OKLAHOMA Transportation

ODOT

Geotechnical Manual

January 2025

PREFACE

The *Oklahoma Department of Transportation (ODOT) Geotechnical Manual* (Manual) has been developed to provide guidance to design, maintenance, and construction personnel in understanding geotechnical data.

This Manual addresses much of the geotechnical information normally required for a project; however, it is impossible to consider every situation that a Geotechnical engineer will encounter. Therefore, project Geotechnical engineers must exercise good judgment on individual projects and frequently be innovative in their approach to Geotechnical engineering. This may require, for example, additional research into available geotechnical literature or different approaches than are described in the following chapters. The detailed geotechnical scope for personnel performing geotechnical work can be found in the latest version of ODOT's Geotechnical Specifications for Roadway and Bridges.

It is also important to recognize that the field of Geotechnical engineering continues to evolve with time. New field methods will be developed, new computer software will become available, and revised or new methods of design will be identified. The methods described in this Manual are not intended to restrict consideration or use of these new developments.

<u>Use of Hyperlinks</u>
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1 ODOT GEOTECHNICAL MANUAL

1.1 GENERAL

1.1.1 Purpose of Manual

The *ODOT Geotechnical Manual* has been developed to provide guidance to design, maintenance, and construction personnel in understanding geotechnical data.

1.1.2 Intended Manual Audience

The intended audience for the Manual includes the following:

- Roadway design engineers
- Pavement design engineers
- Bridge design engineers
- Construction personnel
- Maintenance personnel
- Utility personnel
- Consultants/contractors
- Local highway agencies
- Geotechnical engineers

1.2 COORDINATION

The administration and management of the ODOT geotechnical program requires coordination with many ODOT Divisions as well as entities external to ODOT. Proper communication between the Geotechnical engineer and the ODOT Divisions throughout the project development process and during construction is essential.

1.2.1 Roadway Design Division

The Roadway Design engineer submits the preliminary grade line to the Geotechnical engineer and requests geotechnical survey. The Geotechnical engineer will conduct a field review of the project site to investigate its geotechnical characteristics. The nature and depth of the investigation will be determined on a project-by-project basis. The geotechnical report prepared by the Geotechnical engineer is used by the Roadway Design engineer to develop the preliminary cross sections and by the Pavement Design engineer for the pavement design analysis.

1.2.2 <u>Bridge Division</u>

Bridge Division submits a request to the Geotechnical Engineer to conduct a subsurface investigation of the bridge site. The Geotechnical engineer prepares a geotechnical report for all new bridge projects. The report presents the soil and rock types and bearing capacities.

For hydraulic scour evaluations at existing bridges, the Hydraulics engineer, Geotechnical engineer, and Bridge engineer may work together as an interdisciplinary team to evaluate the existing foundation design and to determine if any corrective actions are warranted.

For cast-in-place concrete walls, the Bridge engineer performs the structural stability design for the wall (e.g., size and spacing of reinforcing steel) using the allowable bearing capacity, expected settlement, and earth pressures provided by the Geotechnical engineer.

Environmental Programs Division

The Geotechnical engineer typically coordinates with the Environmental Programs Division prior to geotechnical work to identify and avoid any environmentally sensitive areas and to obtain any necessary permits such as a United States Army Corps of Engineers Section 404 permit.

Traffic Engineering Division

On a traffic-led project, the Traffic Engineering Division may request that the Geotechnical engineer conduct a subsurface investigation for large, overhead signs, traffic signal poles, high-mast light poles, etc.

Construction Division

Chapter 8 “Geotechnical Earthwork Construction” of this Manual provides additional information on the ODOT’s geotechnical field branch involvement during construction. The following summarizes the geotechnical involvement during construction:

<u>Communication Protocol</u>. During construction, the Geotechnical field branch often serves as a technical advisor to the Resident Engineer, and all communication in the field must go through the Resident Engineer.

<u>Troubleshooting</u>. The Geotechnical field branch is frequently contacted on an as-needed basis by the Resident Engineer to assist with geotechnical-related problems encountered in the field (e.g., soft subgrade, equipment not operating properly within geotechnical elements).

<u>Contract Changes</u>. If there are contractor claims, change order requests, or value engineering proposals, the Resident Engineer may contact the Geotechnical field branch for guidance on the geotechnical issues.

<u>Subgrades</u>. The Geotechnical field branch often advises the Resident Engineer on issues related to subgrades, including:

Modified/stabilized soils

Sulfate bearing soils

Dispersive soils

Unsuitable soils

Soft subgrade materials

Shrink/swell

Groundwater

The Geotechnical field branch is also sometimes involved in the interpretation of field testing on subgrades (e.g., nuclear density, Proctor, gradation, in-situ moisture) and, where appropriate, selecting a course of corrective action.

Embankments. On an as-needed basis, the Geotechnical field branch will spot-check activities related to the construction of embankments (e.g., benching, field tests, material suitability, shrink/swell, compaction, density control). Geotechnical personnel may field inspect embankment placement with respect to the correctness of the contractor's work.

Backfill Around Structures. The Geotechnical field branch sometimes provides assistance and advice to field construction personnel on the acceptability of backfill materials and their placement around structures.

Rock Excavation. Often, the Resident Engineer requests a geotechnical inspection after rock has been blasted to check the stability of the rock slope. The Geotechnical field branch will also review and comment on the contractor's rock blasting plan.

Field Instrumentation. The Geotechnical field branch may be asked to assist with monitoring foundation performance during construction (e.g., settlements, lateral displacements, pore water pressure beneath embankments).

Retaining Structures. The Geotechnical field branch may review and comment on contractor submittals, for example, specifying and reviewing testing required for anchored and/or soil nail walls. The Geotechnical field branch may support other types of wall construction where the Resident Engineer determines that the wall construction is not consistent with the contract documents.

Chapter 2

SUBSURFACE INVESTIGATIONS

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2 SUBSURFACE INVESTIGATIONS

2.1 GENERAL

2.1.1 Overview

ODOT's Geotechnical field branch or a Geotechnical consultant provides the Bridge Design, Roadway Design, Traffic Engineering, Field Construction & Field Maintenance Districts, and other interested parties the data regarding the geotechnical properties, analysis, and design requirements of their respective projects. Subsurface investigations and field tests are conducted by the Geotechnical engineer.

The work which the Geotechnical engineer performs tends to be categorized as either Pre-construction/Design-level or Post-construction work. The pre-construction work performed by the Geotechnical engineer tends to be for either the Bridge Division or Roadway Division. The scope of geotechnical work commonly requested by these divisions can be found in the current specifications for geotechnical work. The post-construction work performed by the Geotechnical field branch tends to be forensic in nature and is typically requested by the various Field Districts. This work may include settlement investigation, slope stability analysis, seepage investigations, repair recommendations and analysis, rippability analysis, report/document review, and more.

Preconstruction work includes subsurface investigations and field test activities. The Geotechnical engineer is responsible for performing these activities or coordinates with other ODOT Divisions to have these activities performed. Following are lists of the tasks which may be required for subsurface investigations and field tests.

1. Pre-Field Activities (typical but not limited to):

    - Performing desk or office reconnaissance, which includes reviewing data such as maps, borings, reports, aerial photography, and any other available existing data
    - Contacting affected landowners before entering property
    - Arranging for traffic control
    - Acquiring necessary permits
    - Ensuring that the utilities location checks have been performed
    - Coordinating with affected railroad companies
    - Arranging for field surveying

2. Field Activities (typical but not limited to):

    - Identifying the location of all borings based on directions from the project Geotechnical engineer
    - Layout of borings
    - Survey of boring locations

- Conformance with the permit conditions
- Selecting and mobilizing the appropriate drilling equipment
- Preparing site for drilling
- Logging drilling activities
- Recovering soil and rock samples
- Performing the necessary field in-situ tests
- Preparing and transporting the soil or rock samples back to the geotechnical laboratory
- Performing, where applicable, the necessary field work for installation of piezometers, monitoring/observation wells, inclinometers, and other instrumentation
- Restoring the borehole site to its original condition to the extent practical

3. <u>Post Field Activities (typical but not limited to)</u>:

   - Organizing field data
   - Presenting boring logs
   - Logging rock core samples
   - Performing laboratory testing
   - Analysis of laboratory and/or in-situ field testing
   - Performing calculations and engineering analysis of the project
   - Writing a final report and providing recommendations

2.1.2 <u>Subsurface Exploration Methods</u>

Subsurface exploration may include many methods sampling soil and rock. A combination of drilling, sampling, in-situ testing, and instrumentation may be performed to evaluate subsurface conditions at the site. A variety of methods for drilling exist but the most common in Oklahoma are drilling with solid stem augers, hollow stem augers, mud rotary drilling, advancing casing, or some combination of these. Disturbed or undisturbed samples may be obtained from a boring. These drilling methods are described in more detail later in this Chapter, but brief descriptions are provided as follows:

1. <u>Solid Stem Augers</u> - The solid stem augers commonly used for geotechnical work have a 4-to-6-inch outer diameter with a solid stem. They are usually 5 feet long. Other sizes and dimensions do exist. These augers are usually used for very shallow explorations (i.e., roughly less than 10 feet deep) or to drill a pilot hole. They are not as versatile as hollow stem augers.

2. Hollow Stem Augers - The hollow stem augers commonly used for geotechnical work have a 6-inch outer diameter with an approximately 4-inch diameter hollow stem in the center. These augers are typically 5-feet long. Other sizes and dimensions do exist. The hollow stem allows instrumentation to be lowered into a boring without the need to remove the augers. Furthermore, this type of auger acts as a temporary casing keeping the hole open and preventing cave-ins within loose strata. Although holes may be drilled as deep as about 60 feet, many drillers prefer to use mud rotary drilling instead for such drilling depths.

3. Mud Rotary Drilling - Mud rotary drilling utilizes a mud rotary bit and drilling fluid, colloquially referred to as "drilling mud," to advance the boring.

4. Casing – Casing comes in a wide variety of lengths, diameters, and thicknesses. A variety of types of casing also exists, all of which have special circumstances in mind behind their design. Casing may be used in a multitude of ways depending on the need. Commonly it is used to simply maintain the stability of the walls of the boring within a loose formation. However, it can be used when drilling through a bridge deck to maintain a relatively leak-tight string of casing ("pipe") until the ground surface beneath the deck is reached and the mud rotary drilling operation begins. Also, casings can be used to advance a hole, but this is typically done in conjunction with mud rotary drilling, as the inside of the casing needs to be cleaned out before any sampling or testing can begin at the target depth.

2.1.3 Subsurface Sample Types

Brief descriptions of disturbed and undisturbed samples are as follows:

1. Disturbed Samples. Disturbed samples are obtained to determine the soil type, gradation, Atterberg limits, consistency, moisture/density characteristics, presence of contaminants, stratification, etc. Disturbed samples may be obtained by hand excavating methods or by mechanical digging or drilling techniques. These samples are considered "disturbed" because the sampling process modifies their natural structure.

2. Undisturbed Samples. Undisturbed samples may be used to determine the in-situ density, strength, compressibility or swell potential, permeability, discontinuities, fractures and fissures of subsurface formations, and many more characteristics. Even though these samples are designated as "undisturbed," in reality they are disturbed to varying degrees. The degree of disturbance depends on the type of subsurface materials, type and condition of sampling equipment used, skill of the drillers, and storage and transportation methods used. Serious and costly inaccuracies may be introduced into the design if proper protocol and care is not exercised during recovery, transporting and storing of samples.

2.1.4 In-Situ Testing

The drilling and sampling program may also be conducted as part of an in-situ testing program, to establish groundwater conditions, and sometimes to support the installation of geotechnical instrumentation. In-situ testing is covered in more detail later in this section of text, but a brief overview is as follows:

1. In-Situ Testing - An in-situ test is a test performed at the site and typically does not require collection/removal of a sample. In-situ testing methods can be used as part of, or supplement to, drilling and sampling methods and/or laboratory testing. These tests include the standard penetration test (SPT), Texas cone penetrometer test (TCPT), cone penetrometer test (CPT), vane shear test (VST), pressuremeter test (PMT), and flat-plate Dilatometer test (DMT) among many others. With the exception of the SPT, these tests

provide information on subsurface soil characteristics without obtaining a sample of the soil. Stratigraphy and engineering characteristics of the soil are interpreted based on the types of information recorded and by using empirical correlations between the test measurement and soil properties. Considerable time and cost savings can often be obtained by supplementing conventional drilling, sampling, and laboratory testing with these in-situ testing methods.

2. Geophysical Testing - Geophysical testing is used to obtain information about the stratigraphy and, in some cases, engineering characteristics of the subsurface material. The seismic refraction method is the most common geophysical method used by the Geotechnical Branch; however, other geophysical tests available to some geotechnical consultants maybe also be considered (e.g., surface resistivity (SR), ground penetrating radar (GPR), electromagnetic conductivity (EM)). Geophysical methods can be useful in establishing ground stratigraphy, detecting sudden changes in subsurface formations, locating underground cavities in karst formations, or identifying underground utilities and/or obstructions. Compression (P-wave) and shear (S-wave) wave velocities obtained from seismic geophysical tests (e.g., downhole, cross hole) can provide information on the dynamic elastic properties of the soil and rock. The shear wave velocity profile can be used for seismic site amplification studies of ground shaking and can be used for soil liquefaction evaluations. The P-wave velocity can be used to estimate if the rock can be excavated by various sizes of equipment or will require blasting.

2.2 PRE-FIELD ACTIVITIES

This section discusses some of the pre-field activities, including review of existing information and performing site reconnaissance. Information from this preliminary work is used to develop a subsurface exploration plan to schedule and conduct the field work. While specifications exist for performing geotechnical work, often information is discovered during these pre-field activities which may warrant modifications to the way the specifications require the work to be performed.

2.2.1 Desk/Office Reconnaissance

The first step in the investigation process is to review existing data. There are a number of helpful sources that should be used in planning subsurface investigations. Review of this information can minimize surprises in the field, assist in determining boring locations and depths, and provide valuable geologic and historical information for inclusion in the geotechnical report.

The following list contains examples of sources of site, geologic, and historic information that should be considered:

- Prior subsurface investigation (historic data) at or near the project site

- Prior construction and structural performance records at the site (e.g., pile length, drivability problems, landslides, excessive seepage, unpredicted settlement, poor pavement performance); some of this information may be obtained from the Field District or Resident Office

- Prior environmental studies

- Site plans showing locations of existing and proposed ditches, driveways, culverts, utilities, and pipelines

- Maps of streams, rivers, and other water bodies crossed by bridges, culverts, etc., including bathometric data

- Aerial photographs (e.g., ODOT Survey Division, USGS, Google Earth)

- US Geological Survey (maps, reports, publications, and website)

- Oklahoma Geological Survey (map, reports, publications, and website)

- "Engineering Classification of Geologic Materials" ("red books," also available on ODOT Materials Division's website)

- US Department of Agriculture Natural Resources Conservation Service (NRCS) soil maps; see NRCS website (Reference 1). These are the same as the former Soil Conservation Service soils maps. These maps only show shallow surficial deposits (i.e., usually limited to a 60-inch depth), but may assist in establishing general soil type, extent of surface soils, general engineering characteristics, and the basis for Pedological Surveys.

- Local university libraries and geology departments

- Water well locations from the Oklahoma Water Resources Board

- US Bureau of Mines data

2.2.2 Site Visit/Preliminary Field Review

It is advisable that the Geotechnical engineer (and others involved in the geotechnical exploration) conduct a reconnaissance visit to the project site to identify topographic and geologic features and to become knowledgeable of access and working conditions. The site visit is most effective when conducted after the available information for the project site has been collected and reviewed. Ideally this site visit is performed before preliminary (30%) plans are completed.

The preliminary field review and/or site visit is a good opportunity to discuss or review the following:

- Design and construction plans

- General site conditions

- Geologic issues

- Geomorphology

- Drilling equipment access restrictions

- Traffic control requirements during field investigations

- Location of underground and overhead utilities

- Type and condition of existing facilities (e.g., pavements, bridges)

- Adjacent land use (e.g., buildings, waterways, irrigation practices, public/private, urban/rural, developed/undeveloped)

- Potential for working hour restrictions
- Right-of-way constraints
- Environmental issues
- Escarpments, outcrops, erosion features, and surface settlement
- Flood levels
- Water traffic and access to water boring sites
- Benchmarks and other reference points to aid in the location of boreholes
- Equipment storage areas and security
- Work safety
- Possible need to modify scope of work (e.g. boring locations, in-situ testing, lab testing etc.)

2.2.3 Subsurface Exploration Plan

The Geotechnical engineer, in coordination with the Bridge, Roadway Design, Traffic Engineering, and Field Construction and Maintenance District, is responsible for determining the number and locations of boreholes, required soil and rock samples, required equipment to collect and test the samples, and field testing and instrumentation requirements. These determinations are based on guidelines from the specifications as well as the type, size and complexity of the project, and geological conditions anticipated. The proposed plan for field work will be documented and compiled within a subsurface exploration plan. This plan also is usually sent to the Environmental Programs Division to initiate the required permitting for drilling near streams, wetlands, or other sensitive areas.

The content of the Subsurface Exploration Plan should include the following information, as applicable:

- Contacts and phone numbers for the landowner, utility companies, railroads, etc.
- Center reference line on centerline
- NRCS county soil survey map (Web Soil Survey)
- Location of borings and test locations overlain on a .kmz file or GIS
- Type of equipment necessary to accomplish the drilling
- Expected minimum depth of borings
- Type of exploration required

- Sampling type and interval
- In-situ testing procedures required
- Instrumentation plans
- Borehole closure requirements
- Site restoration requirements
- Discussions of unusual conditions expected (e.g., hazardous materials, artesian wells, permit conditions, health and safety considerations)

2.2.3.1 Borings

The number, depth, spacing, and type of borings are greatly dependent on the site conditions; the type, scope, and scale of the project; and the project requirements. Consequently, no rigid rules regarding boring programs can be established, despite the existence of geotechnical exploration specifications. Sometimes, the extent of the work is established as the site investigation progresses in the field.

2.2.3.2 Scheduling Drill Crews

Once the subsurface exploration plan has been developed and borings are marked in the field, scheduling of the field work may begin. Scheduling should consider the time required to coordinate with utility companies, railroad companies, and landowners, as well as the time it takes to receive the necessary permits.

2.2.3.3 Permits

If it is known that drilling is required near streams, wetlands, and/or floodplains, the Geotechnical engineer should notify the Environmental Programs Division and other regulatory entities as applicable. If an environmental permit may be required, the Geotechnical engineer should coordinate with the Environmental Programs Division.

2.2.3.4 Traffic Control Requirements

The amount of detail and type of traffic control required for geotechnical field explorations varies depending on the site location, traffic volumes, how close the equipment is located near the traveled way, length of time, etc.

2.2.3.5 Right-of-Entry

Oklahoma State Statute 702 allows entry into private properties for conducting geotechnical survey needed for highway construction, with notice given to the owner prior entry. The statute does not apply to Federal or Tribal property; additional coordination with the Federal or Tribal entity will be required to secure permission to perform work on those properties. Once the Right-of-Entry Permit is received, the geotechnical work may begin. However, it is always good practice to contact the landowner prior to commencing work.

2.2.3.6 Utility Locations

Prior to drilling, it is necessary to determine the location of any utilities within the area by contacting the "OKIE one call system." OKIE will notify Oklahoma on-call members in the local area to locate utilities. For some areas where multiple utilities are known to exist or where the location of existing utilities cannot easily be determined, a subsurface utility engineering survey may be required to determine the location of underground utilities.

2.2.3.7 Railroads

If the subsurface investigation or field work is near or within railroad right-of-way, it is necessary to coordinate the field work with the railroad company. This coordination is conducted through the ODOT Rail Programs Division. Obtaining permission/permits from the railroad company can sometimes be a lengthy process and therefore impact the project schedule.

2.3 FIELD ACTIVITIES

2.3.1 Location and Depth of Explorations

The geotechnical scope provides guidance on the location, number, and depth of boreholes required for various structure types. Actual field conditions may require these locations to be revised. These revisions are determined on a case-by-case basis. The Geotechnical engineer will coordinate the revisions in consultation of the respective ODOT division requesting the work.

2.3.2 Surveying and Location Tolerance

Each exploration must be surveyed to establish the location of the exploration. Surveys are usually conducted following the completion of the fieldwork. Where accuracy of the exploration location is critical, a professional Land Surveyor's services may be necessary to mark the exploration locations prior to and/or after the field work commences. The required accuracy is specified in the geotechnical scope.

2.3.3 Borehole and Test Location Numbering System

Each borehole or test location hole must be identified for recording purposes. Boreholes are usually numbered sequentially in the order drilled. The approximate station and offset are also recorded along with any preliminary survey information available to record the location of the boring.

2.3.4 Soil and Rock Drilling

2.3.4.1 Drilling Equipment

The selection of soil and rock drilling equipment depends upon the anticipated access and/or subsurface conditions as well as the type of work to be performed (e.g. in-situ testing, sampling, coring, etc.).

Following is a list of the most common equipment, and methods used for geotechnical work in Oklahoma:

1. Hollow Stem Augers. Hollow-stem auger methods are used in cohesive or granular soils above the groundwater table and when the boring walls may be unstable. The augers form a temporary casing to allow sampling of the soil below the drill bit. The soil sample may be disturbed or undisturbed depending on the sampling equipment used within the boring. This drilling method tends to be preferred for all soils above the groundwater table.

2. Rotary Wash Boring. The rotary wash boring method is generally the most appropriate method for use in soil formations below groundwater level as well as relatively deep borings (~50 or more feet deep). In rotary wash borings, the sides of the borehole are supported either with drilling fluid or casing advancer. A casing advancer is used to sample soils below the groundwater table or when very loose strata are encountered or expected (e.g. very loose sands). This method is not preferred above the groundwater table because it often results in lower quality samples.

3. Core Drilling. Core drilling methods are used when the rock formations encountered are too hard to be sampled by soil sampling methods, or where higher quality, undisturbed continuous rock samples are desired rather than disturbed samples.

2.3.4.2 Special Concerns for Drilling

The following conditions may require special treatment during the drilling process:

1. Contaminated Soils. If contaminated soil is expected, the Environmental Programs Division will develop plans and determine any health and safety requirements. The Geotechnical engineer will work with the Environmental Programs Division to determine the appropriate mitigation options if contaminated soils are encountered. If hazardous materials are found or anticipated, a certified hazardous waste specialist is required to deal with these materials.

2. Artesian Aquifers. Groundwater in artesian aquifers can be under a higher pressure ("head") than the ground surface. These conditions often occur in valleys next to hills and mountainous areas; however, sometimes the artesian conditions can be several miles from the nearest topographic relief. When an artesian aquifer is penetrated during drilling, large flows of water can develop through the drill hole. These flows can be very difficult to stop, and continued flow can require abandonment of the drill hole and result in environmental damage. If an artesian aquifer is encountered, immediate action may be required. The need for sealing the borehole will be determined on a case-by-case basis.

3. Boulders. Boulders are not common in Oklahoma. However, when rock is encountered during the drilling process, it is typically tested or cored for at least 10 feet to try to ensure it is not a boulder.

If any of the previous conditions are encountered, the details will be noted on the boring log.

2.3.5 Sampling Methods (Soils)

2.3.5.1 General

In general, soil samples obtained for engineering testing and analysis are segregated into the following two main categories:

1. Disturbed Samples. Disturbed samples are those obtained using equipment that alters the macro structure of the soil but does not alter its mineralogical composition or moisture content. Specimens from these samples can be used for determining the general lithology of soil deposits, for identification of soil components and general classification purposes, and for determining grain size, Atterberg limits and compaction characteristics of soils among many other characteristics.

2. Undisturbed Samples. Undisturbed samples are obtained in cohesive or clay soil strata for use in laboratory testing to determine various engineering properties of those soils. Undisturbed samples of granular soils can be obtained, but often specialized procedures are required (e.g., freezing or resin impregnation, block or core-type sampling). It should be noted that the term “undisturbed” soil sample refers to the relative degree of disturbance to the soil’s in-situ properties. Undisturbed samples are obtained with specialized equipment designed to minimize the disturbance to the in-situ structure and moisture content of the soils. Specimens obtained by undisturbed sampling methods are used to determine the strength, stratification, permeability, density, consolidation, dynamic properties, and other engineering characteristics of soils.

The most common types of soil sampling methods are presented in Figure 2-1.

2.3.5.2 Soil Sampling Interval

In general, samples may be taken in both granular and cohesive soils, with thin-walled tube samples taken in cohesive soils. The sampling interval varies between individual projects and between regions. A common practice is to obtain SPT samples at 2.5-ft intervals in the upper 10 ft and at 5-ft intervals below 10 ft. In some instances, a greater sample interval, often 10 ft, is allowed below depths of 100 ft. In other cases, continuous samples may be required for some portion or the full depth of the boring. The sampling methods and intervals preferred by ODOT are within the specifications for geotechnical work. Due to the variable nature of geologic material, access issues, scale of projects, and many other factors, the work within the specifications for geotechnical work may be modified on a case-by-case basis with coordination from the respective divisions.

2.3.6 Sampling Methods (Rock)

The methods used for exploration and investigation of rock profiles include:

1. Drilling and Coring. Continuous core drilling is the primary method used to obtain intact samples of rock for testing purposes and for assessing rock quality and structure. Borings resulting from rock coring are also used to obtain groundwater data, perform in-situ tests, and install instruments. Rotary drilling may be used to advance borings if core (sample) recovery is not desired.

2. Geophysical Methods. Geophysical methods (e.g., seismic refraction) may be used to determine the depth to rock, elastic properties, or rippability.

3. Geologic Mapping. Geologic mapping of rock exposures or outcrops provides a means for large scale assessment of the composition and discontinuities of rock strata, which may be valuable for many engineering applications, particularly rock slope design.

Geographic locations of the geological log or outcroppings must be described from plans, by station to the nearest tenth of a foot, where the information is available. A minimum of a legal description accurate to the nearest 100 ft is required when plans or other detailed location information is not available. GPS locations are acceptable.

Sampler	Disturbed/ Undisturbed	Appropriate Soil Types	Method of Penetration
Bulk	Disturbed	Gravels, sands, silts and clays	Hand tools, bucket auger
Continuous Auger	Disturbed	Cohesive soils	Drilling with hollow stem augers
Split Barrel	Disturbed	Sands, silts, clays and some fine gravels	Hammer-driven (large split spoon)
Thin-Walled Shelby Tube	Undisturbed	Clays, silts, mixed fine-grained and granular soils	Mechanically pushed
Piston (Fixed, Floating)	Undisturbed	Silts and clays	Mechanically pushed
Pitcher Barrell	Undisturbed	Stiff to hard clay, silt, partially weather rock	Rotation and hydraulic pressure
Denison Barrell	Undisturbed	Stiff to hard clay, silt, sand and partially weather rock	Rotation and hydraulic pressure
Block	Undisturbed	Cohesive soils and frozen or resin impregnated granular soil	Hand tools
Continuous Sampler	Disturbed	Cohesive Soils	Mechanically pushed with hollow stem augers

Figure 2-1: COMMON SOIL SAMPLING METHODS

2.3.6.1 Types and Description of Rock

The following are descriptions of the common rock units or lithology found in Oklahoma as outcroppings or as recovered in cores, listed from most to least commonly encountered.

1. Shale. Shales are fine-grained sedimentary rocks consisting of compacted and hardened clay, silt, or a combination of the two particle sizes. Shales normally contain at least 67% clay, with the remainder being silt with a chemical or crystalline material acting as a cementing agent. Shales are by far the most common of the sedimentary rocks and are usually identified in the field by their laminated or fissile appearance. Shales can be any color, with the colors gray, brown, olive or black commonly found in eastern Oklahoma and shades of red often with greenish-gray spots or layers in western Oklahoma. The reddish shales of western Oklahoma commonly do not exhibit strong laminations but are more massive or blocky in appearance and. Shales usually exhibit a smooth, sometimes waxy feel.

2. Sandstone. Sandstones are medium-grained, consolidated sedimentary rocks composed primarily of 85% to 90% quartz grains and 10% to 15% of a cementing agent. Sandstones are commonly reddish in western Oklahoma. In eastern Oklahoma, sandstone is most commonly brown to yellowish, but many are gray. Sandstones usually exhibit a gritty feel.
3. Limestone. Limestones are sedimentary rocks consisting of more than 95% of the mineral

calcite (calcium carbonate) and the remaining percent commonly dolomite. Limestones occur in beds or layers, may be massive or contain visible fossils and are common in southeastern Oklahoma. Limestones are most commonly whitish or cream in color but can range from brown and red to black. Limestones are commonly hard but may be soft and chalky.

4. Gypsum. Gypsum is a rock composed of the mineral gypsum, which is hydrous calcium sulfate. Gypsum occurs as massive layers or beds. It normally does not contain any particles of sand, silt, or clay. It is white in color and may occasionally contain streaks of reddish brown. Gypsum is softer than limestone and is found in the western half of Oklahoma.

5. Anhydrite. Anhydrite is a rock similar to gypsum occurring in beds or layers. As with gypsum, anhydrite is composed of calcium sulfate but lacks the water molecules contained in gypsum. Anhydrite is slightly harder than gypsum, but still a soft rock. Anhydrite generally has the same appearance as gypsum and occurs associated with gypsum in western Oklahoma.

6. Siltstone. Siltstones are consolidated sedimentary rocks that contain at least 70% silt-sized particles with the remainder being clay size particles. They can be any color. Siltstones are usually soft but may occasionally be hard.

7. Conglomerate. Conglomerates are consolidated sedimentary rocks that contain rounded to subangular fragments larger than sand size. Conglomerates are usually brownish to reddish in color. They are usually found in massive beds and associated with sandstones. Conglomerates are usually hard but can range from soft to hard. They are named by the composition of the gravel fragments. For example, if the gravel-sized material is mostly limestone, the rock is described as a limestone conglomerate.

8. Dolomite. Dolomite is sedimentary rock containing more than 50% of the mineral dolomite, which is a calcium/magnesium carbonate. Dolomite commonly contains more than 90% dolomite, with the remaining percentage being calcite. Dolomite commonly occurs with limestone or interlayered with limestone. Most Oklahoma dolomite is white, ranging to light gray or sometimes slightly pinkish. Dolomite is commonly hard, durable rock. Dolomite occurs in all areas of Oklahoma except the panhandle.

9. Caliche. Caliche is a rock-like soil deposit of the high plains of northwestern Oklahoma. It may contain various amounts of gravel, sand, silt and clay, locally. Caliche occurs as a bed or layer at or near the soil surface and can be one to twenty feet thick. Sometimes the caliche is degraded and soft, easily dug with a knife. Caliche is usually whitish or light gray.

10. Granite. Granite is an igneous rock (not bedded) that is hard, dense and crystalline. The rock is composed of crystals of quartz, feldspar and a minor amount of dark minerals. Feldspar gives granite its color. Colors range from salmon in the Wichita Mountains of southwestern Oklahoma, to light reddish purple in the Tishomingo area, to grayish in the Arbuckles. Weathered granite is usually whitish and soft.

11. Gabbro, Anorthosite, Porphyry and Basalt. These rocks are igneous and similar in physical properties and mode of occurrence to granite. These are all hard-dense rocks that are not bedded, except basalt, which appears as a lava flow atop Black Mesa in the extreme northwestern tip of Cimarron County in the panhandle. The other rocks occur in association with granite. Gabbros are hard, dark to black colored rocks occurring in

masses or veins. Anorthosites are composed primarily of feldspar. In Oklahoma, they are hard, dark grayish and occur in masses or veins. Porphyry is a hard, coarsely crystalline rock with the same mineral content as granite or other igneous rock.

12. Chert. Chert, sometimes called flint, is a very hard, dense, siliceous sedimentary rock. It commonly occurs as nodules or concretions in limestones. Chert only occasionally occurs in beds. It is commonly medium to dark gray but, due to impurities, may be brown, black, reddish or whitish. Chert is more commonly found in northeastern Oklahoma.

Rock types are not always pure. For example, sandstones may contain more than just sand grains. If sandstone contains a calcareous matrix or cementing agent, then it is called limy or calcareous sandstone. If limestone is composed mostly of whole or broken fossils, it will be described as a fossiliferous limestone.

Properties of rock are described in more detail below:

1. Hardness. The rock hardness classes most pertinent to engineering are as follows:

    a. Soft. The rock can be worked with a shovel, friable, can be broken by hand in a dry to moist hand specimen, and easily carved with a knife when moist. The red clay shales and mudstones of western Oklahoma are usually soft.

    b. Moderately Hard. The rock cannot be worked with a shovel. It can be worked with a geology hammer or pick. It can be scratched with a penny. Example rocks are the gypsums, anhydrites and caliches of western Oklahoma. Many sandstones and siltstones are among this class, and some black shales in the Ouachita Mountains of southeastern Oklahoma.

    c. Hard. The rock cannot be worked with a pick. It has a ring when struck with a hammer. It cannot be scratched with a penny but can with a knife. Most competent rocks are within this category. Most limestones, sandstones and dolomites are examples.

    d. Very Hard. The rock has a distinct ring when struck with a hammer. It cannot be scratched with a knife. Examples include siliceous limestones of eastern Oklahoma and the granites and other igneous rocks of the Witchitas and Arbuckles.

2. Layering. Most of the rocks of Oklahoma occur as beds or layers. The exceptions are the granites and other igneous rocks of the Wichitas and the Arbuckles.

    a. Fissile. Splits easily along closely spaced planes of 1/16 inch or less. Many shales of eastern Oklahoma are fissile.

    b. Very Thin Bedded. Beds of 1/16 to 2 inch (known as "stringers").

    c. Thin Bedded. Beds of 2 inches to 2 ft.

    d. Thick Bedded. Beds of 2 ft to 4 ft. Beds more than 4 ft are described as “very thick bedded.”

    e. Massive Bedded. Beds exceeding 4 ft. Usually describes homogeneous beds that have little or no evidence of minor joints, laminations or imperfections. Gypsums are commonly massive bedded.

3. Joints, Faults and Fractures. A joint is defined as the surface of a fracture or a discontinuity in a rock mass, without displacement (faulting). If displacement of the sides of the rock, relative to one another, can be observed along the discontinuities, then the feature is by definition a fault. Sedimentary rocks will usually have two sets of parallel joints. All the features defined above are considered discontinuities within a rock mass. Note: Unusual conditions where rocks are observed exhibiting closely spaced (measured in inches or fractions thereof) joints and faults may be described as fractured or highly fractured. Their rating classes are as follows:

    a. Very Low Jointing. A distance of more than 6.5 ft between discontinuities.

    b. Low Jointing. A distance of 2.0 ft to 6.5 ft between discontinuities.

    c. Medium Jointing. A distance of 8 inches to 2.0 ft between discontinuities.

    d. High Jointing. A distance of 2.5 inches to 8 inches between discontinuities.

    e. Very High Jointing. A distance of less than 2.5 inches between discontinuities.

4. Pores. Pores are the open spaces in a rock or soil. Pores are to be observed with the unaided eye. Pores are measured, described and reported where there are obvious features in a hand specimen or length of core.

    a. Very Fine. Less than 1.0 mm.

    b. Fine. 1 mm to 2 mm.

    c. Medium. 2 mm to 5 mm.

    d. Coarse. 5 mm to 10 mm*.

    e. Very Coarse. More than 10 mm*.

    f. Vugs. 5 mm to 30 mm*. If irregular shaped coarse and very coarse pores are present in limestones, dolomites or gypsums, they may be described as having vugs or vuggy (small cavity in a rock).

2.3.6.2 Preservation, Transportation and Storage of Samples

Rock cores from geotechnical explorations are usually stored in structurally sound core boxes made of wood or corrugated waxed cardboard. Prior to closing and sealing the box, a picture of the cores is taken to aid the lab in the event that the cores break during transport to the lab.

2.3.7 Field Classification

Soil description/identification is the systematic, precise, and complete naming of individual soils in both written and spoken forms, while soil classification is the grouping of the soil with similar engineering properties into a category based on index test results. It is important to distinguish between these terms to minimize conflicts between general visual evaluations of soil samples in the field versus more precise laboratory evaluation supported by index tests.

2.3.8 Groundwater Wells

Observations of the groundwater level and pressure are an important part of all geotechnical explorations, and the identification of groundwater conditions should receive the same level of care given to soil descriptions and samples. Determination of groundwater levels and pressures includes measurements of the elevation of the groundwater surface or water table and its variation with the season of the year, the location of perched water tables, the location of aquifers, and the presence of artesian pressures. Water levels and pressures may be measured in existing wells, boreholes, and specially installed observation wells. Piezometers are used where the measurement of the groundwater pressures are specifically required (i.e., to determine excess hydrostatic pressures or the progress of primary consolidation).

Measurements of water level during drilling and at least once following drilling are recorded to obtain the necessary groundwater data, unless alternative methods (e.g., installation of observation wells) are defined in the Subsurface Exploration Plan or by the project Geotechnical engineer.

2.3.8.1 Information from Existing Wells

Oklahoma Water Resources Board (OWRB) requires the drillers of water wells to file logs of the wells. See OWRB website (Reference 2). These are good sources of information regarding the materials encountered and water levels recorded during previous well installations in an area of future development. The well owners, both public and private, may also have records of the water levels after installation that may provide extensive information on fluctuations of the water level.

2.3.8.2 Open Borings

The water level is often recorded in open borings after any prolonged interruption in drilling, at the completion of each boring and, if practical, 24 hours after completion of drilling. Additional water level measurements may be obtained at the completion of the field exploration and other times necessary. The date and time of each observation is recorded.

If the borehole has caved, the depth to the collapsed region is recorded. The collapse may have been caused by groundwater conditions and the caved elevation may indicate the groundwater table elevations at the site.

Drilling fluids obscure observations of the groundwater level owing to filter cake action and the higher specific gravity of the drilling mud compared to that of the water. If drilling fluids are used to advance the borings, the borehole is bailed prior to making groundwater observations. For some projects, consideration can be given to the use of biodegradable drilling muds.

2.3.8.3 Piezometers and Observation Wells

Piezometers and observation wells are used for measuring the groundwater locations at a site and for evaluating the performance of dewatering systems. In theory, a piezometer measures the pressure in a confined aquifer or at a specific horizon of the geologic profile, while an observation well measures the level in the geologic profile regardless of source or location. The term monitoring well applies to wells that are periodically sampled for water quality over the course of a long period of time and monitoring wells are typically only used for environmental purposes. Although different, it is noted that in practice the terms piezometers, observation wells, and monitoring wells are often incorrectly used interchangeably to describe any device for determining water elevation.

2.3.8.4 Water Level Measurements

In general, common practice is to measure the depth to the water surface using the top of the casing or top of the ground as a reference, with the reference point at a common orientation (often North) marked or notched on the well casing. Several devices are used for sensing or measuring the water level in observation wells (e.g., chalked tape, tape with a float, electrical water-level indicator, pressure transducers, data loggers).

2.3.8.5 Well Closure

Unless the soil is contaminated, the well can be closed per the OWRB rules and regulations. For contaminated soils, the Geotechnical engineer will coordinate the well closure with ODOT's Environmental Programs Division.

2.3.8.6 Regulations

Piezometers and monitoring wells must be drilled/installed by drillers licensed by the OWRB. After installing a piezometer or monitoring well, a record must be filed with the OWRB. Observation wells must be removed within a certain number of days after their installation, whereas piezometers and monitoring wells may remain open for an indefinite time period.

Local jurisdictions may impose specific requirements on permanent monitoring wells that must be considered in the planning and installation, such as closure and special reporting of the location and construction. Licensed drillers and special fees also may be required.

2.3.9 <u>Field/In-Situ Tests</u>

The following sections describe the field-testing methods used during subsurface investigations and other common in-situ tests that might be used to address a specific project need.

2.3.9.1 Standard Penetration Test

The standard penetration test (SPT) is performed during the advancement of a soil boring to obtain an approximate measure of the soil resistance, as well as a disturbed soil sample with a split barrel sampler.

A Split Spoon Sampler is used to perform the SPT test. The split spoon sampler is effectively a hollow, very sturdy steel tube, like a straw. It is split lengthwise and can be assembled when a test needs to be performed and disassembled after a test is performed to obtain the soil sample within it.

