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PART A: GENERAL

Part A of this guideline covers definitions, abbreviations, codes, standards and references, as well general geotechnical requirements and conditions (Sections 1 through 5).

Part B covers geotechnical investigation (Sections 6 through 13).

Part C covers geotechnical design (Sections 14 through 20).

Lists of standards and references are in Appendix A (Section 21).

1.0 INTRODUCTION

The provisions of this Guideline apply to the design professional in charge of the geotechnical investigation and geotechnical design work, in whole, or in part. The term A/E used herein shall be taken to apply to the engineer responsible for developing/performing the geotechnical investigation and performing the geotechnical design work.

This Guideline presents the general state of the practice for geotechnical investigations, and provides a general guideline for developing and performing a geotechnical investigation program, both onshore and offshore. It provides a general guideline for geotechnical work and design, and presents the general state of the practice for geotechnical investigation programs, analyses and geotechnical design required for construction of facilities (buildings, transportation, pipelines, etc.). This Guideline presents methodologies for the interpretation of various investigation methods, and the development of appropriate soil and rock parameters for engineering applications as well as subsequent analysis and design.

All geotechnical engineering work is site specific in nature. It is understood that the procedures discussed in this Guideline are subject to local variations. Therefore, it is important for the A/E Lead Discipline Engineer to be thoroughly familiar with local geotechnical investigation and geotechnical engineering practices. The proper execution of a geotechnical investigation program requires a thorough understanding of subsurface investigation techniques, including geophysical methods and laboratory testing of soil and rock samples. Similarly, the proper execution of a geotechnical engineering scope of work requires a thorough understanding of the principles and practice of geotechnical engineering, including design procedures, construction methods and the planned purpose of the facility, supplemented with a working knowledge of geology and hydrogeology.

All geotechnical works shall be undertaken in accordance with the latest Saudi Building Code (SBC) as published by the Saudi Building Code National Committee. Where a code or standard is not referenced within the SBC, but an alternative code is proposed, then the latest issue, supplements, amendments, errata, etc. of the alternative code shall be used, unless otherwise indicated. Errata shall be reviewed for all codes and standards.

Where there is a difference in methodology or terminology between a code or standard and the SBC, the differing methodologies and terminologies shall be reconciled prior to commencement of the geotechnical investigation and subsequent design, and such reconciliation shall be described in the geotechnical data report, the geotechnical design report, specifications, calculations, and other documents.

The notation and terminology of the SBC shall be utilized in all engineering and construction documents.

The A/E shall maintain in each engineering design office, and have readily available, all building codes and standards utilized in the design of the geotechnical works.

Important Note:
In this guidelines, references and design parameters related to SBC and AASHTO were provided based on SBC (2018) and AASHTO (7th Edition). Users shall verify the data/information against applicable latest edition of SBC and AASHTO.

2.0 DEFINITIONS & ABBREVIATIONS

For a list of definitions and abbreviations refer to Volume 6, Chapter 2 (Definitions and References - EPM-KE0-GL-000011). The following definitions and abbreviations apply to this Geotechnical Guideline:

2.1 Definitions

Definitions	Description
Active Zone	is the upper zone of the soil deposit which is affected by seasonal moisture content variations.
Allowable Foundation Pressure	is the vertical pressure exerted by a foundation on a supporting formation that can be safely tolerated without causing detrimental settlement or bearing failure.
Allowable Lateral Pressure	is the lateral pressure on a soil that can be safely tolerated without causing either shear failure or detrimental lateral movement.
Backfill	is soil filling a trench or an excavation under or around a building.
Borehole	is a hole drilled as part of a geotechnical investigation to collect soil and/or rock samples.
Caisson	is a large diameter pier or drilled shaft.
Collapsible Soil	is soil susceptible to large and sudden reduction in volume upon wetting.
Combined Footing	is a footing supporting a line(s) of two or more columns.
Compaction	is the process of increasing the density and hence the shear strength and permeability characteristics of a soil by the use of mechanical means such as rolling or vibration.
Concrete-Filled Steel Pipe and Tube Piles	are constructed by driving a steel pipe or tube section into the soil and filling the pipe or tube section with concrete. The steel pipe or tube section is left in place during and after pouring the concrete.
Contact Pressure	or soil pressure is the pressure acting on and perpendicular to the contact area between a footing and the soil, produced by the weight of the footing and all forces acting on it.
Continuous or Strip Footing	is a footing that is long compared to its width and that is usually used to support continuous walls, e.g., masonry walls and or columns.
Deep Foundation	is a foundation that transfers some or all of the applied loads to soil or rock well below the ground surface. Deep foundations typically extend to depths on the order of 10 m or greater.
Driven Uncased Pile	is constructed by driving a steel shell into the soil and then filling it with concrete. The steel casing is lifted out of the hole while the concrete is being poured.
Enlarged Base Pile	is a cast-in-place concrete pile constructed with a base that is larger than the diameter of the remainder of the pile. The enlarged base is designed to increase the load-bearing area of the pile and hence increase its end bearing.
Expansive Soil	has the potential for swelling or shrinking under changing moisture conditions.
Factor of Safety	applied to bearing capacity is the ultimate bearing capacity divided by the design capacity.
Foundation	is the part of a structure that supports the weight of the structure and transmits the loads to the underlying soil or rock.
Karst Formation	(sometimes known as Karstic Limestone) is a geologic setting where cavities develop in beds of limestone, dolomite, gypsum, etc. by solution in flowing water.
Lateral Sliding Resistance	is the resistance of structural walls or foundations to lateral sliding, and is a function of the interface friction and vertical loading.
Limit State	is a condition beyond which a structure or member becomes unfit for service and is judged to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).
Mat Foundation	is a large reinforced concrete slab supporting a piece of heavy equipment or an array of multiple rows of columns.

Definitions	Description
Pier Foundation	is an isolated cast-in-place concrete structural element transferring loads to firm strata below. The length of a pier is generally less than or equal to 12 times its least lateral dimension. See “Caisson”.
Pile Foundation	is a foundation element consisting of concrete or structural steel that transmits structural loads through weak or loose strata to stiffer underlying soil or rock strata. The length of a pile typically exceeds 12 times its least lateral dimension.
Resistance Factor	is a factor that accounts for deviations of the actual soil or rock strength from its nominal strength, and takes into account the manner and consequences of failure (also called “strength reduction factor”).
Sabkha	is an Arabic term for coastal and inland saline flats built up by deposition of silt, clay, and muddy sand in shallow, albeit sometimes extensive, depressions. Sabkha deposits are usually saturated with brine and often are barren and salt encrusted.
Settlement	is the downward movement of the foundation of a structure or part of a structure due to the applied loading causing compression of the soil.
Shallow Foundation	is one which has a depth typically less than its width, and is not supported by deep foundations.
Slope	is an inclined surface of the earth (manmade or natural).
Spread Footing	is a footing supporting single or isolated column loads.
Underpinning	is the process of adding to or replacing the foundations of an existing structure by using piles, piers, walls, sheet piling, or other supports to preserve the integrity of the structure or to increase its load carrying capacity.
Water Table	is the subsurface elevation or depth at which water is present.

2.2 Abbreviations

Abbreviations	Description
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
API	American Petroleum Institute
ARAMCO	Arabian-American Oil Company, officially Saudi Arabian Oil Company
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
ASTM	ASTM International (formerly known as American Society for Testing and Materials)
BM	Benchmark
CALTRANS	California Department of Transportation
CFA	Continuous Flight Auger
CPT	Cone Penetration Test
DM	NAVFAC Design Manual
EM	USACE Engineering Manual
FHWA	Federal Highway Administration
GDR	Geotechnical Design Report
IBC	International Building Code
LRFD	Load and Resistance Factor Design
NAVFAC	Naval Facilities Engineering Command
NCMA	National Concrete Masonry Association
NHCRP	National Highway Cooperative Research Program
NHI	National Highway Institute
PGA	Peak Ground Acceleration
RMR	Rock Mass Rating
RQD	Rock Quality Designation
SAR	Saudi Rail Company
SBC	Saudi Building Code
SEAPA	Seaports Authority of Saudi Arabia

Abbreviations	Description
SEC	Saudi Electricity Company
SPT	Standard Penetration Test
STC	Saudi Telecom Company
SWCC	Saline Water Conversion Corporation
TBM	Temporary Benchmark
TBM	Tunnel Boring Machine
USACE	U.S. Army Corps of Engineers
USCS	Unified Soil Classification System

3.0 CODES, STANDARDS & REFERENCES

Refer to Volume-6, Chapter 5 (Codes, Standards and Reference)on the list of Codes and Standards and the method of selection. In addition to the above mentioned, the following Codes and Standards apply.

3.1 Codes

- Saudi Building Code: Structural Design
- Saudi Building Code: Testing and Structural Investigation
- Saudi Building Code: Soil, Foundations and Supporting Walls
- American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications
- International Building Code (IBC)

3.2 Design Standards

- All work shall conform to the applicable industry Codes, Standards and Associations.
- The latest revision of the referenced Standards shall be used whenever applicable. In case of conflict, the A/E Lead Discipline Engineer shall propose equipment or methods conforming to one group of Standards.
- The ASTM Standards referenced in this Guideline are listed in Appendix A.

3.3 References

The references used in this Guideline are listed in Appendix A.

3.4 Third Party Standards

All public utilities, power, and communication companies, and any other organization working within the jurisdiction of the Entity shall meet the requirements of this Guideline and all referenced documents herein.

For projects including tunneling (microtunneling and other trenchless methods), port and marine works, overhead electrical distribution networks, potable water and wastewater distribution networks, etc., the requirements of ARAMCO, SAR, SEAPA, SEC, STC and SWCC shall be met.

- For non-building facilities such as bridges, pipeline crossings, tower foundations, etc., the requirements of AASHTO, FHWA, and API shall be consulted in the development of investigation proposals which shall be subject for review and acceptance by the Entity.
- Where there is a conflict between the Entity and public utility/private organization requirements, the more stringent requirements shall govern, as determined by the Entity.

4.0 GEOTECHNICAL REQUIREMENTS

4.1 General Requirements

Geotechnical investigation and design work in the Kingdom of Saudi Arabia shall include an assessment of the subsurface conditions at the project site, including surficial geology and the active zone. The assessment shall include an evaluation of these key geotechnical issues and their potential impact on the proposed development. For the minimum scope of a geotechnical site investigation, including the compliance requirements for geotechnical engineering reports in Saudi Arabia, refer to SBC (Soil, Foundations and Supporting Walls). Further details on geotechnical boring requirements are included in Section 6 of this document. The geotechnical investigation shall take due account of the particular site and ground conditions, notably where problematic soils are encountered in a given project. Reference shall be made to the following sections of SBC dealing with:

- Design for Expansive Soils
- Design for Collapsible Soils
- Design for Sabkha Soils.

4.2 Geotechnical Design Report

A written geotechnical design report (GDR) shall be submitted to the Entity, providing an interpretation of the factual data from the geotechnical data report (Section 13) and shall include, but need not be limited to, provision of the following information, or as otherwise defined in the scope of works.

- Project description.
- A description of the site and its surroundings, including a summary of the site geology from published maps and data.
- A description of the proposed construction works, including loads and actions due to ground conditions, groundwater, structural loads, seismic conditions, etc.
- A summary description of the field and laboratory testing program, including previously completed investigations (detailed description given in the geotechnical data report, Part B).
- Plots showing the locations of all test borings and other field exploration data points (test pits, etc.).
- Descriptions of the soil and rock strata encountered.
- Evaluation of the water table and provision of design groundwater levels, if groundwater is encountered.
- Representative subsurface profiles through the major structures (or along the alignment if a linear project) showing the existing grade, final grade, soil and rock strata, boring locations, measured water table, etc.
- Evaluation of the field and laboratory test results and other supporting documents (including limitations of the data).
- Evaluation of the seismic design parameters.
- Evaluation of soil and water chemistry for foundation and buried utility protection measures.
- Derivation of design properties for each soil and rock stratum, including tabulation and graphical plots of data.
- Assessment of geotechnical design parameters, including use of correlations and their applicability, in accordance with the adopted codes and standards.
- Provision of a design ground model, including justification for adopting stated design values in accordance with the adopted codes and standards.
- Reference to codes and standards used.
- Design recommendations, including assessment of the suitability of the design with respect to the proposed form of construction, cost optimization, and required maintenance and monitoring. Design recommendations shall be provided for the topics covered in Part C of this Guideline (where applicable) - foundations (shallow and deep), retaining walls, soil improvement, seepage, slope stability, earthworks,

and deep excavation and tunneling. Part C contains detailed descriptions of the recommendations to be included in the GDR.

- Geotechnical design calculations (if inclusion is specified in the contract), including soft and hard copies of input and output files from approved spreadsheets and software.
- Details for the development of drawings and specifications, including, where required, items to be checked, site validation proposals, frequency and types of checks and testing, responsible parties, and acceptable limits and durations.
- Description of geotechnical monitoring requirements during construction, including pile installation and load testing (where applicable), earthwork, and continued water level monitoring.

4.3 Computer Software and Spreadsheets

For computer software that has built-in algorithms to implement the provisions of a particular version of the building code, an analysis of the methodology used in the software shall be performed to ensure consistency with the methodology of the selected building code cited in the software. The methodologies adopted in the software shall be reconciled with the selected geotechnical and building code(s) either through the use of appropriate modification to the design parameters, method of analysis, or both. All software shall be subject to validation testing and shall be approved by the Entity.

Where in-house spreadsheets are used for analysis and design they shall be subject to validation by supporting hand calculations and/or commercially available software. Spreadsheets provided to the Entity shall only be used for design and analysis following validation and approval by the Entity.

For recommendations refer List of Recommended Software (EPM-KE0-RG-000009).

5.0 GEOLOGICAL & GEOTECHNICAL CONDITIONS IN SAUDI ARABIA

5.1 Geological Setting

The geological features of the Arabian Peninsula have been detailed by Powers et al. (1966), among others. Saudi Arabia is divided into two main geological zones (Powers et al, 1966; Al-Refeai and Al-Ghamdy, 1994):

- A Precambrian complex of igneous and metamorphic rocks (known as the Arabian Shield) occupying roughly one-third of the Arabian Peninsula in the west.
- A broad expanse of relatively low-relief terrain (known as the Arabian Shelf), in which Tertiary and younger deposits overlie older units, in the eastern two-thirds.

The geology of Saudi Arabia may be divided into the following geographic areas as summarized by Al-Refeai and Al-Ghamdy (1994):

- The western region is mostly mountainous, with low-lying coastal areas along the Red Sea.
- The southern area is covered with extensive sand deposits.
- The central areas consist of the relatively flat terrain of the Central Plateau, with silt, sand, and gravel deposits, and limestone bedrock of Quaternary and Tertiary origin along erosion channels and wadis between escarpments.
- The northern areas form the Nafud sedimentary basin, with largely Quaternary deposits of red to pink and grey marly to calcareous sandstone, sandy marl with pinkish sandy shale, and clayey silty sand.
- The eastern areas are covered by deposits of salt-bearing soils, sand dunes, and outcrops of limestone and calcareous sandstones.

5.2 Geotechnical Issues

The primary geotechnical issues in the Kingdom of Saudi Arabia are outlined in Al-Refeai and Al-Ghamdy (1994), Dafalla and Shamrani (2012), Stipho (1992), and as referenced in SBC (Soil, Foundations and Supporting Walls). These issues are summarized below.

- Collapsible soils will compress when inundated with water, resulting in settlements without additional applied stress, and consequent reduction in bearing capacity.
- Expansive soils are clayey materials that shrink and swell (expand) with changes in moisture content.
- Karst formations are characterized by solution features, with resulting sinkholes and cavities at or near the ground surface, presenting a hazard to structural and geotechnical works.
- Loess is an unstratified recent wind-blown deposit of predominantly silt-size material that has a loose structure and low density, and is highly compressible.
- Sabkha is a salt bearing heterogeneous material, primarily clay, silt, fine sand, and organic matter that is randomly interlayered. Sabkha sediments are generally characterized by high void ratios and low dry densities, and are collapsible.
- Corrosive environment and potential for sulfate attack due to mainly Sabkha, which is often high in water soluble sulfate and chloride, with potential for sulfate attack on concrete foundation elements and corrosion of metallic materials in contact with these soils.

5.3 Seismic Assessment

5.3.1 Seismic Foundation Design Requirements

5.3.1.1 General

Saudi Arabia is a country of relatively low seismic activity. According to the World Health Organization, Saudi Arabia Seismic Hazard Distribution Map, most of the country is classified as having a very low ($PGA \leq 0.02$ g) to low ($0.02 < PGA \leq 0.08$ g) seismic hazard. Here, the peak ground acceleration (PGA) corresponds to a 10% probability of exceedance in 50 years. Local regions have PGA values as high as 0.2 g, namely west of Tarbuk City, north of the City of Al-Madinah, surrounding the City of Jeddah, and the coastal region near the City of Jubail.

5.3.1.2 Seismic Design Category

SBC (Structural Design) classifies a structure based on an Occupancy Category and the severity of the design ground motion at the site. The Seismic Design Categories range from A to D and are determined following SBC (Structural Design). Buildings with a Seismic Design Category A do not require any special seismic design measures. Buildings with a Seismic Design Category D have the most stringent seismic foundation design requirements.

5.3.2 Seismic Site Classification

Seismic Ground Motion Values, of SBC (Structural Design) requires that the site be classified as Site Class A, B, C, D, E or F.

5.3.3 Earthquake Hazard Assessment

As per the requirements of Sections 2.2.5 and 2.2.6 of SBC 303 for structures within Seismic Design Categories C and D, respectively, an investigation shall be undertaken and the GDR shall address the following potential earthquake hazards:

- Slope Instability
- Liquefaction
- Lateral Spreading
- Vertical Displacements
- Surface Rupture.

5.3.4 Seismic Lateral Earth Pressure

For retaining walls assigned to Seismic Design Categories C and D, the combined static and seismic active earth pressure shall be calculated using the method outlined in SBC. This design approach is commonly known as the Mononobe-Okabe method (refer to Seed and Whitman, 1970).

In general the last SBC should be followed.

PART B: GEOTECHNICAL INVESTIGATION GUIDELINES

Part B provides a guideline for performing a geotechnical investigation to obtain geotechnical information at a new site or to supplement geotechnical information at an existing site. The A/E Lead Discipline Engineer shall review all existing geotechnical information for the site, typically contained in a geotechnical data report or a geotechnical design report, or both, to determine what, if any, additional geotechnical investigation is required for the planned project.

