Design Specifications for assumptions and appropriate resistance factors for various modes of failure. Use resistance factors per 11.6.3.7 of AASHTO LRFD Bridge Design Specifications. Use a maximum resistance factor of 0.65 for slopes or walls that support abutment, buildings, critical utilities, or for other installations with a low tolerance for failure.

If a TxDOT retaining wall standard is used for the wall design, it is the designer's responsibility to validate the strength values shown on the retaining wall standard used. If the actual soil conditions show a strength weaker than that shown on the governing standard, the designer must determine what modifications and indicate on plan, if any, are necessary to the standard and if any ground improvements are necessary to ensure wall performance.

Avoid setting retaining wall limits and heights such that the ground slope in front of (base of) the wall is sloped greater than 4:1. When walls must be placed on slopes, conduct both short- and long-term stability analyses using appropriate soil strengths, geometry, and loading conditions (live load surcharge, hydrostatic, etc.). Maintain a minimum horizontal bench width in front of walls founded on slope in accordance with RW (MSE) DD standard.

When retaining walls are placed on fill, evaluate the need for additional ground improvement and engineered fill.

Design Criteria for Specific Wall Types

Spread Footing Walls. The engineer specifying this type of wall for inclusion in the plans is responsible for overall (global) stability of the wall. Ensure that the actual wall geometry and loading conditions apply to the standard drawing selected. Ensure that interruptions to the stem or footing steel by utilities or curved sections of walls do not compromise the design and performance of the wall. Ensure that skewed abutment ends do not pose conflicts with the footprint of the wall. Provide guidance or structural details when deviations from the wall standard drawings are warranted. Standard drawings provide a choice between slope and no slope above wall or surcharge load footings: selection of the appropriate standard drawing is a function of the loading, geometry, and site condition Standard drawings are developed based on the design parameters for foundation and retained soils of a cohesion of zero, a friction angle of 30 degrees for the retained and foundation soil, and a unit weight of 120 pounds per cubic foot for both. Give special consideration to walls subject to inundation. Considerations include drainage and draw-down stability analysis. Standard specification Item 423 governs the design and construction of this wall type.

Provide expansion joints at intervals not exceeding 96 feet and contraction joints at intervals not exceeding 32 feet.

MSE Walls. The engineer specifying this type of wall for inclusion in the plans is responsible for overall (global) stability and for providing information to complete the RW (MSE) DD sheet. MSE wall should be avoided in zones of potential scour or erosion. MSE wall suppliers are responsible for internal stability of the walls and for ensuring that external stability, as defined on the RW (MSE) standard, is met. The friction angle of both the foundation soil and the retained soil must be defined by the wall designer and input on the TxDOT RW(MSE) DD sheet. Default minimum earth reinforcement is set at 8 ft. or 70 percent of the wall height, whichever is greater. The wall designer is responsible for ensuring that the minimum earth reinforcement length selected on the RW(MSE)DD sheet satisfies the resistance factor requirement with the defined friction angle of the foundation soil and the retained soil. To ensure proper performance of the wall in place, evaluate project-specific requirements for wall backfill type, wall embedment, wall drainage, conflicts within the wall reinforced zone, and other considerations as necessary. Give special consideration to walls that are subject to inundation. Type BS backfill is the default backfill for permanent walls. Type DS backfill must be specified for walls that are subject to inundation. Analyze walls subject to inundation for 3 ft. of draw-down. Refer to the RW(MSE)DD standard for guidance on the draw down design condition. Walls to be placed in front of bridge abutments should have a 2 -ft. minimum and 3-ft. desirable clearance from back of wall panel to face of abutment cap to facilitate wall construction. Standard specification Item 423 governs the design and construction of this wall type.

Evaluate MSE walls for total and differential settlement for all applicable dead and live load combinations at Service I limit states in accordance with AASHTO LRFD Bridge Design Specifications. Total settlement should be less than 4 inches unless approved by the TxDOT State Geotechnical Engineer. Limit differential settlement as defined in the AASHTO LRFD Bridge Design Specifications, C11.10.4.1-1. Slip joints may be required to limit effects of differential settlement.

Temporary MSE or Welded Wire Face Wall. Temporary Walls have a service life no longer than 3 years. The engineer who selects this type of wall for inclusion in the plans is responsible for the overall (global) stability of the wall and for providing information to complete the RW (TEW) DD sheet. Temporary MSE wall suppliers are responsible for internal stability of the walls and for ensuring that external stability, as defined on the RW (TEW) standard, is met.

If the site condition soil properties differ from those indicated above, then the RW(TEW) standard will need to be modified to reflect the actual site soil properties.

Set the minimum earth reinforcement length to 6 ft. To ensure proper performance of the wall in place, evaluate project-specific requirements for wall backfill type, wall embedment, wall drainage, conflicts within the wall reinforced zone, and other considerations as necessary. Give special consideration to walls that are subject to inundation. Type C backfill is the default backfill for temporary walls. Specify Type D backfill for walls that are subject to inundation. Analyze walls subject to inundation for 3 ft. of draw-down. Backfill the 2-ft. zone immediately behind the facing with clean coarse rock or cement-stabilized backfill. A designer who prefers to use coarse rock or cement-stabilized backfill must state this in the plan documents.

If a temporary MSE wall will be in service for longer than 3 years, the designer must state this in the plan documents to ensure that the wall supplier provides a design with an adequate service life. Temporary MSE walls placed adjacent to permanent MSE walls must be detailed with earth reinforcement that will prevent corrosion of the permanent earth reinforcements due to contact of dissimilar metals. This may be accomplished by providing galvanized or synthetic earth reinforcements for the temporary MSE walls.

Standard specification Items 403 and 423 govern construction of this wall type.

Concrete Block Walls. The engineer who selects this type of wall for inclusion in the plans is responsible for overall (global) stability of the wall and providing information to complete the RW(CB)DD sheet. Concrete block wall suppliers are responsible for internal stability of the walls and for ensuring that external stability, as defined on the RW (CB) standard, is met.

If the site condition soil properties differ from those indicated above then the RW(CB) standard needs to be modified to reflect the actual site soil properties.

