State of Alabama
Geotechnical Manual

Alabama Department of Transportation

Bureau of Materials and Tests

Geotechnical Division

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1.0 Purpose

1.1 Purpose and Scope

The purpose of this manual is to serve as a guidance document for the performance of geotechnical work for review by the Alabama Department of Transportation (ALDOT) Geotechnical Division. Information and guidance presented in this manual is intended to aid the Engineer in adequately scoping geotechnical explorations, selecting appropriate field and laboratory testing methods, and to direct the Engineer to suitable references for geotechnical analysis methods. Except where specified, the information is presented as guidance and is not exhaustive of the available exploration or analysis methods. Use of this manual does not relieve the user of the responsibility for the results of geotechnical explorations, design of foundation components, or geotechnical activities represented herein.

For purposes of this manual, the term *Engineer* refers to either the State/Assistant Geotechnical Engineer or the Consultant hired to perform the work for the state. The Engineer has primary responsibility for the scoping and execution of the geotechnical exploration, evaluating alternatives, performing analyses, developing recommendations, issuing reports, and providing other support as requested by the Geotechnical Division.

The Consultant will act as the Engineer when contracted by the state for a project. Information/requests from the Consultant to other ALDOT offices as noted in this manual (such as the Bridge Bureau or the Right-of-Way [ROW] Office) will be through the Geotechnical Division. Other information regarding the consulting process is provided in Section 6.2.

This manual is divided into the following sections:

- Section 2 – Overview of the Geology of Alabama: This section provides an overview of the geology of Alabama, divided into the five physiographic provinces in the state, each of which has unique geology and topography. Typical geotechnical considerations for each of the provinces and for the state in general are noted.
- Section 3 – Geotechnical Explorations and Data Collection: This section provides general guidance on the planning and execution of geotechnical explorations performed for the Geotechnical Division, including:
    - Section 3.1 – Planning of Geotechnical Explorations: Guidance on performing site reviews (data review and field reconnaissance) and the development of Geotechnical Exploration Plans.
    - Section 3.2 – Field Exploration and Laboratory Testing Methods: Presents the more commonly employed field exploration methods and laboratory tests.
    - Section 3.3 – Field Exploration Procedures and Considerations: Outlines procedures and requirements for the execution of field explorations.
- Section 4 – Project Types and Scope of Work Guidance: Provides guidance on scoping, geotechnical analyses, and reporting for typical geotechnical project types:
    - Section 4.1 – Current Practice and Overview of Load and Resistance Factor Design (LRFD)
    - Section 4.2 – Bridge Foundations
    - Section 4.3 – Bridge Culverts
    - Section 4.4 – Retaining Walls
    - Section 4.5 – Slope Studies

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  - Section 4.8 – Sign, Lighting, and Signal Pole Foundations
  - Section 4.9 – Sound Barrier Walls
  - Section 4.10 – Other Geotechnical Considerations: For conditions that, if encountered, may require additional exploration, analysis, and/or recommendations (soft soils, ground improvement, sinkholes, rockfall, mine studies, and erosion control).
- Section 5 – Information and Report Management
- Section 6 – Geotechnical Consultant Information
- Section 7 – Construction Submittals

1.2 Standards and References

ALDOT Specifications, Drawings, and Procedures: The Engineer will obtain and review copies of the current versions of ALDOT specifications, drawing, and procedures:

- *Standard Specifications for Highway Construction:* https://www.dot.state.al.us/publications/Construction/Specifications.html
- Standard and Special Drawings: https://alletting.dot.state.al.us/Docs/Standard_Drawings/StdDrawingSelect.htm
- Procedures: https://www.dot.state.al.us/publications/Materials/TestingManual/ProcedureTopics.html

Methods and Procedures: AASHTO (American Association of State and Highway Transportation Officials) and ASTM (American Society for Testing and Materials) references for specific tests or methods are provided in relevant sections of this manual.

Background Information and Historical References: Geology and soils information is available from a variety of sources. As a starting point, the Engineer should refer to those sources as given in Section 2 of this manual.

- Geologic Survey of Alabama (GSA): https://www.ogb.alabama.gov/gsa/geologic/mapping
- US Department of Agriculture (USDA) county level soil survey reports: https://www.nrcs.usda.gov/wps/portal/nrcs/surveylist/soils/survey/state/?stateId=AL
- USDA soil survey mapping tools: https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

Geotechnical Engineering References: The Engineer should base their recommendations, in part, from the guidance, analysis methods, and requirements given in the appropriate reference materials. Primary references are provided below with others noted in relevant sections of this manual.

Additionally, the Engineer may identify additional current reference materials suitable for the project.

- AASHTO:
    - *AASHTO LRFD Bridge Design Specifications*
    - *AASHTO R 13 – Standard Practice for Conducting Geotechnical Subsurface Investigations*
    - *AASHTO R 27-01 – Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications*
    - *AASHTO Manual on Subsurface Investigations*
- Federal Highway Administration (FHWA):
    - *Geotechnical Engineering Circular (GEC) 4 – Ground Anchors and Anchored Systems* (FHWA-IF-99-015)
    - *GEC 5 – Geotechnical Site Characterization* (FHWA-NHI-16-072)
    - *GEC 6 – Shallow Foundations* (FHWA-IF-02-054)
    - *GEC 7 – Soil Nail Walls* (FHWA-NHI-14-007)
    - *GEC 10 – Drilled Shafts: Construction Procedures and LRFD Design Methods* (FHWA-NHI-18-024)
    - *GEC 11 – Design and Construction of Mechanically Stabilized Earth Wall* (FHWA-NHI-10-024 and FHWA-NHI-10-025)
    - *GEC 12 – Design and Construction of Driven Pile Foundations* (FHWA-NHI-16-009, FHWA-NHI-16-010, and FHWA-NHI-16-011)
    - *GEC 13 – Ground Modification Methods Reference Manual* (FHWA-NHI-16-027 and FHWA-NHI-16-028)
    - *Micropile Design and Construction Reference Manual* (FHWA-NHI-05-039)
    - *Rock Slopes Reference Manual* (FHWA-HI-99-007)
    - *Soils and Foundations Reference Manual* (FHWA-NHI-06-088 and FHWA-NHI-06-089)
    - Some FHWA publications are available at: https://www.fhwa.dot.gov/engineering/geotech/library_listing.cfm
- Other references that may be useful to the Engineer:
    - Auburn University, Highway Research Center, 2019. *Evaluation of Landslides Along Alabama Highways.*
    - National Cooperative Highway Research Program (NCHRP), 1998. *NCHRP Report 408: Corrosion of Steel Piling in Non-Marine Applications.*
    - Post Tensioning Institute (PTI), 2014. *Recommendations for Prestressed Rock and Soil Anchors.*
    - Transportation Research Board (TRB), 1996. Special Report 247: *Landslides Investigation and Mitigation.*
    - TRB, 2012. *Rockfall Characterization and Control.*
    - Geotech Tools: https://www.geoinstitute.org/geotechtools/login

1.3 Example Forms, Tables, and Other Documents

Example forms and tables for the documents listed below are provided in Appendix A.

- A-1: SPT/Rock Core Boring Assignment Table
- A-2: Survey Request Form
- A-3: Pre-Construction Investigation Activities Environmental Permits Checklist
- A-4: Driven Pile Foundation Recommendations Table
- A-5: Drilled Shaft Foundation Recommendations Table
- A-6: Landslide Reporting Form
- A-7: Topsoil Survey Summary Table
- A-8: Resilient Modulus Summary Table
- A-9: Soil Classification Summary Table
- A-10: Design Resilient Modulus by AASHTO Soil Classification

2.0 Overview of the Geology of Alabama

The geology of Alabama encompasses rock from the ancient Paleozoic age (greater than 2.5 billion years ago) to the relatively recent Holocene age (~1,800 years ago). The distribution of the geologic formations found in Alabama is controlled by past patterns of ocean coverage, sedimentation, igneous intrusion and volcanism, tectonic forces of folding and faulting, and subsequent erosion.

If available for the project site, the Engineer should refer to the 1:24,000 scale Quadrangle Series (QS) maps issued by the Geological Survey of Alabama (GSA) at https://www.gsa.state.al.us/ogb/publications. Additionally, Alabama's geology has been mapped at a 1:250,000 scale (most recently in 1988) with more detailed geologic mapping available on a series of United States Geological Survey (USGS) topographic quadrangle maps at a 7.5-minute (1:24,000) scale, both of which are available from the Geologic Survey of Alabama.

Near surface site-specific geology and soils information should be developed for each project site. This chapter provides an overview of the geology and soils of Alabama by physiographic province and an introduction to some of the geologic considerations for transportation projects in Alabama. Published sources such as ALDOT, GSA, and the USDA Soil Conservation Service provide a good site-level overview of geology and soils. Links to USDA Soil Survey reports and mapping tools are provided in Section 1.2.

2.1 Physiographic Provinces of Alabama

This report presents the geology of Alabama in terms of its five physiographic provinces. The terminology of physical landscapes follows a hierarchical format, similar to biological classifications. Physiographic regions are the first-order or largest-scale divisions and are based on rock type and age, geologic structure, and history. North America is divided into eight physiographic regions. Each physiographic region is subdivided into physiographic provinces. The physiographic provinces are further subdivided into physiographic sections with each section containing a similar pattern of landforms as other sections in the province and distinguished on the basis of geographic location. For this report, we will discuss the geology and soils of Alabama at the scale of the physiographic provinces; however, for individual projects, the Engineer should consult site-level geologic information for potential impacts on the project.

Alabama contains five distinct physiographic provinces as depicted on Figure 2-1 (Appendix B). The provinces divide the state into areas of similar geology and geomorphology (landforms influenced by the underlying geology). Soil types in Alabama follow the same general distribution. From south to north, the five physiographic provinces of Alabama are:

- East Gulf Coastal Plain,
- Piedmont Upland,
- Alabama Valley and Ridge,
- Cumberland Plateau, and
- Highland Rim.

Each of the geologic formations within these provinces have unique characteristics that influence landforms. Each of the provinces and some of their specific geomorphological features are described in greater detail in the references cited in Section 2.3. The following are outline descriptions of each of the five provinces and their generalized geology and soils.

2.1.1 *East Gulf Coastal Plain*

The East Gulf Coastal Plain (or simply, Coastal Plain) in Alabama is a portion of a larger physiographic region that includes over 42 million acres spanning over portions of five states from southwestern Georgia, across the Florida Panhandle, into Alabama, across Mississippi, and into the southeastern portions of Louisiana and Arkansas. In Alabama, this province occupies 60 percent of the state, including portions of 40 of Alabama's 67 counties. The East Gulf Coastal Plain's northern boundary is the "fall line," and its southern boundary is the Gulf Coast. The "fall line" refers to the approximate trace of the onlap of relatively younger marine deposits onto older rock units of the state's interior. The term "fall line" derives from northeastern states where it is marked by elevation changes and waterfalls with surface water flow towards the coast.

2.1.1.1 Geology of the East Gulf Coastal Plain

The geology of the East Gulf Coastal Plain (Figure 2-2, Appendix B) consists of rock units that are progressively younger to the south. The Coastal Plain developed on Mesozoic-aged to recent sedimentary rocks and sediment (from approximately 140 million years ago to the present). The unconsolidated units are composed mainly of sediments: gravels, sands, silts, and clays. The rock units are mainly composed of chalk (marl), sandstone, limestone, and claystone. The beds of unconsolidated sediments and rock units slope gently southward at approximately 40 feet per mile and are progressively younger from the fall line to the coast. Locally, higher elevations are underlain by more erosion-resistant material (sediment in some areas, sedimentary rock in others), and the lowlands are underlain by softer material.

Many cuestas (a ridge with a gentle slope [dip] on one side and a steep slope [scarp] on the other) are present in the East Gulf Coastal Plain landscape that trend roughly northwest-southeast. Northeast-facing slopes are steeper than the gentler southwest-facing slopes, which is an indication of the dip of the underlying beds. The difference in elevation, or relief, between the top of cuesta and the floor of the valley can be as much as 300 feet.

2.1.1.2 Soils of the East Gulf Coastal Plain

In general, the soils of the East Gulf Coastal Plain (Figure 2-3, Appendix B) are comprised of coastal marshes, flood plains and terraces, coastal plains, and prairies. Most of the soils in this area are derived from marine and fluvial sediments eroded from the Appalachian and Piedmont plateaus. The soils consist of five regions with each region containing specific characteristics that may be important for engineering projects:

- Upper Coastal Plains: Smithdale, Luverne, and Savannah soils are extensive in the Upper Coastal Plains. They have either a loamy or clayey subsoil and a sandy loam or loam surface layer. Savannah soils have a shallow fragipan (a subsurface soil layer that restricts water flow and root penetration).
- Lower Coastal Plains: Dothan and Orangeburg soils are extensive in the eastern part of the Lower Coastal Plains, while Smithdale and Bama soils are extensive in the western part of this region. The reddish soils of the Bama series are the official state soil of Alabama.
- Blackland Prairie: The Blackland Prairie is a curvilinear-shaped area of central and western Alabama known as the "Black Belt" due to of the dark surface colors of many of the soils. Some of the clayey soils have a high percentage of smectitic clays. Smectites are clay minerals that have a three-layer crystal structure that can absorb water within the inter-layer space. Smectitic clays are an important soil from a geotechnical engineering standpoint because they swell when wet and shrink and crack when dry.
- Major Flood Plains and Terraces: Major Flood Plains and Terraces soils are not extensive but are notable when they are found along streams and rivers, as they will replace the original underlying soil type. These soils are derived from alluvium deposited by the streams.

- Coastal Marshes and Beaches: The Coastal Marshes and Beaches soils are not extensive. Found on nearly level and level bottomlands, tidal flats, and beaches along the Mobile River, Mobile Bay, and the Gulf of Mexico, most of these soils are deep and poorly drained. Elevation in these soil areas is from sea level to a few feet above sea level.

2.1.1.3 Geotechnical Considerations for Transportation Projects

Due to the wide variety of subsurface conditions present, a similar variety of geotechnical issues unique to the different formations may need to be addressed on a project-specific basis. Outlined below are some of the usual geotechnical challenges encountered in the East Gulf Coastal Plain.