6.0 INVESTIGATION USING BORINGS

6.1 Boring Requirements

As per SBC boring investigations are <u>not</u> required if <u>all</u> of the following criteria are met:

- The net load pressure on the foundation is less than 50 kPa.
- There are no dynamic or vibratory loads on the building or structure.
- Questionable or problematic soil is not suspected within the proximity of the building or structure footprint.
- Cavities are not suspected underneath the footprint of the building or structure.

If any of the above criteria are not met, a boring investigation must be carried out. Depending on the availability of historical boring information in the area of the proposed development, and its quality, a new boring investigation may not be required. This determination shall be made by the A/E Lead Discipline Engineer.

6.2 Types of Boring Investigations

The type of boring investigation will depend on the anticipated site soils, as well as the objectives of the investigation. Soil and rock boring and sampling requirements are outlined in SBC. More detailed descriptions of various boring types can be found in AASHTO, FHWA (2002a), and USACE (2001). In this section, the term “boring” is used to describe an in-situ investigation technique and is not limited to the definition of “borehole” given in Section 2.1.

Types of boring investigations include the following. More details of the equipment are given in Section 8.3 of this document; samplers used with the equipment are described in Section 8.4 of this document. In-situ investigation techniques are described more fully in Section 9 of this document.

6.2.1 <u>Auger Borings</u>

Auger borings, which can be made with either continuous or discontinuous flight augers, allow for the collection of disturbed soil samples for laboratory testing, but give limited information on the soil strata. They are typically used above the water table (but may be used below the water table in the case of stiff clays).

6.2.2 <u>Drive Borings</u>

Drive borings are advanced into the soil and allow for the collection of disturbed soil samples for later testing; they also give some information on the soil stratification, an advantage over auger borings. Types of drive borings can include percussion and vibratory, with disturbed samples collected using the standard penetration test (SPT) (mainly in sands) and the Becker penetration test (mainly in gravel).

6.2.3 <u>Test Pits</u>

Excavation of test pits allows for determination of soil stratification at shallow depths, at relatively low cost. Samples can be obtained for laboratory testing, including larger samples that contain cobbles and boulders, which may not be obtained through borehole methods. Test pits are typically stopped when rock and/or the water table is encountered. The sides of the test pit must be suitably laid back when deeper than about 1.5 m if entry is required for sampling. Equipment used to dig the test pit must be sized appropriately to obtain the desired depth.

6.2.4 Cone Penetration Testing

Cone penetration testing, or CPT, is an in-situ test where a metal rod tipped with an instrumented cone assembly is advanced continuously into the soil, allowing for interpretation of soil strata and engineering properties without sampling, based on the readings obtained for skin friction, end bearing and porewater pressure on the instrumented cone assembly. See Section 9.2 for the cone assembly dimensions.

6.2.5 Undisturbed Borings

Undisturbed (or intact) soil samples, which are typically obtained using push samplers (such as Shelby tubes) or rotary samplers, allow for a detailed study of stratification, and the determination of properties such as shear strength, consolidation, permeability and density from laboratory tests. Undisturbed samples are usually limited to cohesive soils - specialized soil samplers are typically needed to obtain granular samples. After sampling, the tubes are sealed (often with wax) before being shipped (carefully) to the laboratory. Undisturbed samples are most frequently obtained in rotary wash borings (see Section 8.3.3).

6.2.6 Boring in Rock

Boring in rock is typically carried out through rotary drilling with rock core barrels fitted with diamond- or carbide-tipped bits when rock samples are required, or using percussive methods to install instrumentation, observation wells, etc. Rock core borings can be vertical or inclined, depending on the stratigraphy and properties of the rock. Rock cores are typically 1.5 m in length, and are stored in appropriately labeled core boxes for transportation to the testing laboratory.

6.3 Frequency and Depth of Boreholes

SBC 303 Table 2.1 specifies the minimum requirements for borehole investigations for buildings (see Table 7.11-1). Recommended minimum borehole depths are given in FHWA (2002a) for road infrastructure works (summarized in Table 7.11-2). Borehole depth shall be selected based on the depth below the planned base of the foundation. The selection of boreholes for buildings will depend on the anticipated site soils, as well as the type of proposed development. Boreholes shall fully penetrate all questionable or soft soil layers within the zone of influence of the building loading.

Table 7.11-1: Minimum Borehole Requirements for Buildings (SBC 303)

<table>
<tr><th>No. of Stories</th><th>Built Area (m^2)</th><th>No. of Boreholes</th><th>Minimum Depth of Two Thirds of the Boreholes (m)</th><th>Minimum Depth of One Third of the Boreholes (m)</th></tr>
<tr><td rowspan="3">2 or less</td><td><600</td><td>3</td><td>4</td><td>6</td></tr>
<tr><td>600-5000</td><td>3-10</td><td>5</td><td>8</td></tr>
<tr><td>>5000</td><td colspan="3">Special Investigation</td></tr>
<tr><td rowspan="3">3-4</td><td><600</td><td>3</td><td rowspan="2">6-8</td><td rowspan="2">9-12</td></tr>
<tr><td>600-5000</td><td>3-10</td></tr>
<tr><td>>5000</td><td colspan="3">Special Investigation</td></tr>
<tr><td>5 or higher</td><td colspan="4">Special Investigation</td></tr>
</table>

Where rock is encountered during the depth of a general investigation, a typical rock coring length is a minimum of 1.5 m into competent rock, to ensure auger refusal has not occurred on a boulder. If structures are to be founded directly on rock, a minimum rock core length of 3 m into competent rock is appropriate (FHWA 2002a).

The number and depth of boreholes shall be determined by the A/E Lead Discipline Engineer and subject to review and acceptance by Entity.

Borehole frequency and depth shall be sufficient to define the geologic setting at the project site, and will be dependent on both the anticipated soil conditions as well as the phase of the investigation being performed. For more general planning investigations, a wider spacing may be appropriate, while for detailed design purposes, a closer borehole spacing shall be used.

Table 7.11-2: Minimum Borehole Depths

Area of Investigation	Recommended Minimum Boring Depth (D_b)
Bridge Foundations - Spread Footings with Length L_f and Width B_f	• $L_f \leq 2B_f$: $D_b = 2\ B_f$ below bearing level • $L_f \geq 5B_f$: $D_b = 4\ B_f$ below bearing level • $2B_f \leq L_f \leq 5B_f$: D_b determined by linear interpolation
Bridge Foundations - Foundations Deep	• Soils: D_b = 6 m beyond anticipated pile or shaft tip, or two times the maximum pile group dimension, whichever is deeper • Bearing on Rock: D_b = 3 m of rock core to ensure boring has not terminated on a boulder • Extending into Rock: D_b = 3 m of rock core, or three times shaft diameter (for isolated shafts) or two times maximum shaft group dimension, whichever is deeper, below the anticipated pile tip elevation.
Retaining Walls	Between 0.75 and 1.5 times the height of the wall (below final ground line). If stratification indicates possible deep stability or settlement problems, extend to hard stratum.
Roadways	2 m below proposed subgrade level.
Cuts	5 m below anticipated cut at ditch line. Should be increased in locations of base instability, or where base of cut is below groundwater level.
Embankments	Twice the embankment height, unless hard stratum encountered above this depth. If soft strata are encountered, extend to hard stratum.
Culverts	Use criteria for embankments.

Suggestions for borehole layout and frequency for road schemes are provided in FHWA (2002a).

6.3.1 Establishing Borehole Locations and Elevations

Boring locations and elevations shall be established by a qualified surveyor. If a surveyor is unavailable, the contractor performing the geotechnical investigation under the direction of the A/E Lead Discipline Engineer shall determine the location of all boreholes and other field testing relative to known site features, to an accuracy applicable to the project scope.

Boring elevations shall be to either a geodetic benchmark (BM) or a temporary benchmark (TBM). Elevations are typically reported to ±0.1 m (at a minimum). Borehole elevations shall be referenced to the appropriate datum.

6.3.2 Questionable Soils

Additional boring frequency and procedures are required when questionable soils are suspected beneath the proposed development.

- Expansive Soils - sampling requirements for expansive soils are outlined in SBC.
- Collapsible Soils - sampling requirements for collapsible soils are outlined in SBC.
- Sabkha Soils - sampling requirements for sabkha soils are outlined in SBC.

6.4 Sampling

Soil sampling during a boring investigation will depend on the type of boring investigation being performed, and the information required from the samples. Common sampler types are listed in Section 8.4.1 of this document.

The sampling interval will depend on the type of soil and will vary by project and by region. Common intervals for SPTs using split-barrel (or split-spoon) samplers (which is the most common type of geotechnical sampling) include the following: continuous SPT sampling over the initial 5 m, then 1-m intervals to 20 m, followed by 1.5-m intervals to the bottom of the boring (or top of rock).

In cohesive soils, if encountered, at least one undisturbed sample (such as a thin-walled Shelby tube) shall be recovered from each layer. Shelby tube samples shall alternate with SPT split-barrel samples, or as directed in the scope of works. As required, in very soft to soft soils undisturbed samples will be recovered using thin-wall piston samplers with continuous sampling. In very stiff and hard clay soils, rotary samplers (e.g., Pitcher or Dennison) will probably be needed.

Where samples are taken during a boring investigation, they shall each be assigned a unique number to aid in identification. Care shall be taken to minimize disturbance of undisturbed samples and to maintain the moisture content of all soil samples, except, in most cases, bulk samples. Standard practices for handling field samples are provided in AASHTO.

6.5 Boring Logs

The following information shall be included on all field logs, where applicable, used during the boring investigation, and on all final boring logs after review and approval by the Entity.

- Borehole or test pit number or designation, and ground elevation of the top of the borehole or test pit.
- Driller's name and field representative's name.
- Make, size, and manufacturer's model designation of drilling, sampling, and test pit excavating equipment.
- Type of drilling and sampling operation by depth.
- Borehole diameter.
- Dates and times when drilling and sampling, and test pit operations were started and completed.
- Time required for each rock coring run.
- Drill action, rotation speed, hydraulic pressure, water pressure, tool drops, and any other experience which could indicate the stratum conditions encountered during rock coring.
- Depths at which samples or cores were recovered, or attempts were made to sample or core, including top and bottom depth of each run.
- Classification or description by depths of the materials sampled, cored, or penetrated using the USCS (ASTM D2487), including a description of moisture conditions, consistency and other appropriate properties described in ASTM D2488. This classification or description shall be made immediately after the samples or cores are retrieved.
- Classification and description by depths of rock materials sampled or cored, including rock type, composition, texture, presence and orientation of bedding, foliation, fractures, presence of vugs or other interstices, and the Recovery and RQD for each cored interval.
- Indication of penetration resistance such as SPT blows given in blows per increment (normally 0.25 m or 0.33 m) for driving split-barrel samplers and casing. The hammer type (i.e., donut, safety, or automatic) and the hammer efficiency, where available, shall be reported to enable correction to N_{60} (blow count corrected with respect to 60% hammer efficiency) used for design.
- Weight of drive hammer.
- Length/percentage of sample or core recovered per run.
- Depth at which groundwater is encountered initially and when stabilized.
- Depths at which drill water is lost and regained, and amounts.

- Depths at which the color of the drill water return changes.
- Type and weight of drill fluid.
- Depth of bottom of hole.
- Pressures employed in pressure testing.

7.0 GROUNDWATER ASSESSMENT

7.1 General

Groundwater refers to the water located beneath the earth's surface in soil pore spaces and in the fractures of rock formations. Groundwater has a significant influence on all geotechnical projects. Conversely, development projects often have an impact on groundwater. Groundwater can cause or contribute to failure because of excess saturation and reduction of soil strength due to seepage pressures or uplift forces. In some localities, groundwater may have elevated concentrations of pollutants or may contain constituents in concentrations sufficient to make it aggressive and cause damage to construction materials such as concrete and steel. Groundwater affects the design, performance, and constructability of project elements. Assessment of groundwater conditions involves determination of groundwater levels and pressures, hydraulic conductivity, and water quality in terms of chemical composition.

Experienced geoenvironmental/geotechnical engineers and/or hydrogeologists shall be consulted when planning a groundwater characterization study. Guidelines and reference standards (e.g., ASTM D4750, ASTM D5092) shall be consulted during the planning stages for groundwater characterization studies. Comprehensive reviews and discussions on groundwater and wells are also provided in several reference works including Driscoll (1986), UFC (2004), FHWA (2002a), and Harr (1962).

Characterization of groundwater conditions is an important component of the geotechnical investigation of a project. The geotechnical investigation shall identify groundwater levels and determine the range in seasonal fluctuations. If the geology or the groundwater regime is complex, significant input from an engineering geologist and/or a hydrogeologist is required. Characterization will include determination of whether the water table is the area/regional water table or whether it is a local perched water table produced by local conditions – perched water tables typically do not recharge when lowered.

7.2 Assessment of Groundwater Conditions

Detailed information regarding groundwater observations can be obtained from ASTM D4750 and ASTM D5092. The geotechnical scope of works shall provide requirements for groundwater measurement and monitoring for different applications, including minimum requirements and frequency of monitoring.

The geotechnical scope of works shall assess the types of groundwater investigation required - those used to determine groundwater levels and pressures, and those used to determine the hydraulic conductivity (permeability) of the subsurface materials. Determination of the hydraulic conductivity of soil or rock strata is required in connection with seepage studies (refer to Section 16) for leakage through embankments, yield of wells, piping assessment, groundwater control, temporary shoring design, and assessment of soil susceptibility to liquefaction.

Groundwater levels and pressures may be measured in existing wells, in open borings, and in specially installed monitoring (observation) wells and piezometers. Hydraulic conductivity shall be determined by means of various types of seepage, pressure and pumping tests. Commonly used means of groundwater measurement are given below.

7.3 Groundwater Measurement - Water Levels (Pressures)

7.3.1 <u>Existing Wells</u>

If records (including logs) of wells are available within close proximity of the project, these shall be incorporated into the groundwater study plan for the project. If such information is available, its acquisition shall be coordinated with the owner of the well.

7.3.2 Open Boreholes

The water level shall be measured during drilling and after completion of the borehole (if the borehole does not collapse). Groundwater observations made in open boreholes shall be treated with caution since the length of time required for water levels in boreholes to stabilize at the groundwater level is a function of the hydraulic conductivity of the site soil.

7.3.3 Monitoring Wells

These are wells installed for the purpose of longer-term studies of groundwater levels. Details of the installation procedures are available in the references listed in Section 7.1.

7.3.4 Piezometers

Piezometers are similar to monitoring wells except that the former in general measure water pressures as opposed to water levels. Note: standpipe piezometers are identical in performance to monitoring wells albeit smaller in diameter. Available types of piezometers include pneumatic, vibrating wire, and electrical resistance wire gauge piezometers. Consideration of time lag effects is particularly important in the selection of the appropriate type of piezometers for a given project. For further details on the selection, installation and instrumentation of piezometers, refer to Dunnicliff (1993).

7.4 Groundwater Measurement - Hydraulic Conductivity

In-situ hydraulic conductivity may be estimated using the following methods:

- Falling Water Level Method - for low permeability soils
- Rising Water Level Method - for low permeability soils
- Constant Water Level Method - mainly for high permeability soils
- Packer Tests - mostly for determining in-situ hydraulic conductivity in bedrock
- Aquifer (Pumping) Test - tests hydraulic conductivity of a larger area – see also Section 9.8.

For the selection and specific application procedures for these methods, refer to one or more of the references listed in Section 7.1.

8.0 GEOTECHNICAL EQUIPMENT

8.1 General Field Equipment

General field equipment will be required to successfully complete the field investigation, and shall include logging forms and tools, site information, and appropriate site manuals and permits. A list of general field supplies is provided in Table 2.4 of FHWA (2002a).

The following sections describe personal protective equipment, drilling equipment and sampling equipment. In-situ field testing equipment is described in Section 9, geophysical testing equipment is described in Section 10, laboratory testing equipment is referenced in Section 11, and offshore investigation equipment is described in Section 12.

8.2 Personal Protective Equipment

Investigations shall include all required Personal Protective Equipment (PPE) necessary to complete the work safely. This will include written documents, such as a properly completed Health and Safety plan specific to the job, as well as field gear, such as hard hats, safety footwear, reflective clothing, ear plugs and safety glasses. All safety gear shall conform to the requirements of the appropriate Health and Safety authority.

8.3 Drilling Equipment and Methods

Geotechnical drilling equipment is used to perform borings and obtain soil or bedrock samples, and varies widely depending on the goals of the investigation. A summary of common geotechnical drilling equipment (from FHWA, 2002a), as well as any applicable standards, is provided below.

8.3.1 Solid Stem Continuous Flight Auger (ASTM D1452)

Solid stem augers act similarly to a screw, bringing soils to the top of the boring as they are advanced. Although various sizes are available, the 100-mm diameter auger is the most common. Solid stem augers are usually connected to the drill rig by cotter pins.

8.3.2 Hollow Stem Continuous Flight Auger (ASTM D6151)

Hollow stem augers are similar to solid stem augers, but typically come in larger sizes. They differ primarily in that hollow stem augers have a hollow core, which is typically plugged with a center plug during advancement of the auger. The major advantage of the hollow stem auger is that sampling can be performed through the auger without having to withdraw the auger. The center plug is removed when sampling.

8.3.3 Rotary Wash Boring (ASTM D5783)

Rotary wash borings employ either casing or drilling fluid to support the sides of the drill hole. Drilling fluid is most commonly a mixture of bentonite and water that is circulated by being pumped down the drill hole through the hollow drill rods, discharged through the drill bit, and returned via the annulus between the drill rods and the drill hole. Casing sizes commonly range from 60 mm to 130 mm, and are often chosen based on the diameter of sampling equipment which must be advanced through the casing. Drill bits commonly consist of either drag bits or roller bits. One advantage of this drilling method is the ability to distinguish stratum breaks based on the performance of the equipment (e.g., “rattling” in gravels) and from the cuttings in the circulating drilling fluid; the main disadvantage is the need for an adequate water supply.

8.3.4 Bucket Augers (ASTM D6907)

Bucket augers are used to obtain larger volumes of soil, and consist of a bucket (diameter typically 600 mm - 1200 mm, length typically 600 mm – 900 mm) advanced through the soil or soft/weathered rock with cutting teeth mounted on the bottom. Slots in the base of the bucket allow it to collect samples.

8.3.5 Hand Augers/Excavators (ASTM D1452)

Hand augers are typically used for shallow investigation, with several types available, with the post hole type barrel auger recognized as the most common. Hand held power augers are also available. Hand augers may have to be used when vehicle access is not possible.

8.3.6 Mechanical Excavators

Mechanical excavators, such as backhoes, bulldozers and excavators are commonly used to excavate test pits. Their size will depend on the type of soil, depth of interest, and any site constraints.

8.3.7 Rock Core Drilling (ASTM D2113)

When intact rock samples are required, core drilling equipment is normally used. A summary of rock drilling equipment and procedures is provided in FHWA (2002a). Core drilling barrels are typically single-, double-, or triple-tube, with double-tube being the most common. Drilling fluid is water.