2.3.9.2 Vane Shear Test

The vane shear test (VST), or field vane (FV), is used to evaluate the in-place, undrained shear strength (s_{uv}) of soft to stiff clays and silts. The test consists of inserting a 4-bladed vane into the clay and rotating the device about a vertical axis.

2.3.9.3 Texas Cone Penetrometer

The Texas Cone Penetrometer (TCP) is commonly used in site investigations by ODOT. The TCP test involves driving a hardened conical point into soil and hard rock. From the soil test, a penetration resistance or blow count (N_{TCP}) is obtained, which equals the number of blows of the hammer for the first 6 inches and the second 6 inches of penetration. The Bridge Design Division correlates these values for their design purposes.

2.3.9.4 Cone Penetration Test

The cone penetration test (CPT) is fast, economical and provides continuous profiling of geostratigraphy and soil properties evaluation. Both mechanical and electric cone systems are available; however, for most modern-day investigations the electric cone system is used. The seismic CPTu has added instruments which allow for downhole seismic testing to be performed. A multitude of correlations exist between shear wave velocities and soil properties such as the seismic site class, small strain modulus, shear strength, and many other properties of interest.

The test is very easy and fast to perform and is arguably the best profiling tool for soils and estimating the soil properties if/when soil samples are not obtained. It cannot be advanced into rock or gravelly strata.

2.3.9.5 Pressuremeter Test

The pressuremeter test (PMT) consists of a long cylindrical probe that is expanded radially into the surrounding ground. By tracking the amount of volume of fluid and pressure used in inflating the probe, data can be interpreted to give a complete stress-strain-strength curve. In soils, the fluid medium is usually water (or gas). In weathered and fractured rocks, specialized pressuremeters are used and the fluid used with them is typically hydraulic oil.

2.3.9.6 Borehole Shear Test

While shear strength of soils can be critical for design of earth slopes, laboratory shear tests can be time consuming and expensive. The borehole shear test (BST) provides a convenient method to accurately measure the undrained shear strength of soils in-situ.

2.3.9.7 Flat Plate Dilatometer Test (Marchetti)

The flat dilatometer test (DMT) uses pressure readings from an inserted plate to obtain stratigraphy and estimates of at-rest lateral stresses, elastic modulus and shear strength of sands, silts and clays.

2.3.9.8 Dynamic Cone Penetration Test

This test is most often used in shallow explorations (5 feet or less) related to pavement or pavement design. It can also be used to determine bearing capacity for shallow footings.

2.3.9.9 Falling Weight Deflectometer Test

The Falling Weight Deflectometer (FWD) is a non-destructive testing device used to evaluate the structural properties of a pavement section. It is used for conventional and deep strength flexible, composite. and rigid pavement structures. The use of an FWD helps determine a deflection basin caused by a controlled load. FWD generated data, combined with layer thickness, can be confidently used to obtain the in-situ elastic modulus of a pavement structure and identify variability in the subgrade stiffness. This information may be used in a structural analysis to estimate the expected life and calculate overlay requirements over a desired design life. Selecting the type of rehabilitation to be implemented on a given pavement is of considerable economic significance and to reach that decision without an adequate knowledge of the structural condition of the pavement may have very costly consequences. The lightweight FWD, referred to as LWD, is an instrument for on-site measurement of stiffness to minimize risks and optimize quality.

2.3.9.10 Geophysical Surveys

There are several kinds of geophysical tests used for stratigraphic profiling and delineation of subsurface geometries. These include the measurement of seismic waves (e.g., seismic refraction surveys, crosshole, downhole and spectral analysis of surface wave tests), as well as electromagnetic techniques (e.g., resistivity, magnetometer, radar). The seismic waves are also useful for the determination of elastic properties of subsurface media, primarily the small-strain shear modulus and Young's modulus of the soil or rock. Electromagnetic methods can help locate anomalous regions (e.g., underground cavities, buried objects, utility lines). The geophysical tests do not alter the soil conditions and, therefore, are classified as non-destructive.

2.3.9.11 Seismic Refraction

The seismic refraction method can be used to measure rock velocities and the depths to bedrock. Seismic velocity charts have been developed from field tests conducted in a variety of materials and are one tool that can be used to determine rippability of rocks.

2.3.9.12 Electrical Resistivity Survey (ER) or Surface Resistivity Method

Resistivity is a fundamental electrical property of geomaterials and can be used to evaluate soil types and variations of pore fluid and changes in subsurface media. In general, resistivity values increase with soil grain size. This resistivity technique has been used to map faults, karstic features, stratigraphy, contamination plumes, buried objects, and other uses. Downhole resistivity surveys can also be performed using electronic probes that are lowered vertically down boreholes or are direct push placed. Downhole resistivity surveys are used in detecting fluid contaminants during geoenvironmental investigations.

2.3.10 <u>Borehole Completion and Site Restoration</u>

The general procedure for the plugging of geotechnical borings is governed by the current OWRB specifications "Plugging Requirements for Geotechnical Borings."

2.4 POST FIELD WORK

2.4.1 <u>Boring Log Preparation</u>

The boring log is the basic record of the geotechnical exploration and provides a detailed record of the work performed and findings of the investigation at that particular borehole location. All portions of the field logs are completed in the field prior to completion of the field exploration.

Once the borehole log is completed, copies of the log will be reviewed by at least one other project Geotechnical engineer for quality assurance purposes. A final boring log is the result of the interpretation of the data obtained within that log as well as the interpretation, experience, and judgement of the engineer of record.

2.4.2 <u>Laboratory Tests</u>

If results of laboratory classification tests indicate that the soil or rock classification made in the field needs to be changed, the Geotechnical engineer of record will make changes.

2.4.3 Reports

See Chapter 3 “Geotechnical Reports” for guidance on what information should be included in the final geotechnical reports.

2.5 LABORATORY TESTS

2.5.1 Overview

ODOT uses both in-house and consultant laboratory testing services. The ODOT soil-testing laboratory, a part of the Materials Division, is capable of performing classification, quality control, and engineering property tests. Classification tests include the basic tests of moisture content, density, Atterberg limits, and grain-size analyses. Some of the engineering property test capabilities include triaxial and direct shear strength test, torsional ring shear tests, resilient modulus testing, consolidation tests, permeability tests, unconfined compression tests, and point-load tests on rock samples. Quality control tests support construction operations and include compaction and oversize corrections.

2.5.2 Soil Classification Tests

Soil classification tests are conducted on samples to define the physical characteristics and condition of the soil. Physical characteristics include the moisture content, density, gradation, and plasticity of the soil. Soil samples used for classification testing can be either from disturbed or undisturbed sampling; however, the sample must be representative of the in-place physical and physicochemical characteristics of the soil. The type of classification test depends on the type of soil sample being tested.

Disturbed samples are often obtained from Standard Penetration Test (SPT) sampling but can also be obtained from hand sampling methods (e.g., backhoe test pits, flights augers cuttings). For undisturbed samples, the classification tests can be conducted on portions of Shelby tube samples or on trimmings obtained during preparation of undisturbed samples for engineering property testing.

2.5.2.1 Soil Gradation

Soil gradation is a description of the distribution of the particles within a representative soil mass based on particle size. Soil gradation is one element necessary for soil classification testing and is expressed as a value represented by a certain passing a predetermined or specified size sieve (e.g. “76.1% passing the No. 4 sieve”). In order to determine fractions of sizes smaller than the No. 200 sieve, a hydrometer test is performed. A hydrometer test is often utilized when the fraction of silt vs clay-size particles is to be determined.

2.5.2.2 Atterberg Limits

Atterberg Limits describe the consistency and plasticity of fine-grained soils with varying degrees of moisture. Atterberg limits provide general indices of moisture content relative to the consistency and behavior of soils. The Liquid Limits (LL) defines a liquid/semi-solid change, while the Plastic Limit (PL) is a solids boundary. The difference is termed the Plasticity Index (PI), PI = LL – PL. The liquidity index is LI = (w – PL)/PI where w = is an indicator of stress history; LI = 1 for normally consolidated (NC) soils and LI = 0 for over-consolidated (OC) soils. Largely, these are approximate and empirical values originally developed for agronomic purposes. Their widespread use by engineers has resulted in the development of a larger number of rough empirical relationships for characterizing soils.

2.5.2.3 Moisture Content

This test is to determine the amount of water present in a quantity of soil in terms of its dry weight and to provide general correlations with strength, settlement, workability, and other properties. Determination of the moisture content of soils is the most commonly used laboratory procedure. The moisture content of soils, when combined with data obtained from other tests, produces significant information about the characteristics of the natural, in place soil. For example, when the in-situ moisture content of a sample retrieved from below the groundwater level approaches its liquid limit, it is an indication that the soil in its natural state is susceptible to larger consolidation settlement.

Serious errors may be introduced if the soil contains other components (e.g., petroleum products or easily ignitable solids). When soil contains fibrous organic matter, absorbed water may be present in the organic fibers as well as in the soil voids. The test procedure does not differentiate between the measurement of pore water and absorbed water in organic fibers (although the procedure does suggest evaluating organic soils at a lower temperature of 60°C to reduce decomposition of highly organic soils). Organic soils are not common in Oklahoma, but the measured moisture content of an organic soil can introduce serious errors into the determination of Atterberg limits.

2.5.2.4 Specific Gravity

The specific gravity of solids (G_s) is a measure of solid particle density and is referenced to an equivalent volume of water. Specific gravity of solids is defined as $G_s = M_s/V_s \times \gamma_w)$ where M_s is the mass of soil solids, V_s is the volume of the soil solids, and γ_w is the specific weight of water. Because many sands are comprised of quartz and/or feldspar minerals and many clays consist of the kaolinite and/or illite clay minerals in composition and because the specific gravity of these minerals are confined to a relatively narrow range, The typical specific gravity values of sands and clays are confined to a relatively narrow range, i.e., $G_s = 2.7 \pm 0.1$.

2.5.3 <u>Soil Engineering Property Tests</u>

Engineering property tests are conducted on soil samples to quantify the strength, compressibility, and permeability of the soil sample. In contrast to most classification tests, engineering property tests are usually costlier and time consuming. The results, however, provide specific data required for engineering analyses. These tests are conducted on undisturbed or reconstituted soil samples obtained during the field exploration program. Undisturbed, in-situ samples are representative of not only the physicochemical characteristics of the soil but also retain the structure resulting from the geologic formation of the soil. Reconstituted soils retain the mechanical properties of the soil, but the moisture content and soil structure is changed during the preparation process. Typically, the preparation process for reconstituted samples involves compaction to a specific dry density and moisture content representative of field construction conditions.

2.5.3.1 Strength Tests

Soil shear strength is influenced by many factors including the effective stress state, mineralogy, soil condition, soil hydraulic conductivity, stress history, sensitivity, and other variables. As a result, shear strength of soil is not a unique property. Laboratory-measured shear strength values will also vary because of boundary conditions during the test, loading rates, and direction of loads.

2.5.3.2 Triaxial Shear

The purpose of triaxial testing is to determine strength characteristics of soils and the variation of strength with lateral confinement, porewater pressures, drainage, and consolidation. Strength can be defined in terms of friction angle and cohesion for total or effective stress conditions or in terms of undrained shear strength. Residual strength values and stiffness (modulus) at intermediate to large strains can be evaluated.

Four types of triaxial shear tests are most common:

1. Unconfined Compression (UC) Test. This test is conducted on a cylindrical sample of cohesive soil without a confining pressure.

2. Unconsolidated Undrained (UU or Q) Test. This test is conducted on undisturbed samples of silts and clays in a triaxial chamber in which the confining pressure and axial load are applied quickly without soil sample drainage.

3. Consolidated Undrained (CU or R) Tests. These tests are similar to the UU test except that the sample is consolidated under the confining chamber pressure before it is loaded axially. Typically, the amount of drainage during consolidation is recorded to confirm that the sample is fully consolidated. Porewater pressures are often recorded during shear. Both total and effective stress strength parameters are determined from the test.

4. Consolidated Drained (CD) Tests. This test is similar to the CU test except that the test is conducted with drainage during application of confining pressure and axial load. The intent of the test is to measure soil behavior without excess porewater pressures during shear.

These tests can be conducted on undisturbed or compacted samples of soil. If the tests are intended to establish the strength parameters of in-place soil, it is critical to have high quality samples to test, and it is critical that the tested sample be representative of the layers of interest. Generally, it is possible to conduct triaxial tests on undisturbed samples of cohesive soils and cohesionless soils with sufficient fines content to allow sampling and set up in the test equipment. Triaxial tests can be conducted on both cohesionless and cohesive soils that have been recompacted to a desired density and moisture content.

Triaxial tests have various advantages and disadvantages relative to their use and the information obtained from the test. The unconfined compression test provides an estimate of the undrained strength of cohesive soils. It is the fastest and the least expensive to perform, but the shear strength obtained from this test method can be unreliable for some soils. Unconsolidated undrained tests generally provide a better estimate of undrained strength but also can be unreliable for some soils. The CU test allows determination of strength parameters for cohesive and cohesionless soils under both total and effective stress conditions and, therefore, is widely used for projects requiring estimates of strength parameters under various stress states. The CD test is rarely conducted on
samples other than for research purposes or where special soils occur because the tests are very expensive due to slow testing rates.

Triaxial shear tests are conducted for many different applications on ODOT projects, as summarized below:

1. UC Tests. UC tests are conducted as a measure of the short-term or minimum strength for soils. For soil testing, the UC test should be calibrated against field measurements

(e.g., vane shear test) or augmented with UU or CU test results. UC tests are also conducted on rock samples to define the intact strength of rock.

2. UU Tests. UU tests are conducted to define the undrained, short-term or minimum, strength of the soil for use in foundation analysis and design. Because of the uncertainties with the UU test, it is usually desirable to supplement these tests with either in-situ vane shear tests, cone penetrometer tests or a limited number of CU tests.

3. CU Tests. CU tests are usually the preferred method for determining the strength parameters required during foundation design and for evaluation of slope stability, wherever the effective confining pressure will change from the current conditions. This applies for most foundation design problems, including embankment and cut slope design, shallow foundation design and retaining wall bearing capacity.

4. CD Tests. CD tests are not frequently used because of the time to conduct these tests as noted previously. However, in some cases for special soils (e.g., fissured clays), the CD test may give the best long-term definition of strength parameters for effective stress analyses.

2.5.3.3 Direct Shear Tests

The direct shear test provides an alternative method of evaluating the strength properties of soil. Direct shear testing is commonly performed on granular soils or compacted materials used for embankment fills and retaining structures. This test provides strength results associated with the drained conditions. A multi-reversal method can be performed on clayey soils to estimate the residual shear strength of clayey soils.

The primary use of the direct shear device on ODOT projects is to estimate the friction angle of granular material. There are other special applications where the project Geotechnical engineer might consider the use of the direct shear test:

- Testing of shear surfaces where the likely failure plane will be along a preferred surface (e.g., slickensides for a slide zone)

- To measure geotextile or geomembrane against soil strengths

2.5.3.4 Torsional Ring Shear Test

The torsional ring shear test, commonly referred to as ring shear test, can be used to determine the fully softened shear strength and the residual shear strength of a soil. These two shear strengths are typically of importance when dealing with heavy clays in slope stability problems.

When constructing embankment fills with heavy clays, it is advisable to design it using the fully softened shear strength values, as this represents a longer-term and more realistic scenario which arguably accounts for weathering which will occur over its lifetime.

2.5.3.5 Consolidation Tests

Where loads (e.g., embankments, spread footings) are placed on soils, the soils settle. The settlement can be either very rapid as in the case of granular soils or the settlement can be slow as in the case of cohesive soils. The calculation of settlement involves many factors, including the magnitude of the load, change in stress at the depths, permeability of soil, water table location, and stress history for the soil. Consolidation testing is performed to determine the effects of these factors on the soil's compressibility. One-dimensional consolidation tests are conducted in an oedometer. Tests can be conducted on either cohesionless or cohesive samples, though most consolidation tests are conducted on undisturbed cohesive samples:

1. Cohesionless Soil. Reconstituted samples of cohesionless soil are sometimes tested to determine the compressibility modulus of the soil under one-dimensional loading conditions.

2. Cohesive Soil. For cohesive soils, test either undisturbed or remolded samples. In most cases, the rate and amount of consolidation of undisturbed samples of native soil are of interest as this information is used to estimate the amount and rate of settlement of the soil after construction of new embankments or foundations. The response to load will depend on the permeability of the soil sample and the maximum stress imposed on the sample. Typically, the consolidation test on cohesive soil involves multiple load increments, resulting in a testing duration of 1 to 2 weeks with traditional test equipment. Using automated equipment, this test can be performed within 1 to 2 days.

The project Geotechnical engineer normally requires consolidation tests to be conducted where the foundation soil includes layers of compressible clays located at depths within two to three foundation widths of the ground surface or where the stress increase from the new foundation loads will be greater than 10%. For most projects, both the amount and rate of settlement are required from the consolidation test.

2.5.3.6 Collapse Potential Test

The purpose of this test is to estimate the collapse potential of soils. The collapse potential test is similar to a consolidation test. This type of test is often used when the project site is characterized by loosely compacted or windblown sediments known as loess. At high moisture contents, these soils collapse and undergo sudden changes in volume. The collapse during wetting occurs due to the destruction of clay, lime or calcium carbonate binding which provide the original strength of these soils. Collapse can also occur as a result of compaction of cohesive soils very dry of optimum moisture conditions.

2.5.3.7 Swell Potential Test

The one-dimensional oedometer swell potential test is used to estimate the percentage swell and swelling pressures developed by the swelling soils. This test can be performed on undisturbed or compacted specimens. The potential for swell depends on the mineralogical composition and condition of the soil. The percentage swell of a soil depends on the amount of clay, its relative density, the compaction moisture & density, permeability, location of the water table, presence of vegetation & trees, and overburden stress.

2.5.3.8 Permeability Test

The purpose of permeability testing is to determine the rate of flow of water through soils. Information from the permeability test is used in selecting road subbase material, backfill for retaining walls and, sometimes, in the design of retention ponds. Permeability tests can be conducted on either undisturbed or remolded soil using either flexible wall or fixed-wall systems. Either falling head or constant head tests can be conducted. The selection between the equipment and methods depends on the soil type. Permeability values may also be calculated from consolidation test results on cohesive soils.

2.5.3.9 Double Hydrometer Test

This test method provides an indication of the natural dispersive characteristics of clay soils. Dispersivity refers to the tendency of a soil to easily wash or erode away with the slightest contact of water. This is of particular concern in situations where erosion is not a desirable effect, such as an embankment fill or cut section. Very large portions of an embankment can be washed away after one or two rain events. This test method is applicable only to soils with a plasticity index greater than 4 and soils with greater than 12% fraction finer than 5 microns. This test method may not identify all dispersive clay soils.

2.5.3.10 Crumb Test

The crumb test method provides a simple, quick method for field or laboratory identification of a dispersive clay soil. The internal erosion failures of a number of homogeneous earth dams, erosion along channel or canal banks, and rainfall erosion of earthen structures have been attributed to colloidal erosion along cracks or other flow channels formed in masses of dispersive clay.

The crumb test is a relatively accurate positive indicator of the presence of dispersive properties in a soil. The crumb test, however, is not a completely reliable negative indicator that soils are not dispersive. The crumb test can seldom be relied upon as a sole test method for determining the presence of dispersive clays.

The double-hydrometer test and pinhole test are test methods that provide valuable additional insight into the probable dispersive behavior of clay soils. These test methods provide a qualitative indication of the natural dispersive characteristics of clayey soils. These test methods are not applicable for soils with less than 12% fraction finer than 5 microns and with a plasticity index less than or equal to 4. The crumb test method has some limitations in its usefulness as an indicator of dispersive clay. A dispersive soil may sometimes give a nondispersive reaction in the crumb test. Soils containing kaolinite with known field dispersion problems have shown nondispersive reactions in the crumb test. However, if the crumb test indicates dispersion, the soil is probably dispersive.

2.5.3.11 Pinhole Test

The pinhole test provides one method of identifying the dispersive characteristics of clay soils that are to be or have been used in earth construction. The piping failures of a number of homogeneous earth dams, erosion along channel or canal banks and rainfall erosion of earthen structures have been attributed to the colloidal erosion along cracks or other flow channels formed in masses of dispersive clay. This test method models the action of water flowing along a crack in an earth embankment.

Other indirect tests (e.g., double hydrometer test or the crumb test) that measure the turbidity of a cloud of suspended clay colloids as an indicator of the clay dispersivity, and chemical tests that relate the percentage of sodium to total soluble salt content of the soil, are also used as indicator tests of clay dispersibility.

2.5.4 <u>Quality Control Testing</u>

Quality control testing refers to testing typically performed during construction operations to ensure specifications are being met as the design is being followed.

2.5.4.1 Soil Compaction

The purpose of a soil compaction test is to determine the maximum dry density and optimum moisture content for a given soil and compaction effort. Compaction tests are performed using disturbed, prepared soils with or without additives. Dry density is determined based on the moisture content and the unit weight of compacted soil. The water content at which this dry density occurs is termed as the optimum moisture content (OMC).

In the construction of highway embankments, earth dams, retaining walls, structure foundations and many other facilities, loose soils must be compacted to increase their densities. Compaction increases the strength and stiffness characteristics of soils. Compaction also decreases the amount of undesirable settlement of structures and increases the stability of slopes and embankments. To provide a relative measure of compaction, the concept of relative or percent compaction is used. Percent compaction is the ratio, expressed as a percentage, of the density of compacted or natural in-situ soils to the maximum density obtainable in a compaction test. Often it is necessary to specify the achieving of a certain level of percent compaction (e.g., 95%) in the construction or preparation of foundations, embankments, pavement subbases and bases, and for deep-seated deposits (e.g., loose sands). The design and selection of a placement method to improve the strength, dynamic resistance, and consolidation characteristics of deposits depend heavily on relative compaction measurements.

Where additives (e.g., Portland cement, lime, fly ash) are used to determine the maximum density of mixed compacted soils in the laboratory, care should be taken to duplicate the expected delay period between mixing and compaction in the field.

Compaction tests are performed using either the Standard Proctor method or the Modified Proctor method. These two methods differ according to the amount of compactive effort during the test, with the Modified Proctor method having roughly four times the compactive effort as the Standard Proctor method.

For the same soil, this difference in compactive effort results in a difference in percent compaction. As a rule-of-thumb, a given soil compacted to 95% compactive effort with the Standard Proctor method is equivalent to a Modified Proctor value of roughly 90%.

2.5.4.2 Relative Density

Relative density is a measure of the compacted density of a cohesionless soil. Relative density/unit weight expresses the degree of compactness of a cohesionless soil with respect to the loosest and densest condition as defined by standard laboratory procedures. Only when viewed against the possible range of variation in terms of relative density/unit weight, can the dry density/unit weight be related to the compaction effort used to place the soil in a compacted fill or indicate volume change and stress-strain tendencies of soil when subjected to external loading.

The Standard Proctor compaction test is not appropriate for establishing the compaction parameters for cohesionless soils. Relative density values can vary from 0 to 100%, depending on the condition of the cohesionless soil. Typically, the relative density can be categorized as follows:

Density Value Range	Density Category
0% to 35%	loose
35% to 65%	medium dense
65% to 100%	dense

Ideally, field compaction of cohesionless soils should achieve relative densities of 65% or greater. Determination of relative density requires measurement of the maximum index density (densest state) and minimum index density (loosest state).

2.5.4.3 In-Place Density (Nuclear Gauge)

The Nuclear In-Place Density test method is useful as a rapid, nondestructive technique for in-place measurements of wet density, water content, dry density, and percent compaction of soil and soil-aggregate. The test method is used for quality control and acceptance testing of compacted soil and soil-aggregate mixtures as used in construction and for research and development. The nondestructive nature allows repetitive measurements at a single test location and statistical analysis of the results.

2.5.5 Chemical Tests

2.5.5.1 Soils Tests

2.5.5.1.1 pH Test

AASHTO T289-91 describes the methods for sample preparation and measurement of pH using a 1:1 soil to water ratio test specimen. The major use of the pH is to supplement soil resistivity measurements for corrosion studies. Some metals are more sensitive to the pH of their environment than others; therefore, information on the stability of a metal as a function of pH and corrosion potential is important.

2.5.5.1.2 Sulfates Test

This test measures the soluble sulfate content of soil using colorimetric methods. The results are used to determine whether chemical calcium-based additives (e.g., lime, fly ash, cement kiln dust, Portland cement) would be susceptible to adverse chemical reactions.

2.5.5.1.3 Exchangeable Sodium Percentage and Sodium Absorption Ration Tests

The presence of exchangeable sodium is a contributing chemical factor to dispersive clay behavior. The basic parameter to quantify this effect is exchangeable sodium percentage (ESP), where:

ESP is equal to exchangeable sodium divided by cation exchange capacity times 100

Soils with ESP of 10 or above that are subject to having free salts leached by seepage or relatively pure water are classified as dispersive.

Criteria that have been used to classify dispersive clays using ESP data are:

ESP Value	Degree of Dispersion
<7	Non-dispersive
7 to 10	Intermediate
>10	Dispersive

Another parameter commonly evaluated to quantify the role of sodium with respect to dispersion when free salts are present is the sodium absorption ratio (SAR). The SAR method is not applicable if no free salts are present.

2.5.5.2 Water Tests

2.5.5.2.1 Chloride (CL)

Chlorides in water, particularly water used for compaction, will make adjusting compaction moisture content conditions difficult.

2.5.5.2.2 Sulfate (SO_4)

The determination of sulfate and other dissolved constituents is important in identifying the source of brackish water. This test method covers the turbidimetric determination of sulfate ion.

2.5.5.2.3 Electrical Conductivity Test

The purpose of electrical conductivity test is to measure the ability of a material to conduct electric current. Test methods are applicable for such purposes as impurity detection and, in some cases, the quantitative measurement of ionic constituents dissolved in waters. These include dissolved electrolytes in natural and treated waters (e.g., boiler water, boiler feedwater, cooling water, saline and brackish water). The concentration levels may range from trace levels in pure waters to significant levels in condensed steam or pure salt solutions.

2.5.6 <u>Rock Property Tests</u>

Laboratory rock testing is performed to determine the strength and elastic properties of intact specimens and the potential for degradation and disintegration of the rock material. The derived parameters are used for the design of rock fills, cut slopes, shallow and deep foundations, and the assessment of drainage protection materials (riprap). Deformation and strength properties of intact rock specimens aid in evaluating the larger-scale rock mass behavior that is controlled by joints, fissures and discontinuity features (e.g., spacing, roughness, orientation, infilling), water pressures, and ambient geostatic stress state.

Common laboratory tests for intact rocks include measurements of strength (e.g., point load strength index, unconfined compressive strength), stiffness (e.g., ultrasonics, elastic modulus), and durability (e.g., slaking, abrasion).

2.5.6.1 Sample Requirements and Uses of Rock Tests

The Geotechnical engineer needs to have a clear understanding of the purpose, and requirements of any rock test requested from the laboratory. These needs involve a different set of considerations from those associated with soil testing. Specifically, secondary factors (e.g., fractures, discontinuities within the rock) are often more important for engineering design than the properties of the intact rock. An example of this would be the stability of a roadside cut. Stability of the cut will usually be determined by the secondary factors rather than the intact rock strength. For good practice, the Geotechnical engineer should meet with an engineering geologist before assigning rock tests to discuss the types of tests that would be most useful to the project.

2.5.6.2 Point Load Index (Strength) Test

The purpose of point load index, or strength, testing is to determine the strength classification of rock materials. This type of test provides an indication of strength much more quickly and inexpensively than an unconfined compression test.

2.5.6.3 Unconfined (Uniaxial) Compression Test

The purpose of this test is to determine the uniaxial compressive strength of rock. The uniaxial compression test is the most direct means of determining rock strength.

2.5.6.4 Other Rock Tests

The following describe other rock tests that may be required for an ODOT project:

1. Direct Shear Tests. This type of test is most useful when evaluating the strength along planes of weakness within the rock structure. These weaker seams can serve as failure planes during loading, particularly for slope cuts where the bedding planes dip in a direction with the slope. In this case, it may be important to characterize the strength of the weaker material in the direction of dip.

2. Slake Durability Test. The purpose of slake durability testing is to determine the durability of shale, or other weak or soft rocks subjected to cycles of wetting and drying. When some shales are newly exposed to atmospheric conditions, they can degrade rapidly and affect the stability of a rock fill or cut, the subgrade on which a foundation is to be placed, or the base and side walls of drilled shafts prior to placement of concrete. This test is typically performed on shales and other weak rocks that may be subject to degradation in the service environment.

3. Micro-Deval Test. The Micro-Deval abrasion test is a test of coarse (and fine) aggregates to determine abrasion loss in the presence of water and an abrasive charge. The Micro-Deval abrasion test is useful as part of a quality control or quality assurance process for detecting changes in aggregate produced from an aggregate source.

4. LA Abrasion. This test method is an indicator of the relative quality of various sources of aggregate having similar mineral compositions. Like the Micro-Deval test, it measures the degradation of aggregates when subjected to abrasion, but it does so with dry aggregates instead of wet or soaked aggregates.

5. Aggregate Durability/Soundness. These tests measure an aggregate's resistance to degradation by weathering, specifically freeze-thaw cycles. Durable aggregates are less likely to degrade in the field and cause pavement distress or failure.

2.6 SOIL AND ROCK CLASSIFICATION

One of the objectives of a geotechnical investigation is to define the characteristics, properties, thickness and lateral extent of soils and underlying bedrock within a given project area. Rock lithology, climate, topography, time and geologic history influence the process of soil formation. To define and describe subsurface conditions, it is necessary to use standard classification systems for soil and rock.

2.6.1 Soil Classification

The purpose of soil classification is to enable qualitative assessment of soil engineering properties. The soil classes describe the soil in terms of grain size and plasticity. Soil type in a classification system should be described in terms easily recognized and understood by field and laboratory technicians, geologists and Geotechnical engineers. To be of value, soil classification systems should be standardized and applicable across all geographical regions. Considering engineering applications, two use-type or experience-based classification systems are commonly used for transportation applications — AASHTO Soil Classification System and Unified Soil Classification Systems (USCS). In addition, ODOT uses the US Department of Agriculture (USDA) Soil Taxonomy Classification System in conjunction with a Pedological Survey to identify and classify natural soil profiles in their undisturbed state as developed in their natural environment along a proposed roadway alignment.

2.6.1.1 ASSHTO Soil Classification System

The AASHTO soil classification system is useful in determining the relative quality of the soil material for use in earthwork structures, particularly embankments, subgrades, subbases and bases. According to this system, soil is classified into seven major groups, A-I through A-7. Soils classified under groups A-I, A-2 and A-3 are granular materials where 35% or less of the particles pass through the No. 200 sieve. Soils where more than 35% pass the No. 200 sieve are classified under groups A-4, A-5, A-6 and A-7. These are mostly silt and clay-type materials. The classification system is based on the following criteria:

1. Grain Size. The grain size terminology for this classification system is as follows:

    - Gravel fraction passing the 2-inch sieve and retained on the No. 10 sieve

    - Sand fraction passing the No. 10 sieve and retained on the No. 200 sieve

    - Silt and clay fraction passing the No. 200 sieve

2. Plasticity. The term silty is applied when the fine fractions of the soil have a plasticity index of 10 or less. The term clayey is applied when the fine fractions have a plasticity index of 11 or more.

3. Cobbles and Boulders. If cobbles and boulders (size larger than 3 in) are encountered, they are excluded from the portion of the soil sample on which classification is made. However, the percentage of material is recorded.

To evaluate the quality of a soil as a highway subgrade material, a number called the group index (GI) is also incorporated along with the groups and subgroups of the soil. In general, the quality of performance of a soil as a sub grade material is inversely proportional to the group index.

2.6.1.2 Unified Soil Classification System (USCS)

The Unified Soil Classification System (ASTM D 2487) groups soils with similar engineering properties into categories based on grain size, gradation and plasticity.

2.6.1.3 USDA Soil Taxonomy

Soil classifications describe a soil in sufficient detail to permit engineers to recognize features significant to design and, if need be, to obtain samples in the field. Highway engineers find that the system of soil classification and taxonomy can be used in the general identification of soil, from which they can classify the various soil materials for engineering purposes. The USDA classification system is used primarily for agronomic purposes. However, after testing and correlation with engineering properties and performance, it becomes a system of classification suitable for use by the highway, railroad or airport engineer.

2.6.1.4 Natural Resources Conservation Service (NRCS)

All of Oklahoma has been mapped by the NRCS. The more recently published surveys have the soil series classifications listed therein. Copies of the county soil survey reports are available in the counties where the construction activity is to take place. These can be obtained at the local county NRCS field offices or the County Cooperative Extension Office. The reports contain maps based on aerial photographs. The NRCS Web Soil Survey (see Ref. 1) internet site is also a good source for county soils maps and information.

2.6.2 Rock Classification

Rock classification uses basic measured properties such as Rock Quality Designation (RQD - see ASTM D 6032), rock strength, and observed rock mass characteristics (e.g., layering, bedding, joints, joint condition) to establish rock mass ratings.

Rock Mass Rating: In determining rock strength for transportation facilities constructed in, on or of rock, it is important to account for the presence of discontinuities (e.g., joints, faults, bedding planes). In most conditions, the rock mass strength properties, rather than the intact rock properties must be determined for use in design. The rock mass is the in-situ, fractured rock that will almost always have significantly lower strength than the intact rock because of discontinuities that divide the rock mass into blocks. Therefore, the strength of the rock mass will depend on the shear strength of the surfaces of the blocks, spacing and continuous length of the discontinuities and their alignment relative to the direction of loading.

2.6.3 Report Presentation

The boring logs are the basic record of geotechnical exploration and laboratory analyses. The log provides a detailed record of the work performed and the findings of the field investigations and results from the laboratory tests. Information from the field investigation will have been entered into the computer program gINT (geotechnical/geoenvironmental reporting software) or other programs approved by ODOT. The project Geotechnical engineer will revise the field information according to the laboratory results to produce the final boring log. For additional guidance, see Chapter 3 “Geotechnical Reports.”

2.7 GEOTECHNICAL INVESTIGATIONS

2.7.1 Preliminary Soils Survey

2.7.1.1 Pedological and Geological Survey

A Pedological and Geological Survey is required for new highway alignments, new construction parallel to existing highway alignments, and new construction requiring the raising of grade on and above existing highway alignment.

Pedological soil surveys are performed for new alignments. A map of the known soil series which spans the proposed alignment is obtained, typically from NRCS. The types of soils which are predominant across that new alignment are then sampled and tested according to specifications. Sampling and testing soils in this manner allows one to specifically target the soils known to be predominant in the area as opposed to “blindly” sampling at discrete intervals (say, every 500 feet along the proposed alignment). This in turn greatly reduces the number of total soil samples needed for sampling and testing. All soil series are slightly different and even a given soil series can vary greatly across a given county or state. For example, the topsoil may range in thickness by many inches. Hence, on a project-level/design-level situation, the soil series is sampled to determine its characteristics at that specific project. Knowing the topsoil is up to 19 inches thick along a 5-mile-long project has a major impact on bidding, since per ODOT specifications, topsoil is to be removed.

A Pedological Soils Survey is reliant on knowledge of the soil series mapping units and the corresponding taxonomic classification system established by the Natural Resources Conservation Service (NRCS). The general procedures for conducting pedological activities are outlined in the current specifications for performing geotechnical work.

2.7.1.2 Shoulder Soil Survey

This survey is required for the widening of existing pavements at grade, which includes such projects as adding shoulders, lanes, and medians to existing pavements. The purpose of the shoulder soil survey is to evaluate existing ground conditions in advance of a road widening project. Information gathered is used for pavement design and can highly influence construction costs. The procedure for performing a shoulder soil survey is outlined in the current specifications for geotechnical work.

2.7.1.3 In-Place Soil Survey

This survey is required for new construction grading when the design calls for separation of grading and paving contracts. It may also be used to evaluate the subgrade of existing pavement sections that are to be reconstructed with no change in grade or alignment. The purpose is to evaluate subgrade conditions and gather information for pavement designers. The general procedure for conducting the In-Place Soil Survey is outlined in the specifications for geotechnical work.

2.7.1.4 Pavement and Subgrade Soil Survey

This survey is required when the properties of an existing pavement structure and the underlying subgrade soils are needed for evaluation of the pavement load capacity and for an overlay design. The Falling Weight Deflectometer (FWD) is required for evaluating the pavement structure (surface, base, and subbase). The general procedure for conducting pavement and subgrade soil surveys is outlined in the specifications for geotechnical work.

2.7.1.5 Borrow Pit Investigations

A borrow pit investigation is required where select subgrade topping or borrow is specified. These investigations are conducted to obtain subsurface information at a potential borrow site that can be used by engineers and contractors at the planning, bidding, and construction stages.

2.7.1.6 Resilient Modulus Testing

The Resilient Modulus is a measure of stiffness of the pavement material, that is, the amount of elastic strain energy a material can absorb before it yields. Resilient modulus testing is required for the pavement design of all highway projects, unless waived by the Pavement Design engineer for temporary pavements, shoulder widening, etc.

2.7.1.7 Detailed Soil Investigation

A Detailed Soil Investigation is required for geotechnical problems related to roadway designs, which include embankment and foundation soil settlement and stability, cut and natural slope stability, problem soils related to roadway subgrades and embankments, roadway structures, and construction recommendations. A detailed soil investigation of these problems is typically required in conjunction with the Pedological and Geological Survey. Interpretation and judgment of pedological and geological site conditions is the responsibility of the Geotechnical engineer. The following provides explanations of the various types of detailed geotechnical investigations and when they may be performed:

1. Embankment and Foundation Soil Settlement and Stability (Embankments Between 0 to 10 ft. Above Natural Ground Line). Estimates of embankment and underlying foundation soil settlement, slope stability, and design slopes are required. These estimates are made by assuming reasonable parameters for anticipated embankment and foundation soils based on the soil series types occurring within the project extent. These estimates are required for embankments crossing each soil series encountered along the project alignment.

2. Embankment and Foundation Soil Settlement and Stability (Embankments Greater than 10 ft. Above Natural Ground Line). Estimates of embankment and underlying foundation soil settlement and stability are required. Borings are to be typically spaced every 200 ft (erratic conditions) to 500 ft (uniform conditions), with at least one boring made in each pedological soil unit.

3. Cut and Natural Slope Stability. For cut slopes greater than 30 ft below the natural ground line in soil, both the end of construction and long-term slope stability conditions are analyzed. If slope materials are over consolidated, the residual shear strength is used in the long-term slope stability analysis.

4. Problem Soils Related to Roadway Subgrade and Embankments. Additional field exploration and laboratory testing/analysis are required to determine the long-term performance and/or suitability of the following soil and rock that may be incorporated into the roadway subgrade and embankment or found in the foundation soils below the roadway embankment:

    - Organic soils
    - Normally consolidated clays
    - Expansive clays and shales
    - Dispersive soils
    - Collapsible soil
    - Degradable shales
    - Caliche
    - Mine spoils (all types) and caves
    - River of stream meander loops and cutoffs
    - Karst (e.g., gypsum, limestone)

    Consideration of these soils and conditions is coordinated with the Pedological and Geological Survey and Borrow Pit Investigation. The interpretation and judgment of these soil conditions is the responsibility of the Geotechnical engineer.

5. Roadway Structures. The bearing capacity, settlement, and stability of roadway structures are typically checked. Please reference the current specifications for geotechnical work.

6. Construction Recommendations. Construction and/or general recommendations should be provided as part of any geotechnical report. These recommendations may include things such as recommended further work to be performed, rippability, presence of groundwater and how it affects construction and/or design, or anything else that may be pertinent to the design, bidding, or construction process of the project.

2.7.1.8 Geological Investigation

Typically, a geological field investigation is required for any or all of the following:

- Rock cuts of 10 ft or greater
- Shallow rock mapped with a proposed cut section
- Rock mechanics analysis
- Geological hazards
- Rock fills

A geotechnical field investigation may consist of the following elements:

- Borings
- Slope stability analysis
- Rippability ratings
- Evaluation of geological hazards
- Shear strength of rock fills
- Evaluation of excavated rock for use as a source aggregate
- Geological statements

The geological investigation is done in conjunction with the Pedological and Geological Survey. The investigation may include borings, seismograph surveys, rock stability analysis, rippability, geologic hazards such as sinkholes or landslides, rock fill embankments and geologic assessment.