- Structure Foundations:
    - Due to the unconsolidated nature of most of the formations in the East Gulf Coastal Plain, most bridge foundations will bear on deep foundations supported by a combination of side shear and end bearing.
    - Likewise, scour can be a significant issue for stream and river crossings.
    - Artesian groundwater conditions are common in the East Gulf Coastal Plain and need to be investigated and addressed, particularly where drilled shaft foundations are planned.
    - In the more recent formations (i.e., southernmost), soft, underconsolidated clay layers may be present and should be explored to adequate depth for their consolidation potential and impact to bridge foundations, bridge culverts, and approach embankments. Ancient stream channels, valleys, etc., that were filled in with more recent overlying formations and/or alluvium can be particularly problematic and should be adequately explored.
- Slopes:
    - Stability of bridge approach embankments at water crossings can be an issue, even in the more consolidated marl formations (such as the Selma Chalk), due to pre-existing jointing in the formation. In such cases, residual material strengths (rather than peak strengths) may be more applicable to the design of the embankment slopes.
    - Due to the varying depositional conditions under which the soils were placed/deposited, interbedded clay, silt, and sand layers are common, particularly in the older (i.e., more northern) formations. Although the naturally occurring slopes in the East Gulf Coastal Plain can be gentle, these interbedded layers often lead to perched groundwater levels, which can lead to stability issues for even relatively gentle excavation slopes.
    - Consequently, geotechnical studies for projects on new alignment should include a detailed site reconnaissance with particular attention paid to signs of ongoing slope movement that could be exacerbated by excavation and/or embankment placement.
    - Due to the significant proportion of sand in some of the East Gulf Coastal Plain formations, erodibility of cut and embankment soils should be addressed where these particularly sandy soils are present.
- High-Volume Change Soils:
    - The soils overlying the Selma Chalk Formation ("Black Belt" soils) are typically residual in formation and subject to high volume change with changes in moisture content. The extent of these high-volume change soils will need to be identified for a site during the geotechnical exploration. Adequate measures will need to be provided to remove and replace or stabilize such soils. Where desiccation of such soils may extend deeper (such as where tree growth has

occurred), such removal/replacement may need to extend to depths on the order of 10 feet of the natural soil profile. Otherwise, encapsulation of deeper soils at a stable moisture content may be required to reduce the potential for shrinkage and swelling.

- Other high-volume change soils may also be identified locally outside of the formations known to contain such soils.

- Soft Soils and Marshes:
  - Soft soils and marsh deposits should be delineated. If a structure or embankment is proposed, methods for stabilizing the subgrade must be provided, (e.g., undercutting and replacement, aggregate piers/stone columns, pile supported embankments, etc.).
  - It should also be recognized that ancient, buried stream channels and valleys exist (particularly along existing stream and river valleys), and they typically have soils that vary significantly from the surrounding subsurface profile (i.e., contain softer soils).
- Subgrade and Embankment Soils:
  - With the exception of the high-volume change soils noted above, many, if not most, of the East Gulf Coastal Plain soils are adequate for embankment and roadway subgrade use.
  - However, particular attention should be paid to the presence of shallow groundwater if encountered within a few feet of (or above) the proposed roadway grade in cut sections. In such cases, recommendations will need to be provided to suppress the water table to at least two feet below the proposed subgrade (such as the use of aggregate blanket drains leading to collector pipes at the shoulders) for adequate stability.
- Miscellaneous:
  - Many of the East Gulf Coastal Plain formations have been surface mined for various minerals, such as clay, sand, and gravel. The initial field reconnaissance should include a review of historic aerial photos to check for such manmade conditions that may require additional stabilization recommendations.

2.1.2 *Piedmont Upland*

Moving northward, the East Gulf Coastal Plain physiographic province, described above, transitions to the Piedmont Upland (or simply, Piedmont). The Piedmont Upland occupies approximately nine percent of the state in a triangular area in east-central Alabama and includes the Ashland, Heflin, Phenix City, Auburn/Opelika, and Lake Martin areas. The Piedmont Upland consists of a plateau that slopes from the north (at elevations typically above 1,000 feet) to the south (at elevations of approximately 500 feet) where it comes into contact with the East Gulf Coastal Plain. The southern area of the Piedmont Upland is a plateau while the northern area is characterized by many northeast-trending, steep-sided ridges, which contain some of the highest peaks in Alabama, such as Mt. Cheaha, the state's highest point at approximately 2,407 feet, MSL (mean sea level). The boundary between the Piedmont Upland's rugged landscape and its flatter portions is the Brevard Fault Zone, which generally follows the course of the Tallapoosa River in the vicinity of the Lake Martin area.

More than half of the Piedmont Upland is drained by the approximately 200-mile-long Tallapoosa River and its tributaries. The system drains approximately 3,300 square miles in a series of relatively straight segments broken up by abrupt changes in direction. The Tallapoosa River's gradient (elevation drop per mile) increases at the fall line, the boundary between the Piedmont Upland and the East Gulf Coastal Plain. Four Alabama Power Company dams are stationed along this part of the river that generate power by taking advantage of the gradient changes.

2.1.2.1 Geology of the Piedmont Upland

The geology of the Piedmont Upland (Figure 2-4, Appendix B) developed on northeast-southwest trending belts of Precambrian to Paleozoic (around 1 billion years to approximately 300 million years ago) metamorphic rocks that are highly deformed and bordered by faults. The most common rock types of the Piedmont Upland are slate, phyllite, marble, quartzite, greenstone, schist, amphibolite, and gneiss, some of which are among the oldest rocks in Alabama. A characteristic feature of the Piedmont Upland is the formation of saprolite rock (or "rotten rock"), a type of rock where decomposition of the original rocks to depths of less than 20 to 40 feet alters the rock minerals while retaining their texture and structure.

The Piedmont Upland consists of two districts: the Northern Piedmont Upland and the Southern Piedmont Upland. The Northern Piedmont Upland is underlain by slate, quartzite, phyllite, marble, gneiss, and schist, and the rugged nature and high elevations of the northwestern part of the upland are caused by varying rates of erosion of these rock types. The highest elevations and greatest relief (changes in elevation) occur near the northwestern edge of the Northern Piedmont. Erosion-resistant quartzite comprise the high ridges whereas the flat areas at lower elevations, mainly in the vicinity of Sylacauga, are underlain by marble, which is easily eroded/chemically weathered. The Southern Piedmont Upland is underlain by schist, gneiss, and amphibolite. These rock types tend to erode more equally, resulting in the flatness of the Southern Piedmont Upland.

Historically, the Piedmont Upland was important for its mineral resources, including gold mined from the Northern Piedmont Upland. No extensive gold-mining operations remain in Alabama today. Several Piedmont Upland marble quarries exist near Sylacauga and Auburn. Most of the quarried material is used for crushed stone, but some of the white Sylacauga marble is mined in larger blocks (dimension stone) for use as architectural facing pieces or is ground into a fine powder for use as pharmaceutical additives and paper coatings. Sylacauga marble has been used in works of fine art and as building stone in prominent structures such as the Lincoln Memorial and the United States Supreme Court Building. Marble is the state rock of Alabama.

2.1.2.2 Soils of the Piedmont Upland

Most of the soils in the Piedmont Upland (Figure 2-5, Appendix B) are derived from granite, hornblende, and mica schists, where Madison, Pacolet, and Cecil soils are very extensive. These soils tend to have a red, clayey subsoil and a sandy loam or clay loam surface layer. Topography in this province is rolling to steep.

2.1.2.3 Geotechnical Considerations for Transportation Projects

Outlined below are some of the usual geotechnical challenges encountered in the Piedmont Upland.

- <u>Structure Foundations:</u>
    - Due to the underlying bedrock, most bridges will be founded on deep foundations bearing on bedrock. Care should be taken however to make sure the bedrock is continuous below the tip of the pile, drilled shaft, etc. It is not unusual to have significant thickness of soil present below the top of where rock is first encountered.
    - As with most formations, scour can be a significant issue for stream and river crossings.
    - For bridge culverts, particularly those to be covered with thick embankments, the subgrade for bridge culverts should be explored to bedrock due to the occasionally deep residual soils.
- <u>Slopes:</u>
    - Geotechnical studies for projects on new alignment should include a detailed site reconnaissance with attention paid to signs of historic or ongoing slope movement that could be exacerbated by further excavation and/or embankment placement.

    - In addition, bedrock can be expected to be encountered in cuts of more than a few feet in many areas. Therefore, borings should be drilled for each cut. Relatively steep rock cuts on the order of 0.25H:1V (horizontal:vertical) are stable at most locations not impacted by geologic structure, such as faults, though the stability of cut slopes in rock must be evaluated on a project-specific basis.
    - Due to the significant proportion of sand and silty soils in some of the Piedmont Upland, erodibility of cut and embankment soils should be addressed where these soils are present.
- Subgrade and Embankment Soils:
    - Most of the Piedmont Upland Province soils are adequate for embankment and roadway subgrade use.
    - As with most areas, attention should be paid to the presence of shallow groundwater if encountered within a few feet of (or above) the proposed roadway grade in cut sections. In such cases, recommendations will need to be provided to suppress the water table to at least two feet below the proposed subgrade for adequate stability.
- Miscellaneous:
    - The marble in the Sylacauga area is subject to deep weathering along the joints in bedrock due to the slightly acidic groundwater. These joints allow the overburden soils to migrate into the enlarged joints under the action of groundwater seepage resulting in cover-collapse sinkholes.
    - This sinkhole formation process is exacerbated by the deep drawdown of the groundwater table as part of the marble quarrying process and can extend a mile or more out from the quarry. Consequently, a careful review of data from aerial photos, lidar, etc., should be conducted to look for surface depressions, and particularly lines of depressions (lineaments), suggesting favored locations for existing or future sinkhole formation in areas underlain by carbonate rock.

2.1.3 *Alabama Valley and Ridge*

The Alabama Valley and Ridge (or simply, Valley and Ridge) occupies approximately nine percent of the state. It occurs as a roughly northeast-trending rectangular area in central and east-central Alabama. The Alabama Valley and Ridge is bounded by the Cumberland Plateau to the north and west, the Piedmont Upland to the southeast, and the East Gulf Coastal Plain to the southwest.

The Alabama Valley and Ridge is comprised of seven districts: Birmingham-Big Canoe Valley, Cahaba Ridge, Coosa Ridges, Cahaba Valley, Coosa Valley, Weisner Ridges, and Armuchee Ridges. Detailed discussion of the geomorphology of the seven districts is found in Nielson, 2007 (see Section 2.3). In summary, each of the districts within the Valley and Ridge have landforms controlled by folding, faulting, and differential erosion between the softer carbonate rocks and shales relative to the more erosion-resistant sandstones.

The Coosa River drains approximately 5,350 square miles in Alabama, most occurring in the Alabama Valley and Ridge Province. Meanders in the river are less pronounced in the southern area of the Alabama Valley and Ridge because the dolomites of the Knox Group contain varying amounts of chert that restrict the flow pattern of the river against ledges of the more resistant chert-bearing rock. Logan Martin Dam, located south of Pell City on the Coosa River, was built on one of these ledges. Together, the seven dams on the Coosa River produce more than 50 percent of the hydroelectric power supplied by Alabama Power.

The Red Mountain Formation outcrops on the slopes of Red Mountain in the Alabama Valley and Ridge, as well as along the slopes of the Big Wills, Murphrees, and Sequatchie Valleys of the Cumberland Plateau. The iron-ore deposits within the formation became the center of the iron and steel industry in Alabama during the late-nineteenth and early-twentieth centuries. The red color is imparted by hematite, a red iron-oxide mineral that was a driving force in the establishment and growth of the Birmingham area and it's economy during this time. Extensive and long-term mining took place primarily in and around the Birmingham area. Although Red Mountain iron ore was used briefly during the Civil War, its use did not become common until the late nineteenth century when coke (a gray, hard, porous material formed when coal is superheated to remove volatile materials) replaced charcoal as the main source of energy. Numerous coal seams, particularly the Pratt seam in the nearby Warrior basin, supplied a steady source of coal to be transformed into coke. Limestone and dolomite, used as fluxing (chemical cleaning) agents in iron manufacturing, are also abundant in the Birmingham-Big Canoe Valley. Due to the proximity of iron ore, coal for coke production, and limestone and dolomite for fluxing, mining of the Red Mountain ore continued until the early 1970s. Reportedly, a total of approximately 375 million tons of ore were mined.

2.1.3.1 Geology of the Alabama Valley and Ridge

The geology of the Alabama Valley and Ridge (Figure 2-6, Appendix B), which dictates its landscape of numerous zig-zagging ridges separated by deep, steep-sided valleys, developed on tightly folded and thrust-faulted rock layers. The Valley and Ridge consists of Paleozoic sedimentary rocks that range in age from Cambrian to Pennsylvanian, around 540 to 290 million years ago. The ridges are capped with relatively erosion-resistant Pennsylvanian sandstone belonging to the Pottsville Formation, while the valleys are formed in shale, limestone, and dolomite.

2.1.3.2 Soils of the Alabama Valley and Ridge

The soils of the Alabama Valley and Ridge (Figure 2-7, Appendix B) are derived from of a series of mountains upheld by sandstone or shale along with valleys whose soils are derived from shales and carbonate rocks such as limestone and dolomite. The more level areas are dominated by Nauvoo, Hartsells, and Wynnville soils, which were formed in residuum from sandstone. The more rugged portions of the Alabama Valley and Ridge are dominated by soils such as Montevallo and Townley, which were formed in residuum from shale. These soils have either a very channery loam (channery is the characteristic whereby at least 15 percent pieces of the soil is composed of thin, flat rock) or a clayey subsoil and a silt loam surface layer.

2.1.3.3 Geotechnical Considerations for Transportation Projects

Outlined below are some of the usual geotechnical challenges encountered in the Alabama Valley and Ridge.

- <u>Structure Foundations:</u>
    - Due to the underlying bedrock, most bridges and other major structures will be founded on deep foundations bearing on bedrock. Care should be taken however to make sure the bedrock is continuous below the tip of the pile, drilled shaft, etc. As noted above, there are a wide variety of bedrock types present, each of which may lend itself to a different foundation type. For instance, drilled shafts and driven piles are typically used where sandstones and shales are present, but driven piles and micropiles are better suited to karst (carbonate rock, such as limestone and dolostone) bedrock.
    - The residual overburden can be several tens of feet thick, particularly over the carbonate rock formations, and these soils are often soft enough to consolidate under embankment loads. Thus, significant drag load can be imparted to bridge abutment piles, and recommendations for such additional loads should be provided.
    - For bridge culverts, particularly those to be covered with thick embankment, and due to the sometimes deep residual soil, the subgrade for bridge culverts needs to be explored to bedrock.

  - Although settlement can be an issue for bridge culverts in the Alabama Valley and Ridge, the opposite can also be an issue. If the culverts bear on or very close to non-yielding bedrock and the embankment heights are more than 20 feet or so, the culverts can experience quite high pressures. In such cases, special backfill for the culverts will need to be provided.