Rock cores are typically brought to the surface using either conventional or wireline equipment; conventional equipment requires the entire string of rods and core barrel to be brought to the surface after each run to retrieve core samples, while wireline equipment allows the inner core (on double- and triple-core barrels) to be brought to the surface separately.

Core barrels and other rock sampling equipment come in a variety of sizes, which are denoted by letters. Size NX core (54 mm diameter) is the most common core barrel size, although other sizes are also in use, and may be appropriate depending on the type of rock encountered. Coring bits are attached to the tip of the core barrel, and the types are listed in Section 8.4.2.

8.3.8 Non-Core (Destructive) Drilling

Non-core drilling, also known as destructive drilling, is typically used in situations where rock core samples are not required. The equipment used in such an investigation can include:

- Air-track drills
- Downhole percussive drills
- Rotary tricone (roller bit) drills
- Rotary drag bit drills
- Carbide-tipped bits.

8.4 Sampling Equipment

Sampling equipment depends on the type of investigation being conducted.

8.4.1 Soil Sampling

Common soil sampling methods are listed below:

- Split-Barrel (Split Spoon) (ASTM D1586) - see also Section 8.1.
- Thin-Walled Shelby Tube (ASTM D1587) - typically about 75 mm in diameter, most commonly used for sampling medium stiff to very stiff clays.
- Direct Push Sampling (ASTM D6282) - sampling tube is pushed into the soil without rotary drilling – favored for environmental site characterization.
- Piston (ASTM D6519) - variation of the Shelby Tube - the tube is lowered to the bottom of the drill hole with a steel piston flush with the bottom of the tube. The piston is held in place while the tube is pushed into the soil.
- Pitcher - designed to sample relatively intact soils that are too hard to be sampled with a normal Shelby tube. Operation is more akin to core drilling, but using a Shelby tube as the inner barrel.
- Denison (FHWA, 2002a) - somewhat similar to the Pitcher sampler but more complex.
- Modified California (FHWA, 2002a).
- Continuous Auger (FHWA, 2002a).
- Bulk (FHWA, 2002a).
- Block (FHWA, 2002a).

8.4.2 Rock Sampling

Rock core sampling is typically achieved through the use of coring bits attached to core barrels. Types of coring bits (per FHWA, 2002a) include:

- Diamond (most commonly used)
- Carbide
- Sawtooth.

9.0 IN-SITU TESTING

9.1 Standard Penetration Test

The standard penetration test or SPT (ASTM D1586) is the most commonly used in-situ test. The test allows disturbed soil samples to be obtained for further testing, and a wide variety of correlations between the test penetration resistance (N-Value) and various engineering properties have been established (see below). The test is performed by attaching a split-barrel sampler, with outside diameter about 50 mm and length typically about 500 mm, to the bottom of the drill string, and hammering it into the soil for a distance of 450 mm with a 0.62 kN hammer. The N-value is the number of hammer blows needed to advance the sampler 300 mm, the count beginning after the first 150 mm, i.e., from 150 to 450 mm penetration.

SPT is typically specified in granular soil (although correlations for clay exist) and can be advanced in very weathered rock. The test is useful for preliminary analysis of a site (Bowles, 1996) and can guide selection of depth for obtaining undisturbed samples.

The SPT is well correlated to a number of engineering properties and performance parameters, including unconfined compressive strength (see Mitchell et al., 1978), bearing capacity (see Meyerhof, 1956), settlement (see Terzaghi and Peck, 1967), and liquefaction susceptibility (see Seed, 1979).

The repeatability of the SPT can be questionable, and results shall be analyzed on a qualitative basis (Schmertmann, 1979, and Skempton, 1986). Numerous sampling procedures can affect the results of the SPT, and are summarized in Skempton (1986) and Decourt (1989). Corrected N-values shall be reported and used in subsequent design and analysis to account for factors such as overburden stress, length of drill rod, presence of a liner, diameter of the borehole, and hammer energy efficiency. The efficiency of the manually operated hammer (e.g., rope and cathead) is taken as 60% and the N-value obtained with this hammer is designated N_{60}. The N-values obtained with hammers with different efficiencies (e.g., automatic hammers with efficiencies in the 80% – 90% range) are corrected to provide an equivalent N_{60}.

9.2 Cone Penetration Test

The cone penetration test (CPT) (ASTM D3441 and ASTM D5778) allows for a continuous log of soil properties and stratigraphy. Cone penetrometers typically consist of a steel rod with a cone assembly with a 36-mm diameter body (10 cm^2 projected area), 60° apex angle, and 150 mm^2 friction sleeve. A cone assembly with a 44-mm diameter body (15 cm^2 projected area) and 200 cm^2 sleeve is also available. The rod is typically pushed at a rate of 20 mm/sec.

CPT profiling can be used to determine the soil type (see Douglas and Olsen, 1981; Lunne et al., 1997), the undrained shear strength for clays (see Jamiolkowski et al., 1985; Schmertmann, 1970), and the relative density of sands (see Durgunoglu and Mitchell, 1975; Mitchell et al., 1978; Schmertmann, 1978). CPT shall only be conducted at sites with suitable ground conditions – it cannot typically be performed in dense to very dense granular deposits or hard cohesive deposits. Cone penetrometers with a piezocone element can be utilized to measure porewater pressure, and geophones to measure seismic wave data. See also Section 6.2.4.

9.3 Pressuremeter

The pressuremeter consists of a cylindrical probe expanded radially into the surrounding soil to provide an estimate of a soil's stress-strain relationship. Pressuremeters can be used to test soils which are not suitable for push-type tests (e.g., CPT, field vane), such as hard clay, very dense sands and gravels, and weathered rock. Four types of pressuremeters are common:

- Pre-bored (Menard) Pressuremeter (MPMT)
- Self-boring Pressuremeter (SBP)
- Push-in Pressuremeter (PIP)
- Full-displacement Pressuremeter (FDP).

The most common pressuremeter test is the Menard, which is described in ASTM D4719 and in Baguelin et al. (1978). Further details on types of pressuremeters and calibration can be obtained from Briaud (1989) and Clarke (1995).

9.4 Field Vane

The field vane shear test (ASTM D2573) is used to measure the in-situ strength of clay soils and, where required, shall be specified in the geotechnical investigation scope of works. Although a variety of vane sizes are available, the most common consists of a blade diameter of 65 mm, a height of 130 mm, and a blade thickness of 2 mm. The peak undrained shear strength of the clay can be calculated from ASTM D2573.

The field vane shear test shall be corrected to account for the plasticity index (PI) of the clay (see Bjerrum, 1972 and 1973) and for over consolidation ratio (OCR) (see Aas et al., 1986). Guidelines on the use of field vanes in soft or sensitive clays are provided by Leoueil et al. (1990) and Leroueil (2001). Field vane shear tests can also be used to estimate the OCR of a clay deposit (see Mayne and Mitchell, 1988).

9.5 Flat Plate Dilatometer

The flat plate dilatometer (ASTM D6635) consists of a tapered blade with an 18° wedge tip, which is pushed vertically into the soil at either 200 mm or 300 mm depth intervals. The typical rate is 20 mm/sec, similar to the CPT. Pressures are taken from a 60-mm diameter flexible steel membrane located on one side of the blade (typical dimensions L = 240 mm, W = 95 mm, t = 15 mm).

Further information on the flat plate dilatometer can be obtained from Jamiolkowski et al. (1985), Robertson (1986), Schmertmann (1986), Mitchell (1988), and Lunne et al. (1989).

9.6 Direct Shear Test

Direct shear tests can be performed in-situ where available shear strength data are in doubt, or where thin, soft continuous layers exist within relatively stronger material (USACE, 2001). These tests are typically performed on rock (as opposed to soil).

Further information regarding the in-situ direct shear test can be obtained from Ziegler (1972), Nicholson (1983), and USACE (1993).

9.7 Plate Load Test

The plate load test is used to help determine the bearing capacity of small foundations, or to obtain the modulus of subgrade reaction of the soil immediately beneath a foundation. The test consists of measuring the applied load and penetration of a plate pushed into soil or rock mass; typically, a series of maintained loads of increasing magnitude is applied. The test can be performed either drained or undrained in the case of cohesive soils.

The plate load test is typically valid only for the depth and location of a specified test; to obtain representative results for a larger area or a full soil stratum, the test shall be performed in multiple locations or at multiple depths. A variation on the plate load test is the screw-plate test, which consists of a flat pitch auger which is screwed into the soil at the desired depth. Further information concerning the screw-load test is provided in Janbu and Senesset (1973).

9.8 Aquifer Characterization

The aquifer test method (ASTM D4043) is a controlled field experiment utilized to determine the approximate hydraulic properties of water-bearing material. The hydraulic properties that can be determined are specific to the test method selected, and are dependent upon the instrumentation of the field test, knowledge of the aquifer system at the site, and conformance of the hydrogeologic conditions at the site to the assumptions of the test method. Hydraulic conductivity and storage coefficient of the aquifer are the basic properties determined by most test methods. The appropriate test method shall be selected based on site geological conditions encountered. Refer to Sections 7.3 and 7.4 for information on design and installation of wells.

9.9 Other In-Situ Tests

Less common in-situ tests include the following:

- Becker Penetration Test (BPT)
- Dynamic Cone Penetration Test (DCPT)
- Plate Bearing/Jacking
- Borehole Direct Shear
- Borehole Jacking.

Further information on these tests may be obtained from USACE (2001) and FHWA (2002a).

10.0 GEOPHYSICAL TESTING

10.1 General

Geophysical testing shall be specified where necessary to determine the type, nature, and characteristics of subsurface materials; determine the in-situ engineering properties; and/or detect hidden natural or cultural features. Geophysical testing shall supplement or aid in the planning, execution and interpretation of the geotechnical drilling and testing investigations.

Geophysical testing mainly consists of measurement of mechanical waves (to determine elastic properties of materials) or electromagnetic waves (to locate anomalous objects such as cavities or buried objects).

As per FHWA (2002a, 2002b), the advantages and disadvantages of geophysical testing are as follows:

10.1.1 Advantages

- Non-destructive and/or non-invasive
- Fast and economical testing
- Theoretical basis for interpretation
- Applicable to soil and rock
- Works well for large areas
- Characterizes materials at very small strains – provides accurate low strain elastic properties

10.1.2 Disadvantages

- No samples or direct physical penetration
- Complex models frequently assumed for interpretation
- Measurements affected by water, and by cemented layers or inclusions
- Results influenced by depth from surface
- Less effective when there are small differences in stiffness between adjacent units
- Does not work as well for hard strata layered over softer strata
- Specialized equipment and interpretation expertise often required

10.2 Geophysical Testing Methods

Common methods of geophysical testing are summarized below, along with the appropriate standards.

10.2.1 Mechanical Waves

Mechanical waves include seismic and acoustic waves. There are four main types of mechanical waves: compression (P-waves), shear (S-waves), surface or Rayleigh (R-waves), and Love (L-waves). The first two are the most commonly measured in geophysical investigations, particularly shear waves, which can be directly linked to a soil's or rock's low strain shear modulus.

10.2.1.1 Seismic Refraction Method (ASTM D5777)

This method is used to determine the depth, thickness and seismic velocity of subsurface soil, rock or engineered materials. The method requires a seismic energy source, trigger cable, geophones, geophone cable, and a seismograph. Seismic refraction commonly measures P-waves, although recent technology has allowed S-waves to be measured as well (FHWA 2002a). It is frequently used in determining the top of shallow rock formations over relatively large areas.

10.2.1.2 Crosshole Testing Method (ASTM D4428)

This method is used to determine the seismic velocity of subsurface soil, rock or engineered materials. The method requires a cased pipe (usually PVC and grouted in place), a seismic energy source (downhole hammer), trigger cable, a geophone(s), geophone cable(s), and a seismograph. Typically, 3 boreholes are required, although the test can be performed using 2 boreholes. Crosshole testing measures both P-waves and S-waves traveling horizontally through the stratum. The energy source is inserted to specified depths in one boring, and the wave velocity is determined by measuring the arrival times at those depths in the other 2 boreholes (FHWA 2002a).

10.2.1.3 Downhole Testing Method (ASTM D7400)

The downhole testing method is comparable to the crosshole testing method with the primary difference being that the downhole test measures the wave velocity traveling vertically from the energy source at the surface to the geophone(s) at specified depths in an immediately adjacent borehole – only one borehole is required. The seismic CPT is a frequently used form of downhole testing, with an accelerometer located within the cone assembly.

10.2.1.4 Surface Wave Method

This method is used to determine the seismic profile (shear wave velocity) of subsurface soil, rock or engineered materials. The method requires a pair of geophones in a linear array, a transient force or vibrating mass to generate surface waves, and a spectrum analyzer or other data logging equipment. Surface wave methods analyze Rayleigh waves, which are converted to obtain shear waves (S-waves). Variations include spectral analysis of surface waves (SASW) and multichannel analysis of surface waves (MASW).

10.2.2 Electromagnetic Waves

Electromagnetic methods measure electrical and magnetic properties of the soil to determine subsurface conditions. Types of electromagnetic properties include resistivity, conductivity, magnetic fields, dielectric characteristics and permittivity (FHWA 2002a).

10.2.2.1 Surface Ground Penetrating Radar (ASTM D6432)

GPR is used to interpret geologic or subsurface conditions using measurements of changes in electromagnetic wave properties (i.e. permittivity) that are a function of the subsurface material's type, density, and moisture content. Short impulses of high-frequency electromagnetic waves are transmitted into the ground using a pair of transmitting and receiving antennae. The method uses the transmitting antenna to radiate electromagnetic waves. GPR is most effective in dry granular soils with depths of penetration up to 20 m; in wet saturated clays, effective depth of GPR is limited to about 3 to 6 m (FHWA, 2002a).

10.2.2.2 Direct Current Resistivity Method (ASTM D6431 and ASTM G57)

This method is used to measure the electrical resistivity of subsurface materials as an indicator of the type of subsurface material present. The method requires a source of current (battery or generator), high-impedance voltmeter or resistivity unit, metal stakes for the current and potential electrodes, and connecting cables. The electrical resistivity of a soil or rock is one indicator of the corrosive effect of the material on buried steel – high resistivity normally equates to low corrosivity, and vice versa. The inverse of resistivity, i.e., conductivity, is used in the design of electrical grounding systems.

10.2.2.3 Frequency Domain Electromagnetic Method (ASTM D6639)

This method is used to characterize the subsurface materials and geology based on their ability to conduct, enhance or obstruct the flow of electrical current induced in the ground. The method requires an energy source, transmitter coil, receiver electronics, a receiver coil and connecting cables.

10.2.2.4 Gravity Method (ASTM D6430)

This method is used to characterize the subsurface conditions by measuring variations in the earth's gravitational field caused by differences in density of the subsurface soil or rock or the presence of voids or man-made structures.

10.3 Conducting Geophysical Testing

Geophysical testing shall be conducted in accordance with ASTM or other approved standards. The equipment manufacturer's recommendations for calibration and standardization shall be followed. If no recommendations are provided, periodic checks of the equipment operation shall be conducted, including after each equipment problem or repair and before starting field work each day.

An initial site inspection to evaluate the survey plan shall be conducted, and the instrumentation locations shall be laid out and surveyed. Calibration, standardization and data interpretations shall be conducted by the A/E Lead Discipline Engineer.

An experienced geologist or engineering geophysicist shall be involved in preparing the scope and technical specification for a geophysical investigation. Additional guidelines on geophysical testing are found in FHWA (2002a) and USACE (2001).

10.4 Reporting

When conducting geophysical testing, accurate logs and records of all work accomplished shall be kept. The following information shall be included in the final report for the work:

- Purpose and scope of the survey
- A description of the geologic setting
- Limitations of the survey
- A list of assumptions made
- A description of the field approach, including the equipment used and the data acquisition parameters used
- A to-scale site map showing the instrumentation locations
- A description of the approach used to perform the test, as well as a description of corrections applied to field data and justification for their use
- Results of field measurements, raw record copies, and time-distance plots
- A description of the interpreted results
- A description of the data recording format
- A description of any variance from the Work Plan, Quality Control Plan or the ASTM guide
- A list of references used
- A list of the personnel conducting the field testing and data interpretation and their qualifications.

11.0 LABORATORY TESTING

11.1 General

Laboratory testing shall be specified to determine physical, chemical and hydrogeological properties of soil and rock in order to classify and define engineering properties for geotechnical design. Based on the anticipated ground conditions and required parameters, the A/E Lead Discipline Engineer shall identify all the required tests to determine design parameters. It is then the responsibility of the A/E Lead Discipline Engineer and his/her team to use their experience along with the data to develop geotechnical design properties and to perform their design based on these properties and the Geotechnical Design Guidelines provided in Part C of this document. Procedures for specific laboratory tests are not outlined here; references to applicable standards and procedures for laboratory tests are provided.

11.2 Sample Selection for Testing

The number of samples required is largely governed by the size and type of the project, as well as the subsurface conditions in the area. Areas in which additional indexing of soil and rock is required can be identified as samples are collected and analyzed, and testing requirements can be adjusted as needed. Many of the following minimal criteria to be considered when developing a laboratory testing program have been developed by FHWA (2002a):

- Project type (bridge, embankment, rehabilitation, refinery, power plant, etc.)
- Size of the project
- Loads to be imposed on the foundation soils
- Types of loads (static, dynamic, seismic, etc.)
- Critical tolerances for the project (e.g., settlement limitations)
- Vertical and horizontal variations in the soil profile as determined from boring logs and visual assessments
- Identification of soil types in the laboratory
- Known or suspected issues with the soils at the project location (e.g., swelling soils, collapsible soils, sabkha soils, organics)
- Presence of visually observed inclusions such as solution cavities, cemented zones, etc.

All soil samples shall be classified according to the USCS (ASTM D2487). Moisture contents shall be determined for cohesive soils, and for unsaturated granular soils with 12 percent or more fines. Rock core shall be fully described and logged before laboratory testing.

Most index testing is performed on disturbed samples that have not had any special handling to preserve structural integrity. It is important that samples are properly sealed to prevent drying when collected so that a representative moisture content can be determined in the laboratory. Soils can be sealed in metal tubes, plastic bags or glass jars. Rock cores are typically stored in core boxes where no attempt is made to preserve moisture content, since the moisture content for most rock samples is very low; portions of a rock core can be wrapped in foil or plastic wrap and coated in a thin layer of wax if moisture content determination is required in the laboratory.