Concrete block walls may be classified as either structural or landscape walls. The minimum strap length varies depending on the wall function. Minimum earth reinforcement lengths are 6-ft. for walls designated as landscape walls, and 8-ft. otherwise. To ensure proper performance of the wall in place, evaluate project-specific requirements for wall backfill type, wall embedment, wall drainage, conflicts within the wall reinforced zone, and other considerations as necessary. Type BS backfill is the default for permanent walls. Give special consideration to walls that are subject to inundation. Specify Type DS backfill and analyze these walls for 3 ft. of draw-down. The maximum particle size of the select backfill is limited to ¾" for nonmetallic reinforcements. Consult the RW(CB) and RW(CB)DD standard drawing for guidance on wall definition and design. Standard specification Item 423 governs the design and construction of this wall type.

Tied-Back Walls. The prestressed ground anchors (tie backs) are nearly horizontal elements that are drilled, grouted, and stressed in place. Most common anchored walls are anchored sheet pile walls and soldier pile walls. Determine tied-back loads and soldier pile bending moments from the apparent earth pressure diagrams. Fill and live load surcharges are included in the pressure diagram. Determine loads and moments by the tributary area method. The minimum tie-back length is 25 ft. This length is composed of a minimum 15-ft. debonded length and a minimum 10-ft. bonded length. Minimum tie-back length as specified AASHTO LRFD Bridge Design Specifications Article 11.9 is determined by EOR in the contract plans, yet final length of the tie-back is determined by the wall contractor. Anchor loads and soil conditions may warrant tied-back anchors on the order of 60 to 70 ft. long. The anchors are then stressed to the load specified in the construction drawings. Consider the distance the tie backs will project behind the wall and any potential conflicts with subsurface obstructions or right of way limitation. Ensure that tie backs have a minimum 6-in. clear cover from any obstructions. Obtain permanent easements for tie backs that cross the right-of-way line. Consider equipment accessibility due to horizontal and vertical clearance restrictions. Standard specification Item 423 governs the construction of this wall type and is supported by the special specification Prestressed Ground Anchors.

Soil Nailed Walls. Soil nails are nearly horizontal elements that are drilled and grouted in place. Walls are typically designed using limit state equilibrium software programs such as Goldnail, SNAP-2, Slide2, SnailPlus or SNAILZ. Design in accordance with AASHTO LRFD Bridge Design Specifications Article 11.12. Consider the distance the nails will project behind the wall and any potential conflicts with subsurface obstruction or right of way limitation.

Evaluate soil corrosion for permanent walls per AASHTO LRFD Bridge Design Specifications Article 11.12.8, use the following minimum criteria:

- Hole diameter — 6 in.
- Bar size — #6
- Grade — 75 ksi for permanent walls
- Bars — epoxy-coated or galvanized, Dywidag or Williams threadbar, or equivalent

Standard specification Item 423 Retaining Walls and Item 410 Soil Nail Anchor govern construction of this wall type.

Ensure that nails have a minimum 6-in. clearance from any obstructions. Obtain permanent easements for nails that cross the right-of-way line. The top of the wall should be no more than 2 ft. above existing grade to ensure constructability of the soil nail wall; special design considerations are required when this distance is exceeded. Nail spacing depends on project-specific site and loading conditions. A 3-ft. to 4.5-ft. vertical spacing and a 3.0-ft. to 4.5-ft. horizontal spacing is typical. Soil strengths used in the design of soil nail walls are typically determined from correlations of strength to Standard Penetration Test values conducted through the embankment to be nailed. Use nominal strengths in the analysis. Design walls considering the proposed wall geometry and loading. Limit head strength to avoid an unbalanced design. Unrealistic or high head strength results in shorter nails and causes the lowest nails to carry a disproportionate amount of load.Final verification on design should include a global (overall) check using the analysis mode of the design program used or an independent slope-stability program that is capable of modeling soil nail anchors. Consider equipment accessibility due to horizontal and vertical clearance restrictions.

Rock Nailed Walls. Rock nail walls are used in materials classified as rock and have SPT values that meet refusal criteria. Confirm that site conditions are conducive for rock nails. Rock Nailed Wall design is based on empirical equation and should consider the dip, bedding thickness, Rock Quality Designator, percent recovery, joint spacing, and joint pattern of the rock formation. Smaller holes than those used in soil nail walls, but with a diameter not less than 4 inches, are appropriate for rock nailed walls. Adjust nail lengths to ensure that the nailed rock mas is inherently stable in the primary modes of failure (sliding and overturning). Standard specification

Item 423 Retaining Walls and Item 411 Rock Nail Anchors govern construction of rock nail wall type.

Consider the distance the rock nails will project behind the wall and any potential conflicts with subsurface obstructions or right of way limitations. Ensure that nails have a minimum 6-in. clear cover from any obstructions. Obtain permanent easements for nails that cross the right-of-way line. Locate the top of wall no more than 2 ft. above existing grade to ensure constructability of the rock nail wall; special design considerations are required when this distance is exceeded. Consider equipment accessibility due to horizontal and vertical clearance restrictions.

Drilled Shaft Walls. Drilled shafts are vertical elements that are drilled and concreted in place. They vary in size, diameter, and spacing depending on soil conditions, loading, and wall geometry. Derive wall loading using a Coulomb analysis. Soil information necessary for design includes friction angle, cohesion, and unit weight. Determine soil strengths below the proposed ground line at face of wall from correlations of strength to Standard Penetration Test values. Use nominal strengths in the analysis. The following soil strength reductions can be used in design:

- Reduction based on close shaft spacing (per AASHTO LRFD Article 10.8.3.6), refer to reduction within the following Figure 6-1.
- Reduction of surface soil strength based on expected swelling/softening of the soil