- Slopes:
  - Due to the prevalence of mountains and valleys in the Alabama Valley and Ridge, numerous and tall cut slopes both in soil and rock are common. In addition, due to the tectonic nature of the ridge forming process, the bedrock units often dip at angles of 30 to 40 degrees with corresponding cross joints. Rock cuts must account for this bedding and jointing pattern as rock cuts where the roadway alignment is near parallel (i.e., within 25 degrees) to the strike of the bedding planes are often marginally stable to unstable.
  - The slopes of the various hills in the Alabama Valley and Ridge Province are typically covered with a thick layer of colluvium. These colluvial soils are quasi stable and should not be further steepened without careful analyses.
  - The residual soil overburden can vary from a few feet to upwards of 50 feet or more thick. The residual soils overlying the carbonate rock formations are typically highly plastic, very cherty, and very stiff near the ground surface to very soft near bedrock, and can be, at least initially, cut on slopes of 2H:1V. However, long term stability of soil slopes should be based on the soil's effective (drained) shear strength, which usually limits them to slopes of 3H:1V or flatter.
  - Geotechnical studies for projects on new alignment should include a detailed site reconnaissance with attention paid to signs of historic or ongoing slope movement that could be exacerbated by further excavation and/or embankment placement.
- Subgrade and Embankment Soils:
  - Soils overlying the sandstone and shale formations, such as the Pottsville and Parkwood formations, are typically suited for embankment and subgrade use.
  - The plastic residual soils overlying the carbonate rock formations should be encapsulated with low plasticity soils if used in embankments. Long term consolidation of the more plastic soils should also be addressed.
  - As with most areas, attention should be paid to the presence of shallow groundwater that is encountered within a few feet of (or above) the proposed roadway grade in cut sections. In such cases, recommendations will need to be provided to lower the groundwater level to at least two feet below the proposed subgrade to allow for adequate stability.
- Miscellaneous:
  - As noted above, mining of iron ore and coal has been extensive in the Birmingham area, much of it in the Alabama Valley and Ridge.
  - The coal mining has taken place in the Pottsville Formation, which typically contains enough thick sandstone units to limit upward collapse of the overlying rock (i.e., stoping) even for relatively shallow mines where the coal extraction ratio is less than 50%, such that surface ground subsidence is not common.
  - The iron ore mining, on the other hand, has led to surface subsidence/collapse due to both thinly bedded rock layers in the Red Mountain Formation and the relatively shallow occurrence of some of the mines. A more detailed study of the subsurface conditions and potential for

settlement above iron ore mines is warranted as opposed to coal mines in the Alabama Valley and Ridge.

- Proposed construction on new alignment in the Alabama Valley and Ridge underlain by the coal bearing measures and/or iron ore measures should include a review of available mine maps for mining activity that could impact the project. We note that near vertical air ventilation shafts several feet in diameter are also prevalent above many of these coal and iron ore mines.
- As part of the iron making process, significant quantities of byproducts became available, such as slag from both the blast furnaces and open-hearth furnaces, etc. These materials have been widely used for embankments, particularly in the Birmingham area. Where present, conditions should be investigated for adverse corrosion potential on steel foundation piles, tiebacks, soil nails, etc., and adequate corrosion protection provided. Additional information is provided in *NCHRP Report 408: Corrosion of Steel Piling in Non-Marine Applications* (NCHRP, 1998) and *AASHTO Procedure R 27-01: Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications*. In addition, certain types of slag (e.g., slag from the open-hearth furnace process) are known to swell when exposed to moisture.

2.1.4 *Cumberland Plateau*

The Cumberland Plateau occupies approximately 15 percent of the state and occurs as a roughly northeast-oriented rectangular area in central and northeastern Alabama, encompassing Jackson, DeKalb, Marshall, Blount, Cullman, Winston, Walker, and Jackson counties and continues into Georgia and Tennessee. The Cumberland Plateau is bounded by the Highland Rim to the north, the Alabama Valley and Ridge to the southeast, and the East Gulf Coastal Plain to the southwest. The landscape consists of flat-topped, high-elevation, gently sloping (from northeast to southwest) plateaus separated by deep, steep-sided valleys. The highest elevations are above 1,500 feet in DeKalb and eastern Madison counties; the lowest elevations are approximately 200 feet near Holt Lock and Dam in Tuscaloosa County.

The Cumberland Plateau is divided into eight districts: Sand Mountain, Lookout Mountain, Blount Mountain, Warrior Basin, Jackson County Mountains, Murphrees Valley, Wills Valley, and Sequatchie Valley. The geomorphology of the eight districts of the Cumberland Plateau is described in detail in Neilson, 2007.

2.1.4.1 Geology of the Cumberland Plateau

The geology of the Cumberland Plateau (Figure 2-8, Appendix B) results in landforms that are the result of differential erosion of the underlying Paleozoic rocks, which range from Cambrian to Pennsylvanian in age (approximately 550 to 290 million years ago). Ridges of the Cumberland Plateau are formed from the varying age sandstones as they are the more weather resistant rock units. Pennsylvanian sandstones, belonging to the Pottsville Formation, underlie the major plateaus. The valleys are formed in softer shale, limestone, and dolomite. Of these three, limestone is the most easily weathered and eroded; thus, the deepest valleys are cut through this rock type. The plateaus of Lookout Mountain and Sand Mountain developed on down arches in rock layers known as synclines. The Wills, Murphrees, and Sequatchie Valleys developed where the presence of anticlines, which are upward arched folds in rock layers, exposed the more easily eroded rocks.

The Cumberland Plateau contains economic deposits of Pennsylvanian-aged bituminous coal. The deposits are divided geographically into four regions or fields: the Warrior Basin, Plateau, Cahaba, and Coosa. Much of the coal mining in the twentieth century was surface mining, resulting in distinctive surface-mined/reclaimed landscapes throughout the area. Methane gas, a by-product of coal formation, accumulates in and around the coal beds. It is extracted from several areas in the Warrior Basin, mainly in Tuscaloosa, Walker, and Jefferson counties.

2.1.4.2 Soils of the Cumberland Plateau

Most of the soils of the Cumberland Plateau (Figure 2-9, Appendix B) are derived from sandstone or shale. The more level areas are dominated by Nauvoo, Hartsells, and Wynnville soils, which were formed in residuum from sandstone. They have a loamy subsoil and a fine sandy loam surface layer. Most slopes are less than 10 percent, with elevations of approximately 1,300 feet. The more rugged portions of the Cumberland Plateau are dominated by soils such as Montevallo and Townley, which were formed in residuum from shale.

2.1.4.3 Geotechnical Considerations for Transportation Projects

Outlined below are some of the usual geotechnical challenges encountered in the Cumberland Plateau. We note that they are somewhat similar to those of the Alabama Valley and Ridge Province with the exception of the iron ore mining and the extent of the folding and faulting of the Alabama Valley and Ridge rocks.

- Structure Foundations:
    - Due to the underlying bedrock, most bridges and other major structures will be founded on deep foundations bearing on bedrock. Care should be taken however to make sure the bedrock is continuous below the tip of the pile, drilled shaft, etc.
    - The residual overburden can be several tens of feet thick; although, in most areas it is on the order of 20 feet or less.
    - For bridge culverts, particularly those to be covered with thick embankment, and due to the sometimes deep residual soil, the subgrade for bridge culverts needs to be explored to bedrock.
    - As with the Alabama Valley and Ridge Province, settlement can be an issue for bridge culverts in the Cumberland Plateau Province, and the opposite can also be an issue. If the culverts bear on or very close to non-yielding bedrock and the embankment heights are more than 20 feet or so, the culverts can experience high pressures. In such cases, special backfill recommendations for the culverts will need to be provided.
- Slopes:
    - Due to the prevalence of mountains and valleys in the Cumberland Plateau, numerous and quite tall cut slopes both in soil and rock are common. However, with certain exceptions, the underlying rock bedding is gently sloping, which allows rock cuts to be relatively steep (i.e., 0.25H:1V).
    - The slopes of the various hills in the Cumberland Plateau are typically covered with a thick layer of colluvium. These colluvial soils are generally quasi-stable at best and should not be further steepened without careful analyses.
    - The residual soil overburden can vary from a few feet to upwards of 20 feet or more in thickness and is usually composed of low to moderately plastic soils. However, long term stability of soil slopes should be based on the soil's effective (drained) shear strength, which usually limits them to slopes of 3H:1V or flatter.
    - Geotechnical studies for projects on new alignment should include a detailed site reconnaissance with attention paid to signs of historic or ongoing slope movement that could be exacerbated by further excavation and/or embankment placement.
    - Due to the relatively flat lying geologic formations, those formations that contain significant shale (e.g., the Pennington Formation) and are exposed on the sides of mountains often form benches due to being more resistant to weathering than the overlying and underlying carbonate rock formations. These benches tend to collect colluvium, particularly if the mountains are

capped by sandstone. This colluvium collects over time and is normally quasi-stable at best, leading to flow slides from time to time.

- Subgrade and Embankment Soils:
    - Soils overlying the sandstone and shale formations, such as the Pottsville and Hartselle Sandstone formations, are typically well-suited for embankment and subgrade use.
    - As with the other areas, attention should be paid to the presence of shallow groundwater if encountered within a few feet of (or above) the proposed roadway grade in cut sections. In such cases, recommendations will need to be provided to suppress the water table to at least two feet below the proposed subgrade for adequate stability.
- Coal Mining:
    - Some of the most extensive coal mining in Alabama has taken place in the Cumberland Plateau Province; although, the underground mines are fairly deep and, except in localized zones such as around fault zones, unlikely to influence improvements.
    - The one case where surface subsidence could be common is above long-wall mining extraction as mentioned below in Section 4.10.5.

2.1.5 *Highland Rim*

The Highland Rim is the northernmost and the smallest physiographic province in Alabama, occupying approximately seven percent of the state. It occurs as a roughly rectangular area in northwest and north central Alabama and continues northward into central Tennessee and Kentucky. It is bounded by the Cumberland Plateau to the southeast and the East Gulf Coastal Plain to the southwest. The landscape consists of a prominent east-west-trending ridge separating two valleys. Elevations reach approximately 900 feet along the Alabama-Tennessee border and drop to approximately 420 feet where the Tennessee River flows back into Tennessee. The Highland Rim is comprised of three districts: the Tennessee Valley, Little Mountain, and the Moulton Valley, described in detail in Neilson, 2007.

The Tennessee River flows through the Tennessee Valley in a northwesterly direction. Throughout most of its course, the Tennessee River flows in its own alluvium, a loose material deposited by the river when it floods. Prior to construction of Wilson and Wheeler Dams, the river consisted of rapids, shoals, and shallow water from Florence to Decatur where it intersects the Tuscumbia Limestone and encountered the more resistant Fort Payne Chert. In 1924, the Tennessee Valley Authority (TVA) completed Wilson Dam and its locks at Florence, and Wheeler Dam and its locks were completed upstream in 1936, which made the Tennessee River within the Highland Rim section navigable.

2.1.5.1 Geology of the Highland Rim

The geology of the Highland Rim (Figure 2-10, Appendix B) gives rise to landforms that are the result of differential erosion of the underlying rocks, consisting of middle- to upper-Paleozoic sedimentary rocks (approximately 490 to 323 million years ago), most of which formed during the Mississippian period (353 to 323 million years ago). The main ridge-forming rock is the Hartselle Sandstone, while the valleys are formed through the Bangor Limestone and Tuscumbia Limestone Formations.

Numerous quarries extract limestone from the Bangor and Tuscumbia Limestone Formations. Most of this limestone is used as construction aggregate. Limited quarrying in the Highland Rim produces dimension stone used for architectural and decorative purposes.

2.1.5.2 Soils of the Highland Rim

Most of the soils in the Highland Rim (Figure 2-11, Appendix B) valleys were formed as residuum from the weathering of limestones. Soils of the Tennessee River and Coosa River valleys were weathered from pure limestones and are mainly red clayey soils with silt loam surface textures. Decatur and Dewey soils are extensive throughout the valleys, where the topography is generally level to undulating. Most of the soils of the uplands within the Highland Rim are derived from cherty limestones, and Bodine and Fullerton soils are common in these areas. These soils have a gravelly loam and gravelly clay subsoil and a gravelly silt loam surface layer.

2.1.5.3 Geotechnical Considerations for Transportation Projects

Outlined below are some of the usual geotechnical challenges encountered in the Highland Rim. We note that they are somewhat similar to those of the Alabama Valley and Ridge and Cumberland Plateau Provinces with the exception of the iron ore mining and the extent of the folding and faulting of the Alabama Valley and Ridge rocks.

- Structure Foundations:
    - Due to the underlying bedrock, most bridges and other major structures will be founded on deep foundations bearing on bedrock. Care should be taken to ensure that bedrock is continuous below the tip of the pile, drilled shaft, etc.
    - Unlike the Alabama Valley and Ridge Province where the carbonate rock beds are usually turned up, the Highland Rim bedding is typically nearly level away from major fault zones. However, the joints are still enlarged, leading to significant differences in the top of rock locally.
    - The residual overburden can be several tens of feet thick; although, in most areas it is on the order of less than 20 feet to 40 feet.
    - For bridge culverts, particularly those to be covered with thick embankment and due to the occasionally deep residual soil, the subgrade for bridge culverts should be explored to bedrock.
    - As with the other geologic provinces north of the fall line, settlement can be an issue for bridge culverts, and the opposite can also be an issue. If the culverts bear on or very close to non-yielding bedrock and the embankment heights are more than 20 feet, the culverts can experience quite high pressures. In such cases, special backfill for the culverts will need to be provided.
    - Sinkholes are also common in the overburden soils over carbonate bedrock, especially in agricultural areas where groundwater is pumped for use as a source of irrigation water.
- Slopes:
    - Due to the prevalence of mountains and valleys in the Highland Rim, numerous cut slopes both in soil and rock are common. However, with certain exceptions, the underlying rock bedding is gently sloping, which allows rock cuts to be relatively steep (i.e., 0.25H:1V).
    - The slopes of the various hills in the Highland Rim are typically covered with a thick layer of colluvium. These colluvial soils are generally quasi-stable at best and should not be further steepened without careful analyses.
    - The residual soil overburden can vary from a few feet to upwards of 20 feet or more thick and is usually composed of moderately to highly plastic soils. Long term stability of soil slopes should be based on the soil's effective strength, which often limits them to slopes of 3H:1V or flatter.