Laboratory tests to determine engineering properties (consolidation, triaxial compression, etc.) shall be performed on selected undisturbed samples. The A/E Lead Discipline Engineer shall be responsible for specifying laboratory testing requirements, including quality assurance and quality control requirements. The contractor performing the geotechnical investigation under the direction of the A/E Lead Discipline Engineer shall have control over the shipping and storage of samples. Preserving, shipping and handling of samples shall be in accordance with ASTM D4220. The sample storage facility shall be climate controlled.

Refer to SBC 303 Section 2.5 for proper sampling procedures for expansive, sabkha and collapsible soils.

In order to satisfy project requirements, a suitable number of index and engineering property tests need to be planned to adequately cover the proposed site. If it is found during the course of the sampling program that the field coverage is uneven, or lacking in certain stratigraphic units, the field exploration and sampling program shall be revisited.

11.3 Engineering Classification and Property Tests

11.3.1 Rock

The following engineering classification and property tests are commonly performed on rock samples obtained from geotechnical field investigations (including both intact and crushed rock samples).

Table 7.11-3: Rock Engineering Classification and Property Tests

Test	Applicable Procedure	Purpose	Significance
Specific Gravity and Absorption	ASTM D6473	To determine the bulk or apparent specific gravity and absorption of a rock specimen.	Specific gravity and absorption provide valuable insight into a rock's ability to withstand weathering, and to evaluate potential deterioration.
Water Content	ASTM D2216	To determine the water (moisture) content of a rock specimen by mass.	Provides an indirect indication of porosity of a specimen, or clay content of a sedimentary rock.
Pulse Velocities and Elastic Constants	ASTM D2845 (withdrawn in 2017)	To determine the pulse velocities of compression and shear waves through a rock specimen, as well as the ultrasonic elastic constant.	Useful for characterizing the effects of uniaxial stress and water saturation on pulse velocity.
Rebound Hammer	ASTM D5873	To determine the rebound hardness number of a rock specimen.	A simple and quick index test of hardness, especially useful in the field.
Permeability	ASTM D4525	To determine the permeability (hydraulic conductivity) of a small rock specimen (and a liquid permeability equivalent through extrapolation).	Hydraulic conductivity is important in characterizing fluid flow through a rock mass.
Effective Porosity	ASTM D7063	To determine the effective porosity of a rock specimen.	Porosity is intrinsic to permeability, affecting things such as fluid flow and drill penetration rate.
Petrographic Examination	ASTM C295	To determine the physical and chemical characteristics of the rock specimen.	Provides an indication of rock type, minerals present, chemical reactivity (alkali-carbonate etc.) among other properties.
Durability	ASTM D4644 ASTM D5240 ASTM D5312 ASTM D5313	To determine the durability of a rock specimen exposed to a variety of physical and chemical conditioning.	These tests characterize freeze/thaw durability of rocks (D5240 and D5312), the slake durability of shales and weak rocks (D4644), and wetting/drying durability of rocks (D5313).
LA Abrasion	ASTM C535	To determine a coarse aggregate's resistance to degradation.	Provides an indication of relative quality of various sources of aggregate that share similar mineral compositions.
Point Load Testing	ASTM D5731	To determine the point-load strength index of the rock specimen.	A simple and quick index test of strength, especially useful in the field. Can be correlated to unconfined compressive strength.
Uniaxial & Triaxial Compressive Strength	ASTM D7012	To determine the strength of a rock specimen in uniaxial and/or triaxial compression.	The uniaxial compressive strength is the most direct method of determining rock strength. The triaxial compressive strength test can be used to calculate angle of

Test	Applicable Procedure	Purpose	Significance
			internal friction, angle of shearing resistance and cohesion intercept. The procedure listed provides a method for calculating Young's Modulus and Poisson's Ratio from these two tests.
Direct Shear Strength	ASTM D5607	To determine the shear strength of a rock specimen.	Shear strength is an important design parameter in the construction of foundations and slopes. Discontinuities present in a rock mass make this a difficult parameter to ascertain.
Creep during Compression	ASTM D7070	To determine the strain of a rock specimen as a function of time.	Provides quantitative parameters for stability analysis of underground structures subjected to an approximately constant load.

11.3.2 Soil

The following engineering classification and property tests are commonly performed on soil samples obtained from geotechnical field investigations (including both undisturbed and disturbed samples).

Table 7.11-4: Soil Engineering Classification and Property Tests

Test	Applicable Procedure	Purpose	Significance
Water Content	ASTM D4959	To determine the water (moisture) content of a sample of soil.	Provides significant information about the soil when combined with data obtained from other tests.
Atterberg Limits	ASTM D4318	To determine the liquid limit, plastic limit and plasticity index of the fine-grained portion of the sample.	Provides an indication of soil behavior and consistency relative to moisture content.
Sieve	ASTM D6913	To determine grain size distribution.	Provides a means for classifying soil and aggregate based on the distribution of grain sizes on each sieve.
Hydrometer Analysis	ASTM D422 (withdrawn in 2016)	To determine the distribution of particle sizes smaller than the No. 200 sieve (75 µm).	Provides a means for determining the percentage of silt and clay particles present.
X-ray Diffraction	USACE EM 1110-2-1906 (USACE, 1986a)	To determine the mineralogy of clay particles.	Obtaining clay mineralogy is important for predicting behavior of clay under various conditions.
Permeability	ASTM D5084	To determine the hydraulic conductivity of the soil sample.	Hydraulic conductivity is one of the main parameters used when selecting soil materials for construction (backfill type etc.)
Swell Potential	ASTM D4546	To estimate the potential for wetting-induced swell of the soil.	Mitigating the effects of expansive soils and clay is an important consideration in design.
Collapse Potential	ASTM D4546	To estimate the potential for wetting-induced collapse of the soil.	At high moisture contents, collapsible soils such as loess undergo a sudden change in volume, posing a risk to the integrity of the structure.
Unconfined Compressive Strength	ASTM D2166	To determine the unconfined compressive strength of a cohesive soil.	Provides a good measure of the shear strength of the soil.

Test	Applicable Procedure	Purpose	Significance
Triaxial Strength	ASTM D2850 ASTM D4767	To determine the strength and stress-strain relationship of an undisturbed or remolded cohesive soil.	Certain triaxial tests can provide effective stress parameters of the soil (cohesion and angle of internal friction).
Direct Shear	ASTM D3080	To determine the consolidated, drained shear strength of a granular or cohesive soil.	Provides a simple and easy test for shear strength, although with some shortcomings outlined in the referenced procedure.
Miniature Vane	ASTM D4648	To estimate the undrained shear strength of a fine-grained, undisturbed, remolded or reconstituted soil.	Provides a quick field or laboratory test for estimating shear strength of a soil.
Consolidation	ASTM D2435 ASTM D4186	To determine the one-dimensional consolidation behavior of a cohesive soil.	Provides quantitative parameters for estimating settlement of cohesive soils.
Compaction	ASTM D698 ASTM D1557	To determine the compaction characteristics of soil.	Provides maximum dry density and optimum moisture content of fill material.

12.0 OFFSHORE GEOTECHNICAL INVESTIGATIONS

12.1 Introduction

Offshore investigations (e.g., for deep-water structures and oil production platforms) including those for near shore structures (i.e., breakwaters and sheet pile walls for harbors) are highly specialized, and although the basic procedures for geotechnical site investigation (i.e., refer to SBC and ASTM D 420) shall be adhered to, several other requirements related to Saudi and international waters shall be considered. There are many additional statutes and regulations that apply to offshore geotechnical investigation work (especially those involving international waters). Applicable standards and regulations are included in, but not limited to, the following:

- American Petroleum Institute
- International Association of Oil and Gas Producers
- International Marine Contractors Association
- International Cable Protection Committee/Society for Underwater Technology
- International Marine Contractors Association
- International Organization for Standardization (ISO 19900 series: Petroleum and Natural Gas Industries - General Requirements for Offshore Structures).

In addition, other applicable offshore codes of practice shall be considered if found appropriate for a specific project. Due consideration needs to be given to shipping, harbor and other regulations with respect to developing investigation proposals and carrying out the investigation at the site, and for permission to use the overwater facilities proposed.

For simple cases, such as shallow-water offshore and near-shore investigations, a land-type investigation can be used with additional facilities needed for access, taking into account the depth of water. As the difficulties associated with access and depth of water increase, techniques for offshore investigations shall be specified including provisions to use static and floating platforms.

12.2 Useful Resources

Several reference materials and guidance documents are available containing information on the planning and execution of offshore explorations. A partial list is provided below:

- Geotechnical & Geophysical Investigations for Offshore and Nearshore Developments, by TC 1 of ISSMGE (2005)
- Offshore Geotechnical Engineering: Principles and Practice, by Dean (2010)
- Offshore Geotechnical Engineering, by Randolph and Gourvenec (2011)
- Guidance Notes for the Planning and Execution of Geophysical and Geotechnical Ground Investigations for Offshore Renewable Energy Developments, by the Society for Underwater Technology (OSIG, 2014).

A complete reference to these documents is provided in Appendix A.

12.3 Key Considerations

Since offshore investigations involve greater uncertainties and cost, they require more extensive planning of the scope of works and preparation for mobilization and completion of the works. To control the variables and associated risks/hazards and to ensure that these do not adversely impact a project and subsequent design, it is recommended that a project-specific geological and geotechnical risk register be created and maintained as soon as the project commences. Typical risks/hazards that may be present at an offshore geotechnical investigation site include the following (OSIG, 2014):

- Areas of soft soils (e.g., very recent deposits, channel fill) whose presence may affect foundation placement and installation depths and may also restrict the selection of installation vessels
- Areas where the seabed is mobile; this will affect foundation behavior, loads and installation depths, and may also affect cable routing, installation and long-term burial/protection
- Very hard soils or bedrock, the presence of which may affect foundation installation methods and installation depths, as well as cable routing and burial/protection options and methods
- Areal variation in foundation conditions that may require the selection of more than one foundation type for a development area
- Seabed or buried obstructions, boulders, unexploded ordinance, etc.
- Shallow gas, presence of which may impact foundation stability and the safe execution of geotechnical soil borings
- Seismic risk and the potential for soil liquefaction
- Environmental issues that can impact or be impacted by the project.

12.4 Types of Investigation

In order to manage offshore subsurface investigations effectively, all investigations must be planned with very clear aims and objectives. The sequence of an offshore investigation program, which shall be identified at the planning stage, shall involve the following.

12.4.1 Preliminary (Desktop) Study

This is to determine and compile existing information found in published documents, company archives (if any), online sources, and from technical data providers. The preliminary study shall address the following items, among others (OSIG, 2014):

- Definition of area to be investigated
- Geodetic datum and projection to be used
- Vertical (tidal) datum to be used

- Project requirements
- License and consenting requirements pertinent to the area to be investigated
- Conceptual foundation selection studies
- Existing geophysical and/or geotechnical data
- Existing site investigation data and reports for nearby sites
- Environmental issues (marine mammals, seabed ecology, etc.)
- Public domain data (e.g., winds, waves, tides, weather, climate)
- Any other local experience or knowledge
- A hazard register including all hazards to public safety, the project and the environment.

12.4.2 Geophysical Survey

This involves the use of non-intrusive devices to obtain a general understanding of the nature and characteristics of the seabed. An experienced marine engineering geophysicist shall be involved in preparing the geophysical survey scope and technical specifications. The following are some commonly used offshore geophysical tools:

- Echo sounder - acoustic system, used to measure water depth, hull-mounted
- Side scan sonar - acoustic system, used to identify seabed features, hull-mounted or towed
- Chirper and pinger - high-resolution seismic systems, used to identify shallow soils, 10 to 50 m depth; chirpers are either hull-mounted or towed, pingers are generally hull-mounted
- Boomers and sparkers - high-resolution seismic systems, used to identify intermediate depth soils, 60 to 100 m depth, towed.

12.4.3 Shallow Penetration Geotechnical Investigation

This involves in-situ testing and soil sampling to relatively shallow depths (a few meters) below the seafloor, with subsequent laboratory testing. This step is generally required for pipelines and small structures. More details of this type of investigation are given in the following sections.

12.4.4 Deep Penetration Geotechnical Investigation

This involves a significant amount of field work with in-situ testing and soil sampling to depths of 120 m or greater below the seafloor, with extensive laboratory testing. This is usually performed for sites where fixed offshore platforms are planned. More details of this type of investigation are given in the following sections.

12.5 Geotechnical Drilling and Sampling Techniques

The primary aim of an offshore geotechnical site investigation is to determine the soil/rock layering at the proposed structure locations and to measure engineering properties of the soil and rock, by in-situ testing and testing of laboratory samples. Detailed descriptions of the various offshore geotechnical drilling techniques are provided in ISSMGE (2005) and Dean (2010). Common offshore drilling and coring systems include the following:

12.5.1 Soil Borings

The most commonly used technique for offshore drilling and sampling of deep soils is the rotary wash boring described in Section 8.3.3. Typically, the drilling equipment is larger and more powerful than that used on land. Drilling is normally performed through an opening in the vessel deck (moon pool) that extends through the hull. If drilling from a platform, the drill rig may be cantilevered over the platform edge. A barium compound should be added to the drilling fluid to increase its weight to prevent a blow-out where shallow gas pockets are anticipated. Since drilling may be performed in water depths approaching 100 m in the Arabian Gulf and several hundred meters in the Red Sea, and to depths of more than 100 m below the seabed, wireline sampling of the soils reduces the time (and hence cost) of the investigation significantly. With wireline sampling, clay samples in thin-walled tubes and granular samples in split-barrel samplers are obtained using a down-hole hammer.

12.5.2 Rock Coring

When rock is encountered, it can be sampled using the rock coring techniques described in Section 8.3.7. Wireline coring is used almost universally in offshore investigations. Alternatively, it can be cored using a seabed drill rig as described in the next section.

12.5.3 Seabed Drill Rigs

These specialty drill rigs are commonly remote controlled but can be operated by divers in relatively shallow water (less than about 30 m depth). The large advantage in both deep and relatively shallow water is that the rig is independent of sea conditions – it can be left on the seafloor even if the operating vessel is forced from the site due to bad weather conditions. A major advantage in deep water is the elimination of the drill string from the drill vessel to the seabed. The biggest problem is the difficult logistics involved in retrieving samples. If divers are involved, they can typically stay down for only a limited duration (about 1 hour at 20 m water depth) and thus a team of divers is needed.

12.5.4 Various Corers and Samplers

There are a variety of corers that are designed to obtain samples from immediately below the seabed, to maximum lengths ranging typically from about 3 m to 10 m, depending on the sampler type and the softness/looseness of the soil. Typical diameter is 100 mm, with a range from about 75 mm to 150 mm. The sampler (and any housing) is lowered to the seabed and then activated. Commonly used corers are described below.

12.5.4.1 Gravity Corer

This is the simplest and most basic of the corers. The steel core barrel is fitted with a sharpened replaceable core cutter on the bottom and a replaceable core liner within the core barrel to ease sample removal. The barrel is fitted with external stabilizing fins to ensure verticality during penetration. When the sampler is activated, a set of lead weights fitted on the top of the sampler push the corer into the soil. The number of weights used is determined by the consistency of the sediments and the length of sample required. A core catcher is fitted inside the end of the barrel.

12.5.4.2 Piston Corer

This variation of the gravity corer utilizes a piston inside the barrel and works similarly to the piston sampler described in Section 8.4.1, preventing plugging in the core barrel.

12.5.4.3 Vibro Corer

This variation of the gravity corer has a large electric motor that powers concentric weights to produce sufficient vibration to collect granular cores up to 9 m in length.

12.5.4.4 Box Corer

This is quite different from the corers described above in that it recovers box-shaped samples with dimensions of the largest samples being about 0.5 m x 0.5 m x 0.5m. It has a double-shovel mechanism that results in the sample being completely enclosed after retraction.

12.5.4.5 Grab Sampler

This is one of the simplest forms of seabed sampling equipment – it is essentially a clamshell bucket that can be manually or hydraulically operated. The manually operated sampler can work in depths of water up to 4,000 m.

12.6 In-Situ Testing

In addition to drilling and sampling, various in-situ tests are available for offshore geotechnical exploration. These include the following.

12.6.1 CPT

In deeper water, this test is most commonly performed from the seabed. The weight of the system on the seabed determines the available reaction for performing the test. The rods are advanced hydraulically, typically with a stroke in a range from 1.5 to 5 m. CPTs can also be performed from the deck of a vessel - a jack-up barge is preferred because even a slight horizontal movement can bend the rods, and any vertical movement will give erroneous readings. The test is best carried out within drill rods to prevent buckling of the CPT rods between the deck and the seafloor.

In shallow water, the CPT can be operated from a high-wheeled truck or an amphibious vehicle. ROVs (remotely operated vehicles) have been successfully used in both shallow or slightly deeper water.

12.6.2 Vane Shear Test

This test is particularly suited for use in very soft clays near the seafloor, where undisturbed sampling is challenging. The vane can be operated from the seafloor or from the deck of a vessel, through the drill string.

12.6.3 Other In-Situ Tests

Less commonly performed in-situ testing includes:

- Pressure meter (see Section 9.3)
- Dilatometer (see Section 9.5)
- Flow penetrometers – soil deformation mechanism is symmetrical in the plane perpendicular to the axis of the penetrometer; examples are the T-bar and the spherical ball penetrometers.

For additional information on offshore in-situ tests, refer to ISSMGE (2005) and Dean (2010).

12.7 Vessels, Platforms and Deployment Systems

The choice of deployment systems depends partly on the type of information needed, partly on the environment, and on the availability and cost of suitable equipment. Deployment systems include the following.

12.7.1 Geotechnical Drill Ships

These are vessels designed solely for offshore subsurface investigation, e.g., the Glomar Challenger that was built for the Deep Sea Drilling Program. These are very uncommon.

12.7.2 Converted Ships

The most common ship used for offshore geotechnical investigation is the converted oilfield workboat. These ships have the wheelhouse, crew quarters, etc. at the bow, leaving a long open deck for the drill rig, drill rods, bentonite storage and mixing, etc. Conversion includes fitting a moon pool, and large winches to manage the anchors (see Section 12.8). The vessel has to be fairly substantial – once anchored for drilling it must remain stable under changing wind and tide conditions.

The vessel requirements for geophysical surveys are far less demanding. The majority of measuring equipment either fits in the cabin, is attached to the hull, or is towed by the boat, which is typically far smaller (and often faster) than is used for drilling.

12.7.3 Barges

Although large offshore construction barges can be used for geotechnical investigations, this will typically only occur when no other vessels are available, since renting such barges is very expensive. Smaller barges are typically used for drilling in rivers, lakes and nearshore. These barges are not usually self-propelled but may be fitted with a diesel-powered winch. They can be fitted with a moon pool or the rig can be cantilevered over the side. They are often termed spud barges because they are fitted with spuds – vertical steel piles attached to the barge (usually fore and aft) that are released and penetrate the sea bed to anchor the barge in place.