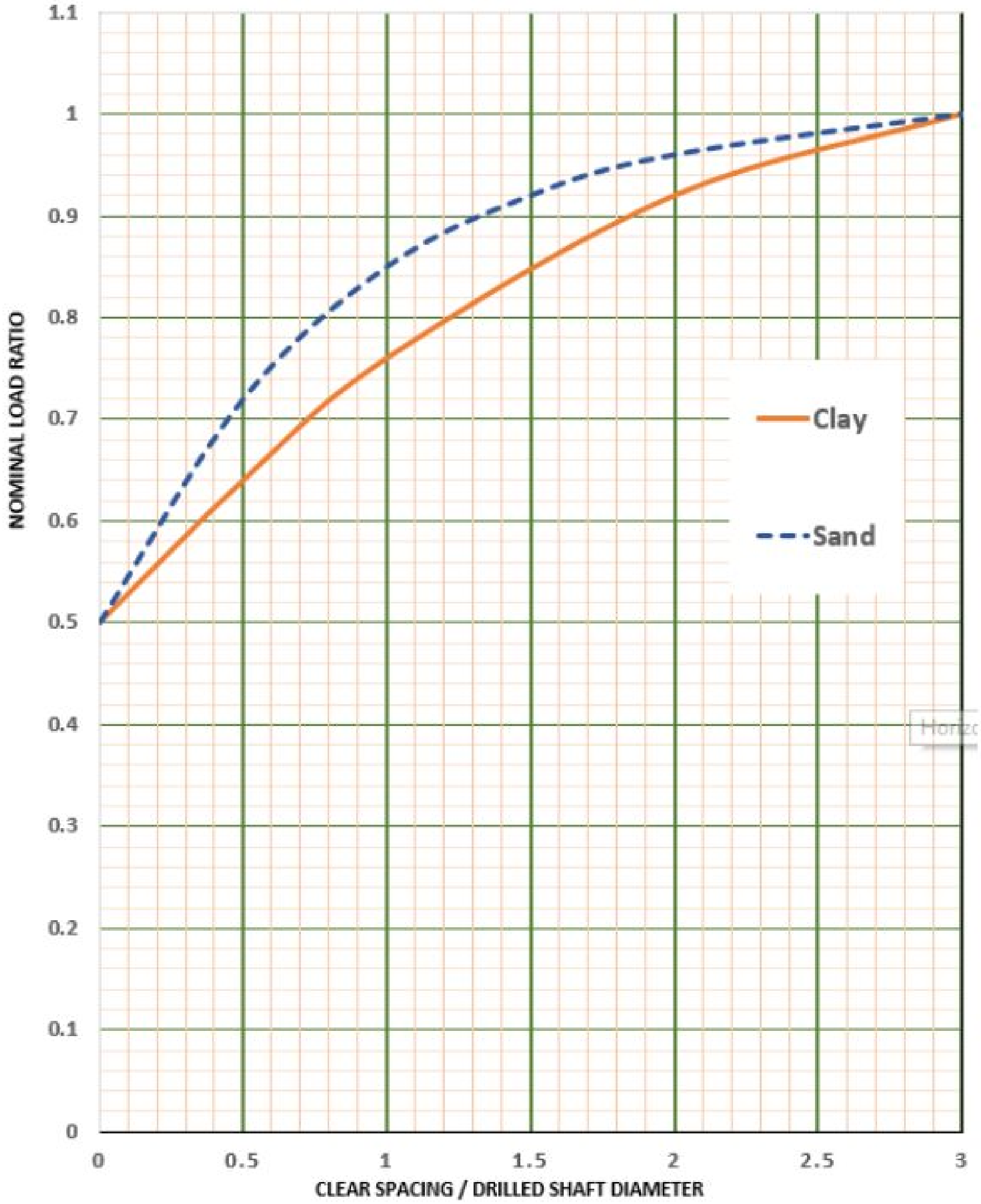


Figure 6–1. Nominal Load Ratio vs. Clear Spacing/Drilled Shaft Diameter

Design the walls iteratively, varying the length of shaft for successive runs. Make a plot of shaft embedment versus top of shaft deflection to determine when additional embedment does not result in a reduced deflection. The minimum embedment length that results in no additional top of shaft deflection is defined as the depth to fixity. An acceptable approach is to terminate the shaft at a depth 33% longer than minimum embedded depth to fixity. Maximum tolerable top of shaft deflection is set at 1% of the exposed wall height. The maximum steel reinforcement within concrete is 2.5% to 3% as limited by reinforcing spacing requirements. Minimum clear spacing between adjacent shafts is set at 1 ft. Design wall fascia to account for the maximum earth pressure at the bottom of the wall. The load applied to the fascia shall be applied through the window between the shafts assuming simple supports at the centerline of the shafts. The Contractor is responsible to ensure that face stability is maintained between shafts throughout construction. Address this by a note in the plans. Consider equipment accessibility due to horizontal and vertical clearance restrictions. Standard specification Item 416 Drilled Shafts and Item 423 Retaining Walls govern construction of this wall type and are supported by special specification Prefabricated Soil Drainage Mat.

Sheet Piles and Soldier Pile Walls. Sheet piles and soldier piles provide lateral resistance through the flexural resistance of structural members through cantilevering and embedment into founding soil. In most conditions, these walls can accommodate an exposed height to a maximum of 15 feet. Exposed height usually depends on the acceptable limit of deflection at the top of the wall. Walls taller than this or with exceeding deflection limits require the addition of anchors in the form of a deadman or tieback. Design in accordance with AASHTO LRFD Bridge Design Specifications Article 11.8 non-gravity cantilevered walls.

Design Criteria for Reinforced Soil Slope

Reinforced Soil Slopes (RSS). The specifying engineer is responsible for the overall stability of slopes as indicated in Chapter 7. Provide reinforcement when the resistance factor (1/factor of safety) for the unreinforced slope is less than the required value. RSS is the method of stabilizing existing slope using internally stabilized fill slopes constructed with alternate layers of compacted soil and extensible reinforcement. If using RSS, the uphill slope may not be steeper than 1.5H:1V. Place reinforcing layers at a vertical distance of 3 feet or less. Ensure there is enough space and no interruptions for laying reinforcement. .Determine reinforcement length based on the evaluation of full range of potential failure surface, including deep seated failure surface. Soil strengths used in the design are determined from soil borings and correlation with Standard Penetration Test values. Typically, RSS requires smaller equipment as compared to retaining wall systems, however temporary special shoring may be required depending on the slope and site condition. Consider benching during excavation and placement of fill materials. Type of materials used in backfill has an important role on the stability of the RSS.

Section 4: Excavation Support

Overview

The Occupational and Safety Health Administration (OSHA) defines an excavation as any man-made cut, cavity, trench, or depression in the Earth's surface by earth removal. A protection system for an excavation includes support systems, sloping and benching systems, shield systems, and other systems that provide protection. The two main types of excavation protection are trench excavation protection (see standard specification Item 402) and temporary special shoring (see standard specification Item 403). The contract plan should show all locations of excavation support and provide note and sketch of estimated length.