2.8 PROBLEM SOILS

2.8.1 Expansive Soils

Expansive soils are typically cohesive soils that undergo volume change with changes in moisture content. Unlike collapsible soils, expansive soils tend to increase in volume (i.e., swell) as the moisture content of the soil is increased and decreased in volume (i.e., shrink) as the moisture content of the soil is decreased. Although the expansion potential of a soil can be related to many factors (e.g., soil condition, soil structure and fabric, environmental conditions), it is primarily controlled by clay mineralogy. Soils that contain low-plasticity minerals tend to exhibit lower shrink/swell potential than soils containing high-plasticity minerals. Expansive soils are found throughout the US; however, damage caused by expansive clays is most prevalent where the climate is considered semi-arid, and periods of intense rainfall are followed by long periods of drought. This pattern of wet and dry cycles results in periods of extensive near surface drying and desiccation crack formation. During intense precipitation, water is added to the deep cracks permitting the soil to swell; upon drying, the soil will shrink. This weather pattern results in cycles of swelling and shrinking that can be detrimental to the performance of pavements, slabs on-grade, and retaining walls.

Deep-seated volume changes in expansive soils are rare. More common are volume changes within the upper few feet of a soil deposit. These upper few feet are more likely to be affected by seasonal moisture content changes due to climatic changes. The zone over which volume changes are most likely to occur is defined as the active zone. The active zone can be evaluated by plotting the moisture content with depth for samples taken during wet and dry seasons. The depth at which the moisture content becomes nearly constant is the limit of the active zone depth, which is also referred to as the depth of seasonal moisture variation. The active zone is an important consideration in foundation design. In the design of piles or drilled shafts, it is important to recognize that full side friction resistance may not be realized in this zone. As the soil undergoes cycles of shrinking and swelling, it may lose contact with the pile or shaft. Alternatively, as the soil swells, it may impose significant uplift pressures on the foundation element.

In the field, the presence of surface desiccation cracks and/or fissures in a clay deposit is an indication of expansion potential. Experience has indicated that the most problematic expansive near-surface soils are typically highly plastic, stiff fissured over consolidated clays. To identify expansive soils in the laboratory, several classification methods have been developed. Currently, there is not a standard classification procedure; different methods are used in various locations across the US. Typically, methods include the use of Atterberg Limits and/or soil suction to qualitatively describe a soil as having low, medium, high or very high expansion potential. Generally, soils with a plasticity indexes less than 15% will not exhibit expansive behavior.

2.8.2 Dispersive Clay

Using dispersive clay soils in hydraulic structures, embankment dams or other structures (e.g., roadway embankments) can cause serious engineering problems if these soils are not identified and treated appropriately. This problem is worldwide and structural failures attributed to dispersive soils have occurred in many countries. Numerous investigations of these materials and their properties have been performed and are reported in international technical literature.

In the past, clay soils were considered highly resistant to erosion by flowing water; however, it is generally recognized that highly erodible clay soils do exist in nature. Some natural clay soils disperse or deflocculate in the presence of relatively pure water and are, therefore, highly susceptible to erosion and piping. The tendency for dispersive erosion in a given soil depends on several variables (e.g., mineralogy, chemistry of the clay, and dissolved salts in the water in soil pores and in the eroding water). Slow-moving water rapidly erodes such clays, even when compared to cohesionless fine sands and silts. When dispersive clay soil is immersed in water, the clay fraction behaves like single-grained particles, that is, the clay particles have a minimum of electrochemical attraction and fail to closely adhere to, or bond with, other soil particles. Therefore, dispersive clay soil erodes in the presence of flowing water when individual clay platelets are split off and carried away. Such erosion may start in a drying crack, settlement crack, hydraulic fracture crack or other channel of high permeability in a soil mass. The principal difference between dispersive clays and ordinary erosion-resistance clays appears to be the nature of the cations in the pore water of the clay mass. Dispersive clays have a preponderance of sodium cations, whereas ordinary clays have a preponderance of calcium, potassium and magnesium cations in the pore water.

2.8.2.1 Geographic and Climatic Factors

In areas of sloping topography where dispersive clays exist, a characteristic pattern of surface erosion is evidenced by jagged, sinuous ridges and deep rapidly forming channels and tunnels. In gently rolling or flat areas, there is frequently no surface evidence of dispersive clay due to an overlying protective layer of silty sand or topsoil from which the dispersive clay particles have been removed. The absence of surface erosion patterns typical of dispersive clays does not necessarily indicate that no dispersive clays are present. Dispersive clay soils can be red, brown, gray, yellow or various combinations of these colors. Black soils with obviously high organic content are not dispersive. Nearly all fine-grained soils tested known to be derived from in-situ weathering of igneous and metamorphic rocks have been non-dispersive, as well as all soils derived from limestone.

2.8.2.2 Engineering Consequences

Most studies reported in the literature have shown that failures of structures built of dispersive clay soils occurred on first wetting. All failures were associated with the presence of water and cracking by shrinkage, differential settlement, or construction deficiencies. These failures emphasize the importance of early recognition and identification of dispersive clay soils; otherwise, the problems they cause can result in sudden, irreversible, and catastrophic failures.

2.8.2.3 Piping Failure Mechanism

Piping failure occurs in an earth embankment dam when a concentrated leak emerging at the downstream side is caused by water flowing through the pores of the soil. The erosion starts first at the discharge end of the leak, causing a local concentration of seepage and erosion forces. Erosion progresses upstream forming a tunnel-shaped passage or pipe until it reaches the water source, at which time, a rapid catastrophic failure may result. Such erosion occurs mainly in cohesionless soils or soils with low cohesion that have little resistance to the plucking forces of seeping water.

2.8.2.4 Identification of Dispersive Clay Soils

Identification of dispersive soils starts with field reconnaissance investigations to determine if there are any surface indications such as unusual erosional patterns with tunnels and deep gullies, concurrent with excessive turbidity in any storage water. Areas of poor crop production and stunted vegetative growth may indicate highly saline soils, many of which are dispersive. However, dispersive soils can also occur in neutral or acidic soils and can support lush grass growth. Therefore, although surface evidence can give a strong indication of dispersive soils, lack of such evidence does not in itself preclude the presence of dispersive clay at depth and further exploration is needed. Dispersive clays cannot be identified by the standard laboratory index tests (e.g., visual classification, grain size analysis, specific gravity, Atterberg limits) and, therefore, other laboratory tests have been devised for this purpose. Clay soils are routinely tested for dispersive characteristics during design studies for hydraulic structures where clay may be subjected to potential erosion and piping.

2.8.2.5 Laboratory Tests

The five laboratory tests most generally performed to identify dispersive clays are the crumb test, the double hydrometer test, the pinhole test and soil pore water chemistry, specifically the sodium absorption ratio (SAR) and exchangeable sodium percentage (ESP).

2.8.3 <u>Shales</u>

Shales are fine-grained sedimentary rocks consisting of compacted and hardened clay, silt or a combination of the two particle sizes. Shales normally contain at least 67% clay, with the remainder being silt with a chemical or crystalline material acting as a cementing agent. Shales are by far the most common of the sedimentary rocks. They are usually identified in the field by their laminated or fissile appearance. Shales can be any color. They are usually gray, brown, olive or black in the eastern half of Oklahoma and shades of red often with greenish-gray spots or layers in western Oklahoma. The reddish shales of western Oklahoma commonly do not exhibit strong laminations but are more massive or blocky in appearance. They usually exhibit a smooth, sometimes waxy feel.

On many transportation projects, construction activities involve the use of potentially degradable materials. Although these materials may at first exhibit rock-like characteristics, they have the potential to degrade to soil-size particles. The gradual, but ultimate degradation of the rock to the original parent soil material can occur within minutes or after several years of exposure to air and/or water. Shale, the most common member of this family of materials, can generically be considered to include claystone, siltstone and mudstone.

In many parts of Oklahoma, high-quality granular material is not locally available for use as borrow material in the construction of earth embankments and/or rockfill. As a result, degradable materials that appear at first to be competent granular materials are used. However, once in contact with water, these materials may degrade causing problems and/or failures during the service life of the structure. Foundations built on these materials are also at risk for failure. Drilling and other construction activities often introduce water, initiating the degradation process. Deep foundations, especially rock-socketed drilled shafts, which may be designed considering intimate contact with and support from the rock interface, may fail as the materials degrade and contact is lost. Additionally, highway cuts are often excavated in degradable rock. The exposure of the cut to atmospheric conditions may result in significant degradation during the service life of the cut slope.

Many rock types are prone to degradation when exposed to the cyclic wet/dry and freeze/thaw weathering processes. Rock types that are particularly susceptible to degradation due to these processes are poorly indurated shale and claystone exhibiting high clay content. The degradation can take the form of swelling, weakening and ultimately disintegration. The effect of degradation on slope stability can range from surficial sloughing and gradual retreat of the face to catastrophic slope failures resulting from the significant loss of strength. In sedimentary rock formations comprising alternating beds of resistant sandstone and relatively degradable shale, the weathering process can develop overhangs in the sandstone and produce a rockfall hazard. In shallow foundations, the assumed bearing capacity of the material may decrease as the foundation material degrades resulting in settlements and/or foundation failures. In deep foundations, the assumed end bearing and/or side friction may decrease over time.

Where potentially degradable materials are encountered, it is essential to establish the anticipated competency of the materials over the service life of the project. An assessment of the time required for significant degradation relative to the service life of the structure should be evaluated. Commonly, the point load test, the slake durability test, and the jar slake test are used to make this assessment. Classification systems and index values have also been developed for identifying the behavior of shales.

2.8.4 Sinkholes, Gypsum, and Abandoned Mines

Sinkholes are a common occurrence in geologic formations where gypsum occurs in layers of varying thickness. The gypsum is dissolved by moving groundwater or infiltrating surface water. As the gypsum dissolves, channels, tunnels or caverns are formed that can collapse and form sinkholes. Collapse of abandoned mines pose similar hazards for geotechnical structures. A thorough subsurface geotechnical investigation, including detailed geophysical studies, is necessary to identify potential sinkhole hazards.

2.8.5 Sulfate Rich Soils

The addition of calcium-based stabilizers to soils reduces their plasticity, shrink/swell potential, and compressibility while increasing their workability, strength, and durability. However, when the calcium-based additive is applied to soils containing soluble sulfate, the resulting reactions can actually increase the volume change tendencies of the soil to extremes. The chemical reaction of a calcium-based additive and the aluminum present in soil particles, water and soluble sulfates results in crystalline growth of the mineral ettringite. Localized concentration of soluble sulfates combined with the other ingredients can result in surface heaving of overlying pavements 3 inches to 12 inches. Natural soil sulfate is most often encountered in the form of gypsum. Gypsum crystals can be visible to the naked eye or can be sub-microscopic. Gypsum can occur below an existing water table where solution cavities may be prevalent. If gypsum occurs above the water table, it commonly precipitates interstitially within the geologic sediment or as varying sized lenses in the sediment profile. Depending on the geologic formation, solution cavities could exist above the water table. Soils and shales containing gypsum are common in Oklahoma. Most of the problems with gypsum occur in the western half of the State.

Gypsum bearing soils and the potential for interaction with transportation structures depend on observed gypsum occurrences. For ODOT projects where gypsum/sulfate-rich soils may be present, especially in Districts 4, 5, 6 and 7, the soluble sulfate content is determined using ODOT OHDL-49 test method. If the soluble sulfate content is greater than 500 ppm, additional samples for soluble sulfate testing should be taken throughout the length of the project represented by the sample. If the soluble sulfate content is greater than 1000 ppm for any of the additional samples, the use of calcium-based additives may not be suitable or additional steps such as staged chemical stabilization, or mellowing may need to be considered. If the soluble sulfate content is greater than 8000 ppm, the use of calcium- based additives is not recommended.

2.9 REFERENCES

References specifically cited in Chapter 2 are numbered below, followed by additional pertinent technical information sources presented alphabetically by author.

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Chapter 3

GEOTECHNICAL REPORTS

ODOT GEOTECHNICAL MANUAL

January 2025

3 GEOTECHNICAL REPORTS

This chapter provides general information on the types of geotechnical reports generated for ODOT. Detailed information regarding the requirements of geotechnical reports for pre-construction, design-level work is provided in the ODOT Geotechnical Specifications. (Reference 1)

3.1 GENERAL

Broadly speaking, geotechnical reports at ODOT fall under the following general categories:

1. <u>Pre-Construction, Design-Level</u>: Design level geotechnical work is often performed years in advance of construction. The Roadway, Bridge, and Traffic Design Divisions are most often involved in this type of geotechnical work. Examples of design projects include new bridges, interchanges, new alignments, retaining walls, embankments, cut and fill sections, etc.

2. <u>Construction Related</u>: This is work performed during construction. This work may be related to quality control/quality assessment type items, such as verifying the friction angle of mechanically stabilized earth (MSE) backfill. Construction related geotechnical work may be needed if questions arise about rippability of rock or to verify if a change of conditions has occurred. Such work is often performed by the Geotechnical Branch in the Materials Division because the information is usually needed quickly.

3. <u>Post-construction and/or Forensic</u>: This type of work usually occurs after construction is complete, sometimes after many years. It is typically forensic in nature after a problem has arisen. Example of this work may include:

    - Slope stability analysis and repair recommendations for slope failures
    - Determining the cause and other issues relating to settlement
    - Determining cause of unexpected/premature pavement failures
    - Data collection for field divisions to help in maintenance decisions

The Geotechnical engineer responsible for preparing the geotechnical report should have broad experience in civil and Geotechnical engineering with respect to transportation work, a good basic knowledge of the soils and geology of Oklahoma, and an understanding of the designs that are required to remedy the problems at a certain specific location(s) on soil types.

The Geotechnical engineer should anticipate possible design and construction issues and provide recommendations and solutions to mitigate or eliminate problems as well as any comments concerning possible limitations or other construction problems. The recommendations should be brief, concise and include the reasons for recommendations and be understood well by a non-geotechnical user.

3.1.1 <u>Pre-Construction, Design-Level Geotechnical Reports</u>

Before a new Bridge or Roadway design project can be finalized, much data must be gathered, including geotechnical data. The detailed minimum requirements for geotechnical work for the various types of Design projects are contained within the ODOT Geotechnical Specifications. Additionally, detailed requirements regarding the reporting of this information are also contained within the ODOT Geotechnical Specifications.

In general, a report written for pre-construction, design-level work, and indeed nearly all geotechnical work, will contain the following elements:

- Executive summary
- Body of report
- Recommendations
- Appendices

In general, the body of geotechnical report would contain the following:

- Purpose of the report
- Project information including location, extents and scope of the project
- Area geology
- Any geotechnical features in the area such as outcrops of bedrock, existing cuts and fills including slope angles, evidence of current or past landslides, surface soils, potential, problem soils (e.g., dispersive or expansive clays, sulfate bearing soils), groundwater conditions (e.g., springs, streams, irrigation), wetland locations, areas that may require sub excavation or other foundation stabilization/drainage measures, locations that may require rock excavations, including areas that may require blasting, locations that will require extensive excavations and/or fills, roadway patching that may indicate subgrade problems, the type of vegetation or lack thereof, location of nearby buildings, drainage structures, bridges, utilities, etc., and other features that may affect the project alignment, right-of-way and/or design
- Subsurface investigation and laboratory test results
- Design and construction recommendations and constraints

In addition, Pedological investigation reports should contain locations of samples, soil series, field logs, and laboratory tests. Shoulder Survey/In Place Soils investigation reports would contain locations of samples, field logs, and laboratory tests. Pavement and Subgrade Soils investigation reports would contain back calculated Falling Weight Deflectometer (FWD) data, core logs with photos, pavement surface conditions, field logs, and laboratory tests.

Design and construction recommendations would include but are not limited to:

- Alignment recommendations based on existing topography
- Additional right-of-way required for cut and fill slopes, landside mitigation, or rock containment
- Subsurface drainage recommendations
- Undercuts

- Unusual erosion control measures
- Shrink swell factors for earthwork calculations
- Rock excavation requirements such as blasting or rippability
- Slope stabilization measures
- Alternative fill design for areas with potential problematic borrow sources such as dispersive clays
- Estimated short- and long-term rates of settlement
- Mitigation for settlement such as wick drains or other solutions
- Recommendations and specifications for monitoring settlement and slope stability.
- Recommendations for slope protections in flood plains resulting from rapid draw down or scour
- Recommendations for landslide mitigation in areas prone to landslides
- Recommendations for dewatering, geotextile fabrics, etc. in areas with drainage issues or low water table
- Recommendations for slope heights and angles
- Recommendations for design and construction bridge foundations, retaining walls, and other structures as needed.

Appendices in the report will include boring logs, lab test results, photographs, maps, and design calculations.

The executive summary will summarize the findings and recommendations.

3.1.2 Supplemental Geotechnical Reports

The project design team, or end-user, may request the Geotechnical engineer to provide information supplemental to the originally requested work/report. This supplemental information is presented in the form of a supplemental report. The supplemental report will address the analysis and alternatives for those elements requested for additional geotechnical review or study. It is possible to have multiple supplemental reports for a given project. Supplemental reports may be generated for any type of geotechnical work.

3.1.3 Construction Related Geotechnical Reports

Questions, issues/problems, or other needs may arise during the construction phase of a project. The nature of construction related issues is vast and highly variable. Examples include:

- Provide borings in relatively tight spacing to determine the limits of rubble buried under ground
- Install monitoring wells to observe ground water

- Perform seismic testing to determine rippability
- Perform and/or review soil stabilization mix designs
- Review CTB mix designs
- Review geotechnical reports and provide clarification
- Provide referee testing

In all these cases, a report is generated detailing the results of observations, lab work, and fieldwork; and applicable recommendations. This type of work and the resulting reports are usually completed by the Materials Division's Geotechnical Branch, with the reports addressed to the Field District's Construction Engineer and/or the Resident Engineer. Due to the unique circumstances under which most of these reports are written, there is not a standard set of requirements or specifications governing their contents or what should be included.

3.1.4 Forensic/Post-Construction Geotechnical Reports

Post-construction geotechnical work is usually forensic in nature. Forensic type work is usually requested by either a Construction Engineer, Resident Engineer, or Maintenance Engineer. In most cases there is usually a problem of some sort (e.g. excessive settlement, pavement rutting, slope failure, etc.) and the requesting engineer needs to gather information in order to make an informed decision as to how to repair the problem. Like construction related geotechnical work, the ODOT's Geotechnical Field Branch is usually contacted to perform this type of work, but consultants may be used as well. As with any geotechnical report, the details of field and laboratory work, modelling, findings, and recommendations will be provided. Due to the unique nature of these types of problems, there does not exist a standard set of specifications governing the contents of these types of geotechnical reports or how to perform this type of work. Frequently, follow-up meetings are held to discuss the findings of these reports in more detail which may result in supplemental work and supplemental reports.

3.2 REFERENCES

References specifically cited in Chapter 3 are numbered below, followed by additional pertinent technical information sources presented alphabetically by author.

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Chapter 4

GEOTECHNICAL ASPECTS OF PAVEMENT DESIGN

ODOT GEOTECHNICAL MANUAL

January 2025

4 GEOTECHNICAL ASPECTS OF PAVEMENT DESIGN

A pavement system consists of layers of subbase, base course, and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed. This chapter addresses the primary geotechnical characteristics of pavement materials and alternatives for improving poorer quality subgrade soils.

4.1 GENERAL

4.1.1 Geotechnical Characterization of Subgrade

The moisture, density, strength, stiffness, and compressibility characteristics of materials underlying the pavement structure can significantly influence pavement performance. The subgrade must be strong enough to resist shear failure and have adequate stiffness to minimize deformation and/or settlement. Stronger and stiffer materials provide a more effective foundation for the riding surface and are more resistant to stresses from repeated loading and environmental conditions.

A critical component of the pavement design is the characterization of the material upon which the pavement structure will be constructed. In cases where the subgrade is inadequate, the project Geotechnical engineer must identify methods of improving the existing subgrade conditions. An important part of this evaluation is the balance between initial construction costs and long-term operations and maintenance costs.

4.1.1.1 Test Methods

Several laboratory and in-situ test methods are available for characterizing the physical characteristics, strength, and stiffness of subgrade soils and base course aggregates.

4.1.1.2 Final Written Report Presentation

For detailed information on data reduction and report formatting, see the current version of the ODOT Geotechnical Specifications for Roadway Design. (Reference 1)

4.1.2 Soil Classification

The Geotechnical engineer needs to determine the physical and chemical characteristics of the subgrade soils for the design analysis. These characteristics include the moisture content, density, gradation and plasticity of the soil.

4.1.3 Resilient Modulus

The resilient modulus (M_R) is a measure of the elastic response of soil recognizing certain nonlinear characteristics. M_R can be used directly for the design of flexible and rigid pavements. M_R is a material property that can be used in mechanistic analysis of multilayered systems for predicting roughness, cracking, rutting, faulting, etc. The method for determining M_R is described by AASHTO. (Reference 2).

ODOT uses the results from the M_R test to evaluate the subgrade strength when designing the thickness of each layer in a pavement section.

4.1.4 Correlations with MR

Different types of correlations are used to estimate M_R of subgrade soils and bases. Several literature sources document these correlations (e.g., NCHRP Synthesis 382 *Estimating Stiffness of Subgrade and Unbound Materials for Pavement Design*) (Reference 3). One common approach develops direct relationships between M_R and various measured soil properties or in-situ test parameters. Some useful correlations of M_R with California Bearing Ratio (CBR) and q_u (unconfined compressive strength, in psi) include the following:

M_R (psi)	=	1500 CBR	(Heukelom and Klump) (Reference 4)
M_R (psi)	=	2554 $CBR^{0.64}$	(Powell, et al,) (Reference 5)
M_R (ksi)	=	$0.86 + 0.31\ q_u$	(Thompson and Robnett,) (Reference 6)
M_R (ksi)	=	$9.98 + 0.124\ q_u$	(Little, D.) (Reference 7)

4.1.4.1 California Bearing Ratio

The California Bearing Ratio (CBR) test (AASHTO T193 and ASTM D 1883) provides a relative measure of subgrade soil or base course aggregate strength and stability.

The M_R and CBR tests are common tests for laboratory characterization of geomaterials (i.e., fine-grained subgrade soils and unbound aggregate materials for roadway pavement analysis). However, there is a major difference between the two tests in terms of materials properties. The CBR test evaluates the potential minimum strength of geomaterials, and the M_R test is a stiffness property obtained under repeated/cyclic loading for actual placement conditions. The determination of resilient modulus requires sophisticated equipment and skilled personnel for laboratory and field testing. Therefore, some agencies use empirical correlations with CBR to estimate M_R for pavement design. However, ODOT does not recommend using these correlations for pavement design.

4.1.4.2 Dynamic Cone Penetration Test

The dynamic cone penetration test (DCP) (ASTM D 6951) is used to assess in-situ strength of undisturbed or compacted soil and stabilized materials. Correlations between M_R and DCP results (e.g., DCI) are generally derived from correlations between CBR and DCI; however, some direct correlations are available in the literature.

4.1.4.3 Falling Weight Deflectometer

The Field Investigation Unit conducts nondestructive field testing of existing pavements using a typical trailer mounted falling weight deflectometer (FWD) in accordance with AASHTO T256 or ASTM D 4694 and ASTM D 5858. The FWD measures pavement deflection due to a simulated wheel load, and from this, can estimate the stiffness of the pavement layers.

The FWD delivers an impulse load to the pavement surface by dropping a weight from a predetermined height to obtain a peak force ranging from 1500 lbs to 24,000 lbs. The load is transmitted to the pavement through a loading plate in the form of a half sine wave with duration of 25 to 30 milliseconds. A load cell measures the magnitude of load. Deflections are measured using seven to nine (depending on the FWD model) velocity transducers (geophones) mounted on a bar that is lowered automatically to the pavement surface with the loading plate. The FWD is equipped with a microprocessor-based control console for storing and processing data. The existing pavement thickness is required to assess the data collected from FWD testing.

Pavement performance and structural capacity are assessed using the FWD through the use of maximum elastic deflection measurements in combination with an indicator of the radius of curvature of the pavement under load. The resilient modulus gives an indication of a soil's potential response to loading by heavy equipment during construction and loading by traffic after the road is constructed. The FWD is also used to measure load transfer across pavement joints, detect areas where subgrade support has been lost and for forensic investigations. FWD testing may be used on a system-wide basis for pavement management purposes.

Additional information, interpretation methods and guidelines for use of the FWD are described in the AASHTO *MEPDG*. (Reference 2)

4.1.4.4 Portable Falling or Light Weight Deflectometer

The portable falling weight or lightweight deflectometer (PFWD or LWD) is used to rapidly assess the in-situ elastic modulus of surface/subgrade soils or base courses. The LWD is portable, and testing is quick. Typical LWDs have a falling mass weighing between 22 lbs and 44 lbs that applies an impulse load to varying diameter loading plate (6 to 12 inches). The basic concept behind the LWD is the impulse load applied by the falling weight to the loading plate and soil. The applied impulse load is either measured or assumed constant based on prior calibration.

The impulse load is 15 msec to 30 msec in duration with peak forces varying between 1575 lbs to 4500 lbs. The response (deflection) of the loading plate or soil surface is measured with a geophone or accelerometer. Using the measured/assumed force and loading plate/ground deflection, a soil modulus can be calculated.

The modulus obtained from LWD testing is derived from elastic theory that is based on full confinement. M_R is a measure of the elastic response of a confined soil sample, where the confining pressure is varied, which results in different M_R values. Limited information is available, but the accepted trend is that M_R and E_{LWD} will need to be correlated for different subgrade soil conditions.

The primary application of the LWD has been quality assurance of compacted fills, subgrades and bases during construction. It can also be used on low-volume roads with thin surface treatments. It does not have adequate test loading capacity for most paved roads.

4.2 SUBGRADE CHEMICAL TREATMENT

4.2.1 <u>General</u>

Subgrade treatment involves placing and compacting one or more layers of a mixture of soil, chemical additive, and water for the purpose of achieving stabilization or modification. ODOT *Standard Specifications* (Reference 8) define stabilization or modification additives as cementitious additive (e.g., Portland cement, fly ash, cement kiln dust) or non-cementitious additive (e.g., lime).

Non-cementitious chemical additives, such as lime, provide a source of cations which interact with the soil minerals in the form of cation exchange and agglomeration/flocculation. However, cementitious chemical additives containing silica and alumina provide both a source of cations for modification reactions and a source of the building blocks for pozzalonic reaction, i.e., a reaction resulting in a strong cementation matrix.

Subgrade soils that are unsatisfactory in their natural state can be altered by admixtures, addition of aggregate, or proper compaction and moisture control, thereby making the soils suitable for subgrade construction. This section addresses admixture stabilization methods, which may be warranted for several reasons, including:

- Soft or weak soils
- High plasticity soils
- Excessively wet soils
- Expansive soils
- Stabilization or salvaging of unpaved roads
- Dust control

4.2.2 Subgrade Stabilization

For subgrade stabilization, chemical additives are incorporated in sufficient quantities to increase the shear strength of subgrade soils and provide structural value for the pavement section. For projects where sulfate bearing soils may be present, review the Materials Division OHD L-49, OHD L-50, and OHD L-51 (References 9, 10, and 11) to determine soluble sulfate content in the soil and applicability of subgrade treatment. Sulfate related problems occur when sulfate in the soil reacts with calcium in commonly used chemical additives and water to produce a chemical reaction product (ettringite), which causes swelling/distortion of the treated subgrade layer and pavement distortion and damage. If sulfate content exceeds the threshold value specified in OHD L-49, OHD L-50 and OHD L-51, the pavement design engineer should consider other alternatives for subgrade treatment.

4.2.3 Subgrade Modification

For subgrade modification, chemical additives are commonly added to change the subgrade plasticity index (PI) and improve the workability of subgrade soils in order to establish a sound working platform for support of construction equipment. No structural value is added to the pavement structure. On projects where sulfate soils may be present, the project Geotechnical engineer should review Materials Division OHD L-49 to determine soluble sulfate content in soil and applicability of subgrade treatment. If sulfate content exceeds the threshold value specified in OHD L-49, OHD L-50, and OHD L-51, do not consider a subgrade treatment.

4.2.4 Geotechnical Considerations

Chemical modification or stabilization of subgrade soils consists of uniformly mixing chemical additives with the soil to improve the strength and workability of soils. The choice of the proper admixture depends upon the desired modification of the subgrade. The quantity of admixture is generally determined by means of laboratory tests or presumptive guidelines. Evaluate the response of the blended material to weather and material handling (abrasion and degradation) and chemical interactions with sulfate bearing soils. Guidelines regarding the application of chemical admixtures are detailed in the ODOT *Standard Specifications*.

4.2.5 Selection of Chemical and Application Rate

Chemical additive selection, whether from OHD L-50 or OHD L-51 or by mix design, requires that the soil be tested and classified according to AASHTO M145. Before selecting chemical additives, test the soil sample for soluble sulfates according to OHD L-49. If the soluble sulfate content is greater than 500 ppm, take additional samples for soluble sulfate testing throughout the length represented by the sample. If the soluble sulfate content is greater than 1000 ppm for any of the additional samples, stabilization with calcium-based additives may not be suitable. If the soluble sulfate content is greater than 8000 ppm for any of the additional samples, stabilization with calcium-based additives is not recommended.

Soil stabilization mix design is determined through one of two methods. The first method is a complete laboratory test procedure to determine the recommended percentage of stabilization additive from the test results. The second method consists of an abbreviated laboratory test procedure with determination of the recommended percentage of stabilization additive from the soil additive percentage tables. Laboratory test procedures should follow the requirements of OHD L-50.

4.3 GEOSYNTHETICS

4.3.1 General

Geosynthetics are used in roadway construction to serve one or more of the following functions:

- Reinforcement
- Separation
- Filtration (cross-plane flow)
- Transmission or drainage (in-plane flow)

Geosynthetics improve the support characteristics and performance of pavement elements. Typical geosynthetic placement locations within the pavement section include:

- Between subgrade and base (or subbase)
- Between, or within, subbase and base
- Between base and surface course

Geotextiles (or geogrids) placed on fine-grained subgrade soil increase stability and improve pavement performance by separating the layers and, to some degree, reinforcing the subgrade-base (or subbase) interface. Geotextiles also provide filtration and drainage by allowing pore water pressures in wet subgrade soils to dissipate into the coarser base material or laterally through the geotextile itself. The separation, reinforcement, and filtration/drainage functions combine to provide a method of mechanically stabilizing poorer quality subgrade soils. Subgrade soils for which geosynthetics (geotextiles, geogrids, some geocomposites) can be helpful in improving performance include:

- Soil Classification:
  + AASHTO classifications A-5, A-6, A-7-5 and A-7-6
  + USCS classifications SC, CL, CH, MO, MH, OL and OH
- Strength:
  + C_u (undrained shear strength) < 15 psi
  + CBR (soaked) < 3
  + M_R < 4500 psi
- Groundwater Table: Near ground surface

- Sensitivity of Soil: High

Under these conditions, geosynthetics serve multiple functions, as discussed in the following text.

4.3.1.1 Design for Separation

When designed for separation, the base course thickness is generally not reduced due to the use of the geosynthetics. The geosynthetic separator, typically a geotextile, prevents intrusion of fine-grained subgrade soil into granular base layers. Geotextile separators may also be used between varying gradation, or grain size, base layers. Most geotextiles will work as separators, if they are strong enough to survive construction. Filtration is a secondary function in this application, so the geotextile should have small enough openings to prevent contamination of the subbase or base layer but have sufficient permeability (more than subgrade soil) to prevent development of porewater pressures in the subgrade soil. See FHWA *Geosynthetic Design and Construction Guidelines* (Reference 12) for design details and criteria.

4.3.1.2 Design for Reinforcement

When designed for reinforcement, both the geosynthetic (geotextile or geogrid) and cover aggregate (base) thickness required to stabilize the subgrade and provide adequate support are evaluated. For roadway/pavement design, the stabilization layer (geosynthetic and aggregate) provides roadbed improvement by causing less subgrade disturbance, providing a granular layer that will not be contaminated by subgrade soil intrusion, and providing potential subgrade improvement with time. The base course thickness required to carry the design traffic loads may be reduced due to roadbed improvement from inclusion of the geosynthetic, depending on the strength of the geosynthetic. The extent of the improvement (e.g., reduced layer thickness) needs to be assessed for different aggregate and geosynthetic properties when assessing reinforcement variables. There are two approaches to stabilization design:

1. <u>Bearing Capacity</u>. The reinforcement function that results from improved bearing capacity is included in the pavement design with no direct contribution from the strength of the geosynthetic (e.g., the geosynthetic acts only as a separator and filter).

2. <u>Tensile Strength</u>. The reinforcement function includes the tensile strength of the geosynthetic and the interface or interlocking behavior of the geosynthetic. The reinforcement mechanisms mobilized in subgrade stabilization are different for geotextiles and geogrids.

See FHWA *Geosynthetic Design and Construction Guidelines* for design details and criteria.

4.3.2 <u>Asphalt Materials</u>

Historically, the use of asphalt materials to improve subgrade soil properties involves two basic application procedures: a) the asphalt material is mixed with the soil, and b) the asphalt material is applied to the surface of the soil layer (e.g., sprayed asphalt membrane).

Asphalt materials that could be used in soil treatment include asphalt cement, asphalt cutbacks, and asphalt emulsions. Asphalt emulsions are the most common product used because of cost, ease of construction, and environmental issues. The mechanisms involved in asphalt treatment of soils (mixed with soil) are different from chemical additive treatment. In asphalt treatment of fine-grained soils, the basic mechanism is waterproofing. Soil clods are coated with asphalt and compacted, which results in a membrane layer that keeps water from penetrating into or through the layer and adversely affecting the properties of the soil. In coarse-grained soil, the basic mechanisms are adhesion and waterproofing. Individual particles are partially or totally coated, which results in the soil particles adhering to one another. When compacted, the asphalt-coated particles form a layer of reduced permeability soil. Sprayed asphalt (emulsion) membranes provide similar waterproofing layers when sprayed on the soil surface. Sprayed asphalt membranes would typically be applied to foreslopes, ditches, and backslopes along the pavement to reduce moisture infiltration into the subgrade.

Fine-grained soils suitable for asphalt treatment (mixed) include soils with less than 25% passing the U.S. No. 200 sieve and PI less than 10. These properties ensure that the asphalt emulsion can be thoroughly mixed with the soil and the mixture can be efficiently compacted.

Granular soils suitable for asphalt treatment (mixed) include well-graded fine gravels (maximum particle size less than ¾ inch) and well-graded sands with minimal fines (e.g., less than 5%) and non-plastic or PI less than 5.

Sprayed asphalt membranes should be applied to smooth surfaces with no clods or debris that could result in holes in the membrane. Application rates depend on soil type and purpose of membrane and range from 15 ft^2 to 25 ft^2 per gallon.

Note: Asphalt treatment is now used on existing constructed roadways but is no longer used for new construction base applications.

4.4 SUBGRADE UNDERCUT

4.4.1 General

Soil is a highly variable material. The interrelationship of soil texture, density, moisture content, and strength are complex and, in particular, behavior under repeated loads is difficult to evaluate. Subgrade soils may vary considerably, so it is necessary to conduct a thorough study of the in-situ soils to determine the pavement design. For some project sites, the in-situ soil will not possess all of the desirable properties. In these cases, subgrade soils may require over-excavation and replacement with suitable select fill. These areas are referred to as undercuts.

4.4.2 Geotechnical Considerations

Because of the complexity of subgrade undercutting, it is not practical to establish rules suitable for all cases. The purpose of this section is to provide the user with general principles that can be incorporated as part of investigation, analysis, and pavement design process.

Soils with excessive moisture can be unsuitable. This soil may be removed, dried and reused, or in some cases, dried and recompacted in place.

Some soils are undesirable under nearly all conditions and should be removed, if practical, prior to construction of the subgrade. These include:

- Soft, weak subgrade soils
- Soils with high sulfate contents
- Soils with high organic content
- Expansive soils
- A-7-5 and A-7-6 soils with high group indices. These soils are especially problematic in cut/fill transition zones and locations where subgrade drainage may be limited.

The geotechnical report should identify subgrade areas requiring over excavation during preconstruction investigations. The designer will specify on the project plans the location and depth of the required sub-excavation, and the specifications will describe the requirements for removal and disposal of unsuitable material as well as backfill requirements.

Preconstruction subsurface investigations may not successfully locate all unsuitable subgrade soils. Where unexpected unsuitable soils are encountered during construction, these soils will be over excavated at the direction of the Resident Engineer. Undercuts generally extend to a maximum depth of 2 ft to 5 ft below top of subgrade (bottom of base course elevation), depending on the thickness and variability of the problem material. Proof-rolling may be incorporated to help identify areas of soft, weak subgrade soils.

4.5 SUBGRADE DRAINAGE

4.5.1 Applications

Geotextiles are used as filters in drain applications (e.g., trench and interception drains, blanket drains, pavement edge drains, structure drains, beneath permeable roadway bases). The filter restricts movement of soil particles as water flows into the drain structure, which collects and transports the water. Geocomposites, consisting of a drainage core surrounded by a geotextile filter, are often used as the drain itself in these applications. Geotextiles are also used as filters beneath permanent erosion control systems. Geotextiles are used to replace graded granular filters in almost all drainage applications. Therefore, they must perform the same functions as graded granular filters:

- Allow water to flow through the filter into the drain, and to continue without clogging throughout the life of the project

- Retain the soil particles in place and prevent their migration (piping) through the filter (if some soil particles do move, they must be able to pass through the filter without blinding or clogging the filter during the life of the project)

Geotextiles, like graded granular filters, require proper engineering design or they may not perform as desired. Unless clogging resistance, constructability, and retention and flow requirements are properly specified, the geotextile/soil filtration system may not perform properly. In addition, construction should be monitored to ensure that materials are installed correctly. In most drainage and filtration applications, geotextiles can be justified over conventional graded granular filter material.

4.5.2 Geotechnical Requirements

Properly designed geotextiles can be used as a replacement for, or in conjunction with, conventional graded granular filters in almost any drainage application. Properly designed geocomposites can be used as a replacement for granular drains in many applications (e.g., pavement edge drains).

The project Geotechnical engineer should begin all geosynthetic designs with a criticality and severity assessment of the project conditions. See Figures 4-1 and 4-2 for guidance on a particular application.

4.5.2.1 Retention Criteria

For steady state flow conditions:

$$\text{AOS or } O_{95(\text{geotextile})} \leq B\, D_{85(\text{soil})}$$ Equation 4.5 (Reference 13)

where:

AOS	=	apparent opening size
O_{95}	=	opening size in the geotextile for which 95% are smaller
AOS	$\cong$	O_{95}
B	=	a coefficient (dimensionless)
D_{85}	=	soil particle size for which 85% are smaller

Critical Nature of the Project

Item	Critical	Less Critical
Risk of loss of life and/or structural damage due to drain failure	High	None
Repair costs versus installation costs of drain	>>>	≤
Evidence of drain clogging before potential catastrophic failure	None	Yes

Figure 4-1: GUIDELINES FOR EVALUATING CRITICAL NATURE OF DRAINAIGE & EROSION CONTROL

Severity of the Conditions

Item	Severe	Less Severe
Soil to be drained	Gap-graded, pipable or dispersible	Well-graded or uniform
Hydraulic gradient	High	Low
Flow conditions	Dynamic, cyclic or pulsating	Steady State

FIGURE 4-2: GUIDELINES FOR EVALUATING SEVERITY OF DRAINAIGE & EROSION CONTROL

The coefficient B ranges from 0.5 to 2 and is a function of the type of soil to be filtered, its density, the uniformity coefficient C_u if the soil is granular, the type of geotextile (woven or nonwoven), and the flow conditions.

For sands, gravelly sands, silty sands and clayey sands (with less than 50% passing the No. 200 sieve per the Unified Soil Classification System), B is a function of the uniformity coefficient, C_u.