12.7.4 Jack-Up Platforms

These come in a variety of sizes, from platforms that can be transported by road and assembled on shore, to ocean going platforms used for oil exploration. The barges are typically towed to their location, the legs are lowered to the seafloor, and the body of the barge is jacked up on its legs until it is sufficiently clear of the water. The feet at the base of the legs will penetrate the seabed until sufficient bearing is achieved. Larger feet can be fitted for operation in soft soils. Before jacking, extra water ballast can be taken on to act as preloading, and discharged before drilling operations begin. The big advantage of the jack-up barge is its stability and lack of movement, even in rough weather.

12.7.5 High-Wheeled/Amphibious Vehicles

These are used in water that is too shallow for a boat, or in swamp/marsh conditions. Often called marsh or swamp buggies, they can have tracks or large tires.

12.7.6 Remote-Controlled Systems

As noted in Section 12.5.3, remote-controlled or diver-operated equipment is available for operating on the seafloor. A wide variety of support vessels can be used, with the main provision being the ability to lower and raise the drilling equipment to and from the seafloor.

12.8 Positioning and Anchoring

Offshore and nearshore geophysical or geotechnical investigations require accurate positioning systems. Positioning of locations is controlled using data from Global Positioning System (GPS), a system of satellites around the globe. Where underwater positioning is required, acoustic positioning systems such as Ultra Short Baseline (USBL), Long Baseline (LBL) or special variations of these may be employed. For more details on offshore positioning, refer to ISSMGE (2005).

For drilling/sampling and in-situ-testing, it is essential that ship movement be minimized during operations. For converted oilfield workboats, a 4-point mooring system is sufficient. Large anchors are attached by cable to winches on both sides of the bow and stern of the boat. The anchors are typically taken and dropped by a support vessel. The cables are tightened using the winches, and the boat is accurately positioned by adjusting the length of each cable. The boat is positioned facing into any swell.

12.9 Laboratory Testing

Laboratory techniques for offshore site investigations essentially do not differ from those applied to investigations for onshore developments and generally follow the same standards (refer to Section 11).

Depending on the scale and purpose of the offshore geotechnical investigation, a soil/rock testing laboratory may be part of the offshore operation. Where the investigation is for a planned deep-water structure such as an oil production platform that needs to be pile supported, then strength testing of the samples can be performed as the samples are retrieved in order to estimate the length of pile required, and thus determine the required depth of the boring. Although some tests are carried out offshore during the field work, long-term tests such as consolidation testing will generally be performed onshore.

13.0 GEOTECHNICAL DATA REPORT

The results of the geotechnical investigation shall be presented in a geotechnical data report, prepared by the contractor performing the investigation and under the direction of the A/E Lead Discipline Engineer. A draft report shall be prepared for review by the A/E Lead Discipline Engineer and the Entity, followed by a final report incorporating all comments.

The geotechnical data report shall include:

- Description of the scope and timeframe of the investigation, and the investigation staff and crew
- Description of the site(s) investigated

- Description of the geology of the site area
- As-built exploration point survey data
- Description of all major field equipment used in the investigation
- Typed boring logs with soil and rock geologic descriptions prepared in accordance with ASTM D2487 and D2488 and guidance provided by the A/E Lead Discipline Engineer
- SPT hammer energy measurement report, if required
- Typed test pit logs, with soil descriptions in accordance with ASTM D2487 and D2488 as applicable
- Typed logs of observation wells and piezometers installed, along with results of hydraulic conductivity tests performed and water level readings
- Log of each CPT performed, showing a graphical presentation of the various measured data (tip resistance, sleeve friction, porewater pressure, interpreted soil type, etc.) versus depth
- Results of other in-situ testing performed (pressuremeter, flat dilatometer, etc.)
- Results of the geophysical survey (content outlined in Section 10.4)
- Summary tables of the results of all laboratory tests performed
- Plots of laboratory test results (grain size curves, stress-strain curves from triaxial strength tests, void ratio versus log pressure curves from consolidation tests, etc.)
- Any other data collected during the course of the subsurface investigation program
- Electronic files of the boring log data, CPT data, etc. in a format specified by the A/E Lead Discipline Engineer.

PART C: GEOTECHNICAL DESIGN GUIDELINE

14.0 INTERPRETATION OF GEOTECHNICAL CONDITIONS

Before geotechnical design can commence, the geotechnical data obtained in accordance with Part B must be analyzed and design (or best estimate) values assigned for the soil and rock properties. The first step in this process is to define the stratigraphy, or layering of the site.

14.1 Subsurface Profiles

Representative subsurface profiles (sometimes termed engineering geological sections in a mainly rock site) shall be drawn through the major structures (or along the alignment if a linear project) showing the existing grade, final grade, each soil and rock stratum, and including boring locations and the measured water table at each location. If the scale allows, the N_{60} values on the profile provide a quick overview of the strength and relative density of each soil stratum. Similarly, RQD values provide an overview of rock quality.

14.2 Soil and Rock Property Design Values

These values form the basis of subsequent analyses of bearing capacity, settlement, pile capacity, slope stability, etc. They are obtained either from field and/or laboratory measurements, or from mainly empirical relationships found in the literature.

14.2.1 Soil

Refer to Table 7.11-4 for the appropriate ASTM standards used to obtain some of the following properties:

- Water content (w) - from laboratory measurements
- Atterberg limits (LL, PL, PI) - from laboratory measurements
- Grain (particle) size distribution - from laboratory measurements
- Unit weight (γ) - from intact samples (usually clays), and from empirical relationships with N_{60} for sand and gravel
- Specific gravity (G_s) - from laboratory measurements, or use 2.7 to 2.75 for most soils if no test results available
- N_{60} - from average of N-value measurements, then adjusted for hammer energy
- Unconfined compressive strength (U) - from laboratory measurements
- Undrained shear strength of cohesive soils (c_u) - from laboratory measurements, or from correlations with CPT measurements; approximate values from correlation with N_{60}
- Internal friction angle (φ) - from laboratory measurements (triaxial or direct shear), or from empirical correlations (mainly with N_{60})
- Relative density (D_r) - from laboratory measurements or from empirical correlations (mainly with N_{60})
- Permeability (k) - from field and laboratory measurements
- Coefficient of consolidation (c_v) - from laboratory measurements
- Shear and compression wave velocity (V_s and V_p) - from field measurements
- Shear modulus, low strain (G_L) - from V_s and γ.
- Shear modulus, high strain (G_H) - from GL using modulus versus strain relationship, or from relationship between G, E, and μ
- Elastic modulus, low strain (E_L) - from relationship between E, G, and μ
- Elastic modulus, high strain (E_H) - usually from empirical relationships with N_{60} (sand) or c_u (clay)
- Poisson's ratio (μ) - from relationship between shear and compression wave velocity, or use 0.25 to 0.35 for granular soils and 0.35 to 0.45 for cohesive soils if no test results available

- Static earth pressure coefficients (K_a, K_p, K_0) - from relationship with φ; for at-rest coefficient K_0, usually use 0.5 if computed value is <0.5
- Coefficient of sliding (tanδ) - for concrete against soil or rock, usually use δ = 2/3φ
- CBR - from either field or laboratory measurements
- pH - from laboratory measurements
- Sulfate and chloride content - from laboratory measurements
- Electrical resistivity - mainly from field measurements but laboratory measurements can be made
- Thermal resistivity - mainly from laboratory measurements but field measurements can be made.

14.2.2 Rock

Refer to Table 7.11-3 for the appropriate ASTM standards used to obtain some of the following properties. Only properties not listed for soils in Section 14.2.1 are given here:

- Recovery - measured from rock coring
- RQD - measured from rock coring
- Point load strength index (I_S) - from laboratory measurements
- Durability (freeze/thaw, wetting/drying, slaking, degradation) - from laboratory measurements.

15.0 FOUNDATION DESIGN

This section provides guidance on the geotechnical design of shallow and deep foundations and retaining walls, and includes input on soil improvement techniques.

15.1 Shallow Foundations

15.1.1 General

Key design issues for shallow foundations include (refer to Poulos et al, 2001):

- Estimation of the ultimate bearing capacity, taking into account the effects of vertical, horizontal, and moment loading.
- Estimation of the total and differential settlements, including time dependence of settlements.
- Estimation of foundation settlements or heave due to moisture changes, taking into account the soil conditions specific to the site.
- Structural design of foundation elements to meet the project requirements.

15.1.2 Embedment Depth

The embedment depth of shallow foundations is generally less than the width or the least lateral dimension of the foundation unit. Common types of shallow foundations include strip footings for walls, spread (pad) footings for isolated columns, combined footings for supporting loads from more than one structural unit, and raft or mat foundations for various types of structures or special conditions (such as areas of swelling soils or erratic soil conditions).

SBC specifies the following minimum depths for footings:

- 1.2 m for cohesionless soils
- 1.5 m for silty and clay soils
- 0.6 to 1.2 m for rock, depending on the strength and integrity of the formation.

In addition, if the shallow subsoils are subject to shifting or moving, deeper bearing depths are required.

15.1.3 Bearing Capacity of Shallow Foundations

15.1.3.1 General

The bearing capacity shall be assessed to determine the ability of soil and rock to safely carry the imposed loads without undergoing bearing failure or excessive settlement. Bearing capacity and settlement are interrelated and are a function of the dimensions of the foundation element, primarily the least lateral dimension, i.e., the width, and the depth of embedment. The bearing capacity increases with the width in granular soils; however, this increase in bearing capacity is limited by the settlement potential of the underlying soils since the depth of influence of wider foundations is greater. Geotechnical design involves achieving a safe loading capacity of the foundation element that also limits total and differential settlement to tolerable values - the result is the allowable bearing capacity value.

15.1.3.2 Conventional Design

The conventional design (factor of safety based) of shallow foundations involves the following (i.e., refer to USACE, 1992).

- Evaluation of the ultimate bearing capacity using accepted methods of allowable stress design.
- Selection of a reasonable factor of safety based on available subsurface information, importance of the structure, past experience, etc.
- Evaluation of the allowable bearing capacity by dividing the ultimate bearing capacity by the selected factor of safety.
- Carrying out a settlement analysis and adjusting the bearing capacity until settlements are within tolerable limits.

15.1.3.3 LRFD Design

In LRFD (Load and Resistance Factor Design), the design approach typically involves the following:

- Evaluation of the ultimate bearing capacity using accepted methods of limit state design.
- Selection of resistance factors (less than unity), usually given in codes and specifications, established based on substantial statistical data combined with calibration or past experience.
- Calculation of the ultimate (strength) limit state (factored geotechnical resistance at ultimate limit state, ULS) by multiplying the ultimate capacity by the resistance factor.
- Evaluation of the serviceability (service) limit state (geotechnical reaction at serviceability limit state, SLS) as the value that will limit the settlements within tolerable movements for the specific project.
- Selection of the LRFD (limit states) value as the lower of the two limit states (i.e., strength and service).
- Proportioning of foundations so that the selected limit state value is not less than the effects of the factored loads (load factors are specified in codes based on required performance levels, etc.).

There are several reference documents and guidelines on bearing capacity and settlement analyses for shallow and deep foundations, including Chapter 5 of NAVFAC (1982), Chapters 4 & 5 of NAVFAC (1986), USACE (1990), USACE (1992), and FHWA (2002b).

15.1.3.4 Bearing Capacity Analysis

The ultimate bearing capacity of a foundation is commonly given the symbol q_u (ultimate bearing capacity pressure). There are various approaches (assumption of failure paths) for evaluating q_u, as described below, from the above references.

Bearing capacity analysis methods include:

- Slip-line method

- Limit equilibrium method
- Limit analysis method
- Numerical method (finite element, finite difference).

Bearing capacity analyses are generally based on limit equilibrium methods. Bearing capacity analysis based on the methods of Meyerhof, Hansen and Vesic are acceptable for routine foundation design (Bowles, 1996). Failure modes associated with bearing capacity analysis include general shear, local shear and punching shear (refer to FHWA, 2002b).

Bearing capacity analysis for any given situation shall consider the following variables, as applicable:

- Foundation shape (strip, rectangular, square, circular)
- Depth of embedment
- Location of groundwater level
- Sloping ground surface or proximity to edge of slope
- Load inclination and/or eccentricity
- Non-homogeneous (layered) soil.

For complex problems including layered soils, approximate methods may be used to obtain solutions (i.e., refer to Das, 1999, Sloan and Yu, 1996). For complex bearing capacity problems and foundation/soil structure interaction solutions, computer programs can be used to find numerical solutions (i.e., finite element, finite difference).

15.1.3.5 Bearing Capacity Design of Shallow Foundations on Soil

The bearing capacity of shallow foundations on soil may be obtained from:

- Bearing capacity equations based on theoretical analysis.
- Empirical correlations based on in-situ (field) tests.
- Presumptive load-bearing values, usually specified in local codes.
- Theoretical analyses for the bearing capacity of foundations on soil (bearing capacity equation) are described in Chapter 4 of NAVFAC (1986), Chapter 8 of FHWA (2006a), and numerous other reference books. Bearing capacity evaluations shall consider the effects of soil type and layering, load eccentricity, groundwater depth, possible changes in the groundwater depth, sloping ground surface, and uplift forces. Consideration shall also be given to potential flooding of the foundation soil and removal of existing overburden by scour or excavation, soil solutioning, collapse potential, infiltration from downspouts, and shedding water from aprons. The method for bearing capacity analysis shall be commonly used in geotechnical engineering practice or based on sound engineering judgment and subject to the approval of the Entity.

Bearing capacity can be evaluated using data from field testing, including the SPT, CPT, and plate load tests. Empirical design charts relate the allowable bearing pressure for shallow foundations on sand to SPT N-values; these relationships are based on limiting the settlement of the foundation unit, typically to 25 mm (refer to Peck et al, 1974). For large mat foundations, the same reference indicates that the total limiting settlement can be increased to about 50 mm, with differential settlement limited to about 20 mm. Use of the CPT in estimating the bearing capacity of soils is described in Schmertmann (1970) and Lunne et al (1997). The bearing capacity of soils can also be estimated using the results from plate load tests provided the soil within the zone of influence of the footing is similar to that influenced by the plate load test.

SBC provides presumptive load-bearing pressures for soils in the absence of site-specific design for footings.

Minimum requirements for the design of shallow foundations are provided in SBC. Refer to sections dealing with:

- Spread Footings

- Foundation Walls
- Combined Footing and Mats.

SBC specifies a minimum factor of safety against bearing failure of 3 for permanent structures and 2 for temporary structures.

Load combinations are specified in SBC for ASD, LRFD and seismic loads.

For problematic soils prevalent in Saudi Arabia (expansive soils, collapsible soils, and sabkha soils), footings shall be designed in accordance with the provisions of SBC. Footings subjected to vibratory loads shall be designed in accordance with the provisions of SBC. Foundations for non-building related structures shall be designed according to relevant FHWA and AASHTO guidelines. Foundation designs shall be performed following all appropriate design standards and subject to approval by the Entity.

15.1.3.6 Bearing Capacity Design of Shallow Foundations on Rock

The design of foundations on rock shall consider the potential impacts of unfavorable rock conditions, including weathered zones, joints, or other defects. Such discontinuities may result in excessive compression in a zone of weakness, or failure along unfavorably oriented planes of weakness.

For foundations on rock, the GDR shall describe both the intact rock and the condition of the rock mass. Typically, the rock mass is comprised of intact blocks of rock separated by discontinuities such as joints, bedding planes, folds, sheared zones and faults. The rock condition may range from being fresh and unaltered to highly weathered or decomposed. In determining the bearing capacity of foundations on the rock mass, the A/E Lead Discipline Engineer shall consider the stability of jointed rock, decomposed rock, horizontally discontinuous rock, and rock masses with solution cavities. In general, typical failure modes within the rock mass are as follows:

- Bearing capacity failure within weak shale or heavily decomposed rock
- Consolidation failure within weathered rock or weathered zones (horizontal, vertical, or subvertical)
- Punching failure where an underlying weak or porous rock is present
- Rock slope failure, in the case of foundations adjacent to slopes
- Subsidence or collapse due to underlying man-made or natural voids, such as solution cavities.

Guidelines on estimating the bearing pressure on rock are provided in Chapter 22 of Peck et al (1974) and Chapter 6 of USACE (1994). These guidelines make use of RQD (Rock Quality Designation) and RMR (Rock Mass Rating – ASTM D5878) to assess the compressibility and strength of a rock mass.

15.1.4 Settlement of Shallow Foundations

15.1.4.1 General

Predicting the settlement of a structure is a critical element of foundation design, especially for shallow foundations. Settlement evaluations for buildings shall consider both total and differential settlement.

For shallow foundations, settlement (as opposed to bearing capacity) typically governs the performance of the foundation element. When assessing the settlement of shallow foundations, the following three items shall be considered (Holtz, 1991):

- Available method of settlement analysis for any given foundation design
- Tolerable movements (settlements)

Available remedial options should the estimated settlements exceed the tolerable settlements.

15.1.4.2 Causes of Settlement

The following causes of settlement should normally be considered:

- Structural (applied) loading

- Embankment loading
- Consolidation/compression due to groundwater lowering
- Settlement due to vibrations (earthquakes/machinery).

If the design involves foundation groups, the settlement analysis shall assess the potential for overlapping stress bulbs.

15.1.4.3 Components of Settlement

In general, there are three components of the total settlement that can be experienced by a foundation element:

- Immediate Settlement - settlement due to the pseudo-elastic settlement of foundation soils caused by the applied loads. This component of settlement is usually proportionally more significant in cohesionless soils and stiff cohesive soils. Empirical and semi-empirical methods and elastic theory will be used for calculating immediate settlement. For detailed procedures, refer to USACE (1990) and Holtz (1991).
- Primary Consolidation Settlement - settlement that occurs in cohesive soils due to dissipation of excess porewater pressure caused by loading. The time rate of consolidation settlement is controlled by the rate at which water can be expelled from the void spaces in the soil. Several reference materials are available for computing the primary consolidation settlement, including: USACE (1990), Holtz (1991), NAVFAC (1982), FHWA (2002b), and Das (1999).
- Secondary Compression Settlement or Creep - settlement that occurs gradually and at a constant effective stress after the completion of primary consolidation settlement. Secondary compression relates to the slowly occurring adjustment of the particle arrangement under sustained loads (creep). Secondary compression typically is limited to certain clays, silts and organic soils.

Note that for most heavily-loaded structures, settlements will be excessive if placed on soils other than medium dense or dense sands/gravels or stiff/hard clays, even if bearing capacity analysis provides an adequate factor of safety against bearing failure. For these heavily-loaded structures on dense or stiff soils, an elastic settlement analysis will normally provide a reasonable estimate of settlement. With these soils, secondary compression settlement is not typically an issue.