For either protection system, the Contractor must be compensated for the method of choice. For example, for temporary special shoring when excavation techniques such as sloped cuts or benching are used to provide the necessary protection, the surface area of payment is calculated based on the area described by a vertical plane adjacent to the structure.

Trench Excavation Protection

A trench is defined as a narrow excavation (in relation to its length) made below the surface of the ground. In general, the depth of a trench is greater than its width, but the width of a trench (measured at the bottom) is not greater than 15 feet. Trench excavation protection is used for the installation of linear drainage or electrical features that will result in trenches deeper than 5 ft. It provides vertical or sloped cuts, benches, shields, support systems, or other systems providing the necessary protection in accordance with OSHA Standards and Interpretations, 29 CFR 1926, Subpart P, Excavations.

Temporary Special Shoring

Temporary special shoring is used for installations of walls, footings, and other structures that require excavations deeper than 5 ft. Temporary special shoring is designed and constructed to hold the surrounding earth, water, or both out of a work area. In general, typically used wall systems for temporary special shoring are: sheet piles, soldier pile walls, soil nails, and temporary MSE walls (Temporary MSE or Welded Wire Face Wall as specified in Chapter 6, Section 3). It provides vertical or sloped cuts, benches, shields, support systems, or other systems to provide the necessary protection in accordance with the approved design. Unless complete details are included in the plans, the Contractor is responsible for the design of the temporary special shoring. The Contractor must submit details and design calculations bearing the seal of a licensed professional engineer for approval before constructing the shoring. The design of the shoring must comply with OSHA Standards and Interpretations, 29 CFR 1926, Subpart P, Excavations. Design structural systems to comply with AASHTO LRFD Bridge Design Specifications. Design shoring subject to railroad loading to comply with railroad Guidelines for Temporary Shoring and any additional requirements of the railway being supported.

Standard specification Item 403 is for cut shoring. When temporary MSE walls are used for fill situations, construct these walls in accordance with the requirements of standard specification Item 423, Retaining Walls, and include the standard sheet RW(TEW) as well as RW (TEW) DD sheet.

Consider temporary shoring concurrently with the permanent wall layout and design or grade change requirements of any given project. The best wall design or project geometry is difficult to execute and may put both workers and the traveling public at risk if proper shoring requirements are not addressed. In extreme cases, the cost of temporary shoring required to construct a wall can exceed the cost of the permanent wall. Avoid this and reduce negative effects with proper planning and proper wall selection.

Design temporary shoring like a permanent retaining wall. Determine the proper design loading that will act on the shoring wall. Consider the effect of surcharges or slopes behind the shoring wall. Due to the impermeable nature of some shoring types such as sheet piling, consider water pressure or additional drainage details in design. In the case of temporary shoring walls: some increase in prescribed resistance factors may be acceptable depending on the site condition and the availability of subsurface soil data.

Consider temporary shoring for the following conditions:

- At the back of fill-type retaining structures in cut situations
- In front of existing structures such as retaining walls, bridge supports, header banks
- On projects with staged construction
- Near railroads
- For bridge footings

Chapter 7: Slope Stability

Contents:

Section 1: Overview
Section 2: Analysis and Design

Section 1: Overview

Overview

Evaluate all slopes, whether a cut or a fill and whether in soil or in rock, for global (overall) stability. Slopes steeper than 3:1 must have a documented evaluation. When warranted, evaluate for both short-term (undrained) and long-term (drained) conditions under Strength I limit state condition per current edition of AASHTO LRFD Bridge Design Specifications. However, the load factors are not compatible with limit equilibrium analysis and resistance factor is yet to be calibrated & implemented in commercially available software, overall stability analysis still be performed under Allowable Stress Design (ASD) methods.

Conditions

Perform slope stability analyses under all applicable conditions using Limit Equilibrium software such as GSTABL, Slide2, etc. At a minimum, evaluate the following conditions:

1) Short-term (undrained) condition,

2) Long-term (drained) condition, and

3) Rapid drawdown (flood) condition.

For embankments and cut slopes consisting of high plastic clay soil, shear strength may degrade due to exposure to weathering action (shrinkage-swelling). Perform long term stability check based on residual shear strength of the soil.

Specific site conditions may require evaluation for additional types of failure, such as bearing capacity, settlement, and undercutting (for rock cuts).

Perform embankment settlement analysis under service limit state to evaluate the performance of the embankment or structure on top of embankment. If deformation will adversely affect the facility, develop ground improvement or mitigation measures. Identify the most appropriate ground improvement method based on project needs and approval from the department. Include ground improvement limits, details, instrumentation and performance requirement in the plan and specification.

Section 2: Analysis and Design

Global Stability Analysis

Use the following data to analyze global stability of a slope:

- Geometry (cross section and loading conditions)
- Groundwater conditions
- Soil/rock stratigraphy
- Soil/rock properties (unit-weight, moisture, Atterberg Limits, undrained and drained shear strength)
- Additional loading conditions (traffic surcharge, railroad live load, etc.)

For global stability of a slope, a minimum factor of safety of 1.3 is required for both the long-term drained condition and the short-term undrained condition. Make the factor of safety 1.5 or greater for slope or walls that support abutment, buildings, critical utilities, or for other installations with a low tolerance for failure.

Experience has shown that most exposed side slope failures begin as shallow slides and then deepen with time. The following table was developed to determine the recommended upper limit on the Plasticity Index for various slope conditions to maintain a factor of safety of 1.3 for the long term or drained soil conditions using an infinite slope analysis accounting for seepage of water parallel to face of slope without the effect of surcharge loading on the surface.

Table 7-1: Plasticity Index Range for Exposed Side Slopes Required for FS =1.3 for the Long Term or Drained Condition

Slope	Plasticity Index (PI) (%)
2.5 H : 1V	< 5
3.0 H : 1V	< 20
3.5 H : 1V	< 35
4.0 H : 1V	< 55
4.5 H : 1V	< 85

Appendices

Contents:

Appendix 1: Legacy Texas Cone Penetration (TCP) Evaluation
Appendix 2: Ancillary Structure Foundations

Appendix 1: Legacy Texas Cone Penetration (TCP) Evaluation

Overview

If required to use Texas Cone Penetration (TCP) data to evaluate capacity of existing bridges, use the charts and correlations from this Appendix to determine skin friction and point-bearing capacity for drilled shafts and piling. Use Figure A1-2 to determine allowable skin friction for soil softer than 100 blows/12 in. Select the curve based on the description of the soil type.