  - Geotechnical studies for projects on new alignment should include a detailed site reconnaissance with attention paid to signs of historic or ongoing slope movement that could be exacerbated by further excavation and/or embankment placement.
  - Due to the relatively flat-lying geologic formations, those formations that contain significant shale (e.g., the Pennington and Pride Mountain Formations) and are exposed on the sides of mountains often form benches due to being more resistant to weathering than the overlying and underlying carbonate rock formations. Where the cap rock is remnant sandstone from the Pottsville formation, these benches tend to collect colluvium. This colluvium collects over time and is normally quasi-stable at best, leading to flow slides from time to time in particularly wet weather.
- <u>Subgrade and Embankment Soils:</u>
  - Soils overlying the sandstone and shale formations, such as the Pottsville and Hartselle Sandstone Formations are typically well-suited for embankment and subgrade use.
  - However, much of the Highland Rim is immediately underlain by carbonate rock of the Tuscumbia and Fort Payne Formations, which typically weather into a plastic residual soil. Such soils when used in embankments may require encapsulation with a lower plasticity soil.
  - As with the other areas, attention should be paid to the presence of shallow groundwater if encountered within a few feet of (or above) the proposed roadway grade in cut sections. In such cases, recommendations will need to be provided to suppress the water table to at least two feet below the proposed subgrade for adequate stability.

2.2 Special Geologic Considerations

2.2.1 *Subsidence and Sinkholes*

Due to the widespread occurrence of carbonate rocks (predominantly limestone and dolostone but also marble), Alabama has areas of karst topography, which are areas susceptible to subsidence due to sinkhole collapse. The mechanism for karst sinkhole collapse involves the chemical dissolution of carbonate rocks, which are rocks comprised of the mineral calcite ($CaCO_3$) or dolomite [$CaMg(CO_3)_2$]. Rainwater is naturally acidic as it can absorb carbon dioxide and sulfur from the atmosphere and from the interaction with vegetation. As the rainwater moves through the ground and into cracks and crevices of the underlying bedrock, the slightly acidic water slowly dissolves the minerals that compose carbonate rock. Over time, as this dissolution continues, the cracks and crevices become enlarged, forming bedrock voids, caves, and caverns beneath the surface. Raveling of overburden soils into these features can result in voids forming in the overburden that can cause subsidence and occasionally collapse, often with little or no warning (i.e., cover-collapse type sinkhole). Additionally, the weight of overlying soil may eventually cause caverns to collapse, resulting in larger-scale sinkholes to form. On a topographic contour map, sinkholes and other depressions are marked by closed contour lines with hatch marks. Some sinkholes may be elongated rather than round, and some may swallow streams or be filled with water and appear on maps as ponds.

Sinkholes are naturally occurring; although, there are both natural and manmade activities that can accelerate sinkhole formation, such as:

- A change in the water table elevation from drought or pumping of nearby wells and quarries;
- Excessive rainfall;
- Poor surface drainage;

- Heavy construction or weight at the ground surface;
- Excavation to reach construction grades, which may expose incipient sinkholes in the overburden soils;
- Underground mine collapse (usually unrelated to karst); or
- Poorly constructed culverts that allow the surrounding backfill to ravel into the conduit and be carried away by the flow of water.

Karst conditions are usually found in most areas of Alabama underlain by carbonate rock; however, some geologic formations or locations are known to be more active for sinkhole formation. Figure 2-12 in Appendix B indicates mapped sinkholes in Alabama, which is also available at https://www.gsa.state.al.us/gsa/geologic/hazards/sinkholes#sinkholeALMap. Additional sinkhole occurrence and susceptibility is provided in the series of maps "Areas in which Sinkholes Have Occurred or Can Occur" by county (USGS, 1977). These county-level sinkhole maps are available for viewing on the University of Alabama website http://alabamamaps.ua.edu/historicalmaps/counties.html, and can be provided at a higher resolution upon request from the Geotechnical Division.

Site-specific data should be gathered prior to proceeding with geotechnical engineering projects in potential karst areas. This should include a review of known sinkholes in the vicinity; see the maps referenced above. Additionally, there are several geophysical techniques engineers can use to identify and assess the potential for sinkhole formation as well as underground voids, including ground-penetrating radar (GPR), seismic reflection methods, and electrical resistivity imaging (ERI).

Subsurface erosion and collapse of roadways into the voids can be due to underground utilities (e.g., raveling of soil into a stormwater pipe), and such roadway collapses are often mistaken for or mis-labeled as sinkholes.

2.2.2 *Landslides*

A landslide is a perceptible downward and outward movement of slope-forming soil, rock, and vegetation under the influence of gravity. Such movement can be barely perceptible or sudden and catastrophic. Landslides can be triggered by both natural and human-induced changes in the environment. These changes may result from a variety of causes such as:

- Weaknesses in composition or structure of the rock or soil;
- High precipitation;
- Changes in groundwater level;
- Seismic activity;
- Construction or mining activity;
- Over-steepening of slopes;
- Changes in surface water runoff; or
- Heavy loads on slopes.

For geotechnical engineering projects, it is important to understand the structure of the rock at the project site, especially where outcrops may reveal planes of weakness or joint patterns that intersect unfavorably with the angle (or dip) of the bedrock bedding plane and/or the roadway cut alignment.

There are several types of landslides and other downhill mass movements. The most common ones seen in Alabama are creep, slides, and rockfalls.

- Creep is a nearly imperceptibly slow downward movement of soil and/or rock on slopes. Along the slope, certain objects may signal this movement: leaning utility poles, J-shaped trees, leaning retaining walls, or offset fences are some signs that an area is undergoing creep.
- Slides are movements of soil or rock along a distinct surface of rupture that separates the slide material from the more stable underlying material. The scarp is the term for the area at the top of the slide where there is a cracked portion of the slope. The toe of the slide is located near the bottom of the slide, where material mounds up creating a bulge in the topography.
- Rock falls are sudden movements of rock, characterized by free-fall, bouncing, and rolling rock. These are more common in areas of very steep or near-vertical slopes such as roadcuts; although, rock cuts can be unstable regardless of slope if the rock mass properties are not favorable for the cut orientation. Rock fall incidents are more common where the intersection of bedding and jointing creates planes of weakness where rock may become dislodged through the action of roots, stormwater weathering, ice-wedging, and gravity.

Refer to *Evaluation of Landslides Along Alabama Highways* (Auburn University, 2019) for mapped landslides in Alabama; however, conditions (particularly manmade conditions) can cause landslides or slope failures to occur in any geology. A few of the naturally occurring conditions as outlined below are particularly susceptible to landslides:

- Formations containing interbedded sands and clays: This condition is particularly prevalent in the older Coastal Plain formations, such as the Tuscaloosa Formation and Coker Formation.
- Formations containing interbedded shales and carbonate rocks (such as limestone or dolomite): The carbonate rocks will often weather faster than the underlying shales, leaving clay layers and perched water tables. These conditions often result in the formation of benches on hillsides that, in turn, collect colluvium from above and are susceptible to the formation of flow slides. For example, this condition is common with the Pennington Formation on hillsides in northern Alabama.
- Formations that tend to be bluff formers, such as the Pottsville Formation: In this condition, the gradual retreat of the bluff-forming material, such as massive sandstone units, results in the collection of the weathered material on the slope below as colluvium that, in turn, gradually moves downhill under the action of gravity and water. Disturbance of this quasi-stable material by natural causes (such as excessive rainfall or stream erosion at the toe of the slope) or manmade actions (such as making cuts into the hillside or adding fill to the slope) can upset the balance resulting in a landslide.
- Formations adjacent to major streams or rivers: Due to the constant erosion along the banks, most slopes adjacent to the streams and rivers are in a quasi-stable state. It should be noted that even in the case of certain indurated formations, such as the units in the Selma Chalk Formation, the riverbank slopes may appear to be stable on relatively steep slopes; however, due to the jointing in the chalk units, they can still be subject to major slides. For example, this condition has occurred extensively along the Black Warrior River downstream from Demopolis.

2.2.3 *Earthquakes*

There are four zones of earthquake activity that affect Alabama: the New Madrid Seismic Zone, the Southern Appalachian Seismic Zone, the South Carolina Seismic Zone, and the Bahamas Fracture Seismic Zone. Most earthquakes recorded in Alabama are associated with the Southern Appalachian Seismic Zone, located along the Appalachian Mountains and extending from the northeastern corner of Alabama into the central part of the state, and the Bahamas Fracture Seismic Zone, which affects southern Alabama.

While the majority of earthquakes in Alabama have small magnitudes on the Richter Scale (<5), a few have been recorded in Alabama that are of medium intensity, including an estimated 5.1-magnitude quake in 1916 in Shelby County. Because the Richter Scale is logarithmic, a 3.0-magnitude earthquake is 10 times stronger than a 2.0-magnitude earthquake, and a 4.0-magnitude earthquake is 100 times stronger than a 2.0-magnitude earthquake. Therefore, the typical earthquakes of low intensity levels occurring in Alabama are not likely to be noticed. Recorded damage from earthquakes in Alabama tends to be limited to weak masonry structure damage or collapses. A 4.9-magnitude earthquake near Fort Payne, Alabama, in 2003 resulted in broken windows, damaged masonry, sinkholes (the earthquake likely precipitated already incipient sinkholes), and minor landslides. An interactive map of recorded earthquakes in Alabama is located on the GSA website at: https://gsa.state.al.us/gsa/geologic/hazards/earthquakes/alquakes.

Geotechnical engineering projects in Alabama should take into consideration the earthquake zone of the project site. Several seismic stations are recording for seismic activity in Alabama at any given time, and that data can be accessed through the GSA website. The U.S. National Seismic Hazards Maps 2008 edition and 2010 revision are available on the USGS website. Along with seismic station output, there are online tools for mapping earthquake hazards and earthquake probability through the USGS website at: https://www.usgs.gov/natural-hazards/earthquake-hazards/seismic-hazard-maps-and-site-specific-data.

2.2.4 *Mining Influences*

Alabama has a long history of mining activity, including coal mining (underground as well as surface mining), iron ore mining, rock quarrying, and to a limited extent, gold mining. Potential mining influences affecting the geotechnical engineering aspects of projects in Alabama are particularly concerned with abandoned underground mines that may result in collapse features, and surface limestone or marble quarries that may trigger sinkhole features. Sinkholes are addressed Section 2.2.1. A sinkhole-like feature caused by underground mine collapse is generally unrelated to karst. Effects vary from open-hole collapse, including trench-shaped features mimicking the shape of the collapsed underground mine workings, to subtle surface fractures and shifting or rotation that may affect structural stability of buildings, bridges, and roadways.

Underground coal mines present the most widespread mining influence factor in Alabama as coal has been mined in at least 20 Alabama counties for over 150 years. Areas of the state where coal is present at or near the surface include the Warrior Coal Field in middle and western Alabama and the Cahaba Coal Field in a thin band of middle Alabama. Additional coal fields in Alabama include the Sand Mountain, Lookout Mountain, and Coosa coal fields.

While some coal mine maps are available online, a more complete paper repository is maintained at the Mining and Reclamation Division of the Alabama Department of Labor in Birmingham, Alabama. These maps were primarily submitted to the collection by coal mining companies as a requirement of their permits under the mining law in Alabama. Others were donated by the GSA or private individuals.

In general, surface effects from abandoned underground mine works will be minimal or undetectable where the mine works are at greater depths; the exact depth will vary site to site, but generally greater than 200 feet below ground surface is sufficiently deep to allow for natural infilling of mine collapse to reduce or avoid surface expression. At shallower mining depths, site-specific drilling programs should be undertaken to assess rock quality beneath a proposed project.

2.3 References

- Geologic Survey of Alabama:
    - Geologic quadrangle maps: https://www.ogb.alabama.gov/gsa/geologic/mapping
    - Mapped sinkholes in Alabama: https://www.gsa.state.al.us/gsa/geologic/hazards/sinkholes#sinkholeALMap
    - Rheams, K.F., Brabb, E.E., and Taylor, F., 1987; *Preliminary Map Showing Landslide in Alabama,* Miscellaneous Field Studies Map MF-1954, scale 1:500,000.
- USDA Soil Conservation Service:
    - County-level publications for soils: https://www.nrcs.usda.gov/wps/portal/nrcs/surveylist/soils/survey/state/?stateId=AL
    - Mapping tools for soils: https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm
- USGS and the Alabama Highway Department, 1977: Series of maps "Areas in which Sinkholes Have Occurred or Can Occur" by county, which is available at http://alabamamaps.ua.edu/historicalmaps/counties.html
- USGS online tools for mapping earthquake hazards and earthquake probability: https://www.usgs.gov/natural-hazards/earthquake-hazards/seismic-hazard-maps-and-site-specific-data
- Neilson, Mike (University of Alabama at Birmingham), 2007: various articles available at http://www.encyclopediaofalabama.org/category/GeographyandEnvironment

3.0 Geotechnical Exploration and Data Collection

Planning is a key component to a successful geotechnical exploration. During the scoping/planning phase, the Engineer must understand the project requirements, develop an understanding of the site and anticipated subsurface conditions, and select field exploration and laboratory testing methods that are suitable to develop required geotechnical information. To that end, the following subsections provide guidance and information to aid the Engineer in planning and executing geotechnical explorations:

- Section 3.1 – Planning Geotechnical Explorations: Guidance is provided on reviewing available data, performing field reconnaissance, and developing a Geotechnical Exploration Plan. Scope considerations for specific project types are provided in Section 4.
- Section 3.2 – Field Exploration and Laboratory Testing Methods: General information is provided on the more common field exploration and laboratory testing methods, including an overview of the methods, what conditions they are suitable for, and the primary geotechnical parameters that each method either measures or correlates to.
- Section 3.3 – Field Exploration Procedures and Considerations: An overview and guidance on the various components of a field exploration.

The guidance provided in this section is neither intended to be prescriptive in planning/scoping nor exhaustive in presenting available field exploration and lab testing methods. Explorations must be scoped by the Engineer and based on their experience and understanding of the project requirements, site conditions, local geology, etc. Adjustments to the exploration scope may be required as the exploration proceeds and information on the actual conditions encountered is developed, and communication between the Engineer and field personnel is critical to adapting the exploration to unexpected conditions.

As noted in Section 1: For purposes of this document, the term *Engineer* refers either to the State/Assistant Geotechnical Engineer or the Consultant hired to perform the work for the State/Assistant Geotechnical Engineer. When work is performed by a Consultant, information/requests from the Consultant to other ALDOT offices as noted in this manual (such as the Bridge Bureau or the ROW Office) will be through the State/Assistant Geotechnical Engineer.

3.1 Planning Geotechnical Explorations

A plan and profile layout of the project site with planned fill heights and design flood elevations will be forwarded to the Engineer. The Engineer, using standard and accepted geologic and geotechnical engineering practices, will investigate the site shown on the map included with the plan and profile layout. The Engineer will first conduct a site review (which includes both a desktop review of available data/information and a field reconnaissance) as outlined in Section 3.1.1. After conducting the site review, the Engineer will develop and submit a Geotechnical Exploration Plan as outlined in Section 3.1.2. Upon approval by the State Geotechnical Engineer, the exploration plan will be executed at the direction of the Engineer.