Therefore, for:

$C_u \leq 2$ or ≥ 8:	$B = 1$	Equation 4.5(1)
$2 \leq C_u \leq 4$:	$B = 0.5\, C_u$	Equation 4.5(2)
$4 < C_u < 8$:	$B = 8/C_u$	Equation 4.5(3)

where:

C_u = D_{60}/D_{10}

Sandy soils that are not uniform, tend to bridge across the openings; therefore, the larger pores may actually be up to twice as large (B < 2) as the larger soil particles because, quite simply, two particles cannot pass through the same hole at the same time. Therefore, use of the criterion B = 1 would be conservative for retention. If the protected soil contains any fines, use only the portion passing the No. 4 sieve for selecting the geotextile.

For silts and clays (with more than 50% passing the No. 200 sieve), B is a function of the type of geotextile:

Wovens:	$B = 1;\ O_{95} \leq D_{85}$	Equation 4.5(4)
Nonwovens:	$B = 1.8;\ O_{95} \leq 1.8\ D_{85}$	Equation 4.5(5)
Both:	AOS or $O_{95} \leq 0.3$	Equation 4.5(6)

Due to their random pore characteristics and, in some types, their felt-like nature, nc geotextiles will generally retain finer particles than a woven geotextile of the same A the use of B = 1 will be even more conservative for nonwovens.

In absence of detailed design, AASHTO M288 provides the following recommended maximum AOS values in relation to percent of in-situ soil passing the No. 200 sieve:

- No. 40 sieve for less than 15 percent passing
- No. 60 sieve for 15 percent to 50 percent passing
- No. 70 sieve for more than 50 percent passing

For cohesive soils with a plasticity index greater than 7, the maximum AOS size is the No. 50 sieve. These default AOS values are based upon the predominant particle sizes of the in-situ soil.

4.5.2.2 Permeability/Permittivity Criteria

Permeability requirements:

For less critical applications and less severe conditions:

$K_{geotextile} \geq K_{soil}$ Equation 4.5 (Reference 14)

For critical applications and severe conditions:

$K_{geotextile} \geq 10\ K_{soil}$ Equation 4.5 (Reference 15)

Permittivity requirements:

$\psi \geq 0.5\ sec^{-1}$ for < 15% passing No. 200 sieve Equation 4.5 (Reference 16)

$\psi \geq 0.2\ sec^{-1}$ for 15% to 50% passing No. 200 sieve Equation 4.5 (Reference 8)

$\psi \geq 0.1\ sec^{-1}$ for > 50% passing No. 200 sieve Equation 4.5 (Reference 9)

where:

K = Darcy coefficient of permeability (m/s)

ψ = geotextile permittivity, which is equal to $k_{geotextile}/t_{geotextile}$ (1/s) and is a function of the hydraulic head

For actual flow capacity, use the permeability criteria for noncritical conservative, because an equal quantity of flow through a relatively thin geotextile takes significantly less time than through a thick granular filter. Even so, some pores in the geotextile may become blocked or plugged with time. For critical or severe applications, Equation 5.4 (Reference 15) is recommended to provide an additional level of conservatism. Equation 5.4 (Reference 14) may be used where flow reduction is judged not to be a problem (e.g., in clean, medium to coarse sands and gravels).

AASHTO M288 presents recommended minimum permittivity values in relation to percent of in-situ soil passing the No. 200 sieve. The values are the same as presented in Equations 5.4 (Reference 16), 5.4 (Reference 8) and 5.4 (Reference 9). The default permittivity values are based upon the predominant particle sizes of the in-situ soil. The project Geotechnical engineer may require performance testing based on engineering design for drainage systems in problematic soil environments.

Determine the required flow rate, q, through the system and select the geotextile and drainage aggregate to provide adequate capacity. Flow capacities should not be a problem for most applications, provided the geotextile permeability is greater than the soil permeability. In certain situations (e.g., where geotextiles are used to span joints in rigid structures and where they are used as pipe wraps), portions of the geotextile may be blocked. For these applications, use the following criteria together with the permeability criteria:

$$q_{required} = q_{geotextile} (A_g/A_t) \qquad \text{Equation 5.4 (Reference 16)}$$

where:

A_g = geotextile area available for flow
A_t = total geotextile area

4.5.2.3 Clogging Resistance

4.5.2.3.1 Less Critical/Less Severe Conditions

For less critical/less severe conditions:

$$O_{95(geotextile)} \geq 3\ D_{15\ (soil)} \qquad \text{Equation 5.4 (Reference 11)}$$

Equation 5.4 applies to soils with $C_u > 3$. For $C_u \leq 3$, the project geotechnical specialist should select a geotextile with the maximum AOS value from Section 4.5.2.1.

In situations where clogging is a possibility (e.g., gap-graded or silty soils), the following optional qualifiers may be applied:

For nonwovens:

Porosity of the Geotextile, $n \geq 50\%$ — Equation 4.5 (Reference 16)

For woven monofilament and slit film wovens:

Percent Open Area, $POA \geq 4\%$ — Equation 4.5 (Reference 11)

Most common nonwovens have porosities much greater than 70%. Most woven monofilaments easily meet the criterion of Equation 5.4 (Reference 17); tightly woven slit films do not and are, therefore, not recommended for subsurface drainage applications.

4.5.2.3.2 Critical/Severe Conditions

For critical/severe conditions, select geotextiles that meet the retention and permeability criteria in Sections 4.5.2.1 and 4.5.2.2. Perform a filtration test using samples of on-site soils and hydraulic conditions. One type of filtration test is the gradient ratio test (ASTM D 5101).

4.5.3 <u>Membrane Cutoffs</u>

Geosynthetic membrane cutoffs are used in highway projects to control vertical or horizontal infiltration of moisture into a subgrade consisting of expansive or moisture-sensitive soil and to prevent moisture changes beneath roadways.

Even though horizontal cutoffs and liners are commonly used, vertical cutoffs and liners are recommended in new construction, provided buried utilities and drainage structures do not impede their proper placement. These types of synthetic barriers are used to either prevent damage to highway pavements and structures, or as part of a water quality enhancement system. Barriers should be engineered to perform their intended function for the particular application and project conditions.

4.5.3.1 Barrier Characteristics and Importance of QA/QC

Membrane cutoff materials may include geomembranes, thin-film geotextile composites, geosynthetic clay liners (GCL), or geotextiles that are field-impregnated with liquid asphalt. Geomembranes are relatively impermeable compared to natural materials (e.g., compacted clay). Leakage, rather than permeability, is the primary concern when designing geomembrane containment structures. Leakage can occur because of poor field seams, poor factory seams, pinholes from the manufacturing process and puncture holes from handling, placement or in-service loads.

4.5.3.2 Moisture Barriers in Roadway Construction

Moisture barrier composites in roadway construction are used to prevent or minimize moisture changes in pavement subgrades and are typically installed above the subgrade and below the base course or in vertical trenches along the roadway.

4.5.4 <u>Maintenance of Subsurface Drainage Facilities</u>

Effective maintenance begins with a policy that ensures all concerned parties are aware of the importance of providing adequate maintenance to the subgrade drainage systems. A complete maintenance program includes routine inspection and monitoring, preventive maintenance strategies, spot detection of an actual or potential problem, repair and continued monitoring, and feedback. Without routine inspection and maintenance, drainage problems may not be identified until damage is done, and early pavement distress becomes visible on the surface.

Some of the preventive maintenance strategies may include inventory, inspection survey, and scheduling. Systematic inspections using appropriate performance indicators is one of the most effective means of ensuring the performance of drains and is one of the essential elements of a preventive maintenance program. Probably the most significant development in drain inspection has been the use of small-diameter, optical tube video cameras with closed-circuit video systems. Video cameras allow the inside of the drain system to be logged and expose the weakness in construction and inspection procedures.

Standard maintenance for edge drains includes flushing the system, cleaning the outlets, and replacing the outlets when damaged. Scheduled periodic flushing and outlet cleaning provide an effective tool for preventive maintenance.

Maintaining a routine schedule of maintenance will help reduce vegetative growth and the collection of debris around pipe outlets; rodent's nests, mowing clippings, and sediment collecting on rodent screens at the end wall; and will allow edge drain systems to work properly.

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Chapter 5

FILL/SLOPE STABILITY

ODOT GEOTECHNICAL MANUAL

January 2025

5 FILL/SLOPE STABILITY

5.1 SYMBOLS

Figure 5-1 provides a list of symbols and their definitions that are used in this chapter.

Symbol	Definition
a_v	Coefficient of compressibility
B	Width of fill
β	Slope inclination (angle)
c	Soil cohesion
c'	Effective soil cohesion
c_μ	Undrained shear strength (cohesion)
C_c	Compression index for layer "i"
C_d	Factor based on shape and rigidity of loaded area
c'_m , ϕ'_m	Mobilized strength parameters of soil at base of block with effective weight
c_d, ϕ_d	Developed shear strength parameters
C_r	Recompression index for layer "i"
$C_{c\varepsilon}$	Slope of the virgin compression portion of the ε_v vs log p plot
$C_{r\varepsilon}$	Slope of recompression portion of ε vs log p plot
C_α	Coefficient of secondary compression
c_v	Coefficient of consolidation (vertical)
D_{crack}	Depth of the crack
E_i	Interslice forces
e	Void ratio
e_o	Initial void ratio for layer "i"
E_s	Young's Modulus value
ε_v	Vertical strain
F_c	Factor of safety for cohesion
FS	Factor of safety
F_ϕ	Factor of safety for friction angle
H	Slope height or average height of the slice (method of slices)
H_c	Layer "i" thickness at beginning of secondary compression
H_d	Drainage thickness of layer
H_o	Initial layer thickness for layer "i"
k_h	Horizontal seismic coefficient
k_v	Vertical seismic coefficient
L_{arc}	The lengths of the circular arc and chord defining the failure surface
L_{chord}	The lengths of the circular arc and chord defining the failure surface
N	Normal force on base of failure mass
N	Uncorrected Standard Penetration Test (SPT) blow count
N_s	Stability number
N_{60}	Standardize SPT blow count
$(N_1)_{60}$	Normalized and standardized corrected blow count
N_{corr}	Corrected SPT blow count
OCR	Overconsolidated ratio
P_a	Active (driving) force
p	Pressure (stress)

Figure 5-1: SYMBOLS & DEFINITIONS

Symbol	Definition
p_c	Preconsolidation (maximum past effective vertical) stress at midpoint of layer "i"
p_f	Final effective vertical stress at mid-depth of layer "i"
p_o	Initial effective vertical (overburden) stress at mid-depth of layer "i"
Δp	Increase in vertical stress at mid-depth of layer "i" due to applied load
P_p	Passive (resisting) force
q_c	Uncorrected cone tip resistance
q_o	Vertical stress at base of fill
R	Radius of failure circle
R_c	Perpendicular distance from the circle center to force, C_m
S	Resisting forces
S_a	Available strength
S_c	Primary consolidation settlement
S_i	Immediate (elastic) settlement
S_m	Mobilized shear strength (resistance)
S_s	Secondary compression settlement
S_t	Total settlement
t	Time for a given percent of primary consolidation settlement to occur for soil layer
T	Driving forces
T_v	Time factor
t_1	Time when 90% primary consolidation has occurred for layer "i"
t_2	Time when secondary compression begins, typically the time when 90% of primary consolidation is completed
$t_{1\text{-lab}}$	Time when secondary compression begins
$t_{2\text{-lab}}$	Any selected time on the deformation vs log t plot, at least one log cycle beyond the time for 90% primary consolidation
U	Average degree of consolidation (consolidated settlement) or total uplift force (slope stability)
V	Poisson's ratio
W	Weight of failure mass
ΔW	Weight of slice
X_i	Interslice shear force
α	Secondary slope angle (Figure 6.3-5)
γ	Soil unit weight
γ'	Buoyant soil unit weight
γ_{sat}	Saturated soil unit weight
γ_w	Unit weight of water
θ	Assumed failure plane inclination (Figure 6.3-E) or included angle for assumed circular failure surface (Figure 6.3-E)
θ_{crit}	Critical failure surface inclination
ϕ	Friction angle
ϕ'	Effective friction angle
ϕ_d, ϕ_m	Friction angle (subscript "d" is for the developed, "m" is for mobilized) strength parameters
σ'	Effective stress
T_d	Shear strength (drained)
Δe	Change in void ratio between the two times

Figure 5-1: SYMBOLS & DEFINITIONS (CONT.)

5.2 FILL (EMBANKMENT) SETTLEMENT ANALYSIS

5.2.1 Design Considerations

Design requirements for a highway fill depend on a number of factors, including the height and width of the fill, the type and condition of soil supporting the fill, and the location of the groundwater table. The following sections summarize some of the issues that should be considered during design of fills.

5.2.1.1 Site Characterization

The ODOT geotechnical specifications provide guidelines for characterizing fill foundation materials. The focus of field investigations will depend on the type of soil encountered:

1. Cohesive Soils. If the foundation or fill soils are cohesive, then primary design issues will be bearing capacity and global slope stability during construction and long-term settlement. These design issues require collecting undisturbed soil samples for laboratory strength and consolidation testing. It may also be desirable to conduct an in-situ vane shear test, Marchetti dilatometer test (DMT), pressuremeter test (PMT), flat-plate dilatometer (DMT), or Cone Penetration Test (CPT) soundings to obtain shear strength data. It may be necessary to perform laboratory triaxial compression tests (e.g., UU, CU) to determine undrained strengths, as well as total and effective stress parameters. Consolidation tests will be conducted to define the pre- consolidation pressure, the compressibility index and the coefficient of consolidation. High-quality undisturbed samples are required if triaxial or consolidation tests are to be performed.

2. Cohesionless Soils. Cohesionless soils are usually less of a geotechnical design concern for static loading. Settlements will generally be small and will occur rapidly during placement of the fill.

The site characterization process becomes more complicated when the site consists of layers of cohesive and cohesionless soils. In this case, the Geotechnical engineer should evaluate the various potential failure mechanisms.

5.2.1.2 Settlement Design Criteria

The Geotechnical engineer should evaluate the amount of total and differential settlement that can be tolerated during and following fill construction to compare to estimated settlement values. The Geotechnical engineer will also be responsible for determining the applicable design criteria on a project-by-project basis considering several factors for the allowable settlement.

These factors involve both roadway maintenance and safety issues resulting from the amount and rate at which the settlement occurs. For example:

1. Roadways. The normal goal is to minimize the amount of settlement that occurs after the pavement surface has been placed. When roadway differential settlement exceeds a few inches (approximately 2 - 3) over a 100 ft distance, noticeable bumps and dips in the pavement surface occur.

2. Bridges. At bridge abutments, the settlement results in a bump at the bridge abutment or a slope change if an approach slab is used. The target maximum amount of settlement during operation should generally be small (i.e., less than an inch).

3. Miscellaneous. Settlements from fills constructed next to buildings, railroads and utilities should be limited to a fraction of an inch. As the design criteria for settlement become more stringent (i.e., consequences of excessive settlement become more serious), the methods used to quantify the amount and rate of settlement need to be more detailed.

The ability to quantify both the magnitude and rate of settlement will depend on the thoroughness of the field investigations, quality of soil samples and laboratory testing, size of the fill, and type and characteristics of the foundation soils. Typically, the primary source of problematic settlements will be over time from the foundation soils and not the fill itself. Usually, the fill soils are well compacted and any settlement within the fill itself may be assumed to be elastic (i.e. "immediate"). As the height and width of the fill increases, the potential for settlement also increases because of the stress change that occurs in the foundation soil. The amount of settlement also increases as the thickness and the compressibility of the foundation soil increases.

The geotechnical specifications provide guidance to the Geotechnical engineer regarding estimating settlement depending on the height of the proposed fill.

5.2.2 Fill Settlement Calculations

The magnitude and rate of settlement are important long-term (operational) considerations. The project Geotechnical engineer should conduct settlement analyses to determine if the amount of settlement after construction is within the project criteria. If settlement is excessive, then ground improvement measures may be required to improve the soil. This section summarizes methods of analysis for cohesive and cohesionless soils for long-term loading conditions.

5.2.3 Soil Settlement

The settlement of soils in response to loading can be categorized as immediate (elastic) settlement and time-dependent settlement. Immediate settlement occurs rapidly (e.g., direct response to loading) and is commonly calculated using elastic theory. The time-dependent load-settlement response or property of soils is referred to as consolidation and occurs in cohesive soils which are saturated or very nearly saturated (saturation of 70% or greater, approximately).

Compressibility properties of cohesive soils are measured using the consolidation test and estimates of consolidation settlement are made using basic strain equations relating laboratory measurements to field conditions. Consolidation settlement consists of two components:

1. Primary Consolidation-Settlement. Primary consolidation settlement resulting from expulsion of water from the soil voids in response to excess pore water pressures generated by the applied load.

2. Secondary Consolidation-Settlement. Secondary consolidation settlement that results from a time-dependent adjustment of the soil structure under relatively constant stress, similar to creep. Secondary compression settlement is typically evaluated from the time-deformation responses of the consolidation test. This type of consolidation settlement typically is of concern with organic soils. Organic soils are extremely rare in Oklahoma and so it is common practice to assume secondary consolidation-settlement is negligible (i.e. equal to zero).

The total settlement (S_t) of a soil layer under a constant applied load is:

$$S_t = S_i + S_c + S_s$$

Where:

S_i = immediate (elastic) settlement
S_c = primary consolidation settlement
S_s = secondary consolidation settlement

Equation 5.2 (Reference 1)

Immediate settlement can be calculated for either cohesive or cohesionless soils and is the only settlement calculated for cohesionless soils. Immediate settlement for saturated/nearly saturated cohesive soils will be small compared to their consolidation settlement. Immediate settlement can be a significant portion of total settlement for unsaturated and highly over consolidated cohesive soils. Primary consolidated settlements are calculated for saturated or nearly saturated cohesive soils only. The component(s) of settlement (i.e. Si, Sc, or Ss) which are calculated for either the fill soil or its foundation are dependent on the conditions encountered and expected to be encountered at the site. For example, if a fill is reasonably expected to never become saturated, then primary consolidation settlement is arguably unnecessary to calculate. However, if that same fill exists within a flood plain and it may become saturated at some point in its service life, then it is arguably necessary to estimate the consolidation settlement of that fill soil. The same applies to the foundation soils of the fill.

5.2.3.1 Immediate Settlement

Immediate settlement results from elastic deformation of cohesive or cohesionless soils, is usually small in magnitude, and occurs concurrent with the loading. Typically, on its own, it is rarely of any concern for two reasons: 1) it occurs immediately/well before end of construction, so it is effectively addressed during the earth work phase of the project, and 2) its magnitude is typically very small (less than 1 or 2 inches, approximately) so it does not cause any problems with estimated quantities/pay items. For these reasons, immediate settlement is not something which affects design decisions or even construction.

5.2.3.2 Primary Consolidation Settlement

Primary consolidation settlement may be the most common culprit behind problematic types of settlement encountered post-construction (ex: the “bump at the end of the bridge”). There are effectively two major aspects of primary consolidation settlement which can affect the design and/or construction phases of a project: 1) time for consolidation to occur and 2) the magnitude of consolidation. These aspects can combine in the following ways:

1. <u>Relatively fast consolidation high magnitude</u>: Sometimes the magnitude of settlement may be very large, i.e., over one foot, but the estimated time for the consolidation to occur is less than 2 months. This may not affect the design of a fill embankment but would certainly affect the quantities of fill necessary to achieve final grade and it would certainly behoove all parties involved to monitor the consolidation settlement as the construction progresses to ensure settlement is occurring as expected/estimated.

2. <u>Relatively slow consolidation high magnitude</u>: Sometimes the estimated magnitude of consolidation settlement is large and the estimated time for that settlement to occur is unacceptably long. “Long” is a relative term and may mean long enough to affect construction sequencing, or it may be years after construction is complete. In either case, if this is the situation facing the designer, special efforts are required to mitigate the rate of consolidation, the magnitude of consolidation, or both. The rate can be increased by means of things such as wick drains. Wick drains allow shorter drainage paths, thus allowing the porewater pressure to dissipate more quickly. The magnitude of consolidation may be reduced by using light-weight fill, thus reducing the magnitude. Also, ground improvement could be utilized to mitigate the magnitude of consolidation as well as

techniques such as deep mixing of grout/soil slurries, aggregate piles, driven piles, dynamic compaction, and many other methods.

3. Relatively fast consolidation, low magnitude: This is rarely a problem. The estimated magnitude of consolidation is acceptably low or even negligible and the rate of consolidation is relatively fast, with “Fast” being a relative term typically associated with the pace of construction or dissipation of pore water pressures

4. Relatively slow consolidation, low magnitude: This is the type of consolidation typically associated with the approximate one to two inches of settlement at bridge approaches (i.e. the bump at the end of the bridge) which develops years after construction. While it is an annoyance, serviceability is not affected and the costs to mitigate it often outweigh the problem it is causing.

Primary consolidation settlement is influenced by the initial physical and engineering properties of the soil layers in the profile, stress history of the soils in the profile, and initial and final (e.g., after loading) stress conditions in the profile.

5.2.3.3 Secondary Compression Settlement

Secondary settlement occurs after excess porewater pressures associated with primary consolidation settlement have dissipated. For most soils, contributions from secondary settlement will be small. However, in soft soils, the amount of secondary settlement can be significant. Secondary settlement is most often a significant problem in regions with organic soils such as peat. Organic soil is extremely rare in Oklahoma, and it is not uncommon to see no estimate provided for the magnitude of secondary settlement in reports.

The decision to calculate and include secondary compression settlement can be made by reviewing laboratory consolidation test results, specifically the deformation vs log t plots and qualitatively assessing the relative steepness of the latter portion of the plot. In unsaturated and moderate to highly over consolidated soils, the contribution to total settlement will be minimal.

Compare calculated total settlements (immediate, primary consolidation, secondary compression) with the allowable settlement criteria for the applicable earth structure. Consider the differential settlement between fills and bridges. Differential settlement between structures founded on deep foundations and fills constructed of and founded on natural soils cause significant problems for roadway performance. Often special settlement mitigation options are required to reduce maintenance costs and preserve ride quality; see Section 5.2.4.

5.2.3.4 Rate of Settlement

The rate that settlement occurs depends on the coefficient of consolidation for the soil and thickness of the clay layer. If the soil layer has significant sand interlayers, the rate of settlement increases significantly because of the reduced drainage path length. Determination of the average drainage path length is an important component of field exploration.

5.2.3.5 Settlement Analysis for Cohesionless Soil

Settlement considerations for cohesionless foundation soils are usually limited at most fill sites. The limited consideration for cohesionless soil sites results from the rapid drainage that occurs when these soils are loaded, meaning the settlement occurs rapidly. Settlement calculations are made for cohesionless foundation soils using elastic theory.

5.2.4 Mitigation of Fill Settlement

5.2.4.1 Fill Soil

The contribution of settlement within the fill to the total settlement of the fill and fill foundation soil is generally considered to be minimal, because the settlement occurs as immediate settlement as the fill is constructed. However, construction practices including inadequate compaction, moisture control, and wetting-induced collapse (hydrocompression) can lead to significant settlements. For fills greater than 10-ft high and constructed of higher cohesion soils, the settlement cannot be considered minimal. For higher and more cohesive fills, the immediate settlement will be greater and the potential for primary consolidation settlement from pavement and traffic loading will be significant. The Geotechnical engineer should estimate the immediate and primary consolidation settlement along with similar calculations for the fill foundation soils and compare the values. If the amount of fill soil settlement is deemed significant, it should be minimized. Options for reducing fill soil settlement are generally limited to modifying the fill construction, specifically by increasing the stiffness of the fill soil by:

1. Compacting the fill soil to higher dry density. To achieve this, use modified Proctor instead of standard Proctor compaction effort for quality control (e.g., 95% compaction).

2. Chemically stabilizing the fill soil using additives (e.g., lime, fly ash, cement kiln dust); see OHD L-50 (Reference 2) for selection of type and amount of additive.

3. Reinforcing fill soil with horizontal geosynthetic (geogrid or geotextile) inclusion layers with vertical spacings of 2 ft to 4 ft.

Fill construction modification would typically be limited to critical areas subject to problems with differential settlement (e.g., approach embankment and bridge abutment area). Apply fill construction modification options within the zone bounded by the top of the approach embankment and extending 100 ft (±) away from the junction with the bridge abutment then sloping downward to the original ground surface at a convenient slope.

5.2.4.2 Fill Foundation Soil

The contribution of fill foundation soil settlement is generally considered to be the majority of the total fill settlement, particularly for higher fills and more cohesive foundation soils. Total fill foundation settlement can be minimized by one or more of the following options:

1. Controlling Settlement Time. Controlling settlement time can be accomplished by either allowing the settlement to occur before construction of roadway elements or by increasing the rate of settlement. Allowing the settlement to occur before construction of roadway elements is often referred to as staged or sequenced construction and involves taking advantage of the typical roadway construction sequence (e.g., grading/drainage, earthwork, roadway structure construction, paving) and coordinating the construction with the rate of settlement calculations previously discussed. The settlement can be monitored over time via the use of settlement plates or settlement cells. If the time between earthwork and roadway structure construction exceeds the time to complete 90% of the primary consolidation settlement, future settlement should be minimal, assuming the remaining 10% of settlement is acceptable. Alternatively, the time for settlement to occur can be accelerated by using the concept of preloading, with or without vertical drains. Preloading without vertical drains involves adding additional fill above final grade to increase the applied load and associated excess pore water pressures that speed up porewater pressure dissipation and settlement. Settlement can be accelerated further by

using prefabricated vertical drains (wick drains), which shorten the drainage path for the soil layers and speed up settlement. Selection and design of preloading, without or with vertical drains, requires an evaluation of the time rate of settlement before and after preloading.

2. Ground Improvement. Ground improvement is accomplished by increasing the density of the soil at significant depths using dynamic compaction or reinforcing the soils using methods such as stone columns, micropiles, or deep soil mixing. Each of these methods are briefly described in the following sections:

    a. Dynamic Compaction. A method of ground improvement that results from the application of high levels of energy at the ground surface. Apply the energy by repeatedly raising and dropping a tamper with a weight ranging between 10 to 20 tons at vertical distances ranging between 30 ft to 90 ft. Lift and drop the tamper by a conventional crane with a single cable plus a winch that has a free spool attachment that allows the single cable to unwind with minimum friction. The tamper's energy of impact at the ground surface results in densification of the soil profile to depths proportional to the energy applied. Depth of improvement generally ranges between 10 ft to 35 ft for light to heavy applications, respectively. Following the high energy level application, the surface of the profile is in a loose condition to a depth equal to the depth of the compaction craters. The surface layer is compacted on a tight spacing grid with a low-level energy application called an ironing pass.

    b. Stone Columns. Columns may be formed with densified gravel or crushed stone in a pattern (equilateral triangle or square) to create a composite foundation of the columns and the surrounding soil. The stiff columns carry a larger load than the surrounding soil, thus increasing strength and load capacity and reducing settlement.

       Typical column spacing varies from 5 ft to 10 ft and typical column diameters vary from 2 ft to 3 ft. Column lengths (depth) can be 50 ft or more. Column construction involves placing and densifying the gravel or crushed stone in the soil using vibrocompaction or dynamic compaction. Increased strength of the soil profile results from the densified column. In cohesionless soil profiles, densification of the soil between the columns can occur because of increased horizontal stress.

    c. Micropiles. Micropiles are small diameter (typically less than 12 in.) drilled and grouted non-displacement piles that are typically reinforced and used in groups. A micropile is constructed by drilling a vertical or battered borehole, placing steel reinforcement (single rebar or small rebar cage), and grouting (cement) the hole. Micropiles can support significant axial loads and moderate lateral loads and may be considered a substitute for driven piles or drilled shafts or as one component of a composite soil/pile network, depending on the design concept. Micropile installation methods cause minimal disturbance to adjacent structures, soil, or the environment and can be used in all soil types and ground conditions. Install micropiles at any angle below horizontal using the same type of equipment used for installation of ground anchors. Micropile structural capacity relies on high-capacity steel elements to resist most or all the applied load. The steel elements may occupy as much as one-half of the drill hole cross section. The drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout/ground interface. The grout transfers the load through friction from reinforcement to the ground in the micropile bond zone in a manner similar to ground anchors. Due to the small micropile diameter, the end-bearing is neglected.

    d. Deep Soil Mixing. This involves ground improvement techniques that mix additives with soils at depth to improve soil properties in-situ, without excavation or removal. The mechanical mixing aspects of the additives with in-situ soils distinguish this ground improvement method from others. Deep soil mixing can be used for ground water cut-off, excavation support, soil stabilization, settlement reduction or foundation support. Deep soil stabilization is performed under a number of different names or acronyms, many are proprietary. The basic concept and procedures are similar for all techniques, the mixed additives and objectives of the soil mixing can be categorized on following basis:

- Method of additive injection (wet (W) or dry (D))
- Method of additive mixing (rotary mix (R) or high-pressure jet (J))
- Location of mixing (near drilling tool (E) or along shaft (S))

Four common categories (WRS, WRE, WJE, DRE) are used in practice and fall into two distinct groups:

i. Deep Mixing Methods (WRS, WRE, WJE). This refers to wet, single or multiple auger, block or wall techniques developed for large scale foundation improvement and containment in any soil. Primary additives are cement-based grouts.

ii. Lime and Lime-Cement Columns (DRE). This refers to a dry, single-auger column technique developed for soil stabilization and reinforcement of cohesive soils. Primary additive is granular or power lime for lime columns and cement or lime-cement mixtures (References 3 and 4).

3. Reducing the Applied Load on the Foundation Soil. Reducing total settlement by reducing the load the fill applies to the foundation soil can be achieved by reducing the grade line (reducing height of fill) or by using lightweight materials to replace the fill soil (e.g., geofoam). Other lightweight materials are available, but not commonly used because of cost or availability. Lightweight, flowable fill, commonly referred to as low-density cellular concrete (LDCC) or lightweight cellular concrete (LCC) is flowable fill with small circular styrofoam cell inclusions to reduce its weight. Another product that is beginning to see use on some transportation projects is lightweight aggregate. This material typically weighs 20 to 30% of the weight of normal concrete. Lightweight foamed glass aggregate is another product that has now been used on a number of transportation projects. Expanded polystyrene (EPS) geofoam is an innovative product that weighs less than 5 percent of the weight of soil that has been used on many transportation projects as embankment material to reduce foundation soil loads. Other lightweight materials that have been used for this purpose include expanded shale, fly ash, wood fibers, and shredded tires.

5.2.4.3 Field Testing

Proper construction and performance of fills requires competent and thorough inspection and monitoring for long-term performance. The Geotechnical design engineer needs to be familiar with construction issues and performance monitoring requirements. Field construction/ inspection personnel need to be familiar with project plans, specifications and quality control procedures and requirements. Communication between design and construction personnel is required for efficient fill construction and desired performance.

5.2.4.4 During Construction

During construction, inspectors should ensure that the fill foundation surface is properly prepared. The *ODOT Standard Specifications for Construction* detail the criteria for clearing and grubbing, erosion & sediment control, and fill foundation preparation. Fills are constructed in layers with the fill soil spread in desired loose lift thickness, moisture content adjusted, then compacted with appropriate equipment to specified conditions. Quality control involves measuring field dry density and moisture content, along with comparing the values to the compaction specification acceptance criteria for pass or fail.

5.2.4.5 Post Construction

Post construction monitoring starts with observation of the fill for indications of settlement or potential erosion problems. Surface elevation monitoring (survey) can confirm that anticipated settlement is occurring at an acceptable rate or alert personnel to a faster rate of settlement. For fills on settlement-prone foundation soils, more detailed/sophisticated monitoring equipment and programs may be required. For example:

- Instrumentation of fill and fill foundation settlement
- Piezometers to monitor groundwater conditions
- Inclinometers to monitor lateral movement
- Load cells to monitor loads on structures

5.3 SLOPE STABILITY

5.3.1 Criteria

This section addresses natural slopes adjacent to roadways or slopes resulting from roadway construction (cut or fill slopes). The stability of natural slopes is determined by local geology. Geology can be either soil or rock, with or without groundwater. The stability of cut and fill slopes is determined by the height, angle, and inherent strength properties of the slope material.

There has been a history of slope movement in natural and cut or fill slopes throughout Oklahoma. This movement has ranged from relatively small failures in over-steepened slopes along roadways to very large landslides and rockfalls. Slope failures in these natural deposits have been attributed to a variety of causes, including:

- Groundwater infiltration
- Oversteepening of existing slopes from natural processes (e.g., erosion) or from new construction
- Upslope loading
- Excavations at the toe of slope
- Poor fill construction procedures

For cut slopes greater than 30 ft below the natural ground line in soil, the Geotechnical engineer should analyze both end of construction and long-term slope stability conditions. If slope materials are over consolidated (OCR > 1), then the residual shear strength should be used for the long-term slope stability analysis. For over consolidated clays, fully softened strength should be used for first time slides and residual strength for areas with previous slides or slicken-sided structure.

5.3.2 Loading Cases

The following loading cases are normally addressed during slope analysis and design:

1. End-of-Construction Loading. This loading case occurs as the slope is constructed. The primary design issue is whether the existing foundation soil can support the new slope loads without undergoing bearing failures, side slope instability or excessive settlement. These conditions are most critical where soft cohesive soils make up the foundation soils. The end-of-construction evaluations may influence the rate of construction, which may need to be controlled to prevent construction failures or include ground improvement methods.

2. Long-Term Operational Loading. This loading case occurs after the slope has been constructed to the final grade and excess porewater pressures have dissipated. The long-term stability of slopes should be analyzed, especially in fine-grained soils. In the event the foundation soils are cohesive and not heavily over-consolidated, settlement will be a design concern. Consolidation settlement and secondary compression can continue for many years and, depending on the thickness of the fine-grained soils and the amount of loading, can be several feet or more. Significant settlement can result in distress to the pavement at the top of the fill slope, as well as bumps and dips in the pavement at cut/fill transitions, and at the transition to bridges.

5.3.3 Evaluation of Slope Stability

The Geotechnical engineer should determine the stability of natural and cut or fill slopes by analyzing the geologic conditions at a site, including the location of groundwater and proposed slope geometry. Consider the factors in the following sections when planning and assessing the stability of natural and cut or fill slopes. Section 5.3.3.6 provides a summary of factors of safety (FS) requirements that must be satisfied when evaluating the stability of roadway slopes at soil site.

5.3.3.1 Overview of Approach for Stability Assessment

The stability analysis of a natural slope or cut slope involves consideration of the following:

1. Soil Strength Parameters. Review the soil strength parameters for each material within the slope. This will include determining the effects of the following:

   - Long-term versus short-term loading (drained versus undrained strength)

   - Selection of strength parameters from test results, including total stress and effective stress

   - Any necessary adjustments for mode of failure (e.g., triaxial compression, simple shear, triaxial extension)

2. Groundwater. Determine the groundwater level (piezometric level) for the average and worse-case conditions. Consider the potential for variations in groundwater levels, artesian effects, perched water, potential for rapid drawdown, and the effects of irrigation.

3. Cross Section. Analyze cross sections along the slope to determine vertical and lateral limits and other details (e.g., soil density) of each soil layer.

4. External Loads. Analyze the effect of external loads on slope stability.

5. Method of Analysis. Consider the pros and cons for each method of analysis, including the iterative nature of the program and the importance of parametric studies.

6. Construction. Consider the construction schedule and any safety issues that may arise during construction.

The following sections provide additional details for conducting stability analysis.

5.3.3.2 Information for Stability Analysis

Stability analysis requires accurate information on geology and groundwater conditions at a site. The field exploration program must:

- identify low-strength soil layers that could serve as slippage planes. CPT, DMT, vane shear and continuous SPT investigations can be particularly valuable for this task at some sites;

- collect quality representative samples. Determine whether undisturbed samples are required and, if so, the type of sampler needed and how the samples are handled; and

- establish the groundwater elevation. This may require installing and reading piezometers over time to determine the fluctuation in groundwater table.

During the planning phase, it may be valuable to perform preliminary stability analyses based on expected soil conditions. Information from preliminary analyses can often help determine the appropriate location and depths of explorations. Seismic refraction lines are sometimes valuable in identifying the depth to rock. If the presence of soft clay layers is possible, it may be desirable to conduct in-situ tests such as vane shear or DMT tests to obtain strength information for soft clay deposits.

5.3.3.3 Loading Conditions

The loading conditions that are normally evaluated during the stability analyses include:

1. Short-Term Loading. This condition is important for cut slopes or slopes that have new loads (e.g., roadway fill slopes). Consider the following:

   a. Cohesive Soils. Undrained strength should be used to determine stability. In this loading state, the porewater pressures do not have time to dissipate and the strength represents the minimal or weakest state of stress.

   b. Cohesionless Soils. If the soil is a relatively clean cohesionless soil, the porewater pressures could dissipate as quickly as the soil is loaded. In this case, the strength should be determined by the drained properties of the soil using the effective stresses in the analysis.

2. Long-Term Loading. This condition governs the behavior of many natural slopes; however, it is also appropriate for determining the long-term strength of a cut or fill slope. In this case, porewater pressures are fully dissipated. For cohesive or cohesionless soils,

effective strength parameters determine the strength of the soil for long-term conditions.

It is not always easy to determine whether soil will behave in a drained or undrained manner. Both the rate of loading and the permeability of the soil will determine whether the soil responds in a drained or undrained state. Because it is often very difficult to predict the rate of loading during design, the best approach is to check both the drained and undrained cases, and to use the more critical strength as the basis for design.

5.3.3.4 Selection of Strength Parameters

Stability analysis requires establishing shear strength and groundwater conditions for each soil layer identified during the field exploration program. Before assigning strength parameters, an accurate cross section of the site should be developed. This cross section should be used to develop potential failure mechanisms and loading conditions that may control stability. The type of soil and the loading condition will determine the method used to develop strength parameters for analyses.

Studies have shown that the primary mode of shear during slope failures is not always consistent with the mode of failure that occurs in laboratory triaxial tests and is dependent upon the location along the potential failure plane. The strength developed along a failure surface may be more closely represented by the strength from a direct shear (DS) test rather than a triaxial test.

5.3.3.5 Groundwater Conditions

Groundwater conditions often are the cause of slope failures in natural or cut slopes. A number of factors determine or influence groundwater conditions in slopes including infiltration (precipitation or irrigation), perched water layers, or simply the groundwater surface. The groundwater level likely will vary with the time of year depending on climatic changes, heavy rainfalls, and changes in river elevation. These changes in groundwater location influence the effective stresses, which will affect the stability of a slope. Consequently, a key step in the stability analysis involves identification of both the current location of the groundwater and potential fluctuations of groundwater. Often this will require installing piezometers at the site and recording groundwater elevation changes with time.

5.3.3.6 Factors of Safety

Results from limit equilibrium stability analyses are normally given in terms of a factor of safety (FS) against instability for the specified load conditions. In these analyses, the FS defines the ratio of forces resisting instability, which is defined by the strength of the soil to the loads causing instability. The loads include not only those from gravity, but also from external sources (e.g., traffic, water forces). A slope is considered theoretically stable if the FS is greater than 1.0. However, because of uncertainties in the method of strength determination, groundwater location, and method of analysis, a margin greater than 1.0 will be required for most designs.

The selected FS will depend upon specific site conditions. For normal conditions, ODOT requires that slopes meet the following FS values:

End of Construction:	FS is greater than or equal to 1.3 to 1.5 (at structures)
Long-term:	FS is greater than or equal to 1.3
Cut Slopes in Clay:	FS is greater than 1.5
Sudden Drawdown:	FS is greater than 1.2 to 1.3

Seismic: FS is greater than 1.1 to 1.2

If these FS values cannot be met, stabilization methods should be considered to improve the FS value, or a lower factor of safety might be accepted with the Geotechnical engineer's approval. Before accepting a lower FS value, the project Geotechnical engineer should evaluate the costs of stabilization versus the cost of accepting additional risk to public safety, economic disruption, or repair costs.

If the consequences of slope instability are significant (e.g., a major bridge being damaged), an FS ≥ 1.5 or higher may be desirable, depending on the specific conditions involved with the instability. The decision to use a higher FS value for design could have significant economic consequences, if stabilization is required to meet the target FS. Therefore, the project Geotechnical engineer should discuss the cost of stabilization versus risk with the designer and field district engineer to determine if the additional costs are warranted.