Use the CH curve in clay soil identified as high-plasticity, or fat clay. Use the CL curve in clay soil identified as low-plasticity, or lean clay. In clay soil, use the CL curve if no specific identification is provided regarding plasticity. Use the SC curve for soil described as either sandy clay or clayey sand. Use the OTHER curve for soils described as silt, sand, gravel or any layers not fitting into one of the previous designations.

Use figure A1-1 to correlate TCP test results to angle of internal friction of cohesionless soils.

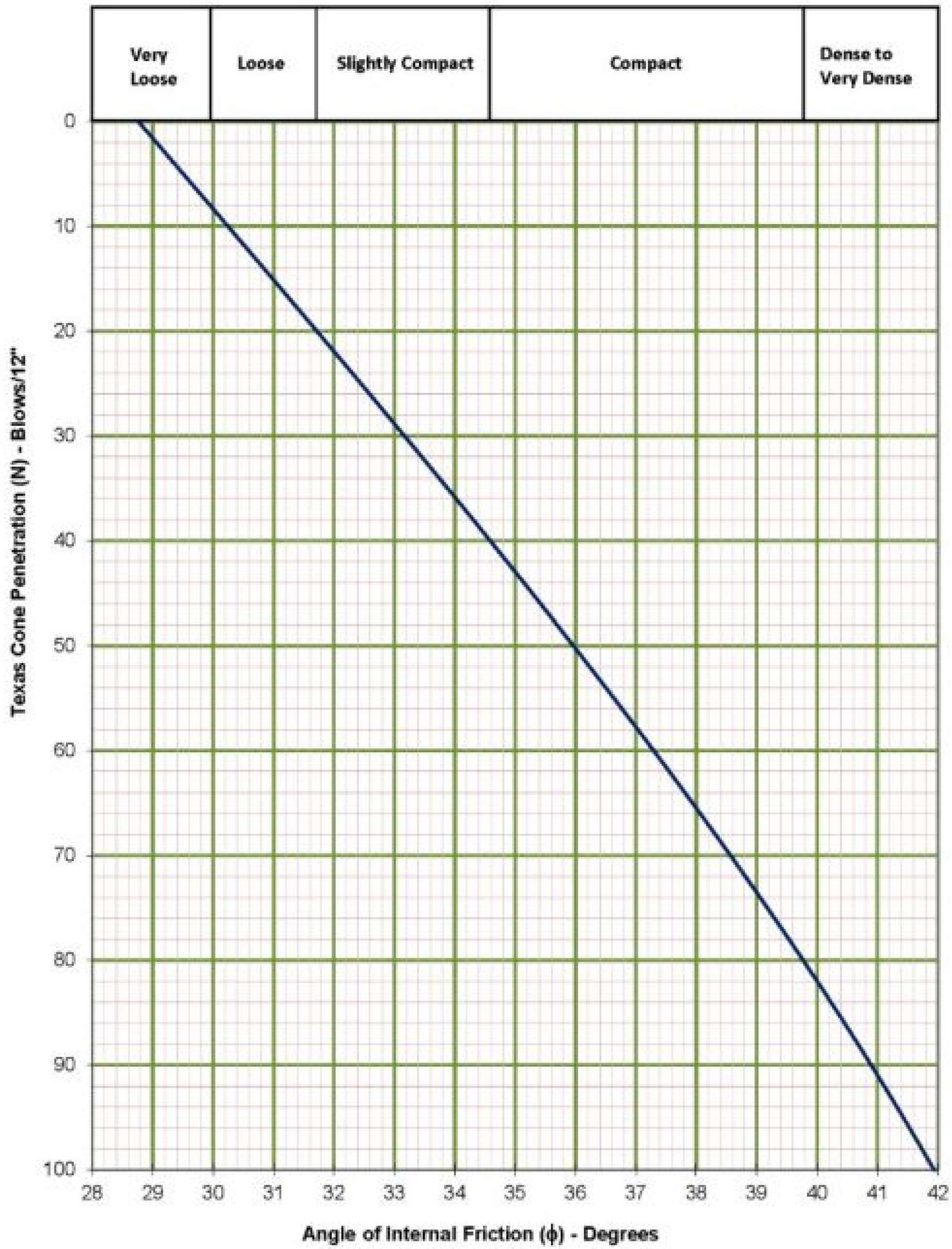


Figure A1–1. Friction Angle Estimates (TCP Values Softer than 100 Blows/12 in.)

Legacy Procedure of Drilled Shafts from TCP

When using TCP information, consider both skin friction and point bearing for drilled shaft capacity. Calculate total allowable skin friction by multiplying the perimeter of the shaft by the unit value for allowable skin friction derived from Figure A1-2, Figure A1-4, or laboratory data. For drilled shafts, apply a reduction factor of 0.7 to allowable skin friction values derived from Figure A1-2 or from laboratory testing to account for disturbance of the soil during drilling. Do not apply the reduction factor to allowable skin friction values obtained from Figure A1-4. Accumulate skin friction along the length of the shaft beginning at the previously defined disregard depth and continuing down to the tip of the shaft. Calculate total allowable point bearing by multiplying the area of the drilled shaft times the unit value for allowable point bearing derived from Figure A1-3, Figure A1-5, or laboratory data. If softer layers exist within two shaft diameters of the proposed tip, use allowable point bearing values for the softer layers. If drilled shafts are to be tipped in very hard material that is overlain by soft strata, the skin friction contribution of the softer strata may be disregarded in design. However, do not ignore the contribution of significant amounts of competent material to tip in rock. In many areas of the state, rock is overlain by thick layers of material that can support considerable loads.

Legacy Procedure of Driven Piles from TCP

When using TCP information for driven piles, designs generally rely solely on skin friction capacity and partial to no capacity is generated by end bearing. <u>Application of the (0.7) reduction factor to the design of driven piling is not necessary</u> as these are displacement piles and do not remove subsurface soils as with drilled shafts.