3.1.1 *Site Review*

A site review will be performed by the Engineer prior to finalizing the Geotechnical Exploration Plan. A thorough site review will serve to familiarize the Engineer with the site and aid in the development of a suitable and complete exploration plan. The site review should proceed in two phases: (1) review of available data and (2) field reconnaissance.

3.1.1.1 Data Review

For the data review phase, the Engineer will obtain and review available data, which should include the following as available:

- Plan and profile layout for the project.
- Available topographic maps, geologic maps, geologic journals, soil survey maps, aerial/satellite imagery, and lidar/digital elevation models (DEMs).
  - The ALDOT Maintenance Bureau's Surveying and Mapping Section has county-wide aerial imagery collected periodically.
  - Topographic maps are available from the USGS National Map Viewer: https://viewer.nationalmap.gov/advanced-viewer/
  - Section 2 provides general geologic information and provides references for the Engineer to locate additional/site specific information. In the absence of historical subsurface information at the site, understanding the site geology can provide the Engineer with a good indication of the likely subsurface conditions, both the bedrock and the overlying soils.
  - DEMs provide elevation data that can be easily overlain with other maps and project plans using the ArcGIS platform. Additionally, DEM data can aid in detecting features such as historic landslide scarps, ground depressions/sinkholes, etc., that otherwise might not be apparent. Publicly available DEMs can be downloaded from the USDA's Geospatial Data Gateway at: https://datagateway.nrcs.usda.gov/. Additionally, counties or municipalities may have DEM data available for use upon request.
- Data on existing structures, such as:
  - Bridge BIN cards for nearby structures, specifically, boring data or subsurface strata depicted on the BIN cards;
  - Construction and pile driving records;
  - Inspection reports, such as scour reports and culvert inspection reports;
  - Well logs;
  - Performance history of nearby existing structures; and
  - Interviews with local ALDOT Division and District personnel who have firsthand experience with local conditions.
- Data from the Alabama Department of Environmental Management (ADEM) or other regulatory agencies where special care must be taken (e.g., locations where boreholes must be grouted) to reduce the potential for groundwater contamination.

3.1.1.2 Field Reconnaissance

A field reconnaissance / site walk will be performed by the Engineer and others as appropriate. Relevant findings and observations from the field reconnaissance will be noted or summarized in the Geotechnical Exploration Plan. The Engineer will note:

- Site hazards, such as open/active sinkholes, evidence of underground mining, or ongoing/historic landslides, etc., that could impact the project.

- Rock outcrops and existing rock cut slopes. When relevant to the project, nearby cuts/exposures through similar geology should be visited as part of the field reconnaissance.
- Accessibility for fieldwork, and potential routes of ingress and egress for drilling/testing equipment. Note locations of overhead obstruction and areas that are not practically accessible to drilling/testing equipment.
- Property ownership, if not on ALDOT ROW, Right-of-Entry (ROE) should be obtained to access required exploratory locations.
    - **Note:** In the case where a consultant is retained to be the Engineer and unless specifically requested otherwise by ALDOT, ALDOT personnel will be responsible for obtaining right-of-entry.
- For bridge replacement projects, the type, size, and visible condition of existing foundations.
- For stream crossings, observed scour around the existing foundations and abutment walls.
- Potential or delineated wetlands.
- Nearby structures.
- Other unusual features or pertinent information that may affect the project.

3.1.2 *Geotechnical Exploration Plan*

After reviewing project information and conducting the site review, the Engineer will submit a Geotechnical Exploration Plan to the State Geotechnical Engineer. The exploration plan will conform to the guidelines below and *AASHTO R-13: Conducting Geotechnical Subsurface Investigation*.

A summary of guidelines on boring frequency and depth are presented below in Table 3-1 with additional guidance and details for scoping explorations for the various project types is provided in Section 4.

Explorations should not be scoped to meet a “minimum” requirement but rather to address the project needs and the expected subsurface conditions. Some preliminary analyses by the Engineer may be warranted to aid in scoping the exploration, particularly in defining competent material type and minimum boring penetrations into competent material. Competent material will be defined in the Geotechnical Exploration Plan by the Engineer based on the project/foundation type, expected geotechnical resistance mobilization mechanisms, and anticipated subsurface conditions.

Table 3-1: Summary of Guidelines on Boring Frequency and Depth

Exploration Type		Boring Frequency Guidance	Boring Depth Guidance
Bridge Foundations	General	• At least one boring per substructure (abutment/bent) • At least two borings per substructure/footing for footing widths >70 feet	• Through unsuitable material and into competent material for foundation support, with competent material to be defined by the Engineer for the project • For all projects, if a review of the available nearby geotechnical information or published geology suggests that compressible layers may be present underlying upper harder/denser layers, sufficient borings drilled deeper than outlined below should be advanced to characterize that possibility.

Bridge Foundations	Driven Piles	• At least one boring within 50 feet of planned piles	• Friction/soil bearing piles:
			1) To depths at which factored resistance is expected to be greater than the pile structural resistance;
			2) To suitable bearing soils and a depth at which the predicted stress from the equivalent footing loading is less than 10% of the original overburden stress;
			3) Generally, borings at least 25 feet into competent material (SPT N-values of ≥50 blows per foot [bpf] will meet the above criteria);
			4) Consider advancing at least one boring per substructure (abutment/bent) or pile group to the shallower of:
			  • A depth of two times the maximum expected pile group width below the anticipated pile tip elevation, or
			  • To SPT refusal (100 blows to advance <2 inches).
			• Rock bearing piles: At least 10 feet into continuous (i.e., ≥95% recovery) bedrock

Bridge Foundations	Drilled Shafts	• At least one boring within 20 feet of redundant shafts where bearing materials are variable • At least one boring at the shaft location per non-redundant shaft	• Soil bearing: As determined by the Engineer based on expected/provided loads • Rock bearing: To at least 30 feet into bedrock for each boring, and with at least one boring per bent to at least 50 feet into bedrock; the bottom 30 feet of cored bedrock should have recovery ≥95% and RQD ≥50%
Bridge Foundations	Micropiles	• Following guidance for shafts	• Following guidance for rock bearing shafts
Bridge Foundations	Spread Footings	• Boring at each corner for variable bearing conditions (such as on hillsides, over karst bedrock conditions, near old stream channels, etc.)	• Soil bearing: Extending below the bearing elevation to depths of 1) At least 2B for L<2B 2) At least 5B for L>5B 3) Interpolating linearly for 2B<L<5B • Rock bearing: Extending below the bearing elevation to a depth of at least 2B
Bridge Culverts		• For L<300 feet, at the culvert ends and midpoint • For L>300 feet, at the culvert ends and approximately every 150 feet along the culvert alignment	• At least 10 feet below the bearing elevation • At least 3 times the culvert width below the bearing elevation or to SPT refusal, whichever is shallower
Retaining Walls		• At least 3 borings per wall • Every 100 feet along the face of the wall • Every 100 feet at the back of the wall or within bond zones for soil nails or tiebacks (approx. 0.7H behind the wall face)	• To the shallower of: 1) At least twice the wall height below the bearing elevation, 2) A depth below the wall where stress increase becomes less than 10% of the existing effective overburden stress (does not apply to walls in cut sections), or 3) To SPT refusal or bedrock.
Slope Studies		• At least 2 borings per slope • Every 200 feet at centerline and in both ditch lines • Quarter points for cuts over 30 feet high	• Fill/embankment: Centerline borings to twice the proposed fill height or to SPT refusal, whichever is shallower; ditch line borings to 3 feet below final grade • Cut slopes: To competent material and depths not normally subject to slope stability issues
Soil Surveys		• Fill sections: Every 300 feet along centerline • Cut sections: Every 200 feet along centerline; boring in both ditch lines for every third centerline boring	• Fill sections: 1.5 times the proposed fill height or SPT refusal, and not more than 10 feet into competent material with N≥15 bpf • Cut sections: 3 feet below ditch line

Note: 1) Refer to Section 4 for additional guidance and information on scoping geotechnical explorations.

2) For bridge foundations, boring depths should be selected so that recommendations for drilled shafts and driven piles, at minimum, can be developed, regardless of the proposed foundation type(s).

3) “L” = length; “B” = width.

The Geotechnical Exploration Plan will be prepared and submitted for approval to the State Geotechnical Engineer. The exploration plan will include the following items:

- An outline of the proposed boring, testing, and sampling program along with justification. Guidance on commonly employed field methods is provided in Section 3.2. Proposed borings will be assigned individual number and/or letter designations by the Engineer, and this nomenclature will accompany the proposed borings for the duration of the project.
    - SPT borings/rock coring: As a minimum, Standard Penetration Test (SPT) borings will be performed. Proposed SPT boring locations and depths meeting the guidance given in Section 4 and applicable sections of the *AASHTO LRFD Bridge Design Specifications*. Boring termination criteria will be provided in the exploration plan and may include the following:
        - Top of bedrock or a minimum penetration into bedrock,
        - A minimum penetration into a competent bearing material with competent material defined based on material type and condition (either SPT N-values or bedrock recovery and/or Rock Quality Designation [RQD]), or
        - An overall minimum boring/coring depth.

      For projects where deep foundations are anticipated, the Engineer will provide termination criteria based on minimum penetration into bedrock or competent material (with a definition of competent material for the project based on material type, SPT N-values, rock recovery/RQD, etc.) and not a target boring depth alone.
    - In-Situ testing: Proposed in-situ testing type(s) and locations/depths, which may be presented either as: (1) general subsurface conditions where the testing will be employed (such as in soft, cohesive soil layers that can be tested with a drill rig-mounted CPT (cone penetrometer test) device) or (2) specific locations and depths where the testing will be employed, particularly where in-situ testing will be performed in lieu of SPT borings. When in-situ testing is proposed in lieu of SPT borings, a suitable number side-by-side comparative SPT borings should be included in the exploration plan.
    - Intact sampling: Approximate quantities and methods for intact sampling and general locations and/or conditions where the sampling method will be employed.
    - Groundwater observation well locations
    - Instrumentation types and locations
    - Other field exploration methods and locations such as geophysical testing, test pits, hand augers, etc.
- A plan figure showing proposed boring, testing, and sampling locations overlain on the provided project plan drawing. In addition, the plan figure should show other available information relevant to the exploration, such as:
    - For projects with difficult access: Potential ingress/egress routes for drilling/testing equipment, including areas where equipment and vehicles are prohibited from traveling.
    - Locations of known overhead and underground utilities/obstructions.
    - Locations of existing property and ROW boundaries.
- An approximate timetable for completion of the exploration.

- A Boring Assignment Table for the proposed SPT borings and in-situ testing locations (see Table A-1 in Appendix A as an example). The Boring Assignment Table will include termination criteria as noted above for SPT borings/rock coring.
- A Survey Request Form for the proposed drilling/testing locations (see comments in Section 3.3.3 and Table A-2 in Appendix A).
- An assessment of the need for an erosion and sediment control plan based on the proposed exploration (see Section 3.3.2).

The exploration plan may require modifications as the exploration progresses. If conditions are encountered that vary from those anticipated or if anomalous conditions are encountered that may impact the proposed construction, the Engineer should modify the exploration, as appropriate with concurrence of the State Geotechnical Engineer. A field reduction in the number of boring/testing locations will only be permitted with prior approval from the State Geotechnical Engineer.

3.2 Field Exploration and Laboratory Testing Methods

The Engineer must select suitable methods and testing frequency/quantity to characterize the subsurface conditions and material properties, as noted in the previous section. This section provides an overview of commonly employed field exploration and laboratory testing methods:

- Section 3.2.1 – Field Sampling and Testing Methods
- Section 3.2.2 – Instrumentation and Monitoring
- Section 3.2.3 – Laboratory Testing

Additional information on these and other methods are presented in *GEC 5 – Geotechnical Site Characterization* (FHWA Publication No. FHWA-NHI-16-072), along with guidance on developing relevant geotechnical parameters from test data.

3.2.1 *Field Sampling and Testing Methods*

This section provides a general overview of field testing and sampling methods. This is not an exhaustive list of available methods but an overview of the more common methods to guide the Engineer in developing the geotechnical exploration scope. Drilling, sampling, and testing should be performed by qualified and experienced personnel. Method specifics are addressed in relevant standards.

- Section 3.2.1.1 – Soil Borings
- Section 3.2.1.2 – Rock Coring
- Section 3.2.1.3 – In-Situ Testing
- Section 3.2.1.4 – Geophysical Methods
- Section 3.2.1.5 – Test Pits

3.2.1.1 Soil Borings

Soil borings are performed to:

- Identify subsurface distribution of materials with distinctive properties,
- Retrieve samples for use in laboratory testing to evaluate engineering properties of soil layers,

- Acquire groundwater data, and
- Provide access for in-situ testing tools.

Test boring methodology, soil sample type and method of recovery, and in-situ field tests methods are key components in providing the above-mentioned objective for geotechnical subsurface exploration.

3.2.1.1.1 Boring Methods

A wide variety of equipment and methods are available to advance soil borings. A method compatible with the soil and groundwater conditions should be used to advance borings to ensure that suitable soil samples are collected. Drilling fluids are often needed to stabilize the sidewalls and bottom of the boring in soft clay or cohesionless soils when drilling below the groundwater table. Without stabilization, the bottom of the boring may heave (loosening soils at the base of the borehole), or the sidewalls may contract, either disturbing the soil prior to sampling or preventing the sampler from reaching the bottom of the borehole. Excess slough or loose soil should be properly removed from the boring prior to sampling. Continuous flight augers (see Section 3.2.1.1.1.1) or rotary wash boring methods (see Section 3.2.1.1.1.2) are typically used to advance soil borings. An alternative to traditional drilling methods is the direct-push method (see Section 3.2.1.1.1.3). For locations that are inaccessible to drilling equipment, hand auger methods can also be used for sites with limited access (see Section 3.2.1.1.1.4).

3.2.1.1.1.1 Continuous Flight Auger (CFA) Method

Continuous flight augers (CFA) utilize auger flights that are continuous along the entire length of the auger with a drill bit attached to the leading section of flight to cut the soil. Drill cuttings are returned to the ground surface via the auger flights. As the augers advance, additional auger sections are added until the required depth is reached. Careful observation of the resistance to penetration and vibration of the drill bit can provide data for interpretation of subsurface conditions. Two types of continuous flight auger are typically used: (1) solid stem and (2) hollow stem.

Solid stem auger methods are generally used to advance soil borings but are not ideal for soil sampling/testing as the augers must be removed from the borehole to allow access to the hole for sample or testing. Solid stem auger drilling methods are commonly used for cohesive soils where the boring walls are stable for the entire depth of the boring. They are not appropriate for sand and soft soils or where the boring may be advanced below the groundwater table. Solid stem auger methods should be performed in accordance with AASHTO T306.