For cohesive soils subject to primary consolidation, both the magnitude and rate of settlement are equally important components of a settlement analysis. For loose coarse-grained soils, such as sands, compression of the soil skeleton results from the rearrangement of the soil particles which generally takes place immediately as the load is applied. Vibratory or dynamic loading may also induce significant settlements in granular soils.

Settlement may also be caused due to the soil's collapse potential, solutioning, heave, and the result of seismic action. It is difficult to estimate the settlement caused by these influences; only the general susceptibility of a particular soil type to settlements caused by these effects can be stated. When the above settlement mechanisms are anticipated, testing and appropriate evaluation shall be carried out to assess the susceptibility of the site soils to these effects.

15.1.4.4 Steps in Settlement Analysis

For detailed procedures for predicting settlements for shallow foundations, refer to Chapter 5 of NAVFAC (1982) or Chapter 3 of USACE (1990). The method used for calculating settlement shall be approved by the Entity. In some situations, more rigorous settlement analyses may be required, including finite-element based computer methods.

Settlements must be within tolerable limits to meet the serviceability requirements of the structure, as stipulated in the project objectives (refer SBC).

The key steps in settlement analysis involve the following (modified from Holtz, 1991):

- Establish the soil profile, including the location of the groundwater table, and identify any compressible layers.
- Establish the total and effective stress profiles with depth.

- Obtain or estimate the magnitude of the applied load(s), and estimate the change in the resulting stress with depth (using stress distribution approaches such as provided in Poulos and Davis (1980)).
- Determine whether the soil is normally consolidated or over-consolidated.
- Calculate the total settlement based on the relevant settlement components.
- Compare the estimated settlement with tolerable settlements for the project.
- If the calculated settlement exceeds the allowed settlement, several alternatives exist including deep foundations (refer to next subsection), resizing the foundation elements, or improving the subsurface soils (refer to Section 15.4).

15.1.5 Computer Models

Several finite element and finite difference based commercial programs are available for bearing capacity and settlement analyses. Some of the most common commercial programs or software suites for bearing capacity and settlement include (shallow and deep foundations): FLAC, PLAXIS, SIGMA/W, GEO5, and LPILE. The selected computer program for any specific project shall be subject to approval by the Entity, including the proposed constitutive soil models.

15.2 Deep Foundations

When the soil conditions near the base of the structure are unsuitable for shallow foundations, deep foundations can be used to transfer the imposed loads to deeper, more competent strata. Deep foundations (piles, piers, and shafts) derive their resistance to loading from their base (end bearing) and/or skin friction acting along their shaft. The depth of the foundation element is substantially greater than the least lateral dimension (width), except in some instances in the case of very large pile-supported mat foundations. For a detailed treatment of foundation type and design, refer to NAVFAC (1986) and Bowles (1996). For design and construction procedures, refer to O'Neill and Reese (1999).

15.2.1 Pile Types

The most common deep foundation types used in Saudi Arabia include the following (refer to SBC):

Driven Piles

- Precast concrete piles
- Pre-stressed concrete piles

  Cast-in-Place Concrete Piles (with or without enlarged base)
- Drilled shafts
- Auger-cast (CFA) piles.

For temporary construction works, steel piles may be used if appropriate corrosion protection measures are implemented.

15.2.2 Design Considerations

The design for deep foundations shall be carried out in conformance with the provisions of the SBC as follows:

- General Requirements for Pier and Pile Foundations
- Driven Pile Foundations
- Cast-in-Place Concrete Pile Foundations
- Pier Foundations.

Key design issues for pile foundations, which shall be addressed in the GDR, include (refer to Poulos et al, 2001):

- Selection of the type of pile and installation method.
- Estimation of the pile size required to provide an adequate margin of safety against failure of both the supporting soil and the pile itself (in compression and tension).
- Estimation of the foundation settlement of non-piled structures adjacent to the piled structure. This will essentially be the differential settlement between adjacent piled and non-piled foundation units.
- Consideration of the effects of lateral loading, and the design of piles to provide adequate margin of safety against lateral failure of the soil and/or failure of the pile in bending, and an acceptable lateral deflection of the pile. This acceptable lateral deflection can be taken as the deflection resulting from half the lateral load that causes 25 mm of deflection (IBC).
- Consideration of ground movements that may occur due to external causes (such as soil settlement and swelling).
- Evaluation of pile performance based on pile loading tests (static and dynamic), and the interpretation of these tests to evaluate parameters that may be used to more accurately predict the performance of the pile foundation.
- The selection of deep foundation type is based on consideration of the types and magnitudes of loads to be supported, the depth required to achieve the needed capacity, the nature of subsurface materials to be penetrated, the desired life of the foundation, and the local expertise with the various methods of construction. ASCE (1993) provides information, foundation exploration and testing procedures, load test methods, analytical techniques, design criteria and procedures, and construction considerations for the selection, design, and installation of pile foundations. When designing piles, group action of piles with respect to pile interaction, lateral loading and settlement shall be considered.

15.2.3 Axial Capacity

Prediction of the axial capacity of deep foundations (i.e. driven piles and drilled shafts) is generally empirical. In the majority of cases, the method of installation will have an effect on the performance behavior of deep foundations. The capacity of the pile also depends on whether the pile is a displacement pile (e.g. a pre-cast concrete pile where considerable soil is displaced during pile installation) or a non-displacement pile (e.g. an H-section steel pile where minimal soil is displaced during pile installation). In soils, the skin friction resistance and end bearing capacity of the displacement pile will be higher.

The axial capacity of deep foundations generally consists of two elements: the capacity of the unit as a structural member in compression, tension, bending, and buckling, and the supporting capacity of the subsurface material. Unless sound and competent bedrock is encountered, the design of deep foundations shall be based on the capacity of the soil medium in which they are founded.

Pile capacities shall be evaluated for (a) a single pile, and (b) a group of piles, normally by using a group efficiency factor which is the ratio of the group capacity to the sum of the individual pile capacities. The group efficiency depends on the pile spacing and size as well as the behavior of the subsurface soil.

The ultimate axial static capacity of deep foundations is the sum of the ultimate shaft capacity (skin resistance) and the ultimate base capacity (end bearing resistance). The allowable pile capacity is obtained by dividing the predicted ultimate capacity by a suitable factor of safety, typically 2 for shaft capacity and 3 for base capacity. The axial static pile capacity analysis shall consider the soil type, drag loads (negative skin friction), pile group effects and pile group settlement, and uplift loads.

15.2.3.1 Solution Techniques

General static pile analysis methods are detailed in NAVFAC (1986) and Bowles (1996). Methods for predicting pile capacity using CPT data are described by Lunne et al (1997). Poulos et al (2001) provide discussions regarding some of the more recent methods employed in estimating static axial pile capacity. A detailed treatment of the design and construction of drilled shafts is provided by O'Neil and Reese (1999).

A detailed treatment of the evaluation of pile capacities can be found in Reese et al. (2005), FHWA (2006b), USACE (1991), Tomlinson and Woodward (2008), Fellenius (2017), Meyerhof (1976), and Hunt (1986). Pile capacity analysis shall comply with the requirements of SBC. For the LRFD design approach, the pile capacity analysis shall comply with the requirements of AASHTO.

Typically, the carrying capacity of soils will be assessed on the basis of:

- SPTs
- CPTs
- Laboratory strength tests (mainly cohesive soils)
- Wave equations analysis.

15.2.3.2 Axial Capacity of Piles in Cohesive (Clay) Soils

Piles installed in cohesive soils derive their axial capacities mainly from the shaft or skin friction resistance, with generally limited end bearing capacity. The capacity of piles founded in cohesive soils may be calculated using the total stress approach (based on the undrained shear strength, also known as the alpha-cu method) or the effective stress approach (also known as the beta method). The pile installation procedure, and the resulting response of the cohesive soils, shall be considered in the determination of the design parameters, notably the skin friction and end bearing resistance.

For piles installed in or through a cohesive deposit that is subjected to consolidation, or volume loss, negative skin friction and the resulting downdrag load shall be considered when calculating the pile capacity (Fellenius, 2017), and determining load testing requirements.

15.2.3.3 Axial Capacity of Piles in Granular Materials

The axial capacity of piles installed in granular soils is obtained through shaft resistance with, typically, a significant end bearing component. The pile installation procedure will inevitably change the density and strength of granular soils.

15.2.3.4 Axial Capacity from Driving Resistance

The capacity of driven (single) piles can be estimated from a dynamic analysis using data recorded as the pile is driven into the ground at the site. Numerous pile driving formulas have been developed; however, most of them predict pile capacity poorly and are site (soil) specific. The wave equation analysis is a better tool for determining the capacity of driven piles, and can be used to assess pile driveability and for hammer selection. The wave equation method applies wave transmission theory to determine the capacity developed by a pile and the maximum stresses that result within the pile during pile driving. Wave equation analyses can be performed using the GRLWEAP software from Pile Dynamics, Inc.

15.2.4 Lateral Capacity

The interaction of a pile-soil system subjected to lateral loading is a complex function of the nonlinear response characteristics of the soil and the typically linear response of the pile material. Detailed treatment of the theory and design methods for laterally loaded piles can be found in Reese and Van Impe (2011). The analysis of laterally loaded piles and pile groups may require finite element or finite difference based computer programs, such as LPILE which uses p-y curves (non-linear lateral load versus deflection curves) for each soil to estimate deflections and stresses versus depth in the pile.

15.2.5 Pile Capacity from Load Tests

Static load testing of deep foundations provides the most accurate method for characterizing load capacity. Static load tests are usually performed on test piles installed during the design stage, to check predicted capacities and to provide data to the pile installation contractor. Pile tests are also carried out as a check (i.e., as proof testing) on production piles installed by the contractor to verify the carrying capacity. These production pile tests are usually performed when the driving characteristics of the production pile changes from that of the test pile, or the contractor mobilizes a different pile driving hammer. Static compressive axial load tests of piles shall be carried out in accordance with ASTM D1143, static tension axial load tests of piles shall be carried out in accordance with ASTM D3689, and static lateral load tests of piles shall be carried out in accordance with ASTM D3966.

Static pile capacity of installed piles may be assessed using the Statnamic load test method which is faster and generally less expensive than the static load test. Guidance on this rapid-load pile load testing can be found in

ASTM D7383. High -strain dynamic load testing (pile dynamic analysis or PDA) in accordance with ASTM D4945 is becoming increasingly prevalent, either in conjunction with static load testing, or as a substitute.

When testing driven piles in predominantly clay soils, it is important to recognize the potential contribution of soil freeze to the measured capacity. After pile driving, excess porewater pressures built up in the soil during driving dissipate (taking several hours to several days) with a consequent increase in the skin resistance component of capacity (freeze). This freeze is permanent (unless the pile is redriven) and can be used in computing pile capacity. The capacity measured in the load test will be greater than that estimated using pile blowcount and the wave equation, since the blowcount reflects the high porewater pressures during driving.

15.2.6 Quality Assurance and Integrity Testing

Non-destructive testing (NDT) methods shall be used for Quality Assurance (QA) integrity testing of piles, especially drilled shaft foundations to identify anomalies. Examples of NDT techniques include Sonic Echo (SE), Impulse Response (IR), Ultra-sonic (US), Cross-hole Sonic Logging (CSL), and Gamma-Gamma Density Logging (GDL).

15.3 Retaining Walls

15.3.1 General

Retaining walls (retaining structures) provide lateral support for soil or rock. Earth retaining systems may be short term (such as for support of excavations) or long term (permanent retaining walls). For temporary construction works, steel elements (sheet piles, steel reinforcing elements, steel soil nails, etc.) may be utilized if appropriate corrosion protection measures are implemented. They may be either internally or externally stabilized systems. For permanent walls, steel shall be avoided, where possible. Types of permanent retaining walls include the following (modified from O'Rourke and Jones, 1990).

15.3.1.1 Externally Stabilized Systems

- In-situ walls (cantilevered, braced, or tied-back)
- Sheet pile (concrete)
- Soldier pile (concrete with timber lagging)
- Cast in-situ (slurry, secant, tangent)
- Soil cement

15.3.1.2 Gravity Walls

- Massive
- Cantilever
- Counterfort and buttress
- Gabion
- Crib wall
- Bin
- Cellular cofferdam

15.3.1.3 Internally Stabilized Systems

Reinforced soils

- Glass fiber
- Geosynthetic reinforcement (geogrids and geotextiles)
- Geocells

In-situ reinforcement

- Reticulated micropiles

Hybrid systems

- Reinforced segmental (masonry concrete) retaining walls, including mechanically stabilized earth (MSE) walls

15.3.2 Design Considerations

15.3.2.1 Factors to be considered in selecting the type of retaining structure shall include (refer to Bathurst and Jones, 2001):

- Soil and groundwater conditions
- Location of the proposed retaining wall with respect to other structures
- Height of the proposed retaining wall and the topography of the ground
- Limitations on ground movements during construction and service
- Availability of materials
- Appearance (aesthetics) requirements
- Time available for construction (speed of construction)
- Anticipated design life (durability) and frequency of maintenance
- Experience and familiarity of the construction contractor with respect to the construction technique
- Environmental issues.

The design of retaining structures shall consider wall performance criteria, wall selection, construction methods (staging) and associated ground movements. Retaining walls in general shall be designed to ensure stability against overturning, sliding, bearing capacity failure, excessive settlement and global failure. Appropriate estimates of the lateral earth pressures (and hydrostatic pressures) behind and in front of the wall are critical to the design. For MSE walls and segmental retaining walls (Modular Walls), additional design checks, such as internal stability checks, shall be performed.

Detailed methods of analysis for retaining structures, including conventional retaining walls and reinforced soil walls can be found in Chapter 3 of NAVFAC (1996), Bathurst and Jones (2001), and Poulos et al (2001). Typical design and construction aspects of retaining structures can be found in Day (1999).

Minimum requirements regarding the design and construction of retaining walls can be found in Chapter 7 of SBC, which specifies the required minimum factors of safety (FS) for retaining walls as follows:

- With respect to bearing capacity: FS $\geq$ 3
- Against sliding: FS $\geq$ 1.5 (cohesionless soils); FS $\geq$ 2.0 (cohesive soils)
- Against overturning: FS $\geq$ 1.5
- Against rotational failure (deep-seated sliding): FS $\geq$ 2.0.

15.3.2.2 Walls shall be designed in accordance with FHWA and AASHTO LRFD design approaches. Design guidelines on Modular Walls can be found in NCMA's TEK (technical) publications.

15.3.2.3 Where steel tie-backs or anchors are used in retaining walls, appropriate corrosion protection measures, including concrete mix design, proper coatings, etc., shall be included in the design detail.

15.3.3 Soil-Structure Interaction

In addition to the design considerations described above, soil-structure interaction shall be considered in retaining wall design to better predict wall performance and to avoid unnecessary over-conservatism. The nature and direction of relative movement between the wall and backfill soil, and the wall-to-soil shear shall be considered.

Typically, friction forces will be mobilized between the soil and wall with very small movements. Consideration shall be given to the effects that such forces will have on the wall safety and overall performance of the wall.

15.4 Soil Improvement Techniques

15.4.1 General

15.4.1.1 Soil improvements are used for:

- Improving stability and bearing capacity by increasing the relative density and/or shear strength of soils
- Controlling soil deformation by reducing compressibility and speeding up consolidation
- Reducing soil permeability to reduce water flow
- Improving homogeneity of soil to reduce differential settlement.

15.4.1.2 Site soil conditions have a profound influence on the success of ground improvement techniques. Adequate geotechnical site exploration (see Part B) must be completed to characterize the subsurface conditions. Evaluation of the appropriateness of a specific soil improvement technique for a given site and application requires consideration of the following factors (refer to Holtz et al, 2001):

- The operational criteria for the facility: stability requirements, allowable total settlement and rate of settlement, seepage criteria, durability and maintenance requirements, etc. These will establish the level of improvement required in terms of strength, stiffness, hydraulic conductivity, etc.
- The area, depth and total volume of soil to be treated or improved
- Soil type and its initial properties; depth to groundwater
- Availability of materials, such as sand, gravel, water, admixtures, etc.
- Availability of equipment and required skills
- Construction: environmental factors such as site accessibility and constraints, waste disposal, erosion, potential water pollution, and effects of and on adjacent facilities and structures
- Local experience and preference; cultural considerations
- Time available
- Cost.

15.4.1.3 In broad terms, soil improvement techniques include:

Foundation soil improvement:

- Lightweight fill
- Removal and replacement
- Consolidation
    - Physical stabilization and densification (vibro-replacement stone columns, vibrocompaction, vibroflotation, deep dynamic compaction, deep soil mixing, etc.)
    - Dewatering and groundwater control
    - Preloading by surcharge
    - Preloading by vacuum
    - Consolidation with vertical (sand) drains
    - Consolidation with prefabricated vertical (wick) drains
    - Chemical/thermal/electrical stabilization including lime and cement stabilization.

Stabilization of slopes:

- Dewatering and groundwater control
- Ground anchors and tiebacks
- Soil nailing

- Reinforced soil system (RSS) with the use of geogrids
- Micropiles, root piles and pin piles
- Biotechnical stabilization.

15.4.2 Local Practices

Soil improvement techniques commonly practiced on problem soils in Saudi Arabia include the following (SBC):

15.4.2.1 For expansive soils:

- Removal and replacement
- Stabilization by chemicals
- Installation of moisture barriers
- Pre-wetting (note: for pre-wetting, strength loss due to wetting shall be evaluated).

15.4.2.2 For collapsible soils:

- Compaction
- Pre-wetting
- Vibroflotation
- Chemical stabilization.

15.4.2.3 For Sabkha soils:

- Stone columns
- Preloading
- Vibroflotation
- Lime stabilization.

Mirza (1992) describes the use of concrete shaft foundations to compact dune sands while Mubarki and Alawaji (1995) describe the use of lime to improve Sabkha soils.

Common ground improvement methods on normal soils in Saudi Arabia include vibrocompaction, dynamic compaction, deep soil mixing and modular columns. The method of ground improvement selected for a site shall be approved by the Entity.

Soil improvement techniques are rapidly advancing, with new methods and enhancements to old methods under development around the world. Engineers tasked with selecting or evaluating soil improvement methods should look for potentially new, innovative solutions.

15.4.3 Verification and Evaluation of Soil Improvement Techniques

Verification and evaluation of the constructed soil improvement system is essential to ensure satisfactory construction and performance of the project. Hence, a well-planned quality control and conformance testing program during construction is essential. In this regard:

- A well-planned inspection, testing, and instrumentation plan shall be put in place.
- Well-trained and competent field inspection personnel are needed during construction.
- The feasibility of instrumentation and field measurements may influence the selection of soil improvement alternatives.