Calculate total allowable skin friction by multiplying the perimeter of the pile by the unit value for allowable skin friction derived from Figure A1-2, Figure A1-4, or laboratory data or a combination thereof. The maximum recommended value for allowable skin friction for piling is 1.4 tons per square foot (TSF). Accumulate skin friction along the length of the pile beginning at the previously defined disregard depth and continuing down to the tip of the pile. If using point bearing, calculate total allowable point bearing by multiplying the area of the pile times the unit value for allowable point bearing derived from Figure A1-3, Figure A1-5, or laboratory data. If softer layers exist within two diameters of the proposed tip, use allowable point bearing values based on the softer layers.

Displacement piling typically refuses to advance once material with TCP values harder than 100 blows/12 in are encountered.

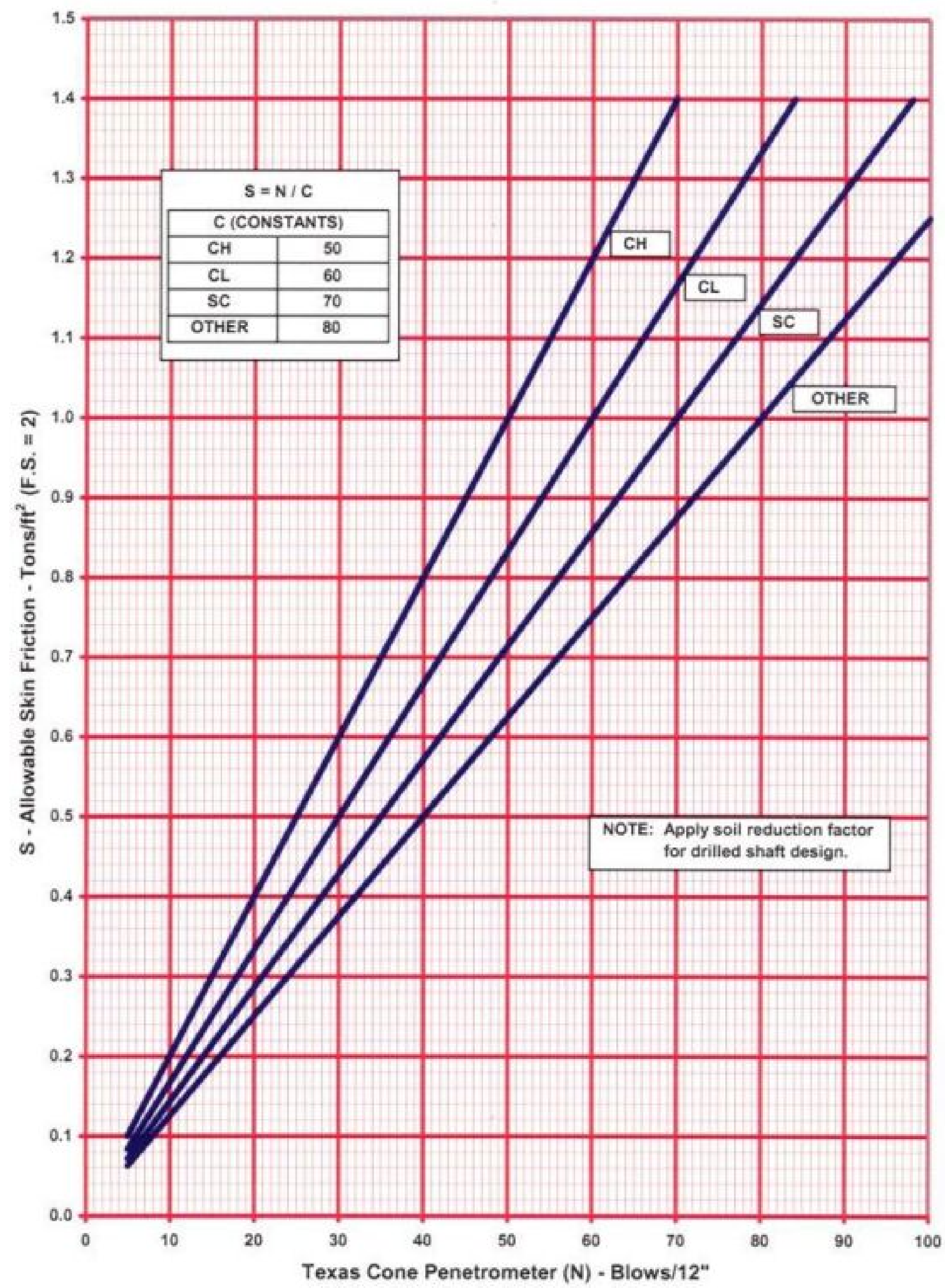


Figure A1-2. Allowable Skin Friction (TCP Values Softer than 100 Blows/12 in.)

Use Figure A1-2 to determine allowable skin friction capacity for soil softer than 100 blows/12 in. Select the curve based on the description of the soil type.

Use Figure A1-3 to determine allowable point bearing for soil softer than 100 blows/12 in. Select the curve based on the description of the soil type, using the criteria noted for the previous chart.