Hollow stem continuous flight augers, or hollow stem augers (HSA), have large, circular, hollow centers that allow sampling through the center of the augers (augers remain in the borehole during SPT testing or intact sample collection). Hollow stem auger methods are commonly used in clayey soils and in granular soils above the groundwater table. They are not recommended for cohesionless soils below the groundwater table due to potential heave at the base of the borehole, which can impact measured SPT N-values due to disturbance. Although drilling fluid can be used to flood the hollow stem auger to balance the head, this approach is less desirable due to difficulties in maintaining an adequate head of water. It is recommended that rotatory wash drilling methods be used for cohesionless soils below the groundwater table. Hollow stem auger methods should be performed in accordance with AASHTO T306.

3.2.1.1.1.2 Rotary Wash Drilling Methods

Rotary wash drilling methods are generally recommended for borings that will be advanced below the groundwater table. Casing or drilling fluids (e.g. water, bentonite, foam, Revert®, or other synthetic drilling product) are used to maintain an open boring while the circulating drilling fluids remove boring cuttings. Observation of the suspended particles in the drilling fluids returned can be used to identify changes in subsurface conditions. Casing should not be advanced below the sampling elevation prior to sampling. Drilling fluid levels should be maintained at or above the groundwater table during drilling, drill rod removal, and sampling. Rotary wash drilling methods should be performed in accordance with ASTM D 5783.

3.2.1.1.1.3 Direct-Push / Geoprobe®

Direct-push methods can be used to advance borings in areas with limited access due to space constraints or softer grounds or for locations where drill cuttings or potential exposure to hazardous waste contaminants should be minimized. The direct-push machine is typically mounted on a track system and utilizes the vehicle weight and a percussion hammer to hydraulically advance a steel mandrel into the subsurface at a steady rate. Disturbed soil samples can be continuously collected in 5-foot increments by a 0.6-inch to 1.5-inch, disposable, plastic liner sampler housed within the steel mandrel. Additional sensors or tools can be attached to the system to collect data for and geotechnical parameters during soil sampling, groundwater sampling, unconsolidated formation sampling, monitoring well installation, and soil vapor sampling.

Direct-push methods can be used in a variety of soil conditions and can be advanced to sampling depths of approximately 50 feet or deeper depending on local soil conditions and geology. Advancement through layers with an abundance of cobbles, gravel, or boulders can be difficult. Direct-push methods should not be used to advance through rock formations without a rock coring system equipped. A drop hammer for SPT, sonic head system for sonic drilling, and soil anchors for CPT can also be equipped to the direct-push machine to perform a variety of testing or drilling techniques. The direct-push machine can also be equipped with traditional drilling equipment (HSA, rotary), material injection (grout, permanganates, peroxides, and other oxidizers) equipment, concrete coring system, and pavement penetration system. The Geoprobe System® is a more commonly used direct-push system.

3.2.1.1.1.4 Hand Auger Methods

For sites that are difficult to access or have limited access due to terrain, hand auger methods can be used to collect soil samples (soil cuttings or tube samples) at shallow depths (on the order of 10 feet or less). In cohesionless soils, especially soils containing gravel or large boulders, hand auger methods may not be practical. Hand auger borings are useful for obtaining disturbed samples of the near surface soils and for performing dynamic cone penetrometer (DCP) testing as an indicator of soil consistency.

3.2.1.1.2 <u>Soil Sampling</u>

The type of samples that can be obtained in soil borings are divided into two categories: disturbed and intact (“undisturbed”) samples.

Disturbed samples are representative samples of subsurface material and are collected using methods that can cause alterations to the soil macrostructure while not affecting the mineralogical composition. Disturbed soil samples are generally used for determining soil lithology, classification, grain size, Atterberg limits, and compaction characteristics. The split-barrel (or split-spoon) sampler, typically used in conjunction with the SPT, is the most commonly used sampler to obtain disturbed soil samples. Details of the SPT are noted in Section 3.2.1.1.2.1.

Intact soil samples are typically performed in cohesive soils and are used for advance laboratory testing to determine engineering properties, such as shear strength, permeability, density, and compressibility of the soils. Though these samples are commonly referred to as “undisturbed samples” (UDs), it should be noted that the samples are not perfectly undisturbed but have a relatively low degree of disturbance to the soil in which the soil integrity is maintained and the in-situ soil properties can be approximated. The term “intact sample” is used in this manual to refer to “undisturbed samples.” Side friction, volume displacement, internal expansion, water content variations, hydrostatic pressure, weather conditions, handling, and transportation are factors that can affect the quality of the intact sample. Therefore, care must be taken to properly collect, handle, and transport the sample to reduce the effects of disturbance. Specialized equipment is used to obtain intact samples to help reduce soil disturbance and variations in moisture content of the soils. Samplers commonly used to obtain intact samples include: (1) direct-push, thin-walled Shelby tubes; (2) piston samplers (for very soft soils); and (3) Pitcher barrel or Dennison samplers (for very hard soils and soft rock). Details of these more commonly used samplers are provided in Section 3.2.1.1.2.2.

3.2.1.1.2.1 Standard Penetration Testing

SPT is the most commonly used in-situ test, as it provides both a physical sample for field classification and laboratory testing and an SPT N-value that has been correlated to a range of engineering parameters. Laboratory tests typically performed on disturbed SPT samples include moisture content and classification (grain size distribution and Atterberg limits); however, other laboratory tests can be performed if they are suitable for disturbed samples. SPT soil samples are not suitable for use for in laboratory testing where the in-situ structure of the soil is important, such as strength and consolidation testing.

SPT can be performed on a variety of soil types and soft rocks with the exception of gravel deposits (due to sampler incompatibility) and very soft clays (SPT N-values cannot be obtained). SPT follows the procedures outlined in AASHTO T206. The test can be performed using either auger or rotary wash drilling methods. The test equipment consists of a calibrated drive-weight assembly and a sampling device. ALDOT requires that automatic hammers be employed, and the hammer energy ratio (ER) measured within the last 12 months. Soil samples depths/intervals will be defined on a project specific basis as part of the Geotechnical Exploration Plan; soil sample intervals of five feet are typically employed.

Once the boring has been advanced to the desired sampling depth, a hollow, thick-walled, tube-shaped sampling device, usually a split-spoon sampler, is lowered into the borehole via drill rods and set at the sampling depth. Care should be taken to gently lower the sampler to the sample depth rather than allowing the sampler/rods to freely drop, which may affect the measured blow count during testing. Once the sampler is set, a 140±2-pound hammer is repeatedly dropped from a height of 30 inches using a hammer drop system to strike the top of the anvil attached to the drill rods to drive the sampler three successive increments of 6 inches. The number of blows to drive the sampler must be recorded for each 6-inch increment, or fraction therefore, until one of the following occurs:

1. A total of 50 blows have been applied during one of the 6-inch increments (the penetration to the nearest 0.1 foot should be recorded for the 50 blows).
2. A cumulative total of 100 blows have been applied.
3. There is no observed advancement of the sampler for 10 consecutive blows of the hammer.
4. The sampler has advanced the complete 18 inches without the limiting blow counts noted above occurring.

The N-value (blow count) should always be recorded as an integer. The first 6-inch increment recorded is known as the "seating drive". The sum of the number of blows to advance the second and third 6-inch increments gives the N-value, or standard penetration resistance, which can be used to correlate with a variety of engineering properties, such as shear strength and bearing resistance. Caution should be exercised when using N-value to correlate with engineering properties due to the potential for high variability, uncertainty, and inconsistencies introduced into the system from various sources. Correlations between SPT N-values and soil unit weights and strengths are shown in Table 3-2 below.

Table 3-2: SPT N-Values, Descriptions, and Correlations to Unit Weight and Strength

Clay				
SPT N-Values (bpf)	**Description**	**Dry Unit Weight (pcf)**	**Buoyant Unit Weight (pcf)**	**Undrained Shear Strength (s_u; psf)**
0 – 2	Very soft	105	43	250 – 500
2 – 4	Soft	110	48	500 – 1,000
4 – 8	Medium	115	53	1,000 – 2,000
8 – 15	Stiff	120	58	2,000 – 3,750
15 – 30	Very Stiff	125	63	3,750 – 6,250
30+	Hard	133	71	6,250 – 8,000
Sand				
SPT N-Values (bpf)	**Description**	**Dry Unit Weight (pcf)**	**Buoyant Unit Weight (pcf)**	**Effective Stress Friction Angle (ϕ)**
0 – 4	Very loose	94	32	29
4 – 10	Loose	111	49	31
10 – 30	Medium	125	63	35
30 – 50	Dense	135	73	39
50+	Very dense	145	83	42

Note: Unit weight, undrained shear strength, and effective stress friction angle correlations with SPT N-values are approximate and should be verified with in-situ or laboratory testing.

Once the soil sampler is removed from the borehole, the soil must be removed from the sampler, field classified by the rig geologist/engineer, and stored in an air-tight, durable container or plastic bag to maintain in-situ moisture content. At a minimum, the following information should be notated on the container:

- Project identification number/name,
- Borehole number/designation,
- Boring location, including staked location information and measured offset,
- Sample identification/number and sample depth,
- SPT blow count for each 6-inch increment, and
- Date sample collected.

3.2.1.1.2.2 Intact "Undisturbed" Sampling

The intact sampling methods noted in this section are the more commonly employed methods in the State of Alabama, and typically collect a 3-inch diameter sample that is suitable for laboratory strength and consolidation testing. The samplers vary in the method of sample collection, and each sampler is suitable for different conditions, as summarized in the following subsections. In general, piston and Pitcher barrel samplers should be considered for soils that are too soft/loose (piston sampler) or too hard/dense (Pitcher barrel sampler) to be collected using a direct-push Shelby tube.

Upon collection, at a minimum, the intact samples should be labeled with the following information:

- Project identification number/name,
- Borehole number/designation,
- Boring location, including staked location information and measured offset,

- Sample identification/number and sample depth,
- Measured sample recovery.

Additionally, the rig geologist/engineer should check the end of the sample tube for dents, buckling, or burring prior to capping or inserting an O-ring packer at the bottom end of the sampler. Damage to the sample tube should be noted on the field logs. To seal the sampler tube and prevent moisture loss, remove approximately one inch of material from the top of the sample, apply an O-ring packer or a microcrystalline (non-shrinking) wax to the top of the sample to seal, and tape the caps to the tube and over holes at the top of the tube.

Intact samples must be handled with care, stored upright and in an environment away from direct sunlight, cannot be allowed to freeze, and should be transported to the laboratory in accordance with ASTM D 4220 guidelines. Sample extrusion will be performed carefully and in accordance with standardized procedures.

3.2.1.1.2.2.1 Thin-Walled (Shelby) Tube Sampler

Thin-walled (Shelby) tube sampling utilizes the direct-push method of penetration to obtain relatively intact soil samples of materials with some cohesion (i.e., enough cohesion to remain in the Shelby tube during collection) and where SPT N-values are generally less than 15 blows per foot. Standard guidelines and practice for the thin-walled tube sampling method is described in AASHTO T207. The commonly used Shelby tube has an outer diameter of 3 inches, tube length of 27.5 inches, and is equipped with a ball check valve vent to relieve air and water pressure buildup within the tube. The check valve must be checked prior to each use to ensure it is clear of mud and soils. Larger diameter tubes can be used where higher quality samples are required and sampling disturbance must be reduced.

During sampling, the Shelby tube should be pushed slowly and in one continuous motion; the total length of push should not exceed 24 inches. If sampling in low density soils or collapsible material, it is recommended that the total length of push range between 12 to 18 inches to prevent the disturbance of the sample. After pushing, driller should wait for approximately 10 minutes before retrieving the Shelby tube sample to allow for sample expansion within the Shelby tube.

3.2.1.1.2.2.2 Piston Sampler

Piston sampling is a type of thin-walled tube sampling method that utilizes a piston, rod, and modified sampler head to create a vacuum to retain samples that are difficult to recover due to softness, saturation, or insufficient cohesion to keep the sample in the tube during extraction from the borehole. The sample recovery percentage and quality are generally higher using Piston samplers compared to direct-push Shelby tube sampling. Piston sampling can be performed in clays, silts, and sandy soils. Standard guidelines and practice for the piston sampling method is described in AASHTO T207.

During sampling, the piston rod is held stationary at the ground surface as the sampler, containing a lockable head and sealed piston at its base that helps to prevent excess soil from entering the sampler, is slowly lowered to the desired sampling depth. At the desired sampling depth, the piston head is locked to the drill rig or casing as hydraulic or mechanical pressure is applied to push the sampler into the soil, creating a vacuum at the top of the sample to aid in sample retention. Once the sampler is retrieved, the vacuum is broken to allow for the removal of the sampling tube.

3.2.1.1.2.2.3 Pitcher Barrel Sampler

The Pitcher barrel sampler (Pitcher sampler) consists of an outer rotating core barrel with a bit and an inner stationary, self-adjusting, spring-loaded, thin-wall sampling tube that telescopes in or out of the core barrel during drilling based on the material hardness. The standard Pitcher barrel sample has an outer diameter of 4.5

inches, an inner diameter of 3 inches, and a length of 3 feet. It was specifically designed to recover high quality, intact samples from formations that are too dense for conventional thin-wall samplers to penetrate or are too brittle, soft, or water-sensitive for conventional core-barrel-type samplers to recover. The Pitcher barrel sampler is used in stiff to hard clays, dense sand, friable shales, soft rocks, and deposits with alternating hard and soft layers. Pitcher samplers are particularly suitable for chalks/marls and soft shales in Alabama.

During sampling, drilling mud is pumped through the inner Shelby tube, flushing away heavy mud and debris at the bottom of the hole as the sampler is being lowered to the sampling depth. Once the Shelby tube comes into contact with the material at the sampling depth, an internal sliding valve moves upwards, blocking the drilling mud from entering the Shelby tube and redirecting its flow down through the annular between the Shelby tube and outer core barrel and up through the sampler and borehole sidewalls, transporting the cutting to the top of the borehole. The sampling process of the pitcher barrel sampler in soft material is similar to the Shelby tube sampling process. The sampling process of the pitcher barrel sampler in hard material requires the assistance of the outer core barrel to obtain the sample. Once the sample has been obtained, the sampler is removed from the borehole, and the Shelby tube is removed from the sampler.

3.2.1.1.2.3 Field Soil Sample Testing

Hand-held devices that can provide immediate estimates/indicators of soil strength in cohesive soils include the pocket penetrometer and the hand Torvane. Pocket penetrometers are typically used in conjunction with SPT samples, and both pocket penetrometers and hand Torvanes can be used on the exposed bottom of intact samples before sealing. The pocket penetrometer provides a pocket penetrometer value (PPV), which is an approximate unconfined compressive strength based on tip resistance and is suitable for firm to very stiff clay soils. The undrained shear strength is estimated as half of the PPV. If used on soft clay soils, a larger foot/adapter is required for an accurate estimate of the unconfined strength.