- For problematic soils (such as expansive and collapsible soils), improvement in strength and/or compressibility may be gradual. Careful monitoring during construction is important to verify that improvement is progressing satisfactorily and/or is occurring uniformly.
- In some applications, measurements of movements and porewater pressures are necessary to verify a particular soil improvement method.
- Post-construction monitoring may be required in some projects, to verify long-term performance.
- Valuable sources of information on geotechnical instrumentation and monitoring are found in Dunnicliff (1993) and Hanna (1985).

16.0 GROUNDWATER AND SEEPAGE ANALYSIS

16.1 General

Groundwater and seepage conditions shall be assessed to determine their potential influence on all geotechnical projects, and conversely, that the development does not adversely impact the groundwater regime, for the following reasons:

- Groundwater can cause or contribute to failure or reduced factor of safety because of excess saturation and consequent reduction of soil strength, and increase of seepage pressures and uplift forces.
- In some localities, groundwater contains elevated concentrations of pollutants or may contain constituents in concentrations sufficient to make it aggressive and cause damage to construction materials such as concrete and steel. In short, groundwater can affect the design, performance, and constructability of project elements.

16.2 Groundwater

16.2.1 Assessment of Groundwater

Assessment of groundwater conditions involves determination of groundwater levels and pressures, hydraulic conductivity, and water quality in terms of chemical composition. The following shall be provided:

- The A/E Lead Discipline Engineer shall identify design groundwater levels and determine the range in seasonal fluctuations. Groundwater levels and pressures may be measured in existing wells, in open borings, and/or in specially installed monitoring (observation) wells and piezometers (see Section 7.3). If the geology or the groundwater regime is complex, input from an engineering geologist and/or a hydrogeologist shall be used to assess the piezometric conditions for design.
- The A/E Lead Discipline Engineer shall determine the hydraulic conductivity of soil or rock strata. Hydraulic conductivity shall be determined by means of various types of seepage, pressure, and pumping tests (see Section 7.4). Hydraulic conductivity estimates will be used to provide an assessment of groundwater seepage, yield of wells, buoyancy, piping potential, slope stability, and soil susceptibility to liquefaction. If required, a plan for groundwater control and temporary shoring design will be proposed.
- Recommendations for instrumentation and monitoring, with frequency and required design water levels and/or pressures to maintain stability, shall be provided in the GDR.

16.3 Seepage

16.3.1 Assessment of Seepage

Seepage is the movement or percolation of water (usually a small quantity) into or through soil deposits or soil structures such as embankments and hydraulic structures. Seepage can also occur during excavation below groundwater level through the sides and/or bottom of an excavation. Seepage shall be assessed in order to mitigate problems during construction. A geotechnical investigation and design report shall consider seepage conditions and potential impacts on a given project, including reduction in factor of safety and mitigation measures.

16.3.2 Seepage Analysis

- Flow Net. This is a graphical method used to study two-dimensional flow of water through a soil, i.e., the graphical solution of Laplace's Equation.
- Flow net sketching. A comprehensive guide to flow net sketching is included in Cedergren (1997). Flow net sketching shall only be used to provide an initial assessment of the flow regime and flow quantities, with subsequent analysis using finite difference or finite element methods.
- Uplift pressures and piping. Seepage analysis shall be undertaken to assess hydraulic uplift forces on structures as well as piping problems. These two phenomena are caused by seepage forces that result from drag forces (viscous friction) between water and solid particles. Uplift pressure and piping assessment are of vital importance in stability analyses of foundations, retaining walls and earth structures subject to the action of flowing water (seepage), and hence shall be considered in any geotechnical investigation of a project.
- Seepage analysis of anisotropic material. Even though the continuity equation and the resulting flow nets are originally derived for isotropic, homogenous ground and steady-state conditions, more complex flow regimes including isotropic soils shall be considered. Flow nets can be utilized for anisotropic conditions and assessment, but computer programs using finite difference or finite element methods (see below) are available to solve complex seepage problems.
- Numerical Analysis. Numerical solutions of Laplace's Equation are possible by use of difference methods which can be programed in computer spreadsheet applications for simple boundary conditions.

16.3.3 Computer Applications in Seepage Analysis

The A/E Lead Discipline Engineer shall be able to apply the computer programs noted below, as required, in addition to sketching flow nets to provide approximate comparisons, or establish anticipated results, of the numerical analysis. Some of the currently available computer programs for seepage and groundwater flow analysis include SEEP2D, SEEP/W, SVFLUX, and PLAXIS 2D PlaxFlow (PLAXIS). These programs are generally based on finite element methods and most can handle three-dimensional problems. Use of these programs shall be subject to approval by the Entity.

16.3.4 Seepage Analysis Factors of Safety

For seepage analyses, target factors of safety shall be determined from published manuals and sound engineering judgment based on the specifics of a given project. Guidance documents on seepage analysis and the minimum factor of safety for piping stability analysis include USACE (1986b) and NAVFAC (1986). The target factors of safety shall be included in the GDR and be subject to the review and approval of the Entity.

The factor of safety for buoyancy shall be at least 1.2 for the highest anticipated buoyant effects. Where the dead weights calculated are well established, such as for concrete components, this factor may be reduced to 1.1.

16.3.5 Groundwater Control

Where groundwater is anticipated to cause problems for the construction and performance of a given development project, the GDR shall include calculations and analysis, and recommend groundwater control measures appropriate for the site. The recommended measures shall be subject to the review and approval by the Entity.

17.0 EARTHWORK CONSIDERATIONS

17.1 Fill and Subgrade Assessment

The GDR shall classify and provide an assessment of materials from site excavations as to their suitability and acceptability for use as fill materials for the project, or whether to dispose of these materials off site. In addition, the GDR shall include an assessment of the subgrade soils to support pavements.

The excavated materials shall be classified in accordance with the USCS using appropriate laboratory test methods, as described in ASTM and AASHTO standards, to determine strength, compressibility and compaction criteria. The GDR shall include the following:

- Assessment of subgrade soils to support pavement
- Assessment of ease of excavation of the in-situ soil and rock
- Suitability of excavated material for re-use as common and/or structural fill
- Environmental impacts of cutting and filling operations
- Bulking potential of the excavated soil
- Recommendations for the earthworks program including requirements for a cut/fill balance
- Where there is a deficit of available fill material, assessment of available sources of fill materials from existing on-land borrow sources and the potential for obtaining and using dredged material
- Methods for utilizing surplus or unsuitable material on site to mitigate disposal costs
- Disposal of surplus materials (both surplus suitable and unsuitable)
- Earthwork monitoring and testing requirements
- Earthwork safety, including stability and maintenance.

17.2 Laboratory Testing of Fill and Subgrade Materials

The classification and acceptability of excavated materials and subgrade soils shall be determined by undertaking the following tests using ASTM methods:

- California Bearing Ratio (CBR)
- Maximum density and optimum moisture content of soils
- Particle-size analysis of soils
- Liquid limit, plastic limit, and plasticity index (Atterberg limits) of soils.

17.3 Evaluation of Fill and Subgrade Materials

The excavated materials shall be evaluated against specified limiting criteria provided in the GDR and required by the project. The GDR shall confirm whether:

- The excavated material is suitable for re-use based on moisture content, plasticity, and/or shear strength.
- The subgrade materials are trafficable and suitable for the support of pavement loads.
- The excavated material can be compacted to specified limiting criteria.
- The earthworks will be stable during and following construction.
- Excessive settlement or heave will, or will not occur.

17.4 Requirements for Fill Selection, Placement, Compaction and Testing

Structural fill shall be used under all structures and roads. Common fill shall be used to raise site grade in areas not supporting structures and roads. All common and structural fill must be approved by A/E Lead Discipline Engineer and follow project specifications. This common and structural fill shall be free of any significant amounts of organic matter, debris or waste, with a maximum size not exceeding 75 mm.

The moisture content of all imported fill materials shall be determined at the source prior to delivery to the fill area. Moisture conditioning of fill materials may be necessary to ensure that the material can be compacted in accordance with the specifications. Such moisture conditioning can be achieved at the fill area after the soil is spread in loose lifts for compaction. The compaction requirements given below specify using the modified Proctor test (ASTM D1557) as the basis of comparison with field density and moisture content measurements. The modified Proctor test is recommended for use under major, critical, and/or heavily-loaded structures. For minor, less critical, and/or lightly-loaded structures, the standard Proctor test (ASTM D698) may be used.

17.4.1 Structural Fill

Structural fill, including imported structural fill, shall be a well-graded granular material with no more than 30% fines. All structural fill shall be compacted to at least 95% of the modified Proctor maximum dry density, and shall be placed in maximum 300-mm loose lifts when compacted with heavy vibratory equipment, or 150-mm loose lifts when using hand-operated equipment. The soil moisture content shall be within 3% of optimum when compacted.

17.4.2 Common Fill

Common fill, including imported common fill, can be granular or cohesive; if cohesive, it shall not have a plasticity index greater than 30. All common fill shall be compacted to at least 90% of the modified Proctor maximum dry density, and shall be placed in maximum 300-mm loose lifts when compacted with heavy vibratory equipment, or 150-mm loose lifts when using hand-operated equipment. The soil moisture content shall be within 3% of optimum when compacted.

17.4.3 Testing Compacted Fill

Frequency of testing of compacted fill shall be approved by the Entity. As a general guideline, the density and moisture content of the compacted fill should be tested at the frequency of 1 test per 1,000 m^2 for large areas, with at least one test per lift and one test per shift. For fill placed by volume (e.g., backfilling trenches), the compacted fill should be tested at the frequency of 1 test per 300 m^3, again with at least one test per lift and one test per shift.

Field density and moisture content testing can normally be performed using a nuclear gauge (ASTM D6938). If this equipment is not approved by the Entity or is not available, the sand-cone method (ASTM D1556) or the rubber balloon method (ASTM D2167) can be used.

18.0 SLOPE STABILITY ANALYSIS AND DESIGN

18.1 General

The stability of natural and manmade (engineered) slopes shall be evaluated for potential failure and its effect on the safety of people and property as well as on the usability and value of the area. Engineered slopes are created for transportation routes (e.g., embankments), urban developments (e.g., levees), dams, mining and municipal waste disposal, and other construction activities requiring the formation of excavations and building of slopes. Where the available space does not permit stable slopes to be created, retaining structures (Section 15.3) may be required.

Several references are available on slope stability analysis and design, including the following: USACE (2003), FHWA (2001), Chapter 5 of NAVFAC (1986), and Duncan and Wright (2005).

18.2 Types of Slope Movements and Modes of Failure

In general terms, the following slope movements or failure types shall be considered:

- Rockfalls or topples. Falls are slope failures consisting of soil or rock fragments that drop rapidly down a slope, bouncing, rolling, and that may even become airborne along the way. A topple is similar to a fall, except that a topple begins with a mass of rock or stiff clay rotating away from a vertical or near-vertical joint or fissure.
- Surficial slope stability. Shallow failure surfaces usually occurring within 1 to 1.5 m depth, generally parallel to the surface. This usually occurs in non-cohesive soils with relatively long slopes, after prolonged rainfall or during a heavy rainstorm.
- Deep seated slope failure. This usually involves a rotational failure surface (of an entire slope) and tends to occur in predominantly cohesive soils.
- Slides (Landslides). Slope movement involving one or more adjacent blocks of earth that move downslope by shearing along well-defined surfaces or thin shear zones. Slides may be rotational, translational or compound.

- Flow (Debris Flow). Involves lateral movement of earth having the characteristics of viscous fluid; the actual consistency of the moving mass may vary from wet to dry.
- Creep is an imperceptibly slow, relatively continuous downward movement of earth material.

18.3 Factors that Cause Slope Failure

- Geometric factors - slope height, angle and shape, disposition of different material types within the slope
- Presence of discontinuities or planes of weakness
- Presence of fluid (air and water) in the soil (groundwater and seepage conditions)
- Process of weathering or ageing, combined with possible long-term fluid pressure changes
- Loading - self-weight and external loads (from foundations, stresses induced dynamically such as from earthquakes or dewatering)
- Construction - additional weight of a constructed embankment or cutting of an existing slope

18.4 Actions Prior to Slope Stability Analyses

- Site investigation and sampling
- Laboratory testing
- Development of soil profile and characterization of soil strength (design soil strength)
- Determination of groundwater location
- Establishing two-dimensional idealization of the cross-section, including the surface geometry and subsurface boundaries between various materials
- Selection of loading conditions

18.5 Loading Conditions

The following loading conditions shall be considered in stability evaluations:

- During construction and end-of-construction
- Short-term, undrained condition including sudden drawdown or inundation
- Long-term, drained (steady-state seepage) condition
- Seismic condition.

18.6 Slope Stability Analysis Methods

18.6.1 Limit Equilibrium Methods

Limit equilibrium methods (LEMs) are the most common methods used to assess slope stability. These methods are based on the principle of static equilibrium in which the summation of moments, vertical forces, and horizontal forces are zero along an assumed potential failure (slip) surface. LEM is the conventional method of stability assessment and shall always be considered. All LEMs result in the computation of a factor of safety which is the ratio of the available shear strength to the shear stress required for equilibrium along the potential slip surface. The most common methods in LEM are listed as follows:

- Design (Stability) Charts
- Infinite Slope Method
- Ordinary Method of Slices (Fellenius's Method)
- Bishop's Simplified Method

- Modified Swedish Method
- Janbu's Simplified Method
- Spencer's Method
- The Wedge (Sliding Block) Method
- Morgenstern-Price Method.

A good summary of these methods can be found in Chapter 10 of Winterkorn and Fang (1975).

Design charts shall only be utilized for preliminary analysis and shall then be checked by undertaking detailed analysis using more rigorous and accurate methods.

For hand calculations, the factor of safety shall be determined using methods approved by the Entity. These may include use of the Bishop's Simplified Method, Janbu's Simplified Method, Modified Swedish Method or Wedge Sliding Method – methods that lend themselves to hand calculations, albeit time consuming. Currently most slope stability evaluations are performed using computer programs which can handle more complex analysis, as described later.

18.6.2 Continuum Methods (Finite Element, Finite Difference)

These methods, especially the finite element method, provide estimates of the deformations caused by the applied loads, including deformation patterns. Both finite element and finite difference methods require considerably more time and effort beyond that required for limit equilibrium analyses, and need additional data related to stress-strain behavior of materials. As such, these methods are generally used only in complex situations. The use of finite element and difference analyses are generally not justified for the sole purpose of calculating stability factors of safety.

18.6.2.1 Probabilistic Methods (Monte Carlo Simulation)

Probabilistic slope stability methods consider uncertainties in the values of the variables and the effect of these uncertainties on the computed values of the factor of safety. Some slope stability computer programs contain Monte Carlo simulation features. For large projects where the uncertainties potentially have severe impacts on the evaluated factor of safety values, probabilistic methods shall be considered. Descriptions of techniques for probabilistic analyses and their application to slope stability studies can be found in USACE (1999). Statistical characterization of the input parameters is discussed by Allen et al. (2005).

18.6.2.2 Limit State Methods

The limit state analysis method (LRFD) for slope stability analysis is gaining widespread acceptance and shall be considered in design when appropriate. The FHWA research work by Loehr et al. (2006) should be reviewed for additional information.

18.7 Factors of Safety and LRFD Procedures

The factor of safety to be used in a slope stability evaluation depends on:

- The method of stability analysis used
- The method used to determine the shear strength
- The degree of confidence in the reliability of subsurface data
- The consequences of failure.

Recommended factors of safety for different applications can be found in FHWA (2001, 2006a) and USACE (2003) and as noted below.

As per AASHTO requirements, the recommended minimum factor of safety for embankment side slopes shall not be less than 1.5 in fine-grained soils and 1.25 in granular soils, but for granular soil this shall be increased to a minimum of 1.3 where slope failure would cause significant damage and remediation costs, such as slopes adjacent to bridge abutments or retaining structures, and where failure would impact regional and critical transportation and communication links.

The factor of safety shall be adjusted to account for the detail of the subsurface investigation exploration, as per AASHTO. Where detailed exploration is available, with soils and rock parameters and groundwater levels defined by in-situ and laboratory tests, the required minimum factor of safety is 1.3 for embankment side slopes and 1.5 for slopes supporting abutments or abutments above retaining walls. The factors of safety shall be increased to 1.5 and 1.8 respectively where there is limited exploration.

For seismic loading, minimum factor of safety is normally taken as 1.1.

For slope stability analysis using LRFD procedures, design shall be consistent with the AASHTO LRFD Bridge Design Specifications (AASHTO).

18.8 Use of Computer Programs in Stability Analysis

Currently, most slope stability evaluations are carried out using computer programs. These provide a rapid and efficient analysis of a wide variety of slope geometries and loading conditions. Computer programs allow the analysis of complex and irregular failure surfaces, seismic loads, surcharge loads, tieback loads, and various other factors. Computer programs that are used to undertake parametric studies can be performed by varying parameters of interest.

Several two-dimensional slope stability programs are available; some complex computer programs are also available for three-dimensional slope stability analysis. For routine slope stability analysis problems, two-dimensional slope stability programs shall be used at a minimum. Commonly available commercial programs for slope stability analysis include, but are not limited to the following: SLOPE/W, ReSSa, SLIDE, SVSLOPE, UTEXAS, FLAC/SLOPE, GEO5, and PLAXIS 2D. CLARA-W (Clara-W) includes three-dimensional slope stability analysis capability. The use of the software to the specific project, together with the proposed slope stability modelling and method of analysis, shall be subject to approval by the Entity.

As noted in the SLOPE/W User's Manual, "Limit equilibrium analysis applied in practice should as a minimum use a method that satisfies both force and moment equilibrium, such as Morgenstern-Price or Spencer methods. With the software tools now available, it is just as easy to use one of the mathematically more rigorous methods than to use the simpler methods that only satisfy some of the statics equations."

18.9 Improving the Stability of Slopes

If the computed factor of safety does not meet the design requirements of a project, methods of improving the stability of the slope shall be considered. The methods used to stabilize slopes should be the most economical and shall consider the following factors:

- Availability and cost of materials
- Construction schedule
- Grading requirements
- Availability of space (right-of-way issues).

Common methods of mitigating slope stability problems are:

- Relocation (if embankment or cut)
- Reducing loads by excavation (removing soil) at the top of slope, i.e., unloading the top of the slope
- Surface drainage control by decreasing infiltration to unstable slope
- Lowering the groundwater table
- Flattening or reducing the slope angle
- Creating a break in the slope by creating benches (benching)
- Construction of stability berms

- Freezing or electro-osmosis
- Reinforcing slope with geogrids.