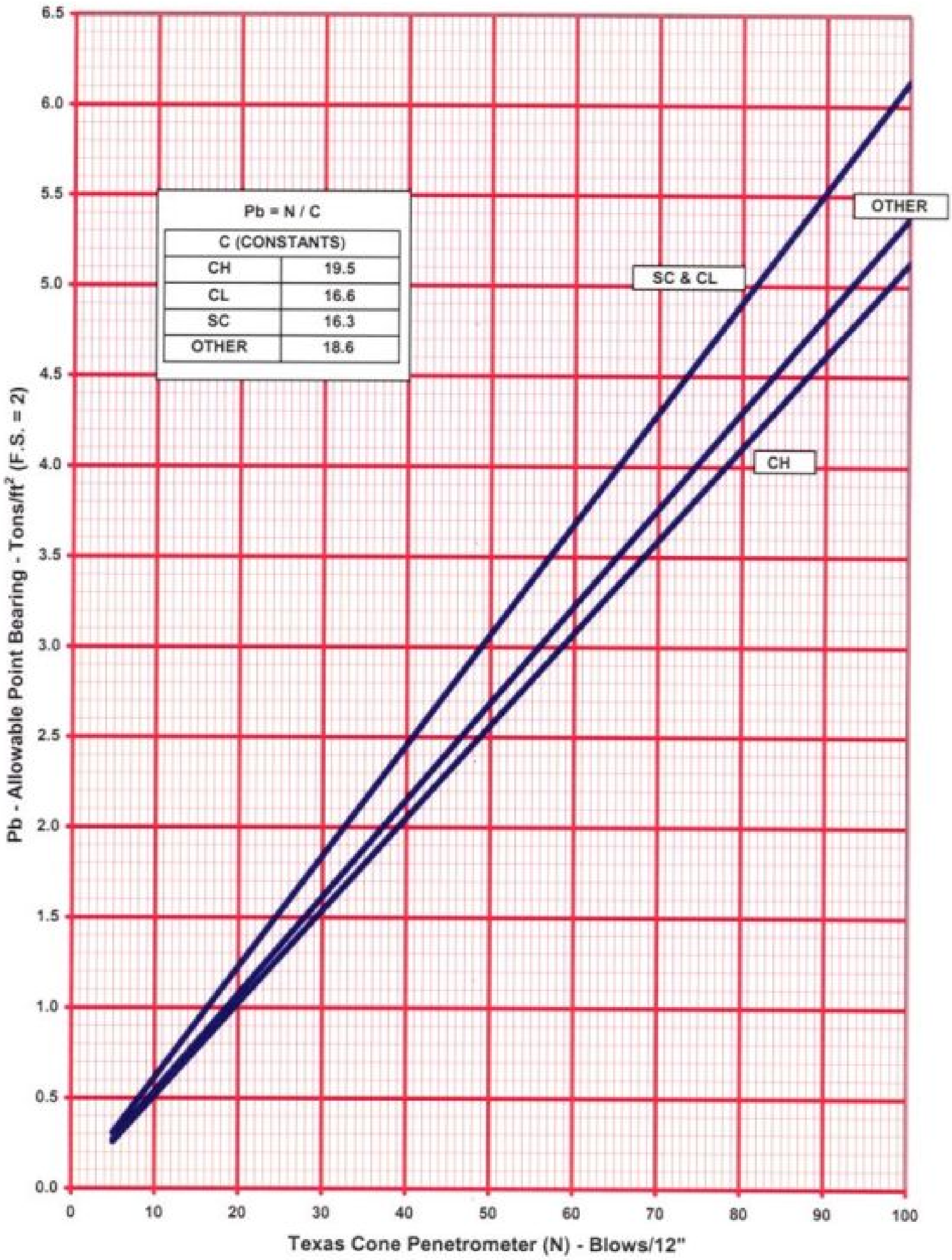


Figure A1-3. Allowable Point Bearing (TCP Values Softer than 100 Blows/12 in.)

Use Figure A1-4 to determine allowable skin friction for soil or rock strata harder than 100 blows/ 12 in. The upper limit of 3.25 tons/ft2 applies for all Texas Cone Penetration values less than 2 in/

100 blows. Do not apply skin friction reduction factor to values obtained from this figure because this figure is derived only for use in drilled shaft design. Piling typically cannot be driven into soil of this strength, so this figure is not generally used for piling.

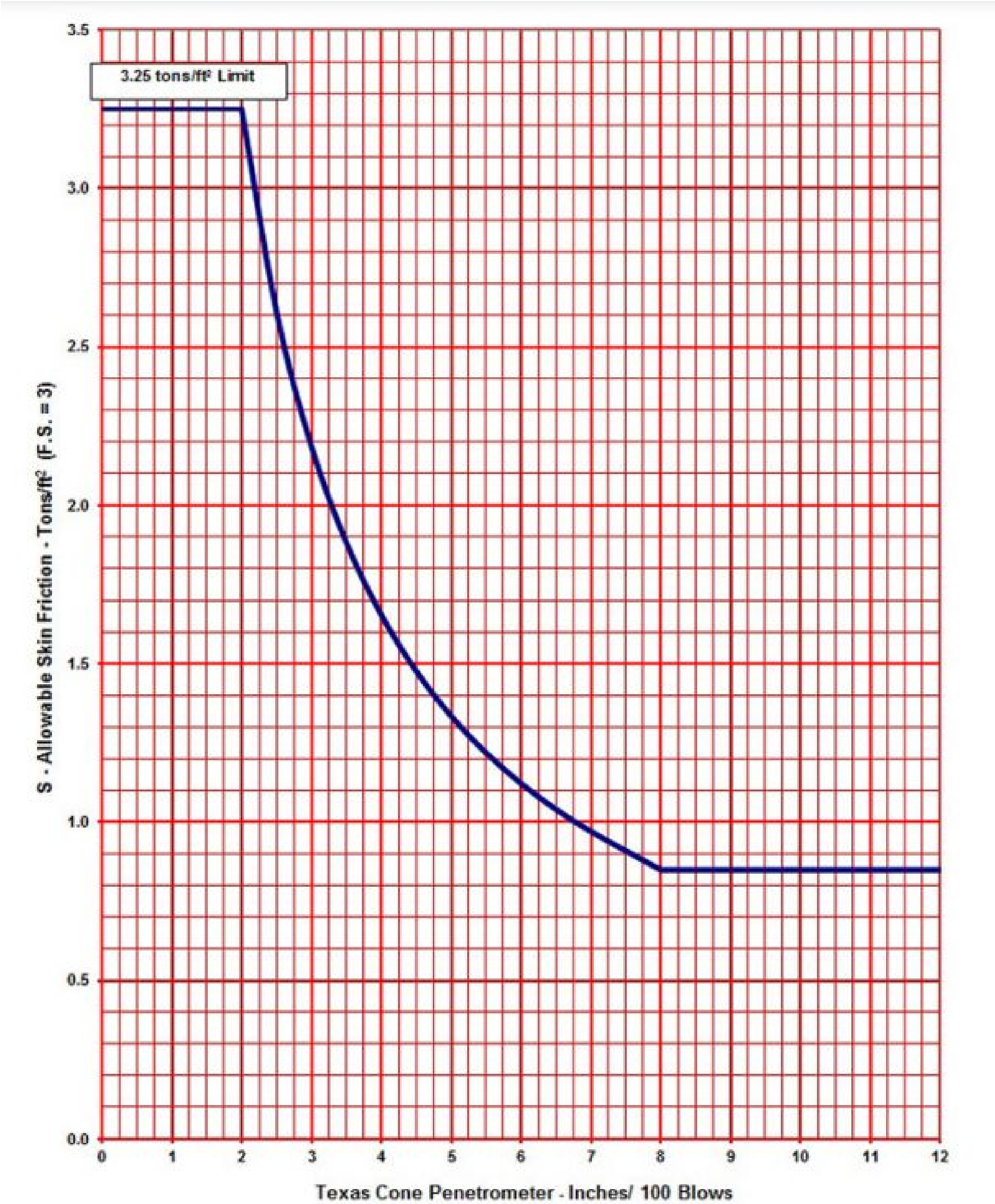


Figure A1-4. Allowable Skin Friction (TCP Values Harder than 100 Blows/12 in.)