5.4 SLOPE STABILIZATION / REMEDIATION METHODS

If the slope stability analysis determines that slopes will not meet the design stability criteria during construction or long-term performance, stabilization measures can be used to improve stability and meet the criteria. Slope stabilization methods used for new slope construction include such alternatives as cross section design, surface and subsurface drainage, foundation soil improvement, and fill soil improvement. Slope remediation methods used for repairing and returning slopes to service include such alternatives as cross section modification, surface and subsurface drainage, buttresses, retaining walls, and reinforcement.

5.4.1 Slope Stabilization Methods

Evaluate the stability of new fill or cut slopes by considering one or more of the following methods. The project Geotechnical engineer should understand that the stability of both the foundation soils and the fill soils are mutually dependent upon each other to ensure a stable slope.

1. Modify Slope. Modify the slope alignment, height, inclination, or load applied to the foundation soil. Relocating the alignment to avoid slide-prone areas or areas where landslides have been a problem before is often difficult because of route selection requirements and the need for extensive alignment studies to identify problem areas. However, the investment may be justified if local experience indicates significant potential stability problems. Changes to cross section features such as decreasing the height of the slope, reducing the slope inclination and, in the case of fill construction, changing the material the fill is constructed of, are all design features that improve slope stability.

2. Drainage. Drainage or control of both surface and subsurface water is an important and efficient stabilization measure for fill or cut slopes because uncontrolled water is often the trigger for many slope failures. Drainage reduces driving forces (e.g., weight of sliding mass) and increases effective weight and resisting forces (e.g., shearing resistance).

   Drainage also reduces or eliminates seepage forces that can cause surface and internal erosion. Selection of surface drainage measures should include temporary and permanent erosion control measures. Surface drainage details are an important aspect of slope design. Surface water should be directed to the ditch with low enough velocities to minimize surface erosion and the ditch should have sufficient gradient to remove the water from the fill or cut slope area without ditch erosion. Subsurface drainage measures should intercept, collect, and remove moving ground water. Commonly used subsurface measures include:

a. Trench Drains. Trench drains with aggregate or geosynthetic drainage materials to facilitate collection and removal of water. Used commonly in cuts.

b. Slope (Blanket) Drains. Slope drains with aggregate or geosynthetic drainage materials typically used beneath fills or buttresses or in cut repair sections.

c. Edge Drains. Edge drains using geosynthetic composite drains to control ground water along pavements.

3. Foundation Soil Improvement. Soils beneath fill may not have sufficient strength to support fills, with a common result being fill slope failure. For new fill construction, the project Geotechnical engineer has the opportunity to incorporate various ground improvement technologies. For example:

   a. Dynamic Compaction, Stone Columns. Deep Soil Mixing, and Micropiles. See Section 5.2.4 for details.

   b. Vibro-Compaction. A technique that uses specifically designed probe-type, vibrators for in-site densification of cohesionless soils. The vibrator is inserted in the soil profile with water or air jetting or by the vibration energy of the probe itself. Water jetting is the most common method used. Once the probe is inserted to the desired depth, granular soil is dumped into the annulus around the probe and systematically densified by raising and lowering the vibrating probe. The suitability of a soil for vibro-compaction depends mainly on grain size. Soils with grain sizes on the coarse side of the U.S. No. 200 sieve are readily compacted by deep vibration (Reference 3)

4. Fill Material Improvement. Soil used for fill construction is typically selected on the basis of specified criteria (e.g., borrow) and generally exhibits better engineering properties than the foundation soil. However, the availability of quality fill soil may be limited thus requiring the soil to be improved in order to get acceptable performance of the fill. The project Geotechnical engineer may select from the following options for fill material improvement:

   a. Strengthen or Stiffen Fill Soil. This is accomplished by increasing the density of the soil through greater compaction energy (95% Modified Proctor vs 95% Standard Proctor) or by chemically modifying or stabilizing the soil. Strengthening by increased compaction increases the density and stiffness of the soil, which accelerates consolidation settlement of the foundation soil and reduces post-construction settlement at the top of the fill. The fill soil strength is improved, which directly increases stability. Placement moisture contents should be near or slightly above optimum moisture content.

   b. Reinforcement. This is accomplished by reinforcement of the boundary between fill and foundation soil, or by placing geosynthetic reinforcement throughout the fill soil. Base reinforcement uses geogrids or geotextiles to distribute the fill load more broadly to the foundation soil, resulting in increased bearing capacity and reduced settlement of the foundation soils. Types and properties of geogrids and geotextiles, along with design criteria and methods, are available in FHWA *Geosynthetic Design and Construction Guidelines*. Base reinforcement is commonly used when the bearing capacity of the foundation soil is exceeded and/or the rate of fill construction is such that excess pore water pressures cannot dissipate efficiently. If the purpose of the geosynthetic is to provide a separation layer between the fill and foundation soil, select a geotextile appropriate for the separation function.

Reinforced fill soil consists of tensile reinforcements added to soil to form a stronger composite soil mass. Reinforced soil structures are generally classified as either a wall when the inclination of the face of the slope is 70˚ or greater from the horizontal (referred to as Mechanically Stabilized Earth Wall (MSEW)), or as a Reinforced Soil Slope (RSS) if the slope face inclination is less than 70˚ from the horizontal. MSE walls are cost effective compared to conventional concrete retaining structures especially for walls in fill cross sections. MSE walls are more flexible than conventional walls and are suitable for sites with poor foundation soils. RSS are a form of mechanically stabilized soil that incorporates planar reinforcing elements in constructed earth structures (e.g., fills). Multiple layers of geogrids, geotextiles and various steel mats are placed horizontally within a soil slope to strengthen the soil and improve stability.

5.4.2 Slope Remediation Methods

Remediation or repair of failed fill or cut slopes is necessary when the slope failure compromises the performance of the roadway structures. Remediation measures vary from slope reconstruction to options that increase stability from external loading (e.g., berms or buttresses) to inclusion of reinforcement elements (e.g., micropiles, soil nails, anchors, cantilevered walls as well as proprietary stabilizations systems such as proprietary plate piling systems, soil nails with proprietary mesh, etc.). As discussed in Section 5.4.1, surface and subsurface drainage is also an important part of slope remediation. Identification of source and removal of uncontrolled surface and subsurface water needs to be addressed prior to selection and design of slope remediation options. The following provides brief descriptions of common slope remediation options:

1. Changes to Slope Alignment or Cross Section Features. Changes typically involve moving the roadway alignment “into” the slope for cuts or reconstruction of the fill in essentially the same location. Where feasible, flattening the slope inclination or lowering the slope height can be included in the adjusted alignment. Stability of the adjusted alignment or cross section will need to be evaluated.

2. Drainage. See Section 5.4.1 for details.

3. Berms or Buttresses. Berms or buttresses provide sufficient external load (weight) and increased shear resistance at the toe of the slope to increase stability of the slope to an acceptable level. The basic design of berms or buttresses is similar to the design for external stability of conventional retaining structures (e.g., overturning, sliding, bearing failure). Conduct a settlement analysis if the foundation soils are compressible to ensure the final grade of the berm or buttress is consistent with the design requirements of the project. Material for berm construction can often be lower quality fill because the primary function is to provide increased load at the toe. Material for a keyed buttress is often a well graded crushed rock because it acts as a retaining structure for the toe of the slope. Other considerations when evaluating berms or buttresses include right-of-way restrictions, location of environmentally sensitive areas, nearby utilities or structures that could be affected by the additional loads, and cost of materials.

4. Slope Reinforcement. This involves the removal of the affected fill within the failing slope area and beyond, followed by reconstruction of the fill using the geosynthetically reinforced soil slope (RSS) techniques described previously. Sufficient material is removed and replaced to either (a) eliminate the failure surface, or (b) allow the reinforced soil to adequately “bridge” the failure surface within the foundation material. This technique can be temporarily disruptive to the site but can be an effective solution where other remediation techniques are less feasible.

Other methods of slope remediation require the use of inclusions that must extend below the identified failure surface. These inclusions serve to tie the failed soil mass and supporting soil together with a network of reinforcing inclusions or to hold the failed mass in place by lateral resistance achieved from essentially simple retaining structures. Examples of such inclusion methods are:

1. Soil Nailing. This consists of the passive reinforcement (i.e., no post-tensioning) of existing ground by installing closely spaced steel bars (i.e., nails), which may be subsequently encased in grout. As construction proceeds from the top to bottom, shotcrete or concrete is also applied on the excavation face to provide continuity. Soil nailing is typically used to stabilize existing slopes or excavations where top-to-bottom construction is advantageous compared to other retaining wall systems. For certain conditions, soil nailing offers a viable alternative from the viewpoint of technical feasibility, construction costs, and construction duration when compared to ground anchor walls. Soil nails are installed with a near horizontal orientation (i.e., inclination of 10˚ to 20˚ below horizontal) and are primarily subjected to tensile stresses. Soil nail systems are used to stabilize natural slopes and excavations. Proprietary slope stabilization systems involving soil nails with a proprietary net or mesh (contraption are also used for slope stabilization projects. An alternative application of soil nails is sometimes used to stabilize landslides. In this case, the reinforcement (sometimes also called nails) is installed almost vertically and perpendicular to the base of the slide. In this alternative application, nails are also passive, installed in a closely spaced pattern approximately perpendicular to the nearly horizontal sliding surface, and subjected predominantly to shear forces arising from a landslide movement.

2. Cantilever Walls. Use to remediate failed soil slopes. Cantilever walls can be used as earth retention walls in slope remediation projects, and can include drilled shaft walls (intermittently spaced, tangent pile walls, or secant pile walls) or soldier pile walls. In the case of intermittently spaced drilled shaft walls, the shafts are spaced with a gap between them. Tangent pile walls are constructed such that each shaft is in contact with adjacent shafts. Secant pile walls are situated such that adjacent shafts overlap each other. Soldier pile walls consist of installing piles, usually at 5- to 10-ft spacing, and lagging which spans the distance between the soldier piles. Lagging is used to retain the soil face from sloughing and to transmit lateral earth pressure to the soldier piles. While soldier piles can be almost any structural member, the most common soldier piles are rolled steel sections, normally H-pile or wide flange sections. For slope stabilization projects, the lagging commonly consists of precast concrete panels. Cantilever walls increase the stability of slopes by increasing the shearing resistance across the failure surface. In order to do this, the shafts must be able to withstand the bending stresses exerted on them from the failure mass and extend a sufficient depth below the failure surface to develop enough passive resistance to withstand the driving force. Cantilever walls can work particularly well at sites with hard soils or rock below the failure surface, enabling efficient resistance to high bending moments. Plies can be installed through the hard strata via pre-drilling.

3. Sheet Pile Walls. Use as an alternative to conventional cantilevered walls as an earth retention wall. The wall consists of interlocking steel sheet pile sections driven to sufficient depth to balance the earth pressures acting on the wall. Sheet pile walls are used for slope remediation as permanent structures designed using conventional earth retaining wall design procedures. Sheet pile walls may be cantilevered or anchored. See the textbook *Foundation Engineering Handbook* (Reference 5) for design details.

4. Ground Anchor. These walls are a structural system that uses grouted anchors to apply a stabilizing force to the slope. The anchor is composed of pre-stressing steel with sheathing and an anchorage. The anchor transmits the tensile force in the pre-stressing steel to the ground via the cement grout. The bond length is the resisting length of the

tieback where the tieback force is transmitted to the ground behind the critical failure surface of the slope or embankment. The tendon bond length is the length of the tendon that is bonded to the anchor grout. Normally, the tendon bond length is equal to the anchor length. The unbonded length of the tendon is the length that is free to elongate elastically. When used to stabilize slopes, the unbonded length is typically the length of the anchor from the wall face to the critical failure surface. For a detailed discussion on the design and construction of permanent ground anchors, see FHWA *Ground Anchors on Anchored Systems* (Reference 6).

5. Helical Anchors. These anchors are an in-situ soil reinforcement technique wherein passive inclusions are placed into the natural ground at relatively close spacing (3 ft to 5 ft) to increase the strength of the soil mass. Helical anchors consist of 1.5-in square solid steel shafts, on which steel bearing plates or helices are welded at regular intervals. Being a true helical shape, the helices do not auger into the soil, but rather screw into it with minimal soil disturbance. Helical anchors are installed using drill equipment with sufficient torque output. The drill is often attached to a backhoe, skid loader or track hoe. The steel used to manufacture the nails is a high-strength alloy that is specifically formulated to resist the installation stresses associated with the high torque applied to the anchors during installations.

There are additional innovative proprietary slope stabilization solutions that are variations of those mentioned in the previous text. Examples include but are not limited to plate pile systems such as SRT® plate pile system, TECCO® mesh system, soil nail launching, and Supernail.

5.4.3 Selecting Stabilization or Remediation Measures

Many stabilization/remediation measures are expensive and can have a significant effect on construction staging and overall schedules. When evaluating stabilization/remediation measures, the project Geotechnical engineer should perform detailed evaluations considering the following:

1. Project Schedule. The construction schedule is one of the most significant considerations. The project schedule will influence which mitigation measures can be used.

2. Construction Access and Constraints. Some stabilization/remediation measures require easy access and large working areas. This could potentially preclude use in some locations.

3. Types and Depth of Soil. The type of soil will have a significant effect on the selection of the stabilization measure.

4. Cost. Specific solutions will range greatly in cost depending on location and on the stabilization method selected.

The preferred approach for selecting the stabilization/remediation measure is to identify possible methods based on state-of-the-art research on ground improvement. After obtaining background information, it is often desirable to contact and meet with potential contractors to determine other issues and factors that could control the success of the method. For stabilization/remediation measures that involve specialty methods, specifications have to be very clear about contractor expectations. It is also critical for the contractor to have the skills and knowledge for the specialty stabilization/remediation method.

The proprietary nature of certain methods requires that the proprietary system owner or representative prepare the design of the overall system, and installation can require certified, specialty contractors affiliated with the system owner.

5.5 ROCK SLOPES

5.5.1 Evaluation of Rock Slope Stability

The stability of rock slopes is an important consideration for roadways that are located on or next to rock slopes or excavated into rock slopes. The objectives of the rock stability analysis are to determine global stability as well as the stability of individual blocks of rock. This section summarizes the modes of failure, methods of assessing stability, determination of rock strengths and acceptable factors of safety.

Rock stability analysis is required when the dip of the geological formation exceeds 20° into the slope face. Alternatively, rock stability analysis should be conducted for dip angles less than 20° when certain site or geologic conditions indicate a greater potential for slope instability. Such conditions include, but are not limited to:

- High groundwater table
- Low RQD values from borings
- Prolonged freeze/thaw exposure
- Elevated seismic risk
- Presence of imbedded material (e.g., shale)

The analysis must meet all the requirements of the kinematic slope stability analysis, using the stereographic projection procedure. This analysis is necessary to determine the slope stability of closely spaced (2 ft or less) rock joints (fractures) and/or tilted (dipping greater than 10°) rock strata of the cut slope. These measurements will allow development of the local structural geology, in three dimensions, required for making this analysis. The data that is required for this analysis is the dip, and dip direction of both the rock strata and of the joints (fractures) in the rock. The equipment necessary to obtain the dip and joint orientation data is a Clar and/or Brunton compass. This device gives magnetic headings and dip angles. Trenching or oriented cores may be necessary in order to expose enough rock strata to make the measurements. Determine the shear strength of the jointed rock based on the Hoek-Brown criteria. If the observations identify joints (fractures) in which shear failures may occur, or fractures that contain soil infilling; then, the shear strength of the infilling or fractures is required to be taken into account in the overall slope stability analysis. In argillaceous massive shales (non-laminar), slope stability analysis is required based on the use of a soil mechanics approach.

To evaluate the stability of rock slopes, it is important to determine:

- Top location of rock, if covered with overburden
- Any variation or discontinuities (fracture/joint) patterns and conditions
- Strength and groundwater conditions

Characterization of rock location and discontinuities can be accomplished through drilling of boreholes, seismic refraction surveys and/or surface mapping of exposed rock faces. Rock strength evaluations can involve laboratory or field testing; however, often strengths are based on a combination of published information and rock mass conditions. Groundwater conditions are established by either monitoring piezometers installed in the rock or observing seeps and flows during field reconnaissance.

5.5.2 Modes of Failure

The failure of rock slopes is controlled primarily by the orientation and spacing of discontinuities (e.g., joints, bedding planes) within the rock mass, and the orientation and the angle of inclination of the slope. These parameters will determine the mode of failure. For analysis purposes, the modes of failure are divided into the following general groups:

1. Sliding Failure. These failures include planar, wedge and circular failures, and involve single or multiple blocks sliding on a bedding plane or a joint set striking approximately parallel to the slope strike. Wedge failures can occur in rock masses with two or more sets of discontinuities whose lines of intersection are approximately perpendicular to the strike of the slope and dip toward the plane of the slope. Global sliding failures can occur along circular slip paths, particularly in highly weathered and decomposed rock masses, highly fractured rock masses or in weak rock (e.g., shales, poorly cemented limestone).

2. Toppling Failure. This failure involves overturning or rotation of columns or blocks of rock about a fixed base. Closely spaced, steeply dipping discontinuity sets that dip away from the slope are necessary for toppling. Toppling failure is usually initiated by layer separation with movement in the direction of the free face or excavation.

3. Sloughing Failure. This failure is generally characterized by occasional rock falls or localized slumping of rocks degraded by weathering. Rock falls occur when blocks become loose and isolated by weathering and erosion. Both rock falls and localized slumping can occur.

Geometric boundaries imposed by the orientation, spacing, and continuity of joints and free surfaces define the potential modes of failure. Several factors can initiate failure including erosion, groundwater, temperature, and state of stress.

5.5.3 Methods of Assessing Stability

The method selected for analyzing the slope depends on the potential failure mode, e.q. sliding, toppling, or localized sloughing. Key considerations include location of joints, bedding planes, and rock strength properties (i.e., intact rock and interface properties). The stability assessment methods are identified as follows:

1. Sliding Stability. Sliding stability of rock slopes is usually evaluated using limit equilibrium methods. Various methods have been developed to analyze sliding stability for planar slip surfaces, three-dimensional wedge-shaped slip surfaces and circular slip surfaces.

2. Toppling Stability. Toppling occurs if two conditions apply:

   a. the projected resultant force (body weight plus any additional applied force) acting on the block falls outside of the base of the block and

   b. the inclination of the surface on which the block rests is less than the friction angle between the block and the surface. A combination of sliding and toppling occur when the slope angle is greater than the interface friction, and the width to height ratio (b/h) of the block is less than tan γ, where γ is the interface friction.

3. Sloughing Stability. This type of stability includes landslides, talus slope failures and debris flows. They are evaluated by conducting hand or computer analyses, similar to methods used to evaluate the stability of soil slopes. Often these slopes are at their angle of repose. Small changes in groundwater conditions or loading can trigger instability in a

slope. Characterization of the strength properties of these materials is particularly difficult because of the wide range of material types.

Specific software can handle water pressures, surcharges, and seismic loading for planar and wedge analyses. Some of the same computer software used for the evaluation of slope stability in soils is also used for these analyses.

5.5.4 Strength and Groundwater Determination

Determination of the appropriate strength to use in the rock slope stability analysis is more complicated for rock than for soil, because of the effects of the rock mass properties. It is difficult and expensive to sample and test large samples of fractured rock. The alternative is to use empirical methods for evaluating friction angle and cohesion intercept of fractured rock masses.

Groundwater that occurs along a discontinuity or within a slope mass will reduce the stability of the rock slope, either by introducing an uplift pressure at the discontinuity or by lowering the effective stresses in broken rock. Changes in groundwater can be either permanent or seasonal.

5.5.5 Rock Slope Stabilization Methods

If the results of the stability analyses indicate that the roadway slope does not meet the minimum factor of safety requirements or displacement limits, then it may be necessary to use slope stabilization methods to improve the slope performance if the slope cannot be economically flattened to achieve an acceptable factor of safety.

Stabilization methods for rock slopes depend on the type of failure mode identified during the field reconnaissance and through stability evaluations. The size of the feature requiring stabilization often is another important consideration when selecting the most cost-effective stabilization method. In many situations, the preferred approach for stabilizing an unstable rock block or mass is to force a controlled failure of the block or mass.

The following stabilization options are available:

1. Tieback Anchors. Tieback anchors are commonly used to stabilize rock slopes. The tieback is similar to that described for soil slopes. Reaction would normally be developed by tensioning the tieback strand against the rock or a concrete pad cast at the face of the rock.

2. Rock Bolts. This stabilization system is similar to the tieback anchor, except that the rock bolts are typically not pretensioned. Rather, the rock bolt is grouted in a borehole and post tensioned. Its primary objective is to tie the rock mass together through the tension and shear capacity of the grout and a high strength reinforcing element.

3. Draped Rock Nets and Fences. Where small rock falls occur or the primary mode of failure is sloughing, a common method of stabilizing slopes is either to use high-capacity rock fences or drape nets. In many cases, the intent of a barrier is to slow the rate of rockfall or catch the rock before it reaches the roadway.

4. Widened Ditches. Widened ditches are used to catch small rock falls. Rock benching is not encouraged.

5.5.6 Factors of Safety

The factor of safety for rock slopes is usually approached on a site-specific basis. For major rock slopes where the consequences of failure are severe, the minimum required calculated factor of safety should usually range from 1.5 to 2.0. For minor slopes or temporary construction slopes where failure would not cause a hazard to individuals or major loss of property, the minimum required factor of safety is 1.3.

For a rock slope to be judged safe with respect to failure, the minimum factor of safety for all potential failure planes must be equal to or greater than the minimum required value. As with the discussion for soil sites, if these FS values cannot be met, stabilization methods can be considered to improve the FS value, or a lower factor of safety might be accepted with the ODOT Geotechnical engineer's approval. Before accepting a lower FS value, the Geotechnical engineer should evaluate the costs of stabilization versus the cost of accepting additional risk to public safety, economic disruption or repair costs.

Also, similar to the soil site, if the consequence of rock slope instability could be significant (e.g., a major bridge being damaged), a FS $\geq$ 2.0 may be desirable depending on the specific conditions involved with the instability. The decision to use a higher FS value for design could have significant economic consequences if stabilization is required to meet the target FS. Therefore, the Geotechnical engineer should discuss the cost of stabilization versus risk with the designer and Field District engineer to determine if the additional costs are warranted.

5.6 SPECIAL STUDIES FOR LANDSLIDES

Landslides present a special set of considerations that the project Geotechnical engineer must address on occasion. Where landslides occur, there is often a need to identify quickly the likely cause of the failure and to develop short- or long-term methods of mitigating the failure. If the failure is blocking or could block an Interstate or heavily traveled roadway, emergency response could be required. The following discussion summarizes some of the considerations during the back analysis of landslides and slope failures.

5.6.1 FIELD EXPLORATION

An appropriate landslide investigation process cannot be defined by a rigid set of procedures. The field investigation and the determination of necessary stabilization methods vary based on site conditions. When investigating landslides, the project Geotechnical engineer should consider the following:

1. Area. The area of an investigation is controlled by the size of the project and the extent of the topographic and geologic features encompassed in the landslide activity. The area studied must be considerably larger than that comprising the suspected active or known movement. This investigation may be accomplished by studies ranging from a simple field reconnaissance to in-depth topographic surveys.

2. Depth. The depth of the subsurface investigation should extend deep enough to identify underlying formations that are likely to remain stable. The investigation should identify materials that have not been subject to past movements but could be involved in future movements. During field investigations, the boring depths vary depending upon the data obtained on-site. The Geotechnical engineer will typically be responsible for determining the appropriate borehole depths.

3. Data Collection. Field data collection may involve a variety of activities ranging from relatively simple reconnaissance studies to sophisticated, specialized instrumentation installations. The Geotechnical engineer determines what data should be collected. The type of data collected determines the appropriate testing equipment required.

5.6.2 Stability Evaluation

Once the landslide or slope failure has been described by one of the previous procedures, the project Geotechnical engineer will normally perform a back analysis to define soil strength parameters existing at the time of failure. The factor of safety is assumed to be 1.0 and soil properties are varied in the slope stability program until FS = 1.0. While this concept is relatively simple to apply, assumptions regarding groundwater conditions, failure geometry, and the spatial variation of soil properties within the landslide or slide failure zone will have a significant effect on the back-calculated properties. Other issues include the degree of drainage during failure and the potential for progressive failures or changing strength parameters with displacement.

Results of the back analysis can be used to develop different stabilization concepts. These concepts can range from simply removing the slide mass to the use of retaining structures or groundwater control. Benefits of a stabilization procedure can be evaluated based on the change in the factor of safety relative to the existing condition. Effects of different assumptions on strength and groundwater should be included in the assessment to account for uncertainties in the back analysis.

5.6.3 Construction Support

The Geotechnical engineer may be requested to provide oversight during repair of the landslide or slope failure, particularly if an emergency condition is identified. This support can range from documenting work done by subcontractors to conducting analyses to help decide the type of repairs that should be implemented. Often this will require close communication with the contractor who is performing the repair.

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Chapter 6

FOUNDATION FOR STRUCTURES

ODOT GEOTECHNICAL MANUAL

January 2025

6 FOUNDATION FOR STRUCTURES

6.1 GENERAL

6.1.1 Overview

The satisfactory performance of a bridge structure depends on the proper selection and design of foundations used to support the bridge. This chapter discusses ODOT specific criteria for the geotechnical design of bridge foundations.

The function of the bridge foundation is to safely support loads from the piers and abutments while limiting settlements to tolerable levels. Knowledge of the loading conditions, environmental and climatic effects over the life of the structure, plus an understanding of subsurface soil conditions, location and quality of foundation material, groundwater conditions, local construction practices, and scour effects is necessary to choose the most appropriate foundation type and size.

6.1.2 Responsibilities

Bridge foundation design involves close coordination between the Structural engineer and the Geotechnical engineer. Where river or stream crossings occur, the Hydraulics division also has a key role in the overall design process. The coordination between these units is summarized in the following sections.

6.1.2.1 Geotechnical and Structural Coordination

The Geotechnical engineer and Structural engineer coordinate to determine the selection and design of bridge foundations as summarized below:

1. Geotechnical Information. The project Geotechnical engineer prepares a geotechnical report for all bridge projects requiring new foundations summarizing the geotechnical information for the site. See Section 3.1.1 for the report contents. The Bridge Division comments on the geotechnical report and works with the Geotechnical Branch to resolve any questions.

2. Foundation Type. The selection of the foundation type is a collaborative effort between the Bridge engineer and Geotechnical engineer based on the geotechnical report, expected superstructure type, scour potential, and other design issues. The Bridge Division provides the Geotechnical Branch with the applicable loads.

3. Structural Design. After the foundation type and basic dimensions are selected, the Bridge Division typically performs the structural design of the foundation.

The bridge designer incorporates the relevant geotechnical information into the bridge design plans. The information from the geotechnical report is transferred to the bridge plans in a form suitable for construction.

Typically, the structural modeling and analysis of the bridge proceeds based on the preliminary structure's foundation type while the project Geotechnical engineer finalizes the subsurface exploration. During this time, the bridge designer coordinates with the project Geotechnical engineer to determine preliminary foundation modeling parameters. The bridge designer determines, verifies
and provides foundation loads (vertical and horizontal) to the project Geotechnical engineer. The bridge designer also provides the elevation at which the foundation loads are applied.

For deep foundations, the Geotechnical Report provides tip elevations, pile resistance, anticipated settlement and p-y soil models of the subsurface soils. The bridge designer performs lateral soil structure interaction analysis using the p-y soil models and uses the information to compute lateral displacements and analyze the structural adequacy of the columns and foundations. The bridge designer uses the lateral soil structure interaction analysis to select the appropriate method (calculated point of fixity, stiffness matrix, linear stiffness springs or p-y nonlinear springs) to model the bridge foundation in the structural design software.

6.1.2.2 Geotechnical Engineer /Hydraulics Engineer Coordination

For bridge crossings over water, the Geotechnical engineer coordinates with the Hydraulics engineer to determine realistic scour depths for foundations. The Geotechnical engineer characterizes the site conditions including the soil type, soil gradation, rock type, depth to rock and competency of bedrock. The Hydraulics engineer evaluates the scour potential based on the information provided by the Geotechnical engineer and estimates potential scour depths. Based on the Hydraulics engineer evaluation, the Geotechnical engineer may present recommendations in the geotechnical report pertaining to scour.

If the Hydraulics engineer determines that the supporting foundation elements are exposed to stream flow from pier and contraction scour, then a redesign of the foundation may be required. When a redesign of the foundation is required, the Bridge Division should resubmit the redesign information (e.g., new foundation layout, sizes, foundation load combinations) to both the Hydraulics engineer and Geotechnical engineer. The Geotechnical engineer analyzes the new foundation and resubmits the necessary geotechnical information to the Bridge engineer. The Hydraulics engineer analyzes the new foundation and confirms that the new design is within acceptable limits of general and contraction scour.

For projects where scour protection is required along the river or stream banks or around in-water foundations, the project Geotechnical engineer works with the Hydraulics engineer to identify the most appropriate scour protection system. The type of system can range from use of rip rap and quarry stone to various types of pre-manufactured scour protective systems. These pre-manufactured systems can include geosynthetic products, flexible concrete mat systems and rock-filled gabions. The project Geotechnical engineer often establishes the bedding and filter requirements for these systems.

6.2 FOUNDATION GEOTECHNICAL DESIGN CONSIDERATIONS

This section provides an overview of general foundation geotechnical design considerations (e.g., foundation selection, geotechnical characterization of the site).

6.2.1 <u>Foundation Type Selection</u>

After reviewing the project site and available geotechnical, the bridge designer will preliminarily select which foundation type is appropriate for that particular location.

6.2.1.1 Drilled Shafts

A drilled shaft is a deep-foundation support element typically constructed by excavating a hole with auger equipment and placing concrete and reinforcing steel in the excavation.

Given the near-surface bedrock common to much of Oklahoma, and given the loading conditions typical of Oklahoma bridges, drilled shafts under piers are generally more economical, better resist lateral loads, and are less susceptible to collapse under scour conditions relative to other foundation systems. Because of this, bridge substructures at piers almost exclusively consist of drilled shafts, whether in water or on land. Drilled shafts are occasionally used under conventional and retaining wall type abutments. The Bridge Division requests a geotechnical field investigation and geotechnical report for the bridge substructures. The bridge designer provides the Geotechnical Branch with the approximate location of the foundations and may provide initial estimates of axial service loads for the superstructure based upon assumed foundation types and sizes. The project Geotechnical engineer uses this information to plan the subsurface investigation.

6.2.1.2 Driven Piles

A pile is a long, slender deep foundation element driven into the ground. Piles are usually used under abutments. Sometimes, piles are used under piers where scour is not a problem, where economically justified, or where drilled shafts cannot be used. H-piles are the most common pile type used on Oklahoma bridges, with HP 10x42 and HP 12x53 being the most common pile sizes.

6.2.1.3 Steel Shell Piles

Steel shell piles are steel tubes infilled with concrete used as deep foundations and columns at bridge piers. The steel tubes are driven into the ground as open-ended pipe piles. Soil inside the core is then removed using an auger prior to placement of concrete. Shell piles are typically 18 to 30 inches in diameter and are beneficial at sites where bedrock is relatively deep and where substantial scour is expected to occur. Shell piles are small enough to be driven by conventional pile diving hammers yet still provide substantial resistance to bending under substantial lateral loading.

6.2.1.4 Spread Footings

Spread footing foundations consist of a reinforced concrete slab bearing directly on the founding stratum. The geometry of the concrete slab is determined by structural requirements and the characteristics of supporting components (e.g., soil or rock). ODOT considers the use of spread of footings for bridge structures at grade crossings, especially at sites where near-surface soils consist of hard clay, intermediate geomaterial, or bedrock.

6.2.2 Tolerable Movements

One of the more difficult tasks of the project Geotechnical engineer is quantifying vertical and lateral deformations of the bridge foundations under the loads developed by the bridge designer. This difficulty is related to the variability of the material in predicting the amount and rate of deformations for foundations and the factors that determine the tolerable movement. These factors can range from the type of bridge superstructure to the type of soil.

Clear guidance on the acceptable vertical and lateral deflections of bridge substructures/ foundations does not currently exist. An acceptable limit for bridge foundation displacement requires engineering judgment based on previous experience, empirical guidelines, and structural analyses.

Tolerable deflections are typically based on structural capacity, load-deflection behavior considerations, rideability, or aesthetics. Designers need to give consideration to seismic aspects as well. There is a distinct difference between loads for the Service Limit State and loads for the Strength or Extreme Event Limit State. LRFD foundation design limits lateral displacements under the Service Limit State but only requires adequate lateral support under the Strength and Extreme Event Limit State. See the *AASHTO LRFD Bridge Design Specifications* (Reference 1) for further discussion of tolerable deformations.

6.2.3 Scour Potential at Streams and Rivers

Scour is localized erosion of the channel bed that occurs around flow obstructions (e.g., piers, bridge abutments), at channel contractions (e.g., bridges), and on the outside of channel bends. It can also be the result of long-term erosion of the channel bed that can occur during the life of a structure.

Scour is a site-specific process that is a function of the flow velocity and duration, geometry of the structural elements exposed to the flow of water, geomorphology of the channel, and properties of the foundation and channel bed materials. A multidisciplinary team of Hydraulic, Geotechnical and Structural engineers should evaluate the risk of scour-induced failure at each structure site.

A scour assessment requires a determination of the cumulative effects of the three main components of scour — aggradation/degradation, contraction scour, and local (or pier) scour. It also requires an evaluation of potential changes in channel geometry and location that may occur during a structure's design life. The amount of scour depends on many factors, including the hydrological characteristics of the site, the hydraulics of the flow, and the properties of the streambed materials.

Use the boring logs to establish the D_{50} values at the streambed surface and to verify depth to bedrock, competency of the bedrock and material conditions. The Hydraulics Division evaluates scour potential based on idealized soil based on the D_{50} of the streambed material.

6.2.4 Geotechnical Subsurface Investigations

Adequate subsurface information is required for foundation design. Lack of this information may lead to construction disputes and claims, overly conservative designs, or unsafe designs. Plan subsurface exploration programs to obtain the maximum possible information at minimum cost. A thorough investigation may result in substantial savings in the cost of a foundation in a particular area. The project Geotechnical engineer must develop a program to obtain an adequate amount of information and data to develop soil parameters for use in design of the foundation system. Soil parameters can be determined from in-situ tests (e.g., SPT, Texas Cone and CPT), laboratory tests, and correlations with index properties.

Chapter 2 "Subsurface Investigations" provides an in-depth discussion on subsurface investigations pertaining to soil testing and measuring soil properties that will apply to bridge foundations. See the following from Chapter 2:

- Subsection 2.3.1 states where borings are located (e.g., spacing) with respect to bridge piers or bents.
- Subsection 2.3.5.2 provides boring and sampling guidelines (e.g., SPT sampling intervals in earth, rock, groundwater).
- Subsection 2.3.9 documents the field-testing methods for the following:
  - Standard penetration test
  - Vane shear test
  - Texas cone penetrometer
  - Cone penetration test

- Pressuremeter test
- Borehole shear test
- Flat plate dilatometer test

- Subsection 2.3.9.10 applies to geophysical surveys.
- Subsection 2.5.2 discusses soil classification tests; ASTM test procedures are used for Bridge Division projects.
- Subsection 2.7.1.7 discusses the requirements for a Detailed Soil Investigation (e.g., foundation soil settlement and stability).

The soil profile should be characterized at each bridge pier or substructure location. Where a drilled shaft foundation is anticipated, it is desirable to leave exploratory borings open for as long as practical to establish whether the hole stays open or caves. This information is useful in helping to determine if temporary casing is required during construction or to what elevation the temporary casing is required.

The following site conditions warrant special consideration during the field exploration phase:

1. <u>Soft and Compressible Soils</u>. If soils are soft and compressible and bedrock is not encountered in the top 60 feet, it is important to collect high quality, relatively undisturbed samples for laboratory evaluations of compressibility and strength. Conduct in-situ tests if undisturbed samples cannot be obtained. This information may be critical for assessing issues including downdrag on piles and shafts, settlement of piles and shafts that are not founded in strong-bearing materials, and differential settlements between bridge supports and approach embankments.

2. <u>Thin Layers of Soft Soil</u>. Thin layers of soft soil can result in downdrag on shafts and driven piles. These layers can also serve as a sliding surface for embankments and slopes. If slope or embankment movement occurs, deep foundations located in the moving soil could be damaged.

3. <u>Groundwater</u>. Evaluate the groundwater conditions in the soil borings during the field investigation. When feasible, install and/or monitor piezometers and/or monitor wells during the various weather and irrigation cycles. As practical, make a determination of all potential groundwater environments beneath the structure (e.g., seasonally high and low groundwater, perched water tables, deep aquifers). Also, evaluate the potential for artesian conditions or cases of excess pore-water pressure because they can reduce the load carrying capacity of the soil and alter the effective stress distribution.

4. <u>Interbedded Layers of Hard Sandstone and Shale</u>. Texas cone penetrometer tests are commonly conducted at 5-ft intervals in the red bed, Permian Age shales of central and western Oklahoma. Interbedded layers of hard sandstone or siltstone that are encountered between Texas cone tests may appear as sand or silt in the boring logs if not accurately recorded or no sample is obtained. This could be confusing to any drilling contractor when reviewing the boring logs and lead to cost overruns or change orders in the field. When encountering this type of geology, it is best practice to obtain an intact sample of the soft rock between Texas cone penetration tests to develop accurate boring logs for bridge pier/pile design and construction. The geotechnical report should include photographs of the intact rock samples obtained.

6.2.4.1 Characterization of Bedrock

Most Oklahoma bridges are supported by piles driven to bedrock at the abutments, and intermediate piers supported by drilled shafts socketed into bedrock. Thus, the characterization of rock is of particular importance for bridge foundations.

6.2.4.2 Shear Strength of Soil

One of the important steps in the characterization process for bridge foundations is the determination of strength properties used for computing the axial and lateral geotechnical resistance of driven piles and drilled shaft foundations. Soil shear strength becomes particularly consequential for bridge foundations at sites where bedrock is deep, resulting in foundations not socketed into rock. LPILE parameters should be provided by the Geotechnical engineer to be utilized by the designer.

Many useful correlations have been established between the engineering properties of soils and various indirect and classification properties. For small projects or preliminary studies, correlations are often used extensively. In other cases, these correlations serve as alternative sources of design information or for checks against laboratory or in-situ tests results.

Generally, the type of strength information depends on whether the foundation is being constructed at a cohesionless soil site, or a site characterized by cohesive soils. Consider the following:

1. <u>Cohesionless Soil Site</u>. When determining the strength of cohesionless soils, it is usually necessary to rely on the results of in-situ testing methods. These methods include SPT and CPT. Other options for in-situ soil property determination include pressuremeter and dilatometer. Empirical correlations are usually used for estimating friction angle of cohesionless soils from in-situ soil measurements (e.g., SPT blow counts, CPT sounding results). Alternatively, methods are available for estimating nominal resistance of driven piles using SPT blow counts and CPT sounding data directly. These direct methods can provide a relatively accurate estimate of resistance, if the soils at the site are consistent with the database used to develop the empirical correlations.

2. <u>Cohesive Soil Site</u>. Either of two methods can be used to determine shear strength at cohesive soil sites. One involves collecting undisturbed samples in the field and then conducting triaxial laboratory tests. Typically, this method is used to obtain both the drained and undrained strength parameters for the soil. The alternative method involves the use of in-situ strength measurements. This approach involves estimating the undrained shear strength of the soil from CPT or Dilatometer soundings or using in-situ vane shear measurements. Relationships have been developed for estimating pile resistance from CPT sounding results. Note that the use of the SPT blow count to estimate undrained shear strength or indirect estimates of pile resistance is subject to large uncertainties and should not be used. Include the use of the Dilatometer.

Determination of shear strength parameters should involve consideration of the type of loading relative to the permeability of the soil. Drained strength parameters are appropriate if the soil drains quickly (e.g., cohesionless soils). Although drained strength parameters can be obtained by laboratory testing, it is very difficult to obtain undisturbed cohesionless soil samples, forcing the laboratory tests to be conducted on reconstituted samples. The reconstitution process introduces enough uncertainties that the preferred approach is to use indirect correlations between in-situ measurements and soil friction angle.