19.0 TRENCHLESS METHODS AND MICROTUNNELING

19.1 General

- Includes techniques for utility line installation, replacement, rehabilitation, etc. with minimum excavation from the ground surface.
- Generally requires surface excavation at the entry and exit locations.
- The design of the alignment for trenchless and microtunneling methods (the drill path) must be developed taking into account the geological setting for the project and geotechnical and hydrogeological issues at the proposed crossing location.
- From a geotechnical perspective, a number of issues should be taken into account, including the distribution and characteristics of the surficial overburden deposits.
- Several resources are available for guidance in carrying out trenchless installations, including the International Society for Trenchless Technology (ISTT) (http://www.istt.com/), the North American Society for Trenchless Technology (NASTT) (http://www.nastt.org/), and Iseley and Gokhale (1998).

19.2 Typical Applications

- Water/wastewater systems
- Gas, petroleum and chemical pipelines
- Electrical and communication networks
- Access ways and other small diameter tunnels

19.3 Common Trenchless Methods

- Microtunneling
- Horizontal Directional Drilling
- Pipe Jacking
- Pipe Ramming
- Auger Boring

The suitability of a given trenchless method to a specific project will generally be governed by the site conditions which will be identified during the geotechnical investigation, critical for the success of any trenchless installation.

19.4 Risk Considerations

Trenchless applications are associated with certain risks and a considerable amount of unpredictability. The A/E's design team shall identify and evaluate the potential risks during the planning stages of the project and develop plans to minimize these risks. The team shall include: the proponent, engineering, geotechnical and environmental consultants, and input from specialist tunneling contractors. It is essential that close consultation with regulatory agencies and land authorities be established during the planning process.

Risks associated with trenchless methods generally fall into three categories (CAPP, 2004):

- Regulatory Risk – this is usually encountered during application/approval.
- Ground and Construction Risk – this can be minimized by conducting sufficient planning and an adequate geotechnical investigation. This risk includes the impact of the selected trenchless method on existing infrastructure being crossed by the trenchless operation.

- Operations Risk – this is generally considered less important from the geotechnical engineering perspective. Potential geotechnical issues include subsidence at entry and exit locations.

Trenchless procedures shall be carried out in compliance with ARAMCO, SAR, SEAPA, SEC, STC and SWCC, and shall be reviewed by the Entity.

19.5 Assessment and Evaluation of Project Options

An assessment of the suitability and reasons for adopting trenchless construction techniques for a selected alignment or alignment options shall be included in the GDR, together with supporting design parameters and criteria, calculations, and guidance for development of the Contract documentation. The assessment shall provide and take into account:

- The geology and the hydrology as characterized by the site investigation.
- The geotechnical design parameters and load cases.
- The design requirements, codes and reference standards to be adopted.
- Trenchless methodologies (and other methodologies as appropriate such as cut-and-cover) appropriate for the prevailing ground, hydrogeological, environmental and site constraints.
- Temporary and permanent support systems (such as starter and receptor pits and lining type).
- Ground and groundwater treatment measures such as grouting and dewatering, and assessing their impact on third parties and the environment (for example groundwater lowering leading to settlement).
- Ground loss and settlement at the ground surface and within the influence distance of the works, and their impact on existing structures and services, such as buildings, buried and surface utilities, and underground structures.
- Mitigation and control measures to assess the impact of the works, including specifying of pre-condition and post-construction surveys, limiting settlement and deflection criteria (with ‘alert’, ‘action’ and ‘alarm’ levels that trigger required responses by the construction contractor in the event of exceedance of these values), and instrumentation and monitoring plans for the detailed assessment of movement of buildings, underground structures, etc., against the limiting movement criteria.
- Details of design and supporting information to be provided by the construction contractor during the tender and construction phases of the works.

20.0 TUNNELING

20.1 General

- Tunnels are constructed to exclude the materials they pass through and are generally required to withstand high pressures.
- Tunnels normally require the provision of shafts at the entry and exit locations.
- The design of the alignment is developed taking into account the geological setting for the project and the hydrogeological conditions.
- From a geotechnical perspective, a number of issues should be taken into account, including the distribution and characteristics of the surficial overburden deposits, presence of rock, discontinuities, size and length of tunnel drive, and type of tunnel.
- Several resources are available for guidance in carrying out tunneling works including publications such as FHWA (2009), Technical Manual for Design and Construction of Road Tunnels.
- Various types of tunneling methods are available including open face without shield, road header, drill and blast, New Austrian Tunneling Method (NATM) with sprayed lining, tunnel boring machine (TBM), cut-and-cover, and immersed tube.

20.2 Selected Tunnel Types

20.2.1 Cut-and-Cover Tunnels

- Typically adopted for limited lengths and where the tunnel is less than 12 m deep below grade, although much deeper cut-and-cover tunnels have been built.
- Constructed using open excavation.
- Temporary support of excavation is utilized during construction operations such as braced sheet pile walls, tied-back concrete diaphragm walls, etc.
- Round or horseshoe shape is the most efficient tunnel configuration but rectangular shapes may also be utilized.

20.2.2 Machine Constructed Tunnels

- Typically adopted where site constraints do not permit the use of cut-and-cover methods, and where ground conditions permit machine construction which, during the planning and design phase, is determined to be cost effective.
- Mostly achieved using TBMs, although road headers are also included as machine construction.
- May require temporary and permanent support to maintain stability and to mitigate future maintenance.
- Typically, a round or horseshoe shape is the most efficient tunnel configuration, but rectangular shapes may also be utilized, notably where a box jacking (tunnel jacking) technique is used.

20.2.3 NATM Tunnels

- This method applies to tunnels in rock and is most suited to situations where there are high in-situ pressures.
- The tunnel is excavated and supported sequentially, with the sequences designed to best address the conditions encountered.
- The strength of the rock around the tunnel is mobilized by allowing controlled deformation of the ground.
- Initial internal support is provided by shotcrete, welded wire reinforcement, etc.
- Permanent support is typically a cast-in-place concrete lining over a waterproof membrane.

20.2.4 Drill and Blast

- Limited to tunneling in rock.
- Drill a pattern of small holes, load with explosives and detonate to create an opening in the rock.
- The drill hole pattern takes into account the rock type, existing rock fractures and the desired final tunnel profile.
- Rock disturbance from blasting typically extends 1 to 2 m into the rock.
- Reduce disturbance using controlled blasting techniques.
- Vibrations from blasting can affect the stability of the tunnel as well as adjacent underground and surface structures.

20.2.5 Jacking in Soil

- Box jacking is carried out within a face shield, and may require face support.
- Temporary support is provided within a compartmentalized face shield which is advanced incrementally followed by jacking of the box structure.

- Additional ground and face support in the form of grouting may be required to stabilize soils, to facilitate arching at the face, and to mitigate face and resulting ground loss and settlement.
- To mitigate drag effects, notably at the top and bottom of the box, an anti-drag system is used to effectively de-couple the box from the ground.
- The box is advanced using thrust jacks against a jacking or reaction slab.

20.3 Types of Tunneling Projects

20.3.1 Water Tunnels

- Usually intake structures or aqueducts
- Tunnels with little or no internal pressure are horseshoe in shape
- Tunnels under internal pressure are circular
- Diameters typically range from about 1.8 m to 15 m
- Concrete lining varies in thickness from about 150 mm to 900 mm depending on external pressure, tunnel design (drained or undrained), soil type, rock stability, and tunnel size.

20.3.2 Sewage and Drainage Tunnels

- Sewage is highly corrosive requiring high quality concrete.
- Generally, these tunnels are close to grade, utilizing cut-and-cover excavation techniques.
- Round shape is most efficient for keeping constant velocity at low flows, thus preventing settling of solids.

20.3.3 Road and Rail Tunnels

- Requires special design considerations for ventilation and safety, such as provision of assisted ventilation, provision for removing refuse, and fire protection measures.
- Generally these tunnels are close to grade, utilizing cut-and-cover excavation or box jacking techniques.
- Round or horseshoe shape is the most efficient tunnel configuration but rectangular shapes may also be utilized, especially where box jacking techniques are used.

20.4 Tunneling in Rock and Soil

20.4.1 Tunneling in Rock

The most important geological items to be determined by the A/E Lead Discipline Engineer in rock tunneling are as follows:

- Type and strength of rock
- Presence of voids, fissures and rock fractures
- Presence of water
- Potential hazards such as gas pockets, weak horizons, etc.

Rock tunneling may include stretches where temporary and permanent shoring will be required:

- Structural steel is normally economical to install due to strength and ease of installation.
- Spacing of shores range from about 1.2 m to 1.8 m, depending on the rock loads.
- Blocking is erected as soon as soon as possible to wedge the bottom of the rock with steel shores.

- Lagging is installed between shores.
- Other support systems include rock bolts, mesh and shotcrete, or reinforced concrete segmental rings.
- In rock tunnels, depending on the stand-up time and nature of the rock, both the temporary and permanent support measures may be omitted, or only localized support measures are required.

20.4.2 Tunneling in Soft Materials

The most important geological items to be determined by the A/E Lead Discipline Engineer when tunneling is soils includes:

- Nature of the soils (strength, relative density, plasticity, moisture content, etc.)
- Presence of free water
- Potential hazards such as gas pockets, weak horizons, presence of expansive soils, etc.

Tunneling in soils typically requires installation of a temporary and provision of a permanent support system, as follows:

- Temporary tunnel shoring may utilize steel supports, mesh and shotcrete, or a reinforced concrete segmental lining installed behind the TBM.
- Without a TBM, forepoling may be required to facilitate tunnel drives. Forepoles are typically made of steel or glass fiber reinforced plastic. They are either driven or drilled and grouted into the soil ahead of excavation.
- Grouting may be required to stabilize soils and reduce inflows.

20.5 Tunnel Lining

20.5.1 General

The tunnel lining provides permanent support and shall be installed as detailed below:

- Tunnels in sound rock or rock which does not deteriorate under exposure to the atmosphere do not need a lining.
- Where rock that deteriorates or unsound rock is present, lining is required.
- Highway and railway tunnels, unless otherwise directed, shall include a permanent lining.
- Lining shall be constructed from concrete materials.

20.5.2 Concrete Lining Details

The concrete lining shall be constructed as follows:

- By pumping concrete between the rock/soil and the temporary steel plate forms, leaving no voids.
- By providing segmental precast lining elements installed behind the tunnel shield or within temporary tunnel supports, with grouting behind the segmental lining to leave no voids.
- The concrete lining shall be designed for the earth pressure, hydrostatic conditions and prevailing loads cases. Concrete thickness and reinforcement is determined by tunnel geometry, rock loading, and the weight of concrete.
- Reference to the GDR is required to analyze the requirements of tunnels with no lining support, or where alternative construction methods such as NATM are proposed.

20.6 Tunnel Shafts

- Shafts serve as starting points for excavation of tunnels.
- Tunnels shall utilize shafts as ventilation for the structure where required by the design codes and standards.
- Shaft design approach shall be similar to the approach used for tunnel design.
- Shallow shafts may be installed using the cut-and-cover method, and utilizing support of excavation with sheet piles or soldier piles with lagging, as appropriate.

20.7 Assessment and Evaluation of Project Tunnel Options

An assessment of the suitability and reasons for adopting tunneling construction techniques for a selected alignment or alignment options shall be included in the GDR, together with supporting design parameters and criteria, calculations, and guidance for development of the Contract documentation. The assessment shall provide and take into account:

- The geology and hydrology of the alignment as characterized by the site investigation.
- The geotechnical design parameters and load cases.
- The design requirements, codes and reference standards to be adopted.
- Tunnel methodologies as appropriate for the prevailing ground, hydrogeological, environmental and site constraints.
- Temporary and permanent support systems (such forepoling, grouting, rock bolting, mesh and shotcrete, and design of the permanent lining system).
- Ground and groundwater treatment measures such as grouting and dewatering and assessing their impact on third parties and the environment (for example groundwater lowering leading to settlement).
- Ground loss and settlement at the ground surface and within the zone of influence of the tunnel works, and their impact on existing structures and services such as buildings, buried and surface utilities, and underground structures.
- Mitigation and control measures to assess the impact of the works, including specifying of pre-condition and post-construction surveys, limiting settlement and deflection criteria (with 'alert', 'action' and 'alarm' levels that trigger required responses by the construction contractor in the event of exceedance of these values), and instrumentation and monitoring plans for the detailed assessment of movement of buildings, services, underground structures, etc., against the limiting movement criteria.
- Details of design and supporting information to be provided by the construction contractor during the tender and the construction phases of the works.

21.0 APPENDIX A

21.1 Standards

The following ASTM Standards are referenced in the Guideline. The full reference for each ASTM Standard listed is the same as for ASTM C295.

1. ASTM C295 : Standard Guide for Petrographic Examination of Aggregate for Concrete. ASTM International, West Conshohocken, PA.
2. ASTM C535 : Standard Test Method for Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine.
3. ASTM D420 : Standard Guide to Site Characterization for Engineering Design and Construction Purposes (withdrawn 2012).
4. ASTM D422 : Standard Test Method for Particle-Size Analysis of Soils (withdrawn 2016).
5. ASTM D698 : Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3)).

6. ASTM D1143 : Standard Test Methods for Deep Foundations Under Static Axial Compressive Load.
7. ASTM D1194 : Standard Test Method for Bearing Capacity of Soil for Static Load and Spread Footings (withdrawn 2003).
8. ASTM D1452 : Standard Practice for Soil Exploration and Sampling by Auger Borings.
9. ASTM D1556 : Standard Test Method for Density and Unit Weight of Soil in Place by Sand-Cone Method.
10. ASTM D1557 : Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3)).
11. ASTM D1586 : Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils.
12. ASTM D1587 : Standard Practice for Thin-Walled Tube Sampling of Fine-Grained Soils for Geotechnical Purposes.
13. ASTM D2113 : Standard Practice for Rock Core Drilling and Sampling of Rock for Site Exploration.
14. ASTM D2166 : Standard Test Method for Unconfined Compressive Strength of Cohesive Soils.
15. ASTM D2167 : Standard Test Methods for Density and Unit Weight of Soil in Place by the Rubber Balloon Method.
16. ASTM D2216 : Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.
17. ASTM D2435 : Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading.
18. ASTM D2487 : Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).
19. ASTM D2488 : Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).
20. ASTM D2573 : Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils.
21. ASTM D2845 : Standard Test Method for Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock (withdrawn 2017).
22. ASTM D2850 : Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils.
23. ASTM D3080 : Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.
24. ASTM D3441 : Standard Test Method for Mechanical Cone Penetration Tests of Soils.
25. ASTM D3689 : Standard Test Methods for Deep Foundations Under Static Axial Tensile Load.
26. ASTM D3966 : Standard Test Methods for Deep Foundations Under Lateral Load.
27. ASTM D4043 : Standard Guide for Selection of Aquifer Test Method in Determining Hydraulic Properties by Well Techniques.
28. ASTM D4186 : Standard Test Method for One-Dimensional Consolidation Properties of Saturated Cohesive Soils Using Controlled-Strain Loading.
29. ASTM D4220 : Standard Practice for Preserving and Transporting Soil Samples.
30. ASTM D4318 : Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.
31. ASTM D4428 : Standard Test Methods for Crosshole Seismic Testing.
32. ASTM D4525 : Standard Test Method for Permeability of Rocks by Flowing Air.
33. ASTM D4546 : Standard Test Methods for One-Dimensional Swell or Collapse of Soils.
34. ASTM D4644 : Standard Test Method for Slake Durability of Shales and Other Similar Weak Rocks.
35. ASTM D4648 : Standard Test Methods for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil.
36. ASTM D4700 : Standard Guide for Soil Sampling from the Vadose Zone.
37. ASTM D4719 : Standard Test Methods for Prebored Pressuremeter Testing in Soils (withdrawn 2016).
38. ASTM D4750 : Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well) (withdrawn 2010).
39. ASTM D4767 : Standard Test Method for Consolidated-Undrained Triaxial Compression Test for Cohesive Soils.

40. ASTM D4945 : Standard Test Method for High-Strain Dynamic Testing of Deep Foundations.
41. ASTM D4959 : Standard Test Method for Determination of Water Content of Soil by Direct Heating.
42. ASTM D5084 : Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter.
43. ASTM D5092 : Standard Practice for Design and Installation of Groundwater Monitoring Wells.
44. ASTM D5240 : Standard Test Method for Evaluation of Durability of Rock for Erosion Control Using Sodium Sulfate or Magnesium Sulfate.
45. ASTM D5312 : Standard Test Method for Evaluation of Durability of Rock for Erosion Control Under Freezing and Thawing Conditions.
46. ASTM D5313 : Standard Test Method for Evaluation of Durability of Rock for Erosion Control Under Wetting and Drying Conditions.
47. ASTM D5607 : Standard Test Method for Performing Laboratory Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force.
48. ASTM D5731 : Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications.
49. ASTM D5777 : Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation.
50. ASTM D5778 : Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils.
51. ASTM D5783 : Standard Guide for Use of Direct Rotary Drilling with Water-Based Drilling Fluid for Geoenvironmental Exploration and the Installation of Subsurface Water-Quality Monitoring Devices.
52. ASTM D5873 : Standard Test Method for Determination of Rock Hardness by Rebound Hammer Method.
53. ASTM D5878 : Standard Guides for Using Rock-Mass Classification Systems for Engineering Purposes.
54. ASTM D6151 : Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling.
55. ASTM D6282 : Standard Guide for Direct Push Soil Sampling for Environmental Site Characterizations.
56. ASTM D6430 : Standard Guide for Using the Gravity Method for Subsurface Investigation.
57. ASTM D6431 : Standard Guide for Using the Direct Current Resistivity Method for Subsurface Investigation.
58. ASTM D6432 : Standard Guide for Using the Surface Ground Penetrating Radar Method for Subsurface Investigation.
59. ASTM D6519 : Standard Practice for Sampling of Soil Using the Hydraulically Operated Stationary Piston Sampler.
60. ASTM D6635 : Standard Test Method for Performing the Flat Plate Dilatometer.
61. ASTM D6639 : Standard Guide for Using the Frequency Domain Electromagnetic Method for Subsurface Investigations.
62. ASTM D6907 : Standard Practice for Sampling Soils and Contaminated Media with Hand-Operated Bucket Augers.
63. ASTM D6913 : Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis.
64. ASTM D6938 : Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth).
65. ASTM D7012 : Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens Under Varying States of Stress and Temperatures.
66. ASTM D7063 : Standard Test Method for Effective Porosity and Effective Air Voids of Compacted Asphalt Mixture Samples.
67. ASTM D7070 : Standard Test Methods for Creep of Rock Core Under Constant Stress and Temperature.
68. ASTM D7383 : Standard Test Methods for Axial Compressive Force Pulse (Rapid) Testing of Deep Foundations.
69. ASTM D7400 : Standard Test Methods for Downhole Seismic Testing.

70. ASTM G:57 : Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method.

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