Use Figure A1-5 to determine allowable point bearing for soil or rock strata harder than 100 blows/ 12 in. The upper limit of 31 tons/ft2 applies for all Texas Cone Penetration values less than 2 in/ 100 blows.

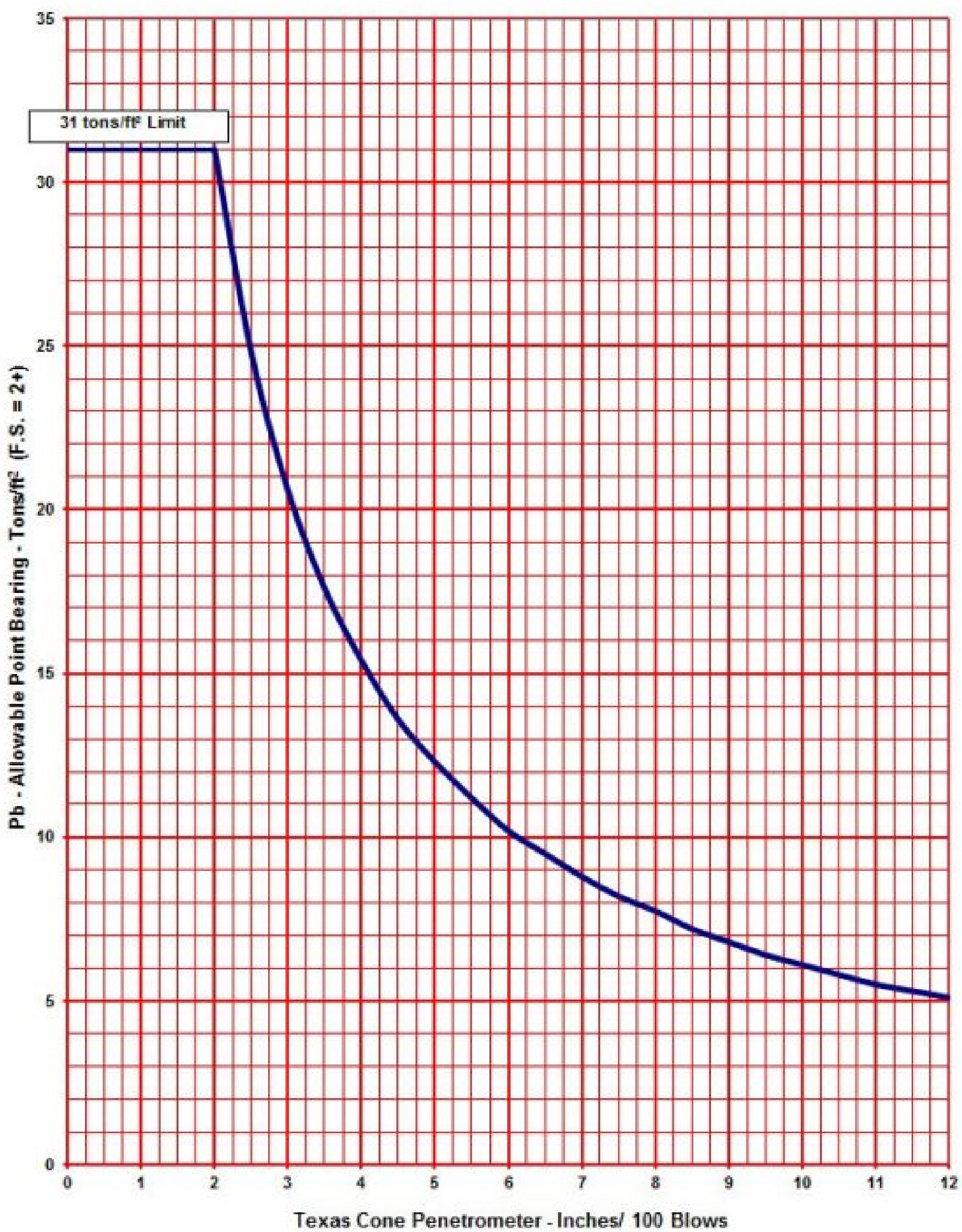


Figure A1-5. Allowable Point Bearing (TCP Values Harder than 100 Blows/12 in.)

TCP Design Verification with Laboratory Test

If additional strength data is available from triaxial or direct shear testing, use this data with TCP results. Determine the ultimate shear strength for each stratum using Coulomb’s formula [Shear Strength = $\tau = c' + \sigma_y' (\tan \phi')$]. Determine allowable skin friction by applying a factor of safety of at least 2.0 to the ultimate shear strength. For drilled shafts, reduce the allowable skin friction value by an additional reduction factor of 0.7 to account for soil disturbance. Determine allowable point bearing by multiplying the ultimate shear strength by a bearing capacity factor of 9 and then dividing by a factor of safety of at least 2.0.

Appendix 2: Ancillary Structure Foundations

When using roadway and traffic standards developed for foundations from TCP information (COSS, High Mast Illumination Poles), use the following correlations (from Touma and Reese, 1972) from SPT values acquired in the drilled boring logs:

In Clay: $N_{TCP} = 1.5 * N_{SPT}$

In Sand: $N_{TCP} = 2.0 * N_{SPT}$

Where, N_{TCP} = equivalent TCP blow counts when using STP information

N_{SPT} = uncorrected blow counts from STP in-situ testing

These correlations apply to the standard foundation embedment selection charts regarding TCP information currently refenced in the standards.