For cohesive soils, the strength under undrained loading is less than the drained strength, particularly if consolidation to a higher stress state occurs. In this case, the strength used in estimating the axial resistance of piles is the undrained strength consistent with the stress state immediately after the load is applied. This long-term drained strength can be estimated based on effective stress parameters, while the short-term strength is determined from total stress parameters. Good practice is to check both cases (undrained and drained) to determine which one controls the design.

6.3 DRILLED SHAFTS

6.3.1 Design Requirements

Drilled shafts are designed for both axial and lateral loading conditions. The two principal design considerations for drilled shafts cast in soil under axial loads are the nominal resistance and settlement. For drilled shafts socketed in bedrock, any settlement is assumed to be negligible. The nominal (strength limit state) resistance of a drilled shaft may be governed by either the structural capacity of the drilled shaft or the geotechnical capacity of the soil. Drilled shafts that are subjected to lateral loads must also be safe against shear failure of the soil or the concrete shaft and excessive lateral deflection.

The project Geotechnical engineer provides design recommendations to the Bridge Division for drilled shaft foundations, which includes the shaft resistance, settlement, lateral response and constructability. The geotechnical report will include the following information:

1. <u>Axial Nominal Geotechnical Resistance</u>. Axial nominal geotechnical resistance in both compression and uplift, where appropriate, for the given design embedment length. Also, include the appropriate resistance factors for the three limit states.

2. <u>Settlement Estimates</u>. Settlement estimates are to be provided if the drilled shafts are founded in rock or extremely dense soils. These settlement estimates should include both the immediate settlement and any long-term consolidation settlement.

3. <u>Lateral Response</u>. The lateral response analyses should show the displacements, moments and shears as a function of depth and for a range of loads that could be imposed on the head of the shaft. The fixity at the head (provided by Bridge Division) of the shaft should be considered in these evaluations.

4. <u>Installation Recommendations</u>. Recommendations are to be provided on shaft installation, including use of temporary or permanent casing. Also, a statement should be provided addressing difficulty of drilling during excavation of bedrock for drilled shafts.

5. <u>Depth</u>. The design tip elevation of the drilled shaft is to be provided in the report.

6.3.2 Site Characterization for Drilled Shaft Foundations

The following site characterization requirements for drilled shaft foundations should be considered or evaluated during the investigation:

1. <u>Cemented Sands</u>. Cemented sands are common in northwestern Oklahoma, particularly within the Ogalala geologic unit. This material can be mischaracterized as weak sandstone if care is not taken to properly characterize this material. If cemented sand is mistakenly characterized as sandstone, confusion may result during construction when material excavated is not identified as sandstone.

2. Cobbles and Boulders. Oklahoma is overall devoid of cobbles and boulders except in very few cases. In these very rare cases, normal practice is to conduct a geotechnical exploration at the center of each shaft location.

3. Gravel. Identify the presence of open gravel layers because these materials may require the use of casing or special drilling muds to avoid hole collapse or excessive loss in drilling muds during construction.

4. Explorations. Explorations should extend at least 30 ft or five shaft diameters, whichever is greater, below the tip of the shaft. If hard bearing material or rock is located less than 30 ft, the depth of exploration can halt 10 ft into the hard bearing material.

5. Rock Sockets. Most drilled shafts for Oklahoma bridges are founded in bedrock. For rock socketed drilled shafts, the exploration should extend at least two shaft diameters below the planned toe elevation of the shaft.

6. Groundwater. As part of the site characterization effort, it is important to establish the location of the groundwater table and whether groundwater is perched or involves artesian conditions. These conditions have an important effect on the drilling methods selected by the shaft construction contractor.

6.3.3 Construction of Drilled Shafts

Details of the construction procedures are critical regarding the performance of the drilled shafts. Therefore, construction methods must be carefully controlled for the foundation to function as designed. Different subsurface conditions warrant different methods of construction. Detailed descriptions of the methods (e.g., dry, casing, wet), and examples of possible construction problems, are described in FHWA *Drilled Shafts: Construction Procedures and LRFD Design Methods* (Reference 2). The project Geotechnical engineer needs to consider the likely method of drilled shaft construction when performing the site investigation.

6.3.3.1 Clay Sites

For installing a drilled shaft into clay, if the clay is homogeneous so that the excavation will remain open and dry, the clay will creep toward the axis of the excavation accompanied by vertical subsidence of the ground surface. The creep and subsidence will be substantial if the clay is weak, but minimal for stronger over consolidated clays. Disturbance and stress relief due to drilling will cause some loss of shear strength along surface of the borehole, which must be addressed during design.

The placement of fluid concrete in the excavation will impose a lateral stress on the sides of the excavation, the magnitude of which is dependent on the fluidity and rate of placement of the concrete. If the excavation is drilled dry, moisture from the fluid concrete can migrate into the clay and cause some additional softening. This problem can be important in concrete that is mixed with a high water-cement ratio in which much more water than is needed to hydrate the cement is used in batching. Whether the excavation in the clay is wet or dry, there is evidence to show that there is an interaction between the clay and particles of cement and/or products of cement hydration, with a consequent strengthening of the bond between the concrete and the clay. This interaction results in a larger strength at the interface than the softened strength that exists just after the concrete placement.

6.3.3.2 Sand Sites

As with clay, the properties of sand around a drilled shaft can be very different from the in-situ properties. Design the subsurface investigation to reveal as practical the in-situ characteristics of the sands, especially its density and grain-size distribution. The parameters selected for the design of a drilled shaft in sand will then be adjusted by the design method according to the best estimate of the properties of the sand that exist around the drilled shaft as built.

If the sand in a drilled-shaft excavation is prevented from collapsing by driving a casing into place, the behavior of the sand around the perimeter (shaft) of the casing is similar to that of a driven pile. The sand heaves at the base of the excavation resulting in lower unit end bearing than for a driven pile. The end-bearing load-deformation behavior may be adversely affected by construction practices that fail to remove cuttings that have been suspended in drilling slurries during borehole excavation. The limit on the amount of loose or disturbed material in the bottom of the shaft is 1 inch after cleaning. It is imperative that the drilled shaft contractor continually sample and test the drill slurry. The contractor should modify it as necessary by exchanging and cleaning it during drilling activities. The contractor should also re-clean the base of the borehole prior to placement of the concrete.

The placing of concrete with high workability (cohesive mixes with high slump) will impose stresses against the sides and base of the excavation that are larger than those from the slurry. The fluid concrete could then cause a slight densification of the sand adjacent to the wall and base of the drilled shaft. Concrete with a low slump will bulk and not collapse under its own weight. In addition to producing potential defects (e.g., honeycomb, voids) in the concrete, this effect causes the lateral stress against the sides of the excavation to be less than would occur had the concrete been fluid. The resistance along the sides is to some extent dependent on this concrete pressure. Low-slump concrete can also have a negative effect on geomaterial resistance.

6.3.3.3 Rock Sites

The requirement to bear on or penetrate rock strata often dictates the use of drilled shaft foundations. One of the important considerations of rock-socketed drilled shafts is the condition of the side of the borehole. High values of side shearing resistance can develop because of dilation that occurs between a rough surface at the boundary of the concrete and the mating surface in the rock. Upward or downward movement of the concrete shaft caused by applying axial loads produces lateral compression of the rock and, as a result, higher lateral stresses along the concrete-rock interface than existed after the concrete was placed. The increased lateral stresses can in turn increase the strength of the rock if pore pressures dissipate rapidly. Either the rock or concrete finally fails by some manner of shearing through the respective asperities at a high value of resistance.

Construction practices that cause the concrete-rock interface to be smooth, rather than rough, can have a profoundly negative effect on the side shearing resistance that develops in rock sockets. For example, in argillaceous (clay based) rock (e.g., shale, mudstone, slate), the presence of free water in the borehole during drilling (e.g., minor inflow of water from a small perched aquifer near the surface, intentional introduction of water by the contractor to aid in excavating cuttings) can cause the surface of the rock to become fully softened or smeared, so that any effect of borehole roughness is almost completely masked. The rough interface with degraded (smeared) rock behaves very similarly to the smooth interface, and the behavior of a drilled shaft with a smeared interface is closer to the behavior of a drilled shaft in a mass of soil that has properties of the degraded rock, rather than one with a rough interface in the original rock. ODOT allows the use of temporary, permanent, and double casing methods of drilled shaft construction, dependent upon site conditions and geology. When permanent casings are utilized, the designer can ignore interface shear in the soil and rock.

6.3.4 <u>Geotechnical Design Considerations</u>

6.3.4.1 Movement

Excessive movements of foundations supporting bridges may lead to discontinuities in the riding surface, damage to the bridge superstructure or substructure, jamming of bearings and expansion joints, aesthetic concerns, or even collapse. It is necessary in bridge design to estimate the maximum settlement and lateral movement anticipated in the foundations and to ensure the deformations fall within tolerable limits. Acceptable lateral deflections generally need to be evaluated on a case-by-case basis in consultation with the bridge designer because acceptable deflections are based upon numerous factors, including loading type (e.g., service, extreme event), type of superstructure, bridge location.

6.3.4.2 Geotechnical Drilled Shaft Resistance

The axial geotechnical resistance of a drilled shaft is the sum of its tip and shaft capacities. During failure, the shear stress at the interface of the drilled shaft and soil reaches a limiting value. This can occur under either compressive or tensile loads.

Drilled shafts in saturated clays are usually designed using total stress analyses where the undrained shear strength of the clay is used. Long-term loads lead to an increase in the shear strength of the clay around the shaft as the clay consolidates with time. Associated with this consolidation will be some settlement of the foundation. There is a possibility that negative pore pressures can develop along the sides of drilled shafts in heavily over consolidated clay or shale, leading to soil softening over time. As a result, total stress methods of analyzing drilled shafts in heavily over consolidated clay or shale may be unconservative. In this case, *FHWA Drilled Shafts: Construction Procedures and LRFD Design Methods* suggests using the undrained shear strength measured in a triaxial or direct test that represents residual strength parameters.

6.3.4.3 Resistance Factors

Resistance factors for side friction and end bearing are available in the *AASHTO LRFD Bridge Design Specifications*. An important task in the design of drilled shafts involves the choice of an appropriate resistance factor for each design mode (i.e., axial or lateral capacity). Specifications are explicit, with resistance factors that vary with method of analysis, geology and form of loading. If load tests are conducted, the resistance factor also changes. Review the *AASHTO LRFD Specifications* for a discussion of these alternatives.

While a minimum of one boring per substructure unit is all that is required, uncertainty can be reduced by performing a boring at the location of every drilled shaft on the project. These situations should be extremely rare.

6.3.4.4 Lateral Loading

The geotechnical nominal resistance is usually not a controlling factor in the design of drilled shafts to resist lateral loads. The governing criterion in lateral load design is usually either maximum tolerable deflection or structural nominal resistance. The lateral deflection of single drilled shafts and groups of drilled shafts may be estimated using the procedures described in NCHRP Report Number 343 *Manuals for the Design of Bridge Foundations* (Reference 3) or by conducting a p-y analysis.

6.3.4.5 Settlement

Evaluate settlement of a drilled shaft foundation as a single shaft or as a group, whichever is applicable. Estimate the settlement considering:

- Short-term (elastic compression) settlement
- Consolidation settlement, if constructed in cohesive soils
- Axial compression of the shaft

Dimensionless load-settlement curves for drilled shafts in cohesive and cohesionless soils are presented in the *AASHTO LRFD Bridge Design Specifications* for both side resistance and end bearing conditions. These curves include elastic shortening of the shaft. They represent the immediate response of the shaft to load. Add consolidation settlement to settlement estimated from the load-settlement curves if the shaft is founded in a compressible clay layer.

Address the group settlement of drilled shafts using the same approach for driven piles; see the *AASHTO LRFD Specifications*.

6.4 DRIVEN PILES

6.4.1 Pile Type Selection

The selection of a pile foundation type for a structure should be based on the specific soil conditions, the foundation loading requirements and final performance criteria. A summary of common pile types is provided in the FHWA *Design and Construction of Driven Pile Foundations* (Reference 4). Foundation piles can also be classified in terms of their method of load transfer from the pile to the surrounding soil mass. Load transfer can be by side (or shaft) resistance, tip bearing resistance, or a combination of both.

6.4.2 Design Requirements

The project Geotechnical engineer provides design recommendations to the Bridge Division for driven pile foundations. The design recommendations include the pile capacity and size, estimated settlement, lateral response to design loads, and constructability of the pile.

The following specific information is provided to the Bridge Division:

- Settlement estimates
- Electronic files from the lateral response analyses
- Recommendations on pile installation, including pile drivability
- Design tip elevation and required capacity during driving

The axial geotechnical nominal resistance in both compression and uplift (where appropriate) at the design tip elevation summarizes the appropriate resistance factors for the three limit states and whether pile driving analyzer tests are required during construction.

6.4.3 Geotechnical Design Considerations

Analysis of pile geotechnical nominal resistance for most practical situations is based on empirical equations, experience, and judgment. Consequently, a host of analytical formulations exist in the literature. Methods of evaluating pile-load nominal resistance represent approximations, because it is difficult to fully account for the variability of soil types and the differences in the quality of construction and pile installation. In this section, the analytical methods described in the referenced publications are recommended. Alternative methods are not necessarily discouraged and, in fact, may be applicable in some situations. For example, for special structures and loading conditions, computer programs based on finite element methods are commonly used. To obtain an estimate of the pile-load nominal resistance, recourse is made to engineering mechanics, experience, measured observations, and to correlations using laboratory and field test results.

6.4.3.1 Single-Pile Axial Geotechnical Nominal Resistance

Static analysis methods are typically used to evaluate pile axial geotechnical nominal resistance during the design phase of a project. These analytical methods use soil strength and compressibility properties to determine pile resistance and performance. Guidelines for conducting static capacity analyses of driven pile foundations are provided in the *AASHTO LRFD Bridge Design Specifications*. These guidelines include semi-empirical methods that use estimated or measured soil properties, and methods that directly apply in-situ test results (SPT or CPT measurements). Consider the following:

- Semi-empirical formulations include the use of either total stress (i.e., σ-method) or effective stress (i.e., β-method) methods for determining shaft and tip resistances. It is useful to determine these resistances versus pile depth.

- In-situ tests are widely used in cohesionless soils because obtaining good quality samples of cohesionless soils is difficult. Two frequently used in-situ test methods for predicting pile resistance are the SPT method and the CPT method. These methods are described in most geotechnical textbooks, manuals and design guides.

- A number of methods in addition to those given in the *AASHTO LRFD Bridge Design Specifications* are available for estimating pile resistances and may be considered for use on a case-by-case basis.

6.4.3.2 Pile Group Axial Capacity

The axial resistance of a pile group in cohesionless soil is taken as the sum of the resistances of all piles in the group. The group efficiency factor, η, is 1.0 in this case, regardless of whether the pile cap is or is not in contact with the ground.

The efficiency of pile groups in cohesive soil may be diminished from that of the individual pile due to overlapping zones of shear deformation that occur in the soil surrounding the piles. In cohesive soils, the resistance of a pile group depends on whether the cap is in firm contact with the ground beneath. Consider the following:

- If the cap is in firm contact, the soil between the piles and the pile group behaves as a unit. In this case, a block type failure mechanism should be evaluated in addition to evaluating the sum of the individual resistances of each pile in the group.

- If the cap is not in firm contact with the ground and if the soil at the surface is soft, the individual resistance of each pile should be multiplied by an efficiency factor, η, taken as:

  - $\eta = 0.65$ for a center-to-center spacing of 2.5 diameters

  - $\eta = 1.0$ for a center-to-center spacing of 6.0 diameters

- For intermediate spacings, the value of η may be determined by linear interpolation

For cohesive soil sites, soil settlement usually occurs after pile installation as excess pore-water from the pile installation dissipates and as secondary compression within the soil occurs. This settlement often results in separation of the soil from the base of the cap such that the cap is no longer in firm contact with the soil. In view of the potential for settlement, normal design practice is to check group capacity assuming the cap is in contact for initial loading conditions and the cap is not in firm contact with the soil. If the capacity with settlement is lower than the capacity for full contact, the lower of the two capacities is used for conservatism.

6.4.3.3 Settlement

Most settlement analyses for driven pile foundations are based on empirical methods and provide only an approximation of the actual settlement because of the complex load-transfer mechanism that occurs with axially loaded deep foundations. Nonetheless, settlement of single piles and pile groups should be calculated and compared to the performance objectives established for the structure to confirm that calculated settlements are within acceptable limits.

Driven piles are not often used as a single or individual foundation element to support a structure; consequently, settlement analyses of single piles are not commonly conducted. If settlement estimates for a single pile are necessary, the USACE *Design of Pile Foundations Manual* (Reference 5) describes elasticity-based methods and the t-z curve method for calculating settlement of single piles. Consider the potential for consolidation-related settlements when making this evaluation.

An empirical approach known as the equivalent footing method is typically used to calculate the settlement of a group of piles. The pile group is treated as an equivalent footing that is founded at an effective depth below the ground surface. For uniform clays sites, the effective depth is two-thirds of the pile embedment in the bearing stratum. For sand sites, the effective depth depends on the soil conditions below the toe of the pile group. After the effective footing depth is defined, procedures for shallow foundation settlement are then applied to the equivalent footing to determine settlement as described in the *AASHTO LRFD Bridge Design Specifications*. Note that the width of the equivalent footing given in the *AASHTO LRFD Specifications* changes according to the soil profile.

An alternative approach to the evaluation of settlements for pile groups is to determine the neutral plane for the pile group and to perform the settlement analysis at this location. The neutral plane is determined on the basis of the resistance along the side and at the toe of the pile, as described in the *J. E. Bowles, Foundation Analysis and Design* (Reference 6). This approach can be particularly useful where non-uniform soil layering occurs. The CFEM refers to this method as the unified method. Capacity and settlement are evaluated together with this approach. The UNIPILE computer program uses this model to evaluate pile capacity and settlement for pile groups.

6.4.3.4 Downdrag Loads

The current *AASHTO LRFD Bridge Design Specifications* require that the foundation be designed so that the available factored geotechnical resistance is greater than the factored loads applied to the pile, including drag loads (downdrag) at the Strength Limit State. The nominal pile resistance available to support structure loads plus downdrag is estimated by considering only the positive skin and tip resistance below the lowest layer contributing to the downdrag. The structure should also be designed to meet settlement limits resulting from downdrag and the applied loads in accordance with the AASHTO *LRFD Bridge Design Specifications* and the structural limits resulting from the combination of downdrag plus structure loads.

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

$$R_n = (\Sigma\gamma_i Q_i)/\varphi_{dyn} + \gamma_p DD/\varphi_{dyn}$$ Equation 7.4 (Reference 1)

Where:

R_n = *nominal bearing resistance of the pile needed to resist the factored loads*

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

Q_p = *($\Sigma\gamma_i Q_i$) = factored load per pile, excluding downdrag load*

DD = *downdrag load per pile*

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

φ_{dyn} = *resistance factor, assuming that a dynamic method is used to estimate pile resistance during installation of the pile (if a static analysis method is used instead, use φ_{stat})*

γ_p = *load factor for downdrag*

The total driving resistance, R_{ndr}, needed to obtain R_n, can be computed using the following equation, which accounts for the skin friction that must be overcome during pile driving that does not contribute to the design resistance of the pile:

$$R_{ndr} = R_{Sdd} + R_n$$ Equation 6.4 (Reference 1)

Where:

R_{ndr} = *nominal pile driving resistance required. Note that R_{Sdd} remains unfactored in the analysis to determine R_{ndr}.*

For projects where downdrag loads have a significant effect on the pile-length determination, the project Geotechnical engineer may want to consider alternatives to the approach given in the *AASHTO LFRD Bridge Design Specifications*. One option is to conduct a load-displacement analysis using a computer program to evaluate the potential effects of downdrag on the pile using a displacement-based approach rather than the limit equilibrium method. Computer programs (e.g., APILE) can be used for this evaluation.

6.4.3.5 Piles on Rock

When a rock mass is relatively close to the ground surface and lateral loads are large, it may be necessary to drive steel pipe piles or H-piles into the surface of the rock or to bore the piles some distance into the rock mass. The term “pilot hole” applies to a boring through soil overburden and then drilled into underlying rock a distance of 1 or 2 pile diameters.

For pile foundations that are driven to rock, the exact area of contact with rock, the depth of penetration into rock, and the quality of rock are largely unknown. Therefore, the determination of load capacity of driven piles on rock should be made based on driving observations, local experience, and load tests. The allowable bearing pressure for design of piles on rock are governed by the rock strength and settlements associated with defects in the rock. For tight joints or joints smaller than a fraction of an inch, rock compressibility is reflected by the rock quality designation (RQD).

Structural resistance typically governs the axial resistance of piles socketed into competent rock. Nevertheless, it is important to check the geotechnical resistance. As a check on the axial resistance of piles driven into rock, the *AASHTO LRFD Bridge Design Specifications* provide an approximate empirical method that reflects the spacing and width of discontinuities within the rock mass and the size and depth of the rock socket. Allowable bearing resistance on unweathered rock is normally based on the strength of intact rock and on the influence of joints and shear zones.

6.4.3.6 Piles in Intermediate Geomaterials

Deposits of intermediate geomaterials (IGM) exist in many locations in Oklahoma and are commonplace in some areas. These materials exhibit a great variety of physical properties. Consequently, early recognition of the presence of IGM and the need for a pile foundation solution are essential for the planning and execution of an effective site investigation and foundation design.

Various terms have been used to describe or classify an IGM, including weak rock, indurated soil, soft rock, and formation material. IGM are intrinsically weak (i.e., they have undergone a limited amount of gravitational compaction and cementation). They are products of the disintegration, weathering, and alteration of previously stronger rocks. Classification of IGM based on material properties is not standardized; however, a common definition is that IGM will have a uniaxial compressive strength in the range of 12.5 ksf to 260 ksf and a stiffness modulus in the range of 2100 ksf to 21,000 ksf.

The following types of IGM have been encountered in Oklahoma:

- Weak shale
- Weak sandstone
- Mudstone, claystone and siltstone
- Very dense sandy gravel
- Caliche

6.4.3.7 Uplift

Where piles are subjected to uplift forces or a moment that results in a net tensile force, investigate the pile group for resistance to pullout, structural ability to resist tension forces, and structural ability to transmit tension forces from the piles to the pile cap or footing. In some cases, uplift resistance determines the minimum pile penetration requirements.

Uplift resistance is typically determined for the pile group as if it acts as a foundation unit, as described in the *AASHTO LRFD Bridge Design Specifications*. In these references, the resistance of a single pile is first determined. The uplift resistance of the group is simply the sum of the uplift resistance of the individual piles. In fine-grained cohesive soils, where loading is assumed to occur under undrained conditions, the single pile shaft resistance is generally considered equal in compression and in uplift. However, in cohesionless or free-draining soils, the relationship between compression and uplift resistance is not as clear. Studies described in the literature indicate the uplift shaft resistance in cohesionless soil may vary from 70% to 100% of the compression shaft resistance.

The AASHTO *LRFD Bridge Design Specifications* provide resistance factors for axial tension, which are lower than those for compression. The reason for the lower resistance factor is that once a pile begins to fail in uplift, the resistance progressively decreases with movement. This behavior is in contrast to most piles loaded in compression where an increase in resistance with movement eventually occurs.

6.4.3.8 Scour

Scour occurs as a result of flowing water eroding away material from the streambed and streambanks. Scour around bridge foundations can create a severe safety hazard. Therefore, design bridge foundations to survive the effects of possible scour.

Perform geotechnical analyses of bridge foundations assuming that the soil above the estimated scour line has been removed and is not available to provide bearing or lateral support. Scour is classified as follows:

1. <u>Local (or Pier) Scour</u>. Local scour affects materials only in the immediate vicinity of a substructure unit. Soil resistance in the scour zone provides resistance at the time of driving that cannot be counted on for long-term support. Consequently, for determining long-term axial capacity, ignore the shaft resistance in the scour zone. However, for conducting drivability analyses, use the full shaft resistance. For pile capacity calculations in local scour cases, only consider the reduction in soil resistance in the scour zone, and the effective overburden pressure is unchanged.

2. <u>Channel Degradation Scour</u>. Channel degradation/contraction scour is where streambed materials are removed over a large area. Soil resistance in the scour zone provides resistance at the time of driving that cannot be counted on for long-term support. Therefore, like the local scour condition, ignore the shaft resistance in the scour zone for long-term pile support considerations, but not for drivability considerations. In contrast to local scour, pile capacity calculations in channel degradation scour cases should also include a reduction of effective overburden pressure due to removal of the streambed materials. This reduction in effective stresses can have a significant effect on the calculated shaft and toe resistances.

These two scour types need to be added together to determine the axial and lateral capacity. Scour is usually evaluated for the 100-year flood. The FHWA recommends that:

- The top of the pile cap should be located below the depth of channel contraction scour to reduce obstruction to flow and to minimize local scour.

- A few long piles should be used rather than many short piles. This results in higher safety against pile failure due to scour.

6.4.3.9 Dynamic Pile Analysis and Testing

Where piles are installed using impact driving methods, evaluate the drivability of the pile foundation design by conducting a dynamic driving analysis. The dynamic analysis is used to:

- Confirm that the design pile section can be installed to the desired depth and ultimate capacity with reasonable size hammers.
- Develop capacity versus blowcount relationships.
- Evaluate compressive and tension stresses during driving to confirm that they are within the allowable driving stresses specified in the *AASHTO LRFD Bridge Design Specifications*.

In projects involving driven pile foundations, conduct the wave equation analyses using computer software. The wave equation analysis is usually conducted at two different phases of the project:

- During design using an assumed pile hammer and system
- At the onset of construction using the actual pile hammer system as submitted by the contractor

Although a wave equation analysis is required, the Gates formula is sometimes used to check wave equation analysis results. The Gates dynamic formula and detailed information on the wave equation analysis are described in *FHWA Design and Construction of Driven Pile Foundations*.

Dynamic monitoring of force and acceleration at the pile head during pile installation may be considered on any project involving driven piles to verify geotechnical capacity, both after driving and later (restrike) to evaluate setup or relaxation. The dynamic monitoring is accomplished with dynamic testing. Results of the dynamic testing include stresses in the pile and an estimate of pile capacity. Dynamic monitoring using signal matching and wave equation analyses (e.g., CAPWAP) may be necessary for piles installed in difficult subsurface conditions, in soils with obstructions or boulders, in weak rock, where piles bear on steeply sloping bedrock surfaces, or other conditions in which uncertainties in the underlying strata exist. Dynamic monitoring may also be necessary on complex projects, projects involving large numbers of piles, or where the structural loads (axial and/or lateral) are relatively high.

6.4.3.10 Lateral Loading

Pile foundations are subjected to horizontal loads due to wind, traffic, bridge curvature, ice jams, vessel impact, and earthquake. Evaluate the nominal resistance of pile foundations to horizontal loads based on both soil/rock properties and pile structural properties. Soil-structure interaction is a vital consideration in lateral pile analyses.

Design of laterally loaded piles should include evaluation of both the pile structural response and soil deformation to lateral loads. Determine the factor of safety against ultimate soil failure. In addition, calculate the pile deformation under the design loading conditions and compare it to the foundation performance criteria. Acceptable lateral deflections generally need to be evaluated on a case-by-case basis in consultation with the bridge designer. Acceptable deflections are based upon numerous factors, some of which include loading type (e.g., service, extreme), type of superstructure, bridge location, etc.

The following two methods should both be used to perform lateral load analyses on piles:

1. <u>Brom's Manual Computation Method</u>. This method is a limit-equilibrium analysis that is used to compute the factor of safety against soil failure. Brom's method provides an estimate of the ultimate lateral load and pile deflections at the ground surface. The *FHWA Design and Construction of Driven Pile Foundations* provides a detailed procedure for conducting Brom's method of analysis.

2. <u>P-Y Method.</u> In this method, P is the soil resistance per unit pile length and Y is the lateral soil or pile deflection. Soil properties have the largest influence on the shape of the p-y curves. However, the p-y curves also depend upon depth, soil stress-strain relationships, pile width, water table location, and loading condition (static or cyclic). Procedures for constructing p-y curves and for conducting lateral pile analyses by numerically solving the beam-on-elastic-foundation equation are well established.

The Geotechnical Branch and Bridge Division use the p-y approach when evaluating the lateral response of driven piles. The analyses are conducted to estimate the deflection, moment and shear as a function of load at the head of the pile and the depth below the ground surface. In this analysis, loads are not proportional to displacements and the law of superposition cannot be used. The recommended technique is to use upper bound and lower bound soil values to check limit states. Upper bound values control for shear in the foundation element, while lower bound values control for deflection and moment.

During the p-y analyses, the following additional factors must be addressed:

1. Fixity at the Head. The fixity at the head of the pile can range from fixed to free. Usually, a fixed-head model is used for smaller diameter piles; however, as loads and pile sizes increase, some rotation at the pile head can occur. This response changes the load-deflection response of the pile. The Bridge Division should provide input on the pile-head fixity and, if this is not provided, the project Geotechnical engineer should request this information from the Bridge Division. If there is uncertainty in the fixity, one option is to provide results for fixed- and free-head conditions.

2. Sloping Ground. Many abutment piles are close to the approach fill slope. This can modify the p-y curves in the direction of the slope. Various methods exist for estimating p-y curves on sloping ground. Because the lateral loads are often the result of seismic event, one common approach is to assume that the p-y curve for level ground is appropriate, based on the assumption that the cyclic response in the longitudinal direction balances between in-slope and out-of-slope response.

3. Repeated Cycles. Repeated cycles of load can affect the lateral response of a pile. The cyclic option in commercially available software was introduced to account for degradation in soil resistance that occurs with thousands of cycles of wave loading to an offshore platform. This cyclic option is not applicable for the limited number of cycles associated with earthquake loading. Normal practice is to use the p-y curves without modification for cycle effects when evaluating seismic response.

4. Pile Stiffness. Consider the stiffness of the pile and how this stiffness changes with load. This issue is particularly important for concrete piles and drilled shafts, where section modulus of the pile or shaft can change from uncracked to cracked during loading, resulting in a change to the predicted deformations, moments, and shears. If steel piles are used, this issue is not critical. However, projects that involve concrete piles, bored piles or drilled shafts, evaluate the effects of modulus change. The Bridge Division can provide direction on this issue.

Another important consideration during the lateral loading of a pile group is the potential contributions of the pile cap. If the pile cap is always embedded, the p-y horizontal resistance of the soil on the cap face may be included as part of the overall lateral resistance of the foundation system. The passive pressure of the soil in front of the cap limits the horizontal resistance at the face of the cap. Use the *AASHTO LRFD Bridge Design Specifications* to estimate the passive pressure and include the wall friction in the friction estimate. The amount of displacement to mobilize this resistance ranges from 0.02 to 0.1 times the cap thickness. A good rule-of-thumb is to assume that the displacement to mobilize passive resistance is 0.05 times the cap thickness for well-compacted granular soils and 0.1 times the thickness for cohesive soils. The contribution from shear along the side of the cap also contributes to the resisting force; however, this resistance is usually small relative to the passive pressure contribution. Base shear should be neglected from the resistance calculation on the basis that the soil usually settles away from the cap.

When analyzing the total from a group of piles and the pile cap, consider the amount of displacement to mobilize the reaction from the cap and the piles. This evaluation often means developing a force-displacement curve for the cap and the pile group and then developing a model that results in compatibility of displacements.

6.4.3.11 Pile Group Lateral Capacity

Multiple rows of piles have less resistance than the sum of the single individual piles because of pile-soil-pile interactions that take place in the pile group. Consequently, piles in pile groups can have less resistance to lateral load than piles in the lead row. The pile cap results in equal displacement of all piles in the cap and, therefore, the pile-soil-pile interaction (also called shadowing effect) results in the lateral capacity of a pile group being less than the sum of the lateral capacities of the individual piles comprising the group. Consequently, laterally loaded pile groups may have group efficiencies less than 1, depending on the spacing of the piles.

When the p-y method of analysis is used to evaluate a laterally loaded pile group, reduce the values of p by a multiplier (p_m), which results in softened (less stiff) soil response curves for the piles. Suggested multipliers are provided in Figure 6-1, which are based on the center-to-center pile spacing (D) and the row number in the direction of loading. An exception to these p_m values occurs when a single row of piles is loaded in a direction that is perpendicular to the row. In this case, the group reduction factor is 1.0 if the pile spacing is 5D or greater. The group reduction factor is 0.7 for a pile spacing of 3D.

Pile Center-to-Center Spacing (in the direction of loading)	Pile Load Modifiers, p_m Row 1	Pile Load Modifiers, p_m Row 2	Pile Load Modifiers, p_m Row 3 and Higher
3D	0.7	0.5	0.35
5D	1.0	0.85	0.7

Figure 6-1: PILE LOAD MODIFIERS, PM, FOR MULTIPLE ROW SHADING

6.5 REFERENCES

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Chapter 7

GROUNDWATER ANALYSIS AND DRAINAGE SYSTEM

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7 GROUNDWATER ANALYSIS AND DRAINAGE SYSTEMS

7.1 GENERAL CONSIDERATIONS

Groundwater is the water present beneath Earth's surface in rock and soil pore spaces and in the fractures of rock formations. Control of groundwater is an important factor in erosion and sediment control, preventing contaminant migration, or keeping groundwater from entering excavations or below grade structures. Poorly planned control of groundwater can lead to time delays and cost increases during construction and can lead to operational or maintenance problems during the lifetime of a structure.

7.1.1 Groundwater in Fill and Cut Sections

Groundwater conditions often contribute to slope failures in natural or cut slopes. Several factors determine or influence groundwater conditions in slopes including infiltration (precipitation or irrigation), confined (artesian) conditions, perched water layers, or simply the phreatic groundwater surface. The groundwater level will vary with time of year depending on climatic changes, heavy rainfall, irrigation, and changes in river elevation. These changes in groundwater pressures influence the effective stresses, which will affect the stability of a slope. Consequently, a key step in the stability analysis involves identification of both the current location of the groundwater and potential fluctuations of groundwater. Often this will require installing piezometers at the site and recording groundwater elevation changes with time.

The project Geotechnical engineer should consider the following items when assigning groundwater conditions at a site during slope stability analyses:

1. Hydrostatic and Steady-State Flow Conditions. With Hydrostatic flow, the fluid velocity is zero, as opposed to Steady-State flow, where the fluid is in motion at a steady rate. Use the maximum anticipated groundwater elevation for design. If, however, artesian or perched groundwater conditions occur, define the groundwater elevation to give the correct effective stress soil conditions and total stress loading conditions within the limits of the analyses.

2. Varying Groundwater Elevations. If transient flow or rapid decreases in water elevation could occur (e.g., adjacent to a river), it may be necessary to examine a rapid drawdown case where the groundwater has a significant gradient toward from the river. In this case, methods used to evaluate transient stress states would be appropriate.

The presence of groundwater in cut sections can require the use of additional drainage features (e.g., drains may be installed on the outside edge of the roadway to intercept water inflow). Pavement underdrains may also be necessary to dissolve porewater pressure and redirect groundwater from underneath the roadway. Seepage forces inside slopes can reduce slope stability.

The project Geotechnical engineer should determine the groundwater level (piezometric level) for the most probable and worse-case conditions. Consider the potential for variations in groundwater levels, artesian effects, perched water, potential for rapid drawdown and the effects of irrigation.

Groundwater conditions often play a role in slope failures, which occur due to an imbalance of driving forces within the slope and resistive forces within the soil. Slope instability may be caused by one or more of the following factors:

- A change in slope profile that adds driving weight at the top or decreases resisting force at the base. Examples include steepening of the slope or undercutting of the toe.

- An increase of groundwater pressure, resulting in a decrease of frictional resistance in cohesionless soil or swell in cohesive material. Groundwater pressures may increase through the saturation of a slope from rainfall or snowmelt, seepage from an artificial source, or rise of the water table.

- Groundwater that occurs along a discontinuity or within a slope mass can reduce the stability of the rock slope, either by introducing an uplift pressure at the discontinuity or by lowering the effective stresses in broken rock.

7.1.2 Other Concerns

Some other typical concerns related to undesirable groundwater flow may include the following:

- Construction can cause blocking and/or lowering of the water table. This can reduce flow in nearby streams, wells and wetlands.

- Changes in the water table due to construction may result in surface water and aquifer contamination.

- Construction excavations through confining layers or through impermeable surfacing can create pathways for surface water or contaminated aquifers to flow into uncontaminated aquifers.

- Road or bridge construction may remove a natural barrier or may block groundwater flow between different sites.

Monitoring is essential in determining baseline conditions and assessing impacts of groundwater movement. This includes monitoring:

- Groundwater levels in monitoring wells and boreholes

- Surface water levels in wetlands, streams, etc.

- Flow from springs and in associated watercourses

- Water quality parameters at springs or in boreholes

7.2 GROUNDWATER CONTROL

Groundwater flowing into a construction zone needs to be properly managed for purposes of facilitating construction, or for purposes of preventing contaminated groundwater from entering the construction site. This can be accomplished by dewatering, or by constructing cut-off walls.

7.2.1 Flow Quantities

Design of a dewatering and pressure relief or groundwater control system first requires determination of the type of groundwater flow (artesian, gravity, or combined) to be expected and of the type of system that will be required. In addition, a complete picture of the groundwater and the subsurface condition is necessary. Determine the number, size, spacing, and penetration of wellpoints or wells and the rate at which the water must be removed to achieve the required groundwater lowering or pressure relief.

In the analysis of any dewatering system, determine the source of seepage and the boundaries and seepage flow characteristics of geologic and soil formations at and adjacent to the site and generalize into a form that can be analyzed. In some cases, the dewatering system and soil and groundwater flow conditions can be generalized into rather simple configurations. For example, the source of seepage can be reduced to a line or circle; the aquifer to a homogeneous, isotropic formation of uniform thickness; and the dewatering system to one or two parallel lines or circle of wells or wellpoints. Analysis of these conditions can generally be made by means of mathematical formulas for flow of groundwater. Complicated configurations of wells, sources of seepage, and soil formations can, in most cases, be solved or at least approximated by means of flow nets, electrical analogy models, mathematical formulas, numerical techniques, or a combination of these methods.

7.2.1.1 Flow Net

Flow nets are valuable where irregular configurations of the source of seepage or of the dewatering system make mathematical analyses complex or impossible. A flow net is a graphical representation of the flow of water through an aquifer and defines paths of seepage (flow lines) and contours of equal piezometric head (equipotential lines). A flow net may be constructed to represent either a plan or a section view of a seepage pattern. See the *NAVFAC Design Manual 7.01, Soil Mechanics* (Reference 1); *Seepage, Drainage, and Flow Nets* (Reference 2); and *Designing with Geosynthetics* (Reference 3) for guidance.

<u>Groundwater Pressures</u>. For steady state flow, water pressures depend on the ratio of mean permeability of separate strata and the anisotropy of layers. A carefully drawn flow net is necessary to determine piezometric levels within the flow field or position of the drawdown curve.

<u>Seepage Quantity</u>. Total seepage computed from flow net depends primarily on differential head and mean permeability of the most pervious layer. The ratio of permeabilities of separate strata of their anisotropy has less influence. The ratio N_f/N_d usually ranges from ½ to ⅔ and, for estimating seepage quantity, a roughly drawn flow net provides a reasonably accurate estimate of total flow. Uncertainties in the permeability values are much greater limitations on accuracy.

For special cases, the flow regime can be analyzed by the finite element method. Mathematical expressions for the flow are written for each of the elements, considering boundary conditions. The resulting system of equations is solved by computer software to obtain the flow pattern.

7.2.2 <u>Drains</u>

Groundwater can be a contributing factor in slope instability, primarily from reduction in effective stresses and reducing shear strength. Using drains to control groundwater provides a means of collecting and removing the water.

7.2.2.1 Typical Configurations

The following are various methods of controlling groundwater in slopes:

1. <u>Horizontal Drains</u>. Horizontal drains involve drilling into a slope and installing a slotted pipe to reduce groundwater elevations and stabilize landslides. Maintenance is typically required to regularly clean horizontal drains in some geologic formations to avoid clogging problems. Most often they are installed as preventative measures in cut slopes. Although they are called “horizontal,” they are typically installed at angles of 2° to 5° above horizontal.