The hand Torvane, or miniature vane shear, is a small diameter vane shear testing device used to estimate shear strength cohesive soils based on torque resistance. Variable diameter vanes are available for use based on soil consistency. Torvane tests should be performed in accordance with ASTM D 4648.

It should be noted that the strength parameters obtained from these tests should only be used as an indicator of the strength of cohesive soils, or for comparison purposes only, and should not be used as absolute values or in place of laboratory test results. In-situ tests and/or laboratory tests on intact samples should be performed when soil strength parameters are required for design recommendations.

3.2.1.2 Rock Coring

Rock coring is a procedure in which a core barrel equipped with a coring bit cuts through a rock mass to recover continuous samples to evaluate bedrock characteristics, such as quality and structure, which can affect foundation construction, excavations/cut slopes, and other construction activities. Qualitative characteristics assessed included weathering, hardness, joint frequency and condition, and discontinuities. Structural characteristics assessed include rock lithology and formation. Laboratory tests can be performed on rock core samples to evaluate compressive strength and elastic modulus.

Where exploration or verification of bedrock is to be performed, rock coring procedures are used when a boring cannot be advanced further using soil drilling methods. Drilling method, type of core barrel, type of drill bit, casing and core barrel size, and drilling fluid used should be considered prior to drilling. Guidelines and procedures for rock coring should be performed in accordance with AASHTO T225. Casing, core barrel, and hole sizes can be selected from the standard sizes provided by the Diamond Core Drill Manufacturers Association (DCDMA). A "WM" series, N size core barrel containing a swivel double tube with a split inner barrel or a triple tube with a diamond drill bit is typically used for rock coring. For ALDOT projects, a barrel equivalent to, or better than, the "WM" series and rock core size of NQ or larger should be used. Deviations from the standard casing type and minimum required core size require approval from the State Geotechnical Engineer.

Once the rock core is recovered and removed from core barrel, it should be logged, marked, preserved, placed, packaged, and transported in accordance with ASTM D 5079. At a minimum, the following information should be noted during coring and logging:

- Size, type, and design of core barrel.
- Length, recovery, and RQD of each core run. Where distinct zones of material are encountered within a core run (e.g., different rock type, different weathering condition, etc.), length, recovery, and RQD should be noted for the different layers within the core run.
- Rock lithology description, which includes the following:
    - Rock type;
    - Color;
    - Texture and grain size/shape, if visible;
    - Stratum thickness;
    - Weathering conditions and alteration, particularly along joints;
    - Hardness; and
    - Other pertinent information.
- Rock structural description, which includes the following:
    - Dip of fractures or joint, if apparent;
    - Presence of discontinuities (core loss, soft seam, cavities/voids, fissures, fracture, etc.);
    - Depth, thickness, and apparent nature of filling within discontinuities; and
    - Other pertinent information.
- Remarks concerning drilling time and character (with drilling depths noted), which include the following:
    - Significant changes in drill operation noted by the driller (down pressure, rotation speed, etc.);
    - Changes in drill bit condition or type;
    - Unusual drilling action (chatter, bouncing, binding, sudden drop); and
    - Loss of drilling fluid, color change of fluid, or change in drill pressure.

Rock core should be placed in a core box with the start and end of the core run notated. Mechanical fractures that occurred during drilling or core extraction should be marked with two parallel lines drawn in marker across the fracture. After core and depths have been marked, photographs of the core, with its surface wet to enhance core color, should be taken. The top and four sides of the core boxes should be labeled with the following information:

- Project identification number/name;
- Borehole number/designation;
- Boring location, including staked location information and measured offset;
- Top and bottom depth of core stored in box;

- Box number; and
- Measured length of core recovery and RQD (labeled on the top side of box only).

3.2.1.3 In-Situ Testing

In-situ tests are conducted with reduced disturbance of the soil at the testing depth. The parameters directly measured from the tests are used to develop engineering properties, such as compressibility, strength, and stress history, through correlations or empirical relationships. Soil samples are not obtained with in-situ testing; therefore, these tests are ideally performed in conjunction with sample borings (SPT or intact sampling). Additionally, in-situ testing should not be considered a replacement for a complete laboratory testing program, but rather as providing supplemental data. The more commonly used in-situ tests are CPT, dilatometer test (DMT), vane shear test (VST), and pressuremeter test (PMT) with SPT presented above as part of Section 3.2.1.1.2. The CPT, DMT, and some types of PMT and VST are called "direct-push" technologies as they are directly pushed into the subsurface for testing by either a drill rig or mobile hydraulic system (i.e., a cone truck). Typically, the PMT and VST are conducted in a borehole advanced using soil drilling methods. Details and general procedures of the tests are discussed in the following subsections.

3.2.1.3.1 Cone Penetrometer Test

CPT is a fast and economical test that consists of a cone penetrometer device, the push system, and the data acquisition system mounted on a CPT truck/track rig. Portable CPT devices can be mounted on a drill rig. The test collects accurate, repeatable, reliable, and continuous vertical subsurface profiles of stress, pressure, seismic, and/or other measurements in clays, silts, sands, and peats. The test is not suitable for testing in soils with an abundance of gravel or boulder deposits or where very dense sand layers are present. It is important to note that although the test results are not as operator dependent as other in-situ tests, a skilled operator is required. The test is performed in accordance with ASTM D 5778.

The CPT is performed using a 10-centimeter squared (cm^2) cross-sectional area cone with a 60-degree apex angled tip and 150-cm^2 sleeve area. The soundings are hydraulically advanced into the ground at a rate of approximately 2 centimeters per second (cm/sec). The sleeve friction is measured directly using a tension load cell. The testing is performed in accordance with methods and procedures outlined in ASTM D 5778. The push system consists of the hydraulic push mechanism attached to the drill rig or cone truck and the rig/truck, which acts as a reaction mass (although ground anchors can be used with lighter equipment). As the cone penetrates the subsurface, data is collected by a data acquisition system connected to the cone penetrometer device through a cone cable threaded through the hollow core/drill rods attached to the rig/truck. During cone penetration testing, CPT parameters (tip resistance, friction resistance, and pore pressure) are recorded at 5-centimeter (cm) depth intervals. Sampling depths are monitored using a potentiometer (wire-spool linear variable displacement transducer [LVDT]), depth wheel, or ultrasound sensor.

Internal sensors and measuring systems on the device can measure tip resistance, sleeve resistance, pore water pressure (CPTu), and shear wave velocity (SCPTu) with the first three parameters being the more common measurements. It should be noted that the porous filter and transducer for the piezocone system must be properly and sufficiently saturated immediately prior to and during testing in order to obtain proper pore pressure responses during soil penetration. The tip resistance is a point stress related to the bearing resistance of the soil. In sandy soils, the tip resistance can be used to estimate effective stress friction angle (ϕ'), relative density (D_r), and effective horizontal stress (σ_h'). In clayey soils, the tip resistance can be used to estimate undrained shear strength (s_u) and preconsolidation stress (σ_p'). The sleeve friction is a shear stress expressed as the friction ratio (FR) and helps to determine soil type. Typically, FR is less than one percent in clean sands and greater than four percent in clays and silts of low to medium sensitivity. In highly sensitive clays, FR may be approximately one percent or less.

3.2.1.3.2 Dilatometer Test

The flat plate dilatometer test (DMT), or dilatometer test, is a simple and economical test performed in very loose to compact sands, very soft to firm cohesive soils, and organic soils. The test measures three specific pressures that can be used to estimate soil type: in-situ lateral stress and soil stiffness, overconsolidation ratio, and shear strength from established correlations. Layers with dense and hard material can be problematic during testing as it is difficult to push the DMT through. Calibrations at the beginning of the tests are required to obtain corrections for the membrane stiffness in air to account for the changes in local geologies. Guidelines for the test can be found in ASTM D 6635 and Schmertmann, 1986.

The DMT system consists of a high strength, flat, rectangular, steel blade measuring 3.7 inches wide, 9.3 inches long, and 0.55 inches thick; a pressure gauge readout unit; special-wire tubing; drill/cone rods; and a gas tank. The steel blade is flush on one side and has a circular, steel diaphragm on the other side. The blade is hydraulically pushed into ground at a rate of 20 millimeters per second (mm/s; 0.87 inches per second [in/sec]) using a drill rig or cone truck to test depth increments of 8 to 12 inches. At the testing depth, the diaphragm is inflated pneumatically using either nitrogen or compressed air. Readings for lift-off pressure, expansion pressure, and closing pressure are collected and recorded by a pressure readout unit connect via special wire-tubing threaded through the drill/cone rods. The lift-off pressure is the pressure required to start moving the diaphragm against the soil. The expansion pressure is the pressure required to move the center of the diaphragm. The closing pressure is the pressure during deflation of the diaphragm. Due to the close interval of the readings, an almost continuous profile of the soil response is established.

3.2.1.3.3 Vane Shear Test

VST, or field vane (FV), is a simple test to determine the in-place undrained shear strength, remolded undrained shear strength, and sensitivity (a ratio of measured peak and remolded strength) of soft to stiff cohesive soils from measured torque. The VST cannot be used in granular soils and can be affected by sand lenses and seams. Although slow and time-consuming, the test has a long history of use in practice, demonstrating that the data and test methods are effective. Various size, shapes, and configurations of test equipment are available to accommodate for variability in soil strength and consistency. Larger size vanes are typically used in softer soils while smaller size vanes are typically used in stiffer soils. Care should be taken when using larger size vanes as it can cause more soil disturbance during testing, be difficult to rotate, or result in loads that overstress the torque wrench capacity. Test procedures should be performed in accordance with AASHTO T223.

The standard VST equipment consists of a four-blade, rectangular vane measuring 65 millimeter (mm) in diameter, 130 mm in height (based on a height to diameter [H/D] ratio of 2), and 2 mm in blade thickness. The blade is attached to a 12.7 mm-diameter rod, which may be thicker when testing in stiffer soils to prevent yield of the rod during rotation, with a torque-measuring device. The soil test is conducted at a regular depth interval of 3.28 feet, or 4B if the test location is at the bottom of a borehole with diameter B, by pushing the vane vertically into the soil and, within five minutes of insertion, applying a torque at a constant rate of 6 degrees per minute. Measurements of torque are typically recorded at 30-second to 1-minute intervals. During testing, rod friction can develop, which should be minimized, and should be recorded by the torque-measuring device. Corrections for rod friction should be incorporated when calculating shear strength. To account for rod friction during testing procedures, a rod sheath, protective shoe, or slip coupling equipment can be used, which will reduce the need to correct for rod friction during empirical calculations.

3.2.1.3.4 Pressuremeter Test

PMT is a slow test that requires a skilled operator and provides four independent pressure measurements from the inflation of a cylindrical probe to estimate total horizontal stress, equivalent Young’s modulus (E_{PMT}), undrained shear strength, effective stress friction angle, bearing resistance, settlement, and soil rigidity parameters for a wide variety of soils and soft rocks. Test procedures should be performed in accordance with ASTM D 4719. Tests can be time consuming due to the complicated procedure and the need for care with the equipment. Additionally, the driller must take care to drill high quality boreholes and PMT test holes to collect accurate test data and maintain equipment quality.

The pressuremeter (PM) equipment consists of a cylindrical, monocell or three-cell, a probe containing an inner membrane, up-hole gas pressure application system, pressure lines, supporting cables, pressure gauge readout device, and a hydraulic jack. The inner membrane is inflatable, flexible, and protected by an outer slotted tube from punctures induced by sharp objects in the borehole. The four basic types of PM devices are:

1. Pre-bored (Menard) type PM (MPMT), which is used in a borehole drilled to sampling depth and prepared for testing after a thin-walled (Shelby) tube sample has been collected.
2. Self-boring PM (SBP), which is advanced from the bottom of the borehole using either cutting teeth or water jetting to the testing depth to minimize ground disturbance and preserve the initial state of stress in the ground at the test location.
3. Push-in PM (PIP), which uses a hollow, thick-walled probe with an area ratio of approximately 40 percent for testing. Although this device is faster compared to the MPMT and SBP, the initial soil disturbance effects are higher, thus causing initial state of rest measurements to be invalid.
4. Full-displacement PM (FDP), which creates a highly disturbed zone of soil at the testing depth. The device often utilizes a cone tip and is thus called the cone pressuremeter (CPMT) or piezocone.

Simple commercial PM systems (Texam®, Oyo®, and Pencel®) that consist of a monocell probe with a displacement-type screw pump for inflation are also available for use.

Testing procedures for each of these devices are similar after the probe has been advanced to and set at the desired testing depth. At the testing depth, the cylindrical probe is inflated incrementally using a pressurized fluid until it comes into contact with the surrounding soil/rock. In soils, pressurized water is used for inflation whereas pressurized air is used in self-boring devices and piezocones. When testing in soft rocks, pressurized hydraulic oil is used. The pressure at which the probe expands into the undisturbed soil is measured and recorded as the lift-off pressure. After monotonic loading, a drained creep test is performed to determine the creep, or yield pressure, of the soil. During this test, the pressure is kept constant while the radial volume changes continue until strain rates of 0.1 percent per minute are recorded. During testing, probe volume should be continuously monitored and recorded at each pressure value measured. Lastly, an unload-reload cycle is performed and the minimum pressure during unloading and yield pressure during reloading is measured. The pressure and volume measured are used to determine compressibility and strength parameters through empirical correlations and cylindrical cavity expansion (CIC) theories.

3.2.1.4 Geophysical Methods

Geophysical methods are fast, economical, non-destructive, and non-invasive testing procedures that utilize the application of physics to investigate and characterize soil and rock properties over areas varying from large to small in size. Cemented layers or inclusions in the subsurface can affect the test methods. Test results can also be influenced by water, clay, penetration depth, and other local conditions. Other drawbacks of the method include difficulty developing an accurate subsurface stratigraphy if a hard material overlays a soft material, the need to interpret the results (which requires experience and familiarity with the test), specialized equipment, and the lack of physical samples.

In geotechnical explorations, geophysical methods can be implemented at various phases of the project for reconnaissance, supplementary exploration, or assessment purposes. Geophysical methods can also be used to supplement borehole data or to gather data in areas where conventional drilling, testing, or sampling methods are difficult due to gravelly or talus deposits or potential soil contamination. Geophysical test data can be used to:

- Establish subsurface soil stratigraphy (where layer contrast significantly) or top of bedrock profile;
- Determine groundwater depths, soil deposit locations, and locations of buried, manmade features such as existing foundations, utilities, and pipes; and
- Investigate the rippability of hard soils and rocks and the presence of discontinuities.