2. Trench Drains. Trench drains are used often in maintenance operations after the elevation of a perched water table has been located and the drain is to keep the water from extending into the roadway subgrade. Trench drains are typically excavated and installed along the top of the slope and filled with #57 aggregate or clean rock to intercept groundwater before it enters the slopes. Either a geotextile or graded filter material is used to keep the trench from eventually clogging with fine-grained soil. Depths of the trenches are determined on a case-by-case basis depending on soil and groundwater conditions.

3. Drainage Blankets or Wicking Geosynthetics. Drainage blankets or wicking geosynthetics are typically placed beneath embankments, under pavements, or at boundaries between cut and fill on supporting slopes. They are particularly important for side-hill embankments and should be used whether the supporting slopes are benched or not. These products intercept groundwater and direct it away from the embankment-slope interface.

7.2.2.2 Materials

Common materials for horizontal drains include slotted PVC or steel pipe with appropriate fixtures to control or direct flow. For trench drains, materials include geotextiles with or without graded drainage material or geosynthetics. For drainage blankets, materials typically include graded drain material with or without geotextiles.

7.2.2.3 Construction Considerations

Successful performance of drains involves installation of all materials with interest and attention to detail on the part of the contractor. Contractors should have a basic understanding of the pipe/soil interactions. This will enable the contractor to anticipate problems that may arise from poor construction practice not otherwise recognized. Consider the following:

- Proper trench excavation and preparation are necessary.

- Standing or flowing water in the trench will soften the toe of the sidewall and increase the possibility of unstable sidewalls and slopes. If installing non-perforated pipe, groundwater control should ensure a dry trench until the trench backfill is sufficient to prevent flotation of the pipe.

- The main trench backfill should be a specifically selected, open-graded aggregate composed of stone chips, pea gravel, or comparable material that is designed to remain stable while allowing unrestricted flow of water to the pipes. #57 aggregate is typically used for this application.

- Uniform compaction of bedding and backfill materials is important. To ensure proper final compacted soil densities, remove trench braces, shields, and boxes prior to the completion of the compaction.

- Uniform support for the pipe is essential. Do not use intermittent supports to establish the grade line. Provide clearance at protruding joints (e.g., bell and spigot, wrap-around joint couplings) of the pipe to prevent heavy and excessive "point" loads to these joints.

- Analyze pipe wall thickness considering construction equipment loading.

- If unsuitable subgrade materials are encountered at the construction site, bridging of the unsuitable materials can likely be achieved by using a properly designed geogrid/

geosynthetic combined with aggregate base and larger rock.

- In areas where there are natural springs, installing a concrete ditch liner can sometimes be difficult. For this situation, using a rip rap to stabilize the slopes can help with installation.

- To assist with water flow around Reinforced Concrete Boxes (RCB's), areas with a shallow water table or existing water issues should consider the installation of weep holes in the RCB.

7.3 REFERENCES

References specifically cited in Chapter 7 are numbered below, followed by additional pertinent technical information sources presented alphabetically by author.

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Chapter 8

GEOTECHNICAL EARTHWORK CONSTRUCTION

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8 GEOTECHNICAL EARTHWORK CONSTRUCTION

8.1 FILL/EMBANKMENT CONSTRUCTION

In most cases, the Materials Geotechnical engineer will not be involved in the routine placement of fill. Observations and testing of the fill will normally be the responsibility of the Resident Engineer. The exception to this would be in special cases where there are unusual fills (e.g., fills with geosynthetic base reinforcement, lightweight fills using pumice, shredded tires, extruded polystyrene, etc.) or construction of mechanically stabilized earth (MSE) walls. These types of fills and walls involve unique placement requirements and may benefit from inspection by the project Geotechnical engineer.

8.1.1 Borrow/Excavation Materials

8.1.1.1 Common Issues

A common construction request to the Material's Geotechnical Branch involves guidance to the Resident Engineer as to the suitability of borrow and excavation material. Questions on material suitability require the Geotechnical engineer to review how the material will be used in construction and whether the proposed material meets engineering design requirements. These requirements range from changes in the friction angle or the compressibility of the material to changes in permeability . These requests will be determined on a case-by-case basis.

For cases involving the suitability of existing materials for foundation support or for re-use at another location, the type of support will depend on the specific construction plans for the location and the design requirements. The project Geotechnical engineer may potentially visit the project site and specifically examine the material and discuss the conditions with the Resident Engineer, if necessary. In cases where strength, compressibility or permeability is in question, the project Geotechnical engineer may require additional laboratory tests of the material.

8.1.1.2 Materials Typically Encountered

Most earthwork (excavation and borrow) materials used in highway construction involve natural soils composed of inorganic, insoluble particles resulting from rock weathering. The soil may be residual (in place over weathering rock) or transported (e.g., water, wind, gravity). Natural soils contain many different grain sizes and mineral types. Coarse-grained soils (e.g., sand, gravel) are bulky, chemically inert grains that when properly compacted, can support large static loads, but may be easily deformed by vibrations. Fine-grained soils (e.g., silt, clay) are flat, plate-like grains with surface chemistry activity that defines their cohesive characteristics. They tend to deform with time under load and lose strength when wet. Suitability of soils for earthwork construction depends on the soil type and properties and intended construction purpose. Borrow material generally falls into the following categories:

1. Select. Select borrow includes soils classified as A-1, A-3 and A-2-4. ODOT may often call for these to be included in the upper part of the subgrade as they typically perform better.

2. Unclassified. Unclassified borrow is any soil that is not specifically called out as Select Borrow.

8.1.1.3 Compaction Factor

Almost all materials change volume in movement from cut to fill. Excavated solid rock will expand so that 1 CY of rock in the cut will occupy 1.15 CY to 1.50 CY in the fill. (Rock Fill is covered in Section 8.7.) If, however, the voids in the rock embankment are filled with earth or other fine material, the volume in the fill will almost equal the combined volumes in the two source locations. Excavated earth will expand beyond its original volume in the transporting vehicle and, finally, occupy only 0.85 CY to 0.65 CY in the fill, depending on its original density and the amount of compaction applied. These changes in volume are referred to as “swell” or “shrinkage.” Excavation, however classified, is commonly, but not always, measured in the space originally occupied. Because of shrinkage or swell, the place and method of measurement must be carefully defined in the specifications.

The shrink/swell factor provided in the soils report does not account for loss of material due to wind, erosion, transport, etc. To determine the actual fill volume, the road designer typically uses a compaction factor of 15%, which accounts for both shrinkage and loss of material.

The roadway designer should coordinate with the project Geotechnical engineer to determine the appropriate compaction factor for the project. The actual compaction factor should be checked and verified by the Resident Engineer during construction. Where soils indicate swell, do not apply an adjustment to the soil quantities. It is possible to have more than one compaction factor within a project. If this occurs, the user should make allowances for this in the volume calculations.

8.1.2 <u>Conditions</u>

The exercise of control over fill construction is a necessity recognized by all road building agencies. It is essential that the Resident Engineer and the Geotechnical engineer thoroughly understand the importance of ground preparation, compaction and stability, to ensure that construction complies with the *ODOT Standard Specifications* (Reference 1).

It is necessary that accurate and timely density determinations be made as construction proceeds to ensure that the specified density is obtained. Tests should be made in a manner to interfere as little as possible with the construction progress. The minimum frequency of density testing will be in accordance with ODOT’s Field Sampling, Testing, and Acceptance Guidelines; however, extra check densities are often necessary. Close observation of the contractor’s work and the performance of equipment on the grade is just as important as testing.

With proper moisture content, the contractor should have little difficulty in obtaining satisfactory densities. Should the field moisture be considerably below optimum moisture content, it may be impossible to satisfy density requirements without the addition of water. Conversely, if the soil is considerably wet of optimum, it may also be impossible to obtain the required density without processing and aeration, or the use of a drying agent (e.g., lime).

Compaction to the specified density is intended to accomplish two purposes. First, it provides the shear strength necessary to resist failure under the applied stresses. Second, adequate density packs the soil grains in such close contact that fill settlement is minimized. Settlement will occur because of the fill’s own mass (weight), or because of structural loading.

Cohesive soils and fine-grained silty soils are often troublesome. On the dry side of optimum, cohesive soils can display excellent shear strength during construction. If left below target density, these soils will become much weaker upon saturation. Soils compacted on the wet side of optimum may achieve target density, but the shear strength will be less than that at optimum.

If the moisture content is too high and compaction efforts are continued without aeration and moisture reduction, the flake shaped clay particles tend to orient themselves parallel to each other rather than in a random single grain relationship. While this orientation may result in some slight density increase, it also results in a reduction in the soil shear strength. Thus, soil with moisture content considerably wet of optimum should be dried prior to compaction.

Silty soils of very low plasticity, for all practical purposes, develop no cohesive bond between the grains. They achieve their load carrying ability from their angle of internal friction and the contact pressure between the soil grains. These soils are particularly sensitive to excess soil moisture. When silty soils are compacted wet of optimum without adequate aeration, they may become susceptible to pumping to the point that the grade is completely unstable.

Regular inspection during construction of the fill should include:

- Detection of bulging side slopes
- Cracks at the top or on the sides of the slopes
- Heaves at the toe of slope
- Any movement adjacent to structures, or distress within the same

8.1.3 Quality Control

Placement conditions for compacted fill material address the issue of what dry density and moisture content to place the fill soil in so as to obtain optimal engineering properties, specifically, shear strength and deformation under load (settlement). Once the placement conditions are selected for the specific soil, the acceptance criteria in the compaction specification sets the limits to which the earthwork contractor's efforts will be measured. Unless directed otherwise through plan notes or special provisions, the acceptance criteria are typically set at 95% of the maximum dry density and a moisture content within two percentage points of optimum moisture determined in AASHTO T 99 compaction test (proctor). For both cohesive and granular soils, the acceptance criteria are confirmed by measuring the in-place dry density of the compacted layer using a nuclear gage (AASHTO T 310). If the field dry densities meet the criteria (pass), the compaction process continues. If the criteria are not met (fail), the appropriate action must be taken to address the failure.

8.2 EROSION PROTECTION

The primary function of geotextiles used for permanent erosion control is to protect the soil beneath the geotextile from erosion caused by surface water flow, runoff, or piping. Geotextiles are used as a part of ODOT Management of Erosion, Sediment and Stormwater Pollution and Control Process; see *ODOT Standard Specifications*.

Examples of permanent erosion control geotextile applications used in combination with riprap include:

- Runoff and erosion protection along drainage channels and high-velocity diversion ditches
- Erosion protection for hydraulic structures (e.g., culverts, drop inlets)
- Erosion protection for highway cuts or fill slopes

8.2.1 Temporary Sediment Controls

Geotextiles, geosynthetic erosion control blankets and other geosynthetic products can be used to temporarily control and minimize erosion and sediment transport during construction. Four specific applications are identified below:

1. Suspended Particles. Use geotextile silt fences as a substitute for hay bales or brush piles to remove suspended particles from sediment-laden runoff water.

2. Turbidity Curtain. Use geotextiles as a turbidity curtain placed within a stream, lake or other body of water to retain suspended particles and allow sedimentation to occur.

3. Retention Blankets. Special soil retention blankets are made of both natural and synthetic grids, meshes, nets, fibers and webbings. They are used to provide tractive resistance and resist water velocity on slopes. These products retain seeds and add a mulch effect to promote the establishment of a vegetative cover.

4. Temporary Erosion Control. Geotextiles held in place by pins or riprap are used for temporarily control erosion in diversion ditches, culvert outfalls, embankment slopes, etc. Alternatively, soil retention blankets are used for temporary erosion control until vegetation can be established in the ditch.

For runoff control, geosynthetic products are designed to help mitigate immediate erosion problems and provide long-term stabilization by promoting the establishment and sustainment of vegetative cover. The main advantages of using geosynthetics for erosion control applications include the following:

- Vegetative systems have desirable aesthetics.
- Products are lightweight and easy to handle.
- Temporary, degradable products improve establishment of vegetation.
- Continuity of protection is generally better over the entire protected area.
- Empirically predictable performance; traditional techniques (e.g., seeding, mulch covers, brush or hay bale barriers) are often less reliable.

8.2.2 Permanent Sediment Controls

Erosion control mats are a type of geosynthetic used in permanent erosion control systems. They are also referred to as a Rolled Erosion Control Product (RECP). These three-dimensional mats retain soil and moisture, thus promoting vegetation growth.

Geotextiles, geosynthetic erosion control mats, and other geosynthetic products can be used to control and minimize erosion and sediment runoff. Possible applications include the following:

- Riprap-geotextile systems are successful applications in protecting precipitation runoff collection and high-velocity diversion ditches.
- Geotextiles in slope protection prevent or reduce erosion from precipitation, surface runoff, and internal seepage or piping.
- Erosion control systems with geotextiles may also be required along stream banks to prevent encroachment of roadways or appurtenant facilities.

- Geotextiles around structures to provide scour protection.
- A riprap-geotextile system can be effective in reducing erosion caused by wave attack or tidal variations where facilities are constructed across or adjacent to large bodies of water.
- Hydraulic structures (e.g., culverts, drop inlets, artificial stream channels) may require protection from erosion. In these applications, if vegetation cannot be established or the natural soil is highly erodible, a geotextile can be used beneath armor materials to increase erosion resistance.

8.3 SUBGRADE CONSTRUCTION

A common construction problem involves soils that are softer than expected, based on the available geotechnical information. If the amount of soft subgrade material is limited, Construction personnel will often determine the type of mitigation that is required, and the project Geotechnical engineer may not be contacted. However, if the depth or extent of the unsuitable material is of such extent that a significant change in the contract may be required, the Materials Division Geotechnical engineer may be contacted to provide recommendations.

A variety of options are available for mitigating the extent of soft material. In most cases, these mitigation measures are site specific and are based on the following:

- Types of soil involved
- Quantity of material that needs to be mitigated
- Availability of specialized contractors, if required
- Availability of materials and/or equipment
- Time requirements
- Costs of potential mitigation measures

The Materials Geotechnical engineer will usually need to travel to the site and inspect conditions to understand the conditions and mitigation requirements. If the depth or extent of mitigation cannot be identified by visual means, it may be necessary to mobilize exploration equipment to quantify the extent of soft materials.

Subgrade areas requiring mitigation may be identified during the geotechnical investigation phase of a project. In this circumstance, the project plans will specifically indicate the location and depth of required mitigation, and the specifications will describe the requirements for mitigation. A number of potential options exist to establish a more suitable subgrade when unexpected poor quality subgrade material is encountered during construction. Often, the most economical and expedient solution, when deemed suitable, is to expand the mitigation of nearby areas that were identified during design.

The following mitigation options are available to address soft soils:

1. Undercutting. At locations where unsuitable soils or soft ground conditions are encountered, soil can be undercut (also called over excavation) at the direction of the Resident Engineer. Undercut depths may extend to a maximum depth of 2 ft to 5 ft below top of subgrade, depending on the thickness and consistency of the soft material. The decision on the amount of undercut is usually made in the field by the construction

inspector in consultation with the project Geotechnical engineer. Undercuts are typically backfilled in compacted lifts using select granular fill. A subgrade reinforcement geosynthetic is often placed prior to placing the backfill.

2. Bridging of Soft Soils. A common approach when soft subgrade soils are encountered is to bridge over the soft ground by placing fill in thick compacted lifts until enough fill has been placed to adequately support equipment. Often a heavy stabilization geotextile or geogrid will be placed before adding the fill. This method can be successful where proposed fill depths are generally on the order of 5 ft or greater. This method has limitations, especially where proposed fill heights are low, and the bridging material is at or near the design subgrade elevation and is unsuitable to support the pavement section.

3. Geosynthetics. Stabilization geosynthetics in conjunction with imported granular fill are often used for soft, very moist to wet subgrade conditions. Geosynthetics for this application must function as a separator, a filtration layer and, to a minor extent, as a reinforcement layer. Do not use woven, slit-film geotextiles (i.e., geotextiles made from yarns of a flat, tape-like character) for this application. This technique is attractive in that this method:

    - is not dependent upon potential chemical reactions with admixtures
    - usually is economical in that geosynthetics are often already being used elsewhere on the project and/or are readily available from local sources
    - provides a relatively quick and generally simple construction technique for successful implementation

4. Subgrade Admixtures. Subgrade stabilization admixtures (e.g., lime, cement, cement kiln dust (CKD), flyash) are generally limited to treatment of the upper 1 ft of the subgrade and, therefore, may not be adequate for treating deeper deposits. These methods are often not suitable or, at a minimum, need to be evaluated very carefully including performing sufficient laboratory tests to evaluate the potential for sulfate/calcium reactions between the soil and stabilizing agent. These methods require special equipment, specialized knowledge and experience by the contractor, and require additional time for chemical reactions to occur.

Occasionally unsuitable soils (e.g., peat/muck, collapsible soils, expansive soils) may be unexpectedly encountered during construction. Addressing these cases when they occur is site specific and often includes removing the unsuitable soils unless the vertical and lateral extent is extensive in which case mitigation will require other options.

8.4 CHEMICALLY MODIFIED/STABILIZED SOILS

Subgrade modification/stabilization admixtures (e.g., lime, cement, CKD, flyash) are generally limited to treatment of the upper 1 ft of the subgrade and, therefore, may not be adequate for treating deeper deposits. These methods are often not suitable or, at a minimum, need to be evaluated very carefully, including performing sufficient laboratory tests to evaluate the potential for sulfate/calcium reactions between the soil and stabilizing agent (e.g., cement, lime). These methods may also require special equipment, specialized knowledge and experience by the contractor, and require additional time to allow the chemical reactions to occur. See Section 2.8.5 for testing of sulfate rich soils. See Section 4.2.2 and the ODOT Standard Specifications or additional information on soil stabilization.

8.5 BACKFILL AROUND STRUCTURES

Embankment construction adjacent to structures is important. Even the casual observer cannot help but notice that the settlement of pavement placed on backfills over culverts or adjacent to bridges and retaining walls is one of the most frequent defects to occur in highway construction. It is unfortunate that the problem of working in a confined space should occur, where the need for a good fill is greatest. The cramped working space, the relatively small volume of fill involved, and the backfill material locally available are contributing factors to improper backfilling.

The Resident Engineer may request the Material's Geotechnical engineer to evaluate the acceptability of certain materials for use as backfill around structures. Evaluations of this type most often occur when the structure is unique or where atypical loading conditions are applied to the backfill.

8.5.1 Acceptable Materials

To avoid poor water pressure behind abutment, wing or retaining walls, backfill materials should be free draining, and a drainage system included to allow water pressures to dissipate. The best backfill materials are stone, gravel, or sand. These materials are best, not only because of their drainability, but also because of their high angle of internal friction, which contributes to their shear strength. These materials are more easily placed and compacted than the fine-grained soils and have low compressibility if properly placed. Silty or clayey sand is less desirable because of the reduced permeability, the increased difficulty in obtaining compaction, and a lower shear strength associated with a lower angle of internal friction. With fine-grained cohesive or silty soils, increased care in placement is a necessity to obtain density; otherwise, excessive forces can be exerted on the wall. Drainability of cohesive backfill materials is difficult, if not impossible, and requires the inclusion of special drainage features behind the wall.

A cohesive backfill is least desirable, not only due to problems with drainage, but also due to volume changes caused by cyclic wetting and drying with time or settlement. Compaction of these materials in a confined area is difficult. Inadequate compaction results in excessive settlements and wall distress.

8.5.2 Placement Conditions

The inspection and approval of material placement around structures is normally the construction inspector's responsibility. Inspectors observe placement and conduct density tests as needed. The Resident Engineer may request that the project Geotechnical engineer provide guidance on substitute materials based on their judgment and experience. Because the decision will ultimately be made by the Resident Engineer, the Materials Division Geotechnical engineer should advise the Resident Engineer on the implications of using alternative materials.

8.5.3 Clay Plating

Clay plating is a method used to protect soil embankment slopes constructed of potentially erosive or dispersive soils. It can also be used as the top layer of an embankment constructed of more granular (sand or gravel) soils to minimize infiltration of surface water. The clay plate acts as a protective layer reducing erosion forces from surface water or as a leveling layer and deterrent to water infiltration on the top of an embankment.

8.6 ROCK EXCAVATION

8.6.1 General

Some roadway construction projects in Oklahoma require the contractor to perform rock excavations. The contractor normally determines the rock excavation method. Depending on the logistics for the site and the type of rock, the excavation methods can range from rock hammering, ripping with a dozer, to blasting.

The role of the Materials Geotechnical engineer during the construction phase includes the review of new rock slopes for stability. Review and inspection activities provided by the project Geotechnical engineer or geologist confirm that:

- rock excavation will be accomplished in a manner that meets the intent of the geotechnical design while avoiding risk to nearby structures

- the excavated slope meets long-term stability requirements while minimizing long-term maintenance problems from rock instabilities

A significant amount of practical experience is required when providing support to the Construction staff for typical rock excavation projects. In most cases, personnel with significant experience in this type of work from the Materials Division Geotechnical Branch should be involved in this review and inspection work. If blasting is required, an industry expert should be called in by the Contractor to review and/or carry out the blast plans.

8.6.2 Review and Inspection Activities

The specific review and inspection activities will depend on the amount of excavation (e.g., volume, height) and type and condition of rock structure at the site. These conditions vary for each site; therefore, before providing any review and inspection services, carefully review the geotechnical report to develop a clear understanding of the site geology and any special issues associated with rock excavation.

In general, the review and inspection services will be more involved if blasting is required because of the potential for over-break of the rock during blasting, damage to nearby structures and safety risks. Typical review and inspection services are summarized below:

1. Pre-Blast Survey. The contractor should conduct a pre-blast survey of both on-site and off-site conditions. The project Geotechnical engineer or geologist may be requested to participate in this survey. The on-site survey involves a review of the contractor's proposed blasting plan and an evaluation of any potential concerns or problems relative to the blasting operations. The role of the project Geotechnical engineer or geologist in the on-site survey is to identify rock characteristics that could be important for developing an excavation plan and geologic conditions that could be affected by vibrations from blasting (e.g., location of potential rock falls on steep slopes, loose sands that could densify). An off-site survey should be conducted of nearby structures to assess any existing damage (i.e., damage that existed prior to any blasting activities on the project). The survey should include photos and attaching crack gauges to existing cracks in buildings.

2. Blasting Plan. The contractor will submit to the Resident Engineer a blasting plan before drilling and blasting operations begin and whenever there is a change in the proposed drilling and blasting methods. The Geotechnical engineer or geologist will review, but not approve, the contractor's blasting plan and may require, through the Resident Engineer, that the contractor clarify and then revise the drilling and blasting methods. As part of this review, the lift heights should be checked to confirm that the heights will allow scaling with track hoes and other similar equipment or will allow for installation of rock bolts if required.

3. Testing Blast. Prior to conducting full-scale operations, the blasting contractor is required

to conduct test blasts to determine the adequacy of the proposed blasting plan. The project Geotechnical engineer or geologist will likely be asked to attend the test blast to observe the effects of the blast on the geologic formations and, in particular, the amount of over-break for the proposed program.

4. Production Blasting. The project Geotechnical engineer or geologist may be requested to observe blasting activities during full-scale operations to ensure satisfactory results are obtained. Of particular importance is the over-break that occurs during the blast and whether long-term stability issues could result. As part of the blasting operations, the contractor is required to submit a plan to establish vibration control and monitoring.

5. Post-Blast Survey. If blasting methods are used to excavate the rock and there are nearby structures, the contractor may be asked to provide a post-blast survey if complaints of damage or annoyance are made. The intent of this survey should be to determine whether the blasting methods resulted in any damage to nearby structures. This damage could include cracks in plaster and settlement of structures.

6. Inspection and Support after Blasting or Mechanical Excavation. Support may include the following:

    a. Scaling Inspection. The Resident Engineer may request the project Geotechnical engineer or geologist to travel to the project site and inspect the face of the excavated rock, either during blasting/mechanical excavation or afterwards. The objective of this inspection is to evaluate the stability of the excavated face. This inspection identifies fractures in the rock that could result in sliding or topping of rock blocks or fractures that could collect water in the winter and freeze, potentially resulting in future failures. The project Geotechnical engineer or geologist will document the field survey in the field report and photograph all features that may fail. The project Geotechnical engineer or geologist may also be requested to participate with the Resident Engineer and contractor in deciding whether reshooting of the excavation slope is required.

    b. Mitigation Methods. If the inspection identifies rock features that could pose a future risk to vehicular traffic, buildings, or other structures, the project Geotechnical engineer or geologist should document these risks and recommend methods for stabilizing the rock surface. The stabilization procedures could range from use of stabilization berms to rock bolts. If the unstable features are relatively small, the project Geotechnical engineer could recommend further rock scaling to remove the loose or unstable features. If rock bolts are required, the Materials Division Geotechnical engineer or geologist should observe rock bolt or rock anchor testing and determine whether test requirements are satisfied.

Whenever field inspections of rock excavations are performed, the project Geotechnical engineer or geologist should use the inspection opportunity to confirm that the original design assumptions and approaches are still valid based on the exposed rock conditions. If the characteristics of the exposed rock suggest a potential for long-term stability issues, it is critical that the issue be brought to the attention of the Resident Engineer and Geotechnical engineer and that a consensus be reached on how to handle the issue.

8.6.3 Rippability of Rock

The rippability of rock material is a measure of its ability to be excavated with conventional excavation equipment. Rippability is determined by unconfined compressive strength, degree of weathering abrasiveness and spacing of discontinuities or a refraction seismograph. The seismograph must be capable of providing valid, usable signals for calculating the depth to bedrock to the nearest foot. It must be capable of sensing rock layers to the depth of the proposed cut. Rock rippability is calculated from the resulting sound wave velocities. Report the rock rippability rating of each layer as rippable, marginal, or non-rippable.

8.7 ROCKFILL

8.7.1 General

Rock fills are used to control embankment erosion in areas of high hydraulic impact and to prevent embankment softening due to capillary action. The weight and interlocking characteristics of rock fill allow it to resist hydraulic impact and capillary forces. Rock fill material should be free-draining with rock-to-rock contact structure that can be effectively compacted by vibrating compaction equipment. Rock fill materials are usually obtained by ripping or blasting and may or may not be sized before construction. Sizing will be based on desired maximum rock size in the rock fill materials (i.e., 8 inches to 24 inches) as referenced in the *ODOT Standard Specifications*. Rock fill materials can be used in construction of embankment, rock berms or buttresses, or as erosion protection layers (e.g., rip rap).

Special rock equipment and procedures are required for rock borrow development, hauling, placing, and compacting to produce a stable and acceptable engineered rock fill. The conventional earth fill test methods for controlling lift thickness, gradation, moisture content, and compaction are not applicable to rock fills.

8.7.2 Materials

Rock fill materials should be hard, sound, durable rock with limited fines. Weathered, soft or poorly cemented sedimentary rocks should not be used as rock fill because of their susceptibility to weathering and degradation during service. In Oklahoma, many sandstones, some limestones and most shales appear sound during excavation and placement but break down during compaction and service. The project Geotechnical engineer must identify these materials as early in the design process as possible and, if included in the embankment, these materials should be moisture conditioned, reduced to 4-inch maximum size particles, and compacted as soil.

Section 2.6 discusses the common laboratory tests for intact rocks including measurements of strength (e.g., point load strength index, unconfined compressive strength), stiffness (e.g., ultrasonics, elastic modulus) and durability (e.g., slaking, abrasion).

8.7.3 Construction Considerations

Vibratory compactors are best suited for rock fill construction because they can more efficiently compact the granular material into stable layers/embankments as they increase the fill strength. Smooth drum vibratory rollers varying in weight between 30 tons and 50 tons are preferred. Operating frequencies of 20 Hz to 25 Hz with ground speeds of 1.5 mph to 3 mph for 4 to 8 passes are generally recommended for efficient layer compaction. Loose lift thicknesses vary with maximum rock size but typically are between 1 ft and 3 ft and must be greater than the nominal maximum rock size. A rule-of-thumb is that the maximum rock size should be 2/3 of the loose lift thickness. ODOT uses some prescriptive guidelines on rock fill construction.

See the *ODOT Standard Specifications for Highway Construction* for further guidance.

8.8 INSTRUMENTATION

Two groups of geotechnical instrumentations are typically used on ODOT projects:

1. Subsurface Investigations. The first group of instruments is used in subsurface investigations prior to and during construction to determine soil and rock properties (e.g., strength, compressibility, permeability) using instruments such as cone penetrometers, pressure meters, dilotometers, and groundwater monitoring systems.

2. Monitoring. The second group is for monitoring soil and rock performance during construction and operational service life of a project. In addition, it is also possible to use performance-monitoring instruments to validate the design. Performance monitoring during construction can involve measuring groundwater levels or porewater pressures, soil stresses, soil or rock deformations, and load or strain in structures. Instrumentation monitoring during design can include the construction of test fills to obtain information about settlement properties at a site, or pile-load tests to address significant issues regarding the constructability or the performance of piles in a specific geologic formation.

8.8.1 General Role of Geotechnical Instrumentation

Every geotechnical design involves uncertainties and every construction job involving soil or rock runs the risk of encountering surprises because of uncertain soil conditions or soil behavior. These circumstances are the result of working with materials created by nature, which seldom provide uniform conditions. The inability of exploratory procedures to detect all possible properties and conditions of natural material requires the project Geotechnical engineer to make assumptions and select equipment and construction procedures without full knowledge of what might be encountered. Field instrumentation can reduce these uncertainties and can aid in the selection of appropriate field equipment and construction procedures.

8.8.2 Objectives of Geotechnical Instrumentation Program

The objectives for instrumentation during construction will change depending on the size and type of construction, the geotechnical conditions, and the project schedule. Some types of instrumentation monitoring are a required part of construction (e.g., testing of soil nails, tieback anchors). Other types of instrumentation monitoring can only be implemented if construction durations are sufficiently long to make the data collection useful and relevant. Preload monitoring is an example of this category.

After establishing a clear set of objectives, the project Geotechnical engineer identifies the potential need for instrumentation monitoring and communicates the preliminary instrumentation plans with the Design Project Manager to confirm that the objectives of the instrumentation work are justified and fit within the likely construction plans.

8.8.3 Types of Instrumentation

Geotechnical field instrumentation can range from settlement and porewater pressure devices used for monitoring the construction of fills to slope deformation systems in landslide areas. Geotechnical instruments can involve simple mechanical measurement systems where data are manually recorded, or they can involve complex electronic systems where data are recorded on a datalogger with remote downloading capabilities. As the sophistication of the measuring instrument increases, so do the costs. Consequently, the project Geotechnical engineer will need to judge the benefits of the different systems, relative to costs for purchasing and recording data.

A variety of situations may warrant the use of geotechnical instrumentation for field performance monitoring on ODOT projects, including:

1. Monitoring Settlements, Lateral Displacements and Porewater Pressures Beneath Fills. These measurements can be made to control slope stability. In this case, the next lift of fill may not be added until porewater pressures have dissipated to a level that would

preclude slope instability or bearing capacity failure.

2. Monitoring Earth-Retaining Structure Load and Deformations. These measurements are used to determine if active or at-rest pressures are developing, if deformations could jeopardize facilities behind the walls, or if the wall is performing in a suitable manner.
3. Monitoring Slopes in Landslide Areas or Where Slope Stability is Marginal. These measurements are used to either identify the rate of movement and location of failure planes within the moving soil or rock mass or confirm that construction fills or excavations are not causing excessive ground movements.

4. Monitoring Groundwater Elevations. These measurements are used to evaluate initial effective stresses and changes in effective stresses caused by loading the soil, or simply the change in groundwater elevations due to seasonal fluctuations caused by snowmelt, heavy rainfalls or irrigation.

8.8.4 Instrumentation Selection, Installation, Monitoring & Interpretation

After the decision is made to include field instrumentation monitoring, plans are developed to select and install the appropriate instrumentation and collect and interpret the data. This step is often the most difficult to accomplish. The project Geotechnical engineer leads the planning process. Implementation and data collection could be the responsibility of the Geotechnical engineer or the contractor, depending on the specifics of the project. The project Geotechnical engineer or other designated representative will be responsible for collecting the data. The project Geotechnical engineer will be responsible for interpreting the data or reviewing the data collection if performed by an independent consultant.

The project Geotechnical engineer will usually develop the instrumentation plan during design, and will often be required to prepare drawings and specifications describing the type, installation and monitoring requirements. During construction, the project Geotechnical engineer should be prepared to:

- review instrumentation submittal information prepared by the contractor
- observe or coordinate the installation of the instrumentation
- collect or review data from the instrumentation
- interpret the data
- troubleshoot when unusual data are acquired

Guidelines for selecting, installing, monitoring and interpreting instrumentation data can be found in the FHWA *Geotechnical Instrumentation Manual* (Reference 2) and *Geotechnical Instrumentation for Monitoring Field Performance* (Reference 3). Companies selling instrumentation typically have detailed information describing equipment capabilities, installation procedures and monitoring alternatives. The project Geotechnical engineer should conduct thorough research before selecting the instrumentation. The success of the instrumentation program will often be determined based on instrument suitability, instrument performance, and data quality.

Two important considerations during the instrumentation selection process are as follows:

1. Redundant Measurements. Redundant measurements are critical to account for the effects of load variations and soil variability on the measurements, the result of damage during installation or from construction equipment, or simply from malfunctions that occur

with time (e.g., leaks in sealed pressure cells, instrumentation cables). Consider all of these possibilities during selection of type, number, and location of the instrumentation. One guiding principle during instrumentation planning is to assume that the instrument most important to design will be the one that fails and, therefore, an appropriate contingency must be identified. This contingency planning can often be handled by redundant measurements.

2. Instrument Calibration. Calibrate each instrument before use. Instruments purchased from a supplier will usually come with instrumentation calibration curves or may be calibrated to a standard atmospheric pressure. It is often good to check these calibration curves in a bench test before installation to confirm that shipping has not changed the calibration or, where applicable, to perform calibrations to account for elevation changes that will affect the atmospheric pressure calibration. If instrumentation is being reused on a project (e.g., vibrating wire piezometer in a standpipe piezometer), it is critical to check the calibration before deployment.

When planning the instrumentation program, anticipate the requirements for baseline measurements. Make these measurements after installation of instruments and before construction begins. The duration of baseline recordings will depend on the particular type of instrument. For example, some instruments (e.g., settlement target on a building) require little to no time between installation and data monitoring, while other equipment (e.g., groundwater piezometers) could require several weeks of monitoring. The project Geotechnical engineer needs to identify the baseline periods for each instrument when planning the program.

Consider the sensitivity of the data measurement during the planning process. The sensitivity will depend on the range of values that could occur during the recording period. Often these values will be determined as part of the engineering design process. For example, if constructing a lift of the fill is expected to cause a 5-psi porewater pressure increase, the sensitivity of the recording should be roughly 10% of the measurement. At the same time, the total range of porewater pressure measurement could be much higher at deeper depths, thereby requiring more sensitive transducers at the ground surface than at deeper depths.

Requirements for data interpretation should be considered in the instrumentation planning process. Too often data are collected and not reviewed until it is too late to implement a remedial plan. For this reason, the project Geotechnical engineer must decide not only what to monitor and where to monitor, but also how the information is going to be used once it is collected. This step could include estimating the amount of staff time that will be required to interpret the data each day. For some projects, an action plan must be developed that tells the field personnel or contractor what actions will be taken if, for example, porewater pressures or displacements reach a predetermined level. One of the primary responsibilities of the project Geotechnical engineer is to ensure data interpretation in a timely manner.

8.9 REFERENCES

References specifically cited in Chapter 8 are numbered below, followed by additional pertinent technical information sources presented alphabetically by author.

1. **ODOT.** *ODOT Standard Specifications for Highway Construction.* Oklahoma Department of Transportation. Oklahoma City, OK. Latest edition.

2. **FHWA.** *Geotechnical Instrumentation Reference Manual.* Federal Highway Administration. Washington, DC. FHWA-HI-98-034. October 1998.

3. **Dunnicliff, J.** *Geotechnical Instrumentation for Monitoring Field Performance.* Wiley-Interscience. New York, NY. 1993.

4. **FHWA.** *Geosynthetic Design and Construction Guidelines Reference Manual.* Federal Highway Administration. Washington DC. FHWA HI-95-038. April 1998.

5. **ODOT.** *Roadway Design Manual.* Oklahoma Department of Transportation. Oklahoma City, OK. Latest edition.

Chapter 9

EARTH RETAINING STRUCTURES

Chapter 9

ODOT GEOTECHNICAL MANUAL

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9 EARTH RETAINING STRUCTURES

9.1 OVERVIEW

Earth retaining structures (ERS) or retaining walls, are used to hold back earth and maintain a difference in elevation of the ground surface. In general, the cost of constructing a retaining wall is high compared with the cost of forming a new slope. Thus, the need for a retaining wall is assessed carefully, and an effort is made to keep the retained height as low as possible.

In highway construction, retaining walls are used along cuts or fills where space is inadequate for construction of cut slopes or embankment slopes. Bridge abutment and foundation walls, which must support earth fills, are also designed as retaining walls.

ODOT earth retaining systems are designed per AASHTO LRFD requirements. This manual discusses general design considerations for walls common to ODOT projects. For more detailed information regarding the design of retaining walls, refer to the most current version of the AASHTO LRFD Bridge Design Specifications.

Common earth retaining structure applications for highway construction in Oklahoma include:

- new or widened highways in developed areas
- new or widened highways at hills or steep slopes
- grade separation
- bridge abutments, wing walls and approach embankments
- culvert walls
- stabilization of new or existing slopes and protection against rockfalls

9.2 CLASSIFICATION OF EARTH RETAINING STRUCTURES

Retaining walls are commonly classified by multiple characteristics, including classification by load support mechanism (e.g., externally or internally stabilized walls), construction concept (e.g., fill or cut walls), system rigidity (e.g., rigid or flexible walls), and permanence (e.g., temporary or permanent walls).

In this chapter, walls are organized by two main categories: externally and internally stabilized systems. An externally stabilized system uses an external wall against which stabilizing forces are mobilized, and an internally stabilized system involves reinforcements installed within and extending beyond the potential soil failure surface. Some wall systems are hybrid systems, combining the elements of both externally and internally supported walls.

9.2.1 <u>Classification by Construction Concept</u>

When classifying according to construction concept, “fill” wall construction refers to a wall that is constructed from the base of the wall to the top “Cut” wall construction involves top-to-base construction, concurrent with excavation operations.

9.2.2 Classification by System Rigidity

Classification according to wall system rigidity is useful when considering the development of earth pressures. A wall is rigid if it moves as a unit overall and does not experience appreciable bending deformations (most gravity walls can be considered rigid walls). Flexible walls are those walls that undergo bending deformations in addition to the rigid body motion. These deformations cause a redistribution of lateral pressures from the more flexible to the stiffer portions of the system. Most wall systems, aside from gravity walls, may be considered flexible.

9.2.3 Temporary and Permanent Wall Applications

ODOT builds both temporary and permanent walls. Permanent walls are generally considered to have a service life of 75 to 100 years. Wall systems commonly selected as permanent walls include cast-in-place walls, mechanically stabilized earth (MSE) walls, soldier pile walls, and soil nail walls due to their long-term durability. When walls are needed only temporarily, such as during construction phasing, wall system solutions such as sheet pile walls, soldier pile walls, or geosynthetic-reinforced soil walls are common due to their lower cost and their ability to temporarily meet project requirements.

9.3 GEOTECHNICAL PARAMETERS FOR EARTH RETAINING STRUCTURES

Soil and rock parameters are needed to complete Earth Retaining Structures (ERS) design, and to evaluate ERS constructability. Such parameters include subsurface stratigraphy, as well as index and performance (strength, permeability) properties of in-situ soils, wall backfill material, and rock (as applicable). Such parameters for retaining structures can be obtained from in-situ tests, laboratory tests, and from correlations with index properties.

The Geotechnical engineer of record uses his or her knowledge of soil mechanics, retaining wall design and project-specific design constraints, and of general site conditions/restrictions to formulate an appropriate subsurface investigation program. The information obtained must be sufficient to perform appropriate retaining wall engineering evaluations, which may include internal stability, external stability, global stability, total and differential settlement, horizontal deformation, lateral earth pressures, bearing resistance, chemical compatibility with soil and wall materials, pore pressures behind wall, obstructions, and dewatering/drainage. In performing engineering evaluations, consideration is given to both short- and long-term performance of the wall, as appropriate.

The subsurface investigation should be designed to provide the following parameters or information, as appropriate:

- Subsurface profile (soil, groundwater, rock)
- Coefficients of lateral earth pressure
- Interface shear strengths of soil, wall structures, and reinforcements
- Shear strength parameters of foundation soil (and wall fill material if native soil is used as fill)
- Compressibility and time-rate consolidation parameters
- Chemical composition of soils

- Hydraulic conductivity of soils directly behind wall

- Geologic mapping including orientation and characterization of rock discontinuities

In-situ field tests including SPT, CPT, dilatometer, and rock coring are the field tests that are most common to Oklahoma ERS projects. Common laboratory tests for ERS structures include index tests (grain size distribution, Atterberg limits, moisture content, unit weight), 1-D consolidation, triaxial testing, pH/resistivity, and soil chemical tests.

Geotechnical investigations are typically planned so that exploration locations are spaced at 100-200 ft intervals with locations either along the back face of the wall or alternating from in front of the wall to behind the wall. For more specific information and requirements on subsurface investigations for earth retaining structures, refer to Chapter 2 of this manual or the current ODOT geotechnical specifications.

9.4 LATERAL EARTH AND WATER PRESSURES

A wall system is designed to resist lateral earth and water pressures that develop behind the wall. For walls typically constructed by ODOT, earth pressures develop due to loads induced by the weight of the backfill and various surcharge loads. Evaluation of lateral earth and water pressures are performed to assess forces acting on the wall from the backfill or retained ground.

9.4.1 Lateral Earth Pressures

For purposes of ERS design, three different lateral earth pressures are usually considered: (1) at-rest earth pressure, (2) active earth pressure, and (3) passive earth pressure.

1. At-rest earth pressure refers to the lateral pressure that exists in level ground wherein there is no lateral deformation.
2. Active earth pressure is developed as a wall moves away from the backfill or retained soil. This movement results in a decrease in lateral pressure relative to the at-rest condition. A relatively small amount of lateral movement is necessary to reach the active condition.

3. Passive earth pressure is developed as a wall moves towards the backfill or retained soil. This movement results in an increase in lateral pressure relative to the at-rest condition. The movements required to fully activate the passive condition are approximately ten times greater than those required to activate the active condition.

The magnitude of active and passive pressure are functions of the soil shear strength, the backfill geometry, the wall geometry, the soil weight, and the friction and cohesive forces that develop along the wall-soil interface as the wall moves relative to the retained ground.

The magnitude of active and passive pressure is also affected by the flexibility of the wall. A relatively stiff wall (e.g. a gravity wall such as that shown in the figure above) attracts load and imposes a deformation condition on the soil which affects the pressure distribution. In contrast, a more flexible wall (e.g., an anchored sheet pile wall) bends under loading) lowers the pressure at the mid-height of the wall between the anchor and the bottom of the excavation while simultaneously causing higher pressures to develop at the location of the anchor.

Surcharge loads on the backfill surface near an ERS cause increased lateral earth pressures on the structure. Typical surcharges considered in design include loading due to highways, railroads, sign/light structures, construction equipment, and material stockpiles. Load cases of particular interest in determining lateral earth pressures due to surcharge loads include uniform surcharge, point loads, line loads parallel to the wall, and strip loads parallel to the wall.

In walls supporting fill, compaction of the backfill can generate significant horizontal pressures on the wall. For this reason, it is customary to reduce the compaction criteria for the zone immediately adjacent to the wall. In cases where a high level of compaction is needed even adjacent to the wall, the horizontal pressures generated by that compaction are evaluated and considered in the design of the wall.

9.4.2 Water pressure

It is standard practice in retaining wall design to provide a drainage system on the back side of the wall such that water pressures do not develop behind the wall. However, there may be cases where it may not be feasible to drain the water from behind the structure, such as may be the case with a temporary sheet pile wall wherein a sheet pile is driven into the ground followed by excavation down to the design excavation depth. In such instances, the ERS is designed for both lateral earth pressure and water pressure.

9.5 EARTH RETAINING SYSTEM TYPES

Figure 9-1 provides illustrations of common retaining wall systems used on Oklahoma highways. In recent decades, various wall systems have been developed to utilize unique products, technologies, and/or specialized construction methods. For example, mechanically stabilized earth and soil nailing have changed the ways we construct fill or cut walls by providing economically attractive alternatives to conventional construction methods. Another example is the increased use of polymeric products to reinforce soil and control drainage. The advent of these products and technologies has led to the development of an array of different earth retention schemes. Each unique system has its specific benefits and constraints. Understanding the various wall system types and their application to highway facilities will help the design engineer in selecting the best possible system.

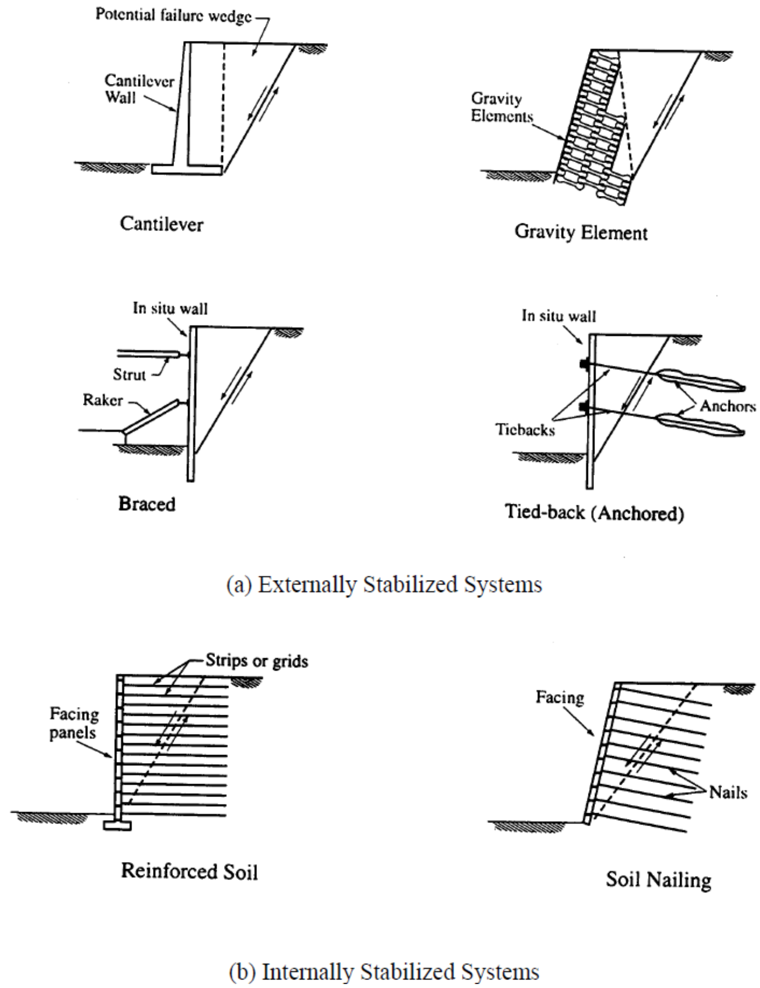

Figure 9-1: VARIETY OF RETAINING WALLS

Retaining wall systems that are routinely used as part of Oklahoma Transportation projects include cast-in-place (CIP) cantilever walls, mechanically stabilized earth (MSE) walls, and nongravity cantilevered walls. Geosynthetic reinforced soil (GRS) walls are used as temporary walls and as abutment walls for off-system bridges. A brief description of the various wall types common to ODOT projects is given in the following subsections.

9.5.1 CIP Gravity and Semi-Gravity Walls

Cast-in-place (CIP) gravity walls have been used as low-height retaining walls for roadway cut and fill applications. They are generally trapezoidal in shape (Figure 9-2) and are generally constructed of unreinforced or minimally reinforced mass concrete. CIP gravity walls are rigid walls that rely entirely on their self-weight to resist overturning and sliding.

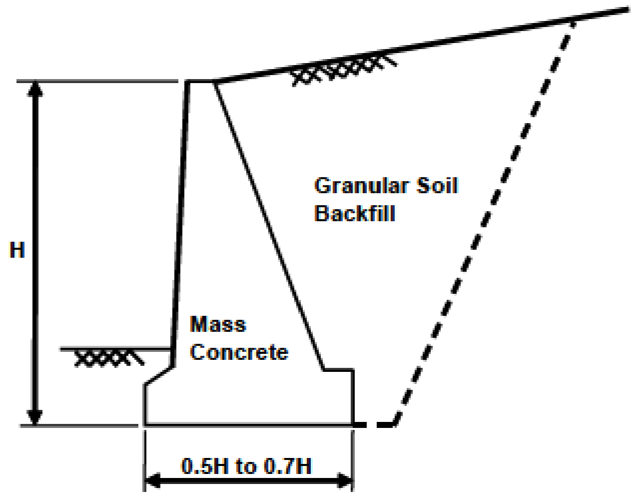

Figure 9-2: CAST-IN-PLACE GRAVITY WALL

CIP semi-gravity walls are commonly used to retain earth in roadway cut fil, and in bridge abutment applications. They can also be used in cut situations, but for these applications a temporary support system is often needed. In addition to its own weight, this type of wall uses its structural bending action to resist lateral forces on the wall. CIP gravity walls include cantilever, counterfort, and buttress walls.

At sites with competent soils, the base of the CIP wall can be designed as a spread footing bearing directly on foundation soils. At locations where bearing resistance or settlement is of concern, the CIP wall can be supported by deep foundations, with the base of the wall acting as the pile cap.

Advantages of CIP walls include:

- well-established design procedures and construction methods
- versatility (can be constructed with or without deep foundations)
- minimal wall movements
- resistance to degradation
- contractors have more control over construction schedule as compared with MSE walls, which often require dependence on third-party fabricators

Some limitations or disadvantages of CIP walls include:

- construction is labor intensive and relatively time consuming
- “Bottom-up” sequence of construction is required. This commonly leads to the need for temporary excavation support during construction, a larger work area footprint, and increased excavation and backfill quantities
- low tolerance to differential settlement and higher cost

9.5.1.1 CIP Cantilever Walls

CIP cantilever walls are the most common type of CIP walls constructed on ODOT projects. They are constructed in the form of an inverted "T" wherein the projecting structural members act as cantilever elements. The soil resting on top of the heel of the wall contributes to overturning resistance, while the structural properties of the system produce the cantilever effect. Thus, cantilever walls are considered semi-gravity walls.

Due to their simplistic design and construction processes, these walls are often more economical than other wall types when the retained height is relatively small. Although they are generally suitable for wall heights up to 30 ft, cantilever walls are most common for applications involving less than about 100 yd^2 in wall surface area or less than about 10 ft of wall height as they usually become less economically feasible than other wall types when exceeding these dimensions.

9.5.1.2 CIP Wall Construction

As mentioned above, construction of CIP walls can involve substantial labor and material costs relative to other wall types. The typical construction process involves site excavation (and installing any temporary shoring), foundation preparation (including removing unsuitable materials, leveling, and proof-rolling the foundation area), placing reinforcing steel and formwork for concrete, then placing the concrete. It is common to place concrete in two stages, with the first stage being the concrete footing, and the wall stem in the second stage. Following removal of formwork, drainage systems are placed behind the wall, followed by placement and compaction of select backfill in front of and behind the wall. For cases where temporary shoring is required, cost can become even more of a factor in wall type selection.

9.5.1.3 CIP Wall Design

Design of CIP walls involves evaluating both the structural integrity as well as the external stability of the structure. Concrete and steel are designed to resist bending, compression, and tensile forces within the wall, and the external stability of the wall is achieved when requirements for sliding, bearing resistance, and limiting eccentricity (failure due to overturning) are met. The overall design process is iterative in nature and is based on pertinent project requirements and constraints including wall geometry, loads, performance criteria, and construction constraints. Appropriate design parameters are selected based on information and recommendations available in the geotechnical report and required load and resistance factors, and structural and external stability evaluations are then performed based on those parameters. Often the initial assumptions for wall geometry are iterated upon until an optimal design is found. For more information regarding the design of CIP walls, refer to the most current version of the AASHTO LRFD Bridge Design Specifications.

9.5.2 Modular Gravity Walls

The purpose of this section is to introduce the concept of modular gravity walls and to discuss their use in Oklahoma.

Although somewhat less common to on-system ODOT projects, modular gravity walls have been used in a variety of applications, including highways, bridges, railroads, channels, dikes, and others. Common types of modular gravity walls include gabion walls (comprising rock-filled wire baskets), concrete module walls, bin walls, and crib walls. Of these common types of modular gravity walls, gabion walls and concrete module walls are the most common on ODOT projects.

As mentioned previously, the rise of unique products and technologies in recent decades has resulted in various proprietary wall systems available for construction. There are some proprietary systems, such as the so-called "Gravix Precast Wall System" or the "T-wall", that combine features of both the modular gravity wall and the MSE wall.

Advantages of modular gravity walls include low cost, fast and easy construction, relatively low labor, wall erection unaffected by temperature, "plant-controlled" quality of prefabricated units, tolerance to differential settlements, no need for architectural finish, and unit can be disassembled and re-used economically. Disadvantages include the requirement for bottom-up construction, potential corrosion of exposed metallic elements (e.g., gabion walls), required storage space at site for larger prefabricated units, possible unavailability of required fill (e.g., rock fill for gabion walls), susceptibility to vandalism (wire baskets can be damaged more easily than concrete structures), not suitable for use with deep foundations, and many available items are proprietary.

Design and construction considerations for modular gravity walls have many similarities to those of CIP walls, with the exception that evaluation of the interaction of the individual units is also involved, and the engineering properties of those units differ from those of CIP wall materials. Design and construction of modular gravity walls are discussed further in Earth Retaining Structures (Reference 1), and in the most current version of AAHSTO LRFD Bridge Design Specifications.

9.5.3 MSE Walls

Mechanically Stabilized Earth (MSE) walls are constructed by placing alternating layers of reinforcing elements and compacted backfill behind a facing element. The soil and the structural elements act together to form a composite structure that constitutes the wall. This composite structure is flexible and can generally accommodate large horizontal and vertical movements without excessive structural distress. MSE walls are typically constructed in fill situations.

The modern methods of soil reinforcement for retaining wall construction originate from France in the 1960s. The first wall to use this technology in the U.S. was in the 1970s. Since the 1980s, MSE walls have been used with increasing frequency in Oklahoma due to their many advantages. The principal components of an MSE wall are the retained and selected backfills, reinforcing elements, and facing (Figure 9-3).

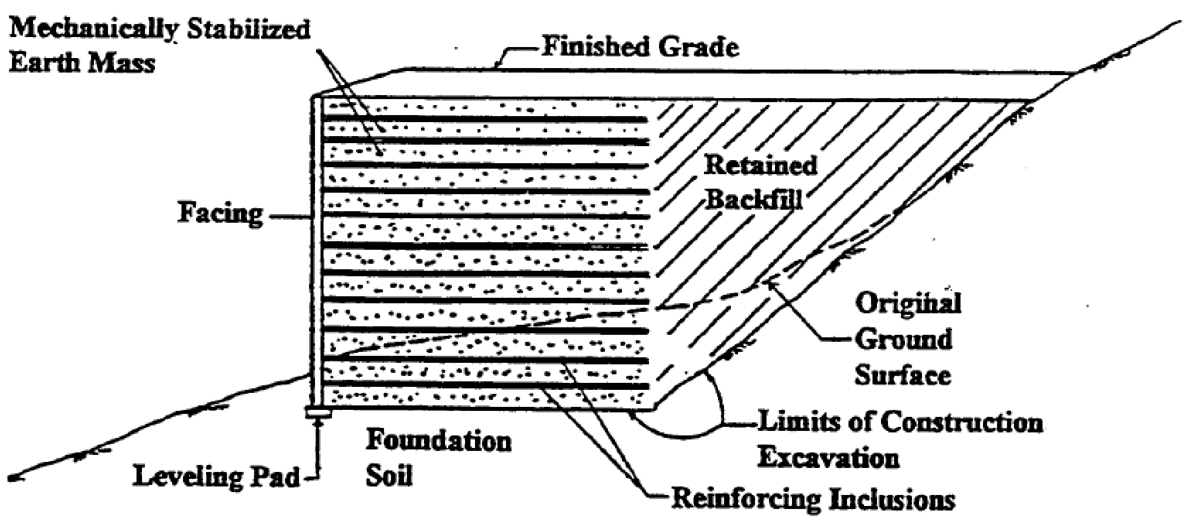

Figure 9-3: PRINCIPAL COMPONENTS OF A MECHANICALLY STABILIZED EARTH WALL

Retained backfill is the fill material located between the mechanically stabilized soil mass and the natural soil. The select backfill is used to construct the mechanically stabilized earth mass and is required for durability, good drainage, constructability, and good soil reinforcement interaction. The facing is not technically part of the composite structure, but it prevents backfill erosion at the wall face. Common facings include precast concrete panels, dry cast modular blocks, metal sheets, gabions, welded wire mesh, shotcrete, and wrapped sheets of geosynthetics.

Like modular gravity retaining wall systems, within the MSE wall construction industry, essentially all manufactured MSE wall components are proprietary in nature, each with its own unique attempt to provide the most value within their retaining wall system. Such components can include the reinforcement elements, the reinforcement connections, the wall facing elements, and the construction methods.

MSE walls do not require a structural foundation. A leveling pad of unreinforced concrete is used to serve as a guide for facing panel erection but is not intended as a structural foundation support.

Figure 9-4 illustrates common applications of MSE walls.

MSE walls have many advantages relative to conventional reinforced concrete and concrete gravity retaining walls, including:

- simple and rapid construction procedures, without need for large equipment
- less site preparation required
- less space in front of structure for construction operations
- rigid foundations not required due to tolerance of MSE structures to deformations
- technical versatility allows them to be constructed to heights of up to 100 ft
- flexibility and capability to absorb deformations due to poor subsoil conditions in the foundations
- precast concrete facing elements for MSE walls can be made with various shapes and textures (with little extra cost) for aesthetic considerations
- The relatively small quantities of manufactured materials required, rapid construction, and competition among owners of the different proprietary MSE wall systems have resulted in a cost reduction relative to other types of retaining walls, making MSE walls likely to be more economical than other wall systems for walls higher than about 10 ft or where deep foundations would be required for a CIP wall.

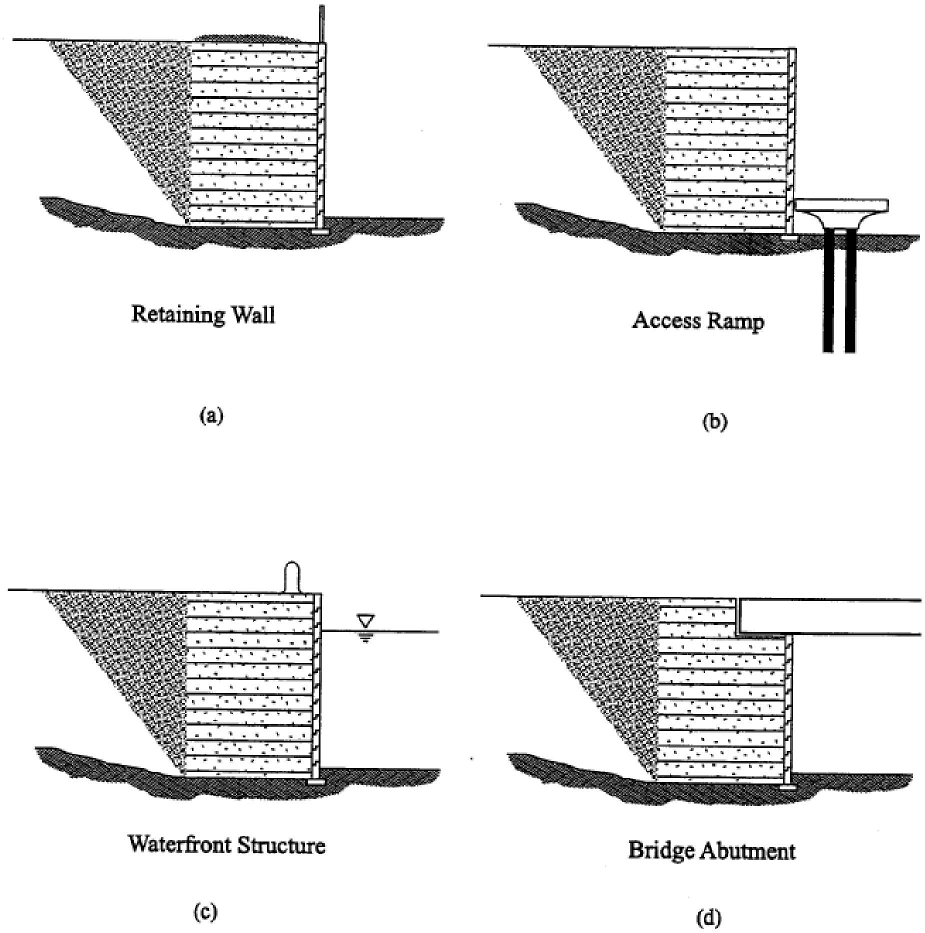

Figure 9-4: COMMON APPLICATIONS OF MSE WALLS

(a) Retaining wall; (b) Access Ramp; (c) Waterfront Structure; (d) Bridge Abutment (REFERENCE 4)

The following general limitations may be associated with MSE walls, although these limitations may be addressed as part of detailed design:

1. MSE walls require a relatively large space behind the wall face to obtain enough wall width for internal and external stability.

2. MSE walls require select granular fill; costs for importing suitable fill can be high at sites where granular soils are not readily available.

3. Suitable design criteria are required to address corrosion of steel wall system elements, deterioration of geosynthetics due to ultraviolet rays, and potential degradation of polymer reinforcement in the ground, and MSE walls should not be used under the following conditions:

    a. when utilities other than highway drainage must be constructed within the reinforced zone, requiring cutting of reinforcement layers for future access for repair

    b. when floodplain erosion may undermine the reinforced fill zone, or where depth to scour cannot be reliably determined

9.5.3.1 Materials for MSE Walls

There are a variety of material types and formats of those materials used in different MSE wall systems. To follow is a summary of MSE wall materials.

1. Backfill. Backfill material should generally meet ODOT specifications for granular backfill to ensure durability, good drainage, constructability, and to exhibit proper engineering properties. For applications where inundation of the wall is anticipated, design for more rapid drainage requires use of crushed stone. For applications where settlement is of concern, lightweight backfill in the form of lightweight cellular concrete or lightweight foamed glass aggregate can be considered.

2. Reinforcing Elements. Reinforcing elements are supplied by the wall system supplier. They may be metallic or nonmetallic and may be linear unidirectional (steel or coated geosynthetic strips over a load-carrying fiber), composite unidirectional (grids or bar mats—with spacing greater than 6 in.), or planar bidirectional (sheets of geosynthetics, welded wire mesh, and woven wire mesh—with spacing less than 6 in.). Reinforcing elements may also be characterized by extensibility. Inextensible reinforcements typically refer to steel reinforcements. Extensible reinforcements include geosynthetic reinforcements.

   - Steel Reinforcements. Most MSE wall systems using precast panels use steel reinforcements that are typically galvanized, including steel grids and steel strips.

   - Geosynthetic Reinforcements. Most modular block wall (MBW) systems use geosynthetic reinforcement, principally high-density polyethylene (HDPE), or PVC coated polyester geogrids, but geotextiles are also used.

3. Wall Facing Elements. These elements come in a variety of material types and configurations (see Figure 9-13), including the following:

   - Segmental Precast Concrete Panels. These units have a minimum thickness of 5.5 in. and are typically 5 ft high and 5 to 10 ft wide. Temperature and tensile reinforcement are required. Vertically adjacent units are usually connected with shear pins.

   - Dry Cast MBW Units. These units are specifically designed and manufactured for retaining wall applications. They typically weigh 75 to 100 lb. and are typically 8 in. high, 8 to 18 in. long, and 8 to 24 in. deep. Full height cores are filled with aggregate during erection, and units are normally dry stacked (without mortar). Vertically adjacent units are often connected with shar pins, lips, or keys.

   - Welded Wire Grids. This facing type is used for temporary structures and may be appropriate for permanent structures where difficult access or difficult handling complicates the use of heavier facing elements. Welded wire grids are galvanized, as needed, to comply with project design life constraints.

   - Geosynthetic Facing. Various types of geosynthetic reinforcement are looped around at the facing to form the exposed face of the retaining wall. These faces are susceptible to U-V degradation, vandalism, and damage due to fire. They are used in temporary soil retainage applications and have also been used as abutments for off-system bridges.

- Post-Construction Facing. Sometimes walls anticipating substantial settlements are constructed in multiple phases to allow substantial settlement to occur prior to placing the wall facing. The initial phase of construction can occur using geotextile, geogrid, or wire mesh as the facing. The final facing can consist of full-height tilt-up panels, shotcrete facing, cast-in-place facing, or conventional segmental precast concrete panels.

9.5.3.2 Design and Construction of MSE Walls

Design and construction considerations for MSE walls have similarities to those of other wall types. Design involves evaluating both internal stability (which addresses the volume of material within reinforced earth zone) and external stability (which addresses the volume of material outside the reinforced earth zone) of the wall. Design and construction of MSE walls are discussed in greater detail in Reference 2 and the most current version of AAHSTO LRFD Bridge Design Specifications.

The process of delivering retaining wall construction projects at ODOT is unique for proprietary wall systems in that for proprietary wall systems, ODOT does not typically provide a design for the wall as is the case with CIP walls. Instead, general minimum requirements are given including wall location, geometry, aesthetic requirements, MSE reinforcement lengths, and whether ground improvement is required to satisfy external stability based on the geotechnical investigation and recommendations. With these project requirements, contractors can then select from one of ODOT's approved proprietary wall systems and have a design prepared and submitted for ODOT's approval that addresses wall facing, reinforcing elements, and internal stability.

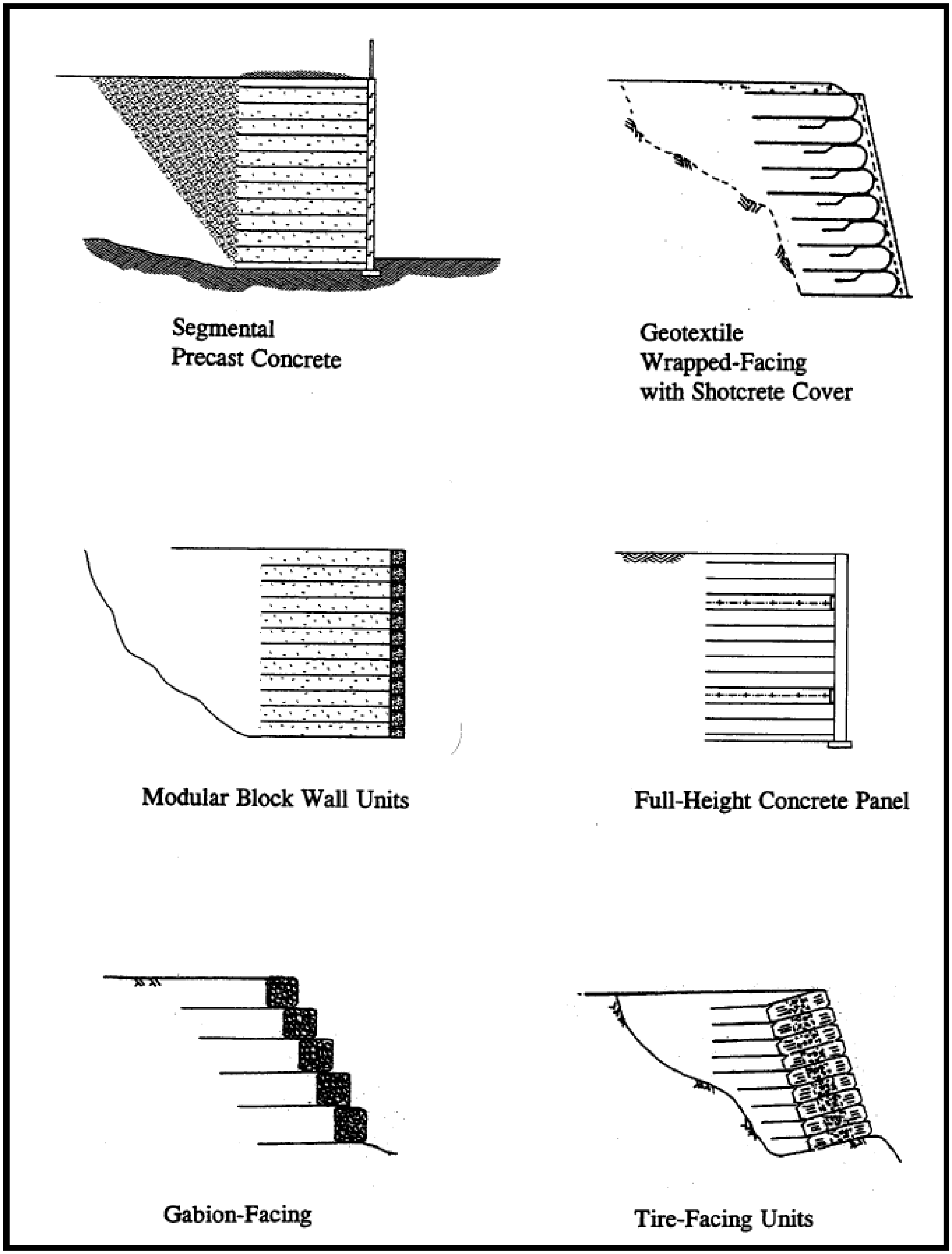

Figure 9-1: VARIOUS TYPES OF MSE WALL FACING

9.5.4 Non-gravity Cantilevered and Anchored Walls

Non-gravity walls are those walls that have no component contributing to the stability of the wall due to its self-weight. They are externally supported and rely on the bending resistance of a vertical structural element to resist the applied lateral loads. These walls may consist of discrete vertical elements (e.g., soldier piles) spanned by structural facing, or maybe a continuous structure (e.g., sheet pile wall, tangent pile wall, slurry wall, jet-grouted wall, and deep mixing method).

The wall configurations include nongravity cantilevered walls and structures with single or multiple levels of support. Wall support may be provided by struts or rakers on the exposed side of the wall, or soil anchors installed through the wall

Externally supported walls can be used for both temporary and permanent wall applications. Typical applications for such walls are listed below.

Walls for temporary excavation support include:

- cofferdams for footing construction
- excavation support for CIP wall construction
- grade separation during staged roadway construction
- excavation support for cut-and-cover roadway tunnels and culverts
- excavation support for utility trenches

Permanent wall applications include:

- roadway cuts
- roadway fill containment
- abutments
- cut-and-cover tunnel walls
- slide stabilization

The primary advantages of externally supported structural walls (as compared to other types of walls) include:

- top-down method of construction (i.e., wall elements are installed prior to excavation) which generally results in reduced ground displacement;
- when used as permanent walls, they require:
  - Reduced quantity of excavation
  - Reduced quantity of backfill
  - Reduced work area

- faster construction time
- can be used to provide a seepage barrier (e.g., for a depressed roadway)
- can effectively support large vertical and lateral loads

The primary limitations of externally supported structural walls (as compared to other types of walls) include:

- typically require more specialized construction techniques
- complicated soil-structure interaction may make analysis and design more difficult and costly
- performance of the completed walls is more dependent on the construction method and quality of work
- metallic components, especially anchors, if in contact with soil or rock, are more susceptible to corrosion

The following is an introduction to some externally supported structural walls that are most common to Oklahoma projects. For additional information on these and other types of externally supported walls, including the design and construction of these walls, refer to FHWA (2008) and the most current version of the AASHTO LRFD Bridge Design Specifications.

9.5.4.1 Sheet Pile Walls

A sheet pile wall consists of a series of interlocking sheet piles driven side-by-side into the ground to form a continuous vertical wall. Sheet pile walls are common for temporary earth support applications and for waterfront structures (as scour countermeasures) but can also be used as permanent walls for highway structures. Sheet piling can also be useful for stabilizing ground slopes and for cofferdam construction.

Although sheet piles can be of a variety of materials, typical sheet piles used on Oklahoma projects are made of steel due to their durability, their light weight, their ability to be driven and extracted with ease for reuse, and their high bending resistance. Limitations of steel sheeting include the inability to penetrate hard layers, limited length, and susceptibility to corrosion.

9.5.4.2 Soldier Pile and Lagging Walls

Soldier pile and lagging walls are commonly used for temporary excavation support in dense or stiff soils where sheet pile walls may not be suitable. They are also often used as permanent earth retaining structures.

Soldier pile and lagging walls consist of soldier piles usually set at 5-ft to 10-ft spacing and lagging which spans the distance between the piles. The lagging is used to retain the soil face from sloughing and transmit the lateral earth pressure to the soldier piles.

Soldier piles most commonly comprise rolled steel sections but can be almost any structural member such as pipe or channel sections, or even precast concrete. Wood is the most common type of lagging, but light steel sheeting or precast concrete are also common depending on the application.

Lagging may be omitted in hard clays, soft shales, and soils with natural cementation, provided the piles are spaced sufficiently close together, and with appropriate steps taken to protect against erosion and spalling of the soil at the face.

9.5.4.3 Tangent Pile Walls

A tangent or secant pile wall consists of a line of drilled shafts. If the shafts are tangent to each other, the wall is called a “tangent pile” wall. If the shafts overlap each other, the wall is called a “secant pile” wall. Another variation of this wall type is called an “intermittent” wall in which the shafts are installed at a spacing exceeding the shaft diameter; this type of wall can be considered only if the ground is sufficiently stable for the application, or if secondary elements, such as in-place grouting, are used to provide a continuous wall.

Tangent pile walls are stiff continuous walls that are constructed by the top-down method. These walls can be used when it is necessary to minimize groundwater lowering outside the excavation or to reduce ground displacements. Tangent pile walls can be used for either temporary or permanent ground support.

Tangent pile walls can be particularly advantageous in conditions that simplify drilled shaft construction, such as in relatively firm cohesive soils which not only provide good lateral support for the shafts but also resist caving during excavation. The drilled shafts are constructed in a staggered pattern to avoid disturbing relatively fresh concrete in an adjacent shaft. Sometimes, reinforcement is provided in every shaft, but it is more common to size steel such that it is required only in alternate shafts so as to reduce the amount of steel cutting required.

The principal advantage of tangent pile walls relative to the other externally supported walls is the ability to be constructed using conventional drilled shaft excavation and procedures. Limitations include rough and irregular exposed face and more labor-intensive connections for bracing members, ground anchors, or wall precast wall facing.

9.5.5 <u>In-Situ Reinforced Walls</u>

In-situ reinforced walls are constructed from the top-down to support temporary and permanent excavations. Construction of these walls involves insertion of reinforcing elements in the in-situ soil to create a composite earth structure.

Soil nail walls are the most common application of in-situ reinforced walls on ODOT projects. However, there exist other types of in-situ walls, such as micropile walls. Information regarding micropile walls can be found in FHWA (2008).

The main components of a soil nail wall are the in-situ material, the reinforcing inclusions, and the wall facing. The reinforcing inclusions typically consist of metal bars. Shotcrete, welded-wire mesh, cast-in-place concrete, or precast concrete panels are typically used for the facing. These walls can be used in a wide range of ground conditions.

Soil nail walls have been used for a variety of applications including temporary and permanent walls for excavations in urban areas, cut-slope retention for roadway widening and construction of depressed roadways, support of existing bridge abutments for roadway widening projects, landslide protection and stabilization of natural slopes, and repair or reconstruction of existing retaining walls.

Not all soil conditions are favorable to soil nailing. Installing soil nails in unfavorable conditions can make soil nails too costly when compared with other techniques. Soil nailing has proven economically attractive and technically feasible when:

- the soil in which the excavation is constructed can stand unsupported in a 3-ft to 6-ft high vertical for one to two days

- all soil nails within a cross section are located above the groundwater table

- if the soil nails are below the groundwater table, the groundwater does not adversely affect the face of the excavation, the bond strength of the interface between the grout and the surrounding ground, or the long-term integrity of the soil nails (e.g., the chemical characteristics of the ground do not promote corrosion)

It is also advantageous, but not required, if ground conditions allow drillholes to be advanced without requiring drill casing and for the drillhole to remain unsupported for a few hours to allow for installation of the nail bars and grouting of the hole.

Soil conditions are presumed to be favorable for soil nail walls when results from field tests indicate competent soils. For example, stiff to hard fine-grained soils, dense to very dense granular soils with some apparent cohesion, and weathered rock with no weak planes are all considered well suited for soil nailing applications.

Dry, poorly graded cohesionless soils; soils with high groundwater; soils with cobbles or boulders; highly corrosive soil; weathered rock with weakness planes or karst; or soft to very soft fine-grained soils are all unfavorable soil types or ground conditions for soil nails. Soil nail walls may be built in these conditions, but with higher design and construction costs when compared with favorable ground conditions or other wall types. Engineered fill and residual soils, although not ideal, are conditions intermediate to the more favorable and less favorable conditions for soil nail wall construction.

Soil nail walls have advantages when compared to alternative top-down construction techniques, including:

- less disruptive to traffic and to the environment

- structural elements do not extend below bottom of the excavation

- easy adjustments of nail inclination and location can be made when obstructions are encountered

- overhead requirements are small. soil nails do not require installation of soldier piles

- relatively small and mobile equipment required enables practical installation at more remote sites

- soil nail walls are relatively flexible and can accommodate significant total and differential settlements

- are usually more economical than conventional concrete gravity walls when conventional soil nailing construction procedures are used

- shotcrete facing is typically less costly than the structural facing required for other wall systems

Some potential limitations of soil nail walls include:

- may not be appropriate for applications where very strict deformation control is required, as the system requires some soil deformation to mobilize resistance (post-tensioning of soil nails can overcome this shortcoming in most cases, but this step increases project cost)
- location of utilities may restrict location, inclination, and length of soil nails in the upper rows
- not well suited where substantial groundwater seepage may occur
- permanent soil nails require permanent, underground easements
- construction of soil nail walls requires specialized and experienced contractors

A more comprehensive coverage of soil nail walls is provided in Geotechnical engineering Circular No. 7 (Reference 5), and FHWA (2008).

9.6 WALL SELECTION

Prior to the 1970s, the predominant types of earth retaining walls for permanent structures were gravity and cantilever. Selection of a wall system in 2023 is considerably more complex because the number of wall types available has increased significantly.

This chapter presents a systematic wall system evaluation and selection process. The objective of wall selection is to determine the most appropriate wall type that is cost-effective, practical to construct, stable, and aesthetically and environmentally consistent with project requirements.

Factors affecting wall selection:

- Ground type
- Groundwater
- Construction considerations (availability of materials, equipment, etc.)
- Speed of construction
- Right-of-way
- Aesthetics
- Environmental concerns
- Durability and maintenance
- Tradition
- Contracting practices
- Cost

9.7 REFERENCES

References specifically cited in Chapter 6 are numbered below, followed by additional pertinent technical information sources presented alphabetically by author.

1. **FHWA**. *Geotechnical Engineering Circular No. 2 – Earth Retaining Systems*. Federal Highway Administration. Washington, DC. FHWA-SA-96-038. February 1997.

2. **O'Rourke, T.D. and Jones, C.J.F.P.** *Overview of Earth Retaining Systems: 1970 – 1990*. Ithaca Section, ASCE, Cornell University. Ithaca, NY. Design and performance of earth retaining structures, proceedings of a conference. June 18-21, 1990.

3. **Teng, W.C.** *Foundation Design*. Prentice-Hall, Inc. January 1, 1962.

4. **FHWA**. *Geotechnical Engineering Circular No. 11 - Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes.* Federal Highway Administration. Washington, DC. Volume I. FHWA-NHI-10-024. November 2009.

5. **FHWA.** *Geotechnical Engineering Circular No. 7 - Soil Nail Walls.* Federal Highway Administration. Washington, DC. FHWA-IF-03-017. March 2003.

6. **AASHTO.** *LRFD Bridge Design Specifications.* American Association of Highway and Transportation Officials. Washington, DC. 7th Edition. January 1, 2015.