Geophysical data should always be correlated with available drilling data when possible.

The more commonly used geophysical methods are electrical resistivity (ER), seismic, and ground penetrating radar (GPR), which are discussed below.

3.2.1.4.1 Electrical Resistivity

ER, or surface resistivity, measures the voltage, or difference in electrical potential, of an applied current at different spacing by embedding an array of electrode into the ground surface along a line across the site to determine the conductivity of the soil, or soil resistivity. Resistivity, usually measured in ohm-meters, is an electrical property of geomaterials and is the measurement of electrical resistance of the material. Procedures and guidelines for the field electrical resistivity test can be found in ASTM G 57. Though the test can be performed on a wide variety of soils and terrain, the test is labor intensive and time consuming. The test should be limited to shallow depths and homogenous deposits as test resolution quality can be negatively affected with increasing test depth or in highly heterogenous deposits.

Electrical resistivity test can be used to:

- Determine depth to bedrock, groundwater table, and anomalies in the subsurface;
- Evaluate subsurface stratigraphy where hard material overlays soft material; and
- Locate prospective borrow material sources, faults, karst features, contamination plumes, and buried features.

Test results can be viewed immediately with appropriate software and field equipment.

3.2.1.4.2 Seismic Methods

Seismic methods measure the seismic waves travelling through the subsurface to determine the physical properties of rock and soil, such as small-strain shear modulus, and map soil stratigraphy, water tables, top of bedrock profiles, and shear wave profiles. Seismic waves are produced from the impact loads, usually generated by a hammer strike, applied to the ground surface. The deflection of various waves (traveling time and amplitudes) is collected by geophones positioned at specific locations along a line to map the subsurface. Seismic surveys can be performed in a variety of soils and types of terrain and can be performed rapidly if ground conditions are suitable. If the survey is performed in an urban setting, surrounding seismic noise can influence the survey results. Processing survey data requires geologic knowledge and expertise. Once the data is processed, subsurface tomography can be created for the testing site. Seismic refraction, seismic reflection, surface wave analysis, and seismic piezocone test (SCPTu) are the more commonly used seismic surveys.

Soil refraction methods are generally used to determine depth to very hard and dense layers and rippability of rock. Guidelines and procedures for the method should be performed in accordance with ASTM D 5777. Soil reflection methods are generally used to determine geological delineations at depths below 10 feet as the method is not constrained by layers of low seismic velocity. This method is especially useful in areas with rapid change in stratigraphy.

Surface wave methods include spectral analysis of surface waves (SASW) and multichannel analysis of surface waves (MASW). The SASW method utilizes a two-receiver approach while the MASW method utilizes a multichannel approach. Compared to the SASW method, the MASW method is more effective and is more commonly used, especially for pavement explorations as the method can be conducted on roadway surfaces. The MASW method evaluates the thickness and stiffness of various subsurface soils and pavement layers; investigates underground mine voids, map voids, and sinkholes; and can produce a top of bedrock profile. The test can be conducted quickly and survey data from the multichannels at multiple locations produce a 2D shear wave velocity versus depth profile at a single point that can be used to determine elastic soil properties of the subsurface strata.

The SCPTu is a type of downhole seismic survey that collects penetration and seismic wave data as a function of depth in clays, sands, silts, and peats during a sounding. The seismic survey is performed as an addition to the CPT to collect shear wave velocity data at an interval of 3.28 feet (see Section 3.2.1.3.1). The seismic waves are generated by striking a steel bar (pushed into firm contact with the ground by the cone truck) horizontally, rather than vertically, or by an electronic wave generator attached the CPT rig. The latter method can help reduce physical strain and testing time and increase repeatability of the test. The data is collected by geophones housed within the CPT cone.

3.2.1.4.3 Ground Penetrating Radar

GPR is a quick and easy to perform procedure that utilizes a pair of transmitting and receiving antennas in a GPR track cart to pulsate high-frequency electromagnetic (EM) energy into the ground, in a grid pattern, to detect changes in permittivity, or dielectric properties, in the subsurface environment to locate shallow soil and rock strata boundaries, groundwater table, underground manmade structures (utilities or tanks), and voids. The procedure can also determine the thickness of pavements and map steel reinforcements in concrete floors, decks, and walls. Data collected is viewed in real-time and requires an experienced operator. The procedure is less effect in the presence of water and clayey soils.

3.2.1.5 Test Pits

Test pits can be performed:

- To obtain detailed, full length vertical and horizontal variations in subsurface strata conditions at shallow depths, especially if thin critical seams (such as water bearing or water restrictive seams) are suspected;
- In areas that are inaccessible to drill rigs;
- Where conventional sampling methods cannot be used due to physical barriers (i.e. bounders, cobbles, debris);
- To identify and/or measure buried features;
- To obtain bulk samples of near surface soils; or
- To obtain depth to shallow bedrock.

Excavators of various sizes are generally used to excavate test pits to depths typically between 6 to 10 feet deep, and no deeper than 16 feet as it is considered unsafe and/or uneconomical at deeper depths. Excavation depth can be limited by the groundwater table. Test pit excavation methods will be performed in accordance with Section 3.1.1. in the FHWA Manual on Subsurface Investigation (2002). A “competent person” as defined by OSHA is required to be onsite to perform oversight of the excavation and check that personnel adhere to the trench safety guidelines specified in Section V, Chapter 2 of the OSHA Technical Manual, *Excavations Hazard Recognition in Trenching and Shoring*. Personnel should not enter an excavation over 4 feet deep without adequate shoring.

3.2.2 *Instrumentation and Monitoring*

Instrumentation can be outfitted at various phases during the design, construction, and service life of a project to monitor site conditions for critical parameters, such as lateral deformation, vertical deformation, or porewater pressure. Factors to consider when selecting instrumentation for monitoring include ground and environmental conditions; instrument life, quality, and performance; and data acquisition requirements. This section provides a general overview of instrumentation and monitoring methods. This is not an exhaustive list of available methods but an overview of the more common methods to guide the Engineer in developing the geotechnical exploration scope. For state projects as outlined in this manual, instrumentation will be primarily used for the following:

- Piezometers: To record water table elevations over long periods of time (such as during fill placement or surcharging) or where rapid changes in water table are expected (such as for groundwater pumping in close proximity to bridge foundations or other structures that may be impacted).
- Settlement Systems: To measure settlement of soils as new embankment is placed or during surcharging.
- Inclinometers: To measure horizontal movements for potential or ongoing landslide/slope failures.
- ShapeArray: To measure deformations over horizontal distances, such as for retaining walls, embankments/roadways that may be at risk for sliding, etc.

Depending on the project needs (e.g., required reading frequency, site location/accessibility, etc.), each of these instruments can be installed with automatically reading equipment with data remotely transmitted to the project team. Additional information on instrumentation methods and tools and an approach to planning and executing an instrumentation and monitoring program can be found in FHWA-HI-98-034.

3.2.2.1 Piezometers

A piezometer is a device that is installed and sealed within an aquifer at the depth of interest to measure and monitor groundwater pressure, porewater pressure, or piezometric head of a specific zone of soil or rock mass. Piezometers can also be used to monitor water flow patterns and estimate effective stress. Water pressure measurements can be used to:

- Determine and control a safe rate of fill placement, especially over soft soils;
- Monitor soil consolidation;
- Aid in slope stability analysis;
- Design and build for lateral earth and uplift pressures; and
- Monitor the effectiveness of drainage systems.

Measurements of pore pressures in soils should be conducted in accordance with AASHTO T252. The two standard piezometer types are open standpipe and diaphragm.

It should be noted that a piezometer is different from an observation well and monitoring well. An observation well does not have a subsurface seal and creates a vertical connection between strata. A monitoring well is used to monitor groundwater levels and water quality by sampling water that is removed from the well.

3.2.2.1.1 Open Standpipe (Casagrande) Piezometers

An open standpipe piezometer consists of a PVC riser/standpipe with a porous filter element attached at the bottom and set within a water-intake zone. The porous filter element is typically a perforated or slotted PVC pipe. The water-intake zone is comprised of sand to allow for water to flow to the porous filter element. A bentonite seal is placed above the water-intake zone to ensure that the porous filter element only responds to the groundwater pressure at the depth of interest and is not affected by the groundwater pressure at other elevations, thus controlling the water level in the standpipe. The integrity of the bentonite seal can be checked after installation by performing a falling head permeability test. As porewater pressure surrounding the porous filter element fluctuates, water flows in or out of the porous filter element and causes the water level within the standpipe to rise or fall, providing a direct measurement of the water level. The water level within the standpipe is typically measured with a water level indicator, which requires direct access to the top of the pipe and provides a depth-to-water measurement. If more frequent readings are required, the standpipe can be equipped with a porewater pressure transducer and a datalogger for automated readings.

Standpipe piezometers can be placed at both shallow and deep depths and are typically more responsive in permeable layers with stable water flow compared to impermeable layers or layers with rapid groundwater changes. Care should be taken to protect exposed standpipes, or equipment, from potential damage from construction equipment. Furthermore, standpipes must be maintained above the ground surface to provide access for water level measurement. Therefore, standpipe extensions should be prepared in advance for planned fill activities.

3.2.2.1.2 Diaphragm Piezometers

Diaphragm piezometers include pneumatic, vibrating wire, and multipoint piezometers. The more commonly used diaphragm piezometer is the vibrating wire piezometer for its accurate results and reliable data.

The vibrating wire (VW) piezometer consists of a pressure transducer attached to a signal cable connected to a data logger or portable readout. The pressure transducer houses a diaphragm, electrical coil, and vibrating wire that interact with each other to produce frequency readouts that are logged and calibrated to obtained pore pressure readings. The VW piezometer can be installed in a borehole or be directly pushed into the subsurface to the depth of interest. If installed in a borehole, the piezometer can be placed in a sand intake zone and sealed with a grout plug, or the entire borehole length can be backfilled with a bentonite-cement grout (grout-in installation procedure). If the grout-in installation procedure is utilized, multiple piezometers can be installed in the same-hole. If installed using push-in techniques, care must be taken to prevent over-ranging the sensors. For both methods of installation, the piezometers must be saturated prior to and during installation.

VW piezometers provides rapid responses, and data collection can be easily automated with the data logger. The signal cable can be buried, causing minimal interference to construction activities. A grounding wire should be installed with the piezometer for lightning protection.

3.2.2.1.3 Field Installation Logs

Items such as the following should be noted in the field installation logs for piezometer installations:

- Project identification number/name,
- Borehole/piezometer number/designation,
- Depth of installation,
- Method of installation used,

- Piezometer type installed, and
- Material encountered at installation depth.

Items such as the following should be noted in the field installation logs for an open standpipe piezometer:

- Porous filter element type, length, and diameter,
- Intake zone material used,
- Intake zone bottom and top depth,
- Bentonite type used,
- Bentonite seal bottom and top depth,
- Grout type used,
- Grout seal bottom and top depth, and
- Riser/standpipe bottom and top depth and diameter.

Items such as the following should be noted in the field installation logs for a VW piezometer:

- Transducer capacity and product number;
- Cable length;
- Readouts during saturation testing prior to installation, during installation, and after installation;
- Intake zone, bentonite seal, and grout seal bottom and top depth (standpipe installation procedure used); and
- Bentonite-to-grout ratio (grout-in installation procedure used).

3.2.2.2 Inclinometers

An inclinometer is a device used to measure and monitor subsurface deformation both laterally and vertically. Measurements of lateral deformation can be used to evaluate slope and embankment stability and verify the performance and safety of structures. Measurements of vertical deformation can be used to evaluate settlement in soft grounds. The device typically consists of a sensing probe housed within an inclinometer casing oriented vertically (in a borehole) or horizontally (in a trench). The inclinometer casing extends through the zone of suspected movement and is anchored or secured in-place to allow for minimal movement, thus providing a surface to measure deflection from. Within the inclinometer casing are grooves that control the orientation of the probe. Care must be taken when installing the casing to ensure spiraling of the casing does not occur as this will result in incorrect orientation of the grooves. A spiral-checking sensor can be used to check casing and groove orientation. If the casing is placed in highly compressible soils, telescoping couplings should be used to prevent damage to the casing. When placed horizontally, the ends of the casing must be aligned vertically. Either a traversing probe or in-place sensor inclinometer system is used to measure deformation.

For the traversing probe system unit, the sensing probe is connected to a readout unit and data is manually collected at two-foot intervals, which requires direct access to the inclinometer casing and can be time consuming. This system can provide a full detail survey of the entire length of the inclinometer casing. In lateral inclinometers, a full depth profile can be used to identify multiple shear zones and provide context to understanding deformation in those zones. Standard guidelines and practice for the traversing probe system is described in AASHTO R45. Care should be taken to ensure that the probe is located properly and uniformly at each interval for accurate displacement calculations.

For the in-place sensor system, the sensing probe is connected to a data logger, and data is collected instantly and automatically, allowing for real-time readings viewable from a remote location. Multiple sensing probes can be installed in a large zone, making this system ideal for projects that require continuous monitoring or deformation monitoring of a large zone.

Software for data processing, reduction, and graphing are available for use. The software can also display sensor locations, alarm status, trend plots, and data readings.

3.2.2.3 Settlement Monitoring

Settlement systems are devices that measure vertical deformation (settlement or heave) of embankments and surcharges to monitor and verify predicted soil consolidation rates and magnitudes, to evaluate the performance of foundations, to predict and adjust final elevations, and to determine the need and timing of corrective measures. They should be installed prior to embankment/fill placement and at the original ground surface or base of excavation; otherwise, it can be difficult to interpret the degree of consolidation. Settlement systems can be used in conjunction with other instrumentation (typically piezometers but also inclinometers or extensometers) for a more complete understanding of settlement. Settlement systems are often either manually surveyed settlement plates or remotely monitored (hydraulic) settlement systems.

3.2.2.3.1 Settlement Plate System

The settlement plate system consists of a flat, base plate (made of either plywood, concrete, or metal) with a fitting with threaded end connections at its center and a metal reference rod (riser pipe) attached to the fitting. As embankment, fill, or surcharge material is added to the area, extensions are added to the riser pipe using threaded connectors. If the settlement plate is installed on highly compressible material or more than 26 feet of material will be placed on top of the settlement plate, a sleeve pipe, typically PVC casing, is required to be placed around the riser pipe to protect the riser pipe from bending distress or distortion caused by from additional soils. Elevation surveys of the base plate and riser pipes should be conducted prior to fill placement to obtain baseline elevations and before and after a new riser pipe section is added. Elevation surveys should be surveyed to a fixed reference datum outside of the construction area, preferably to a known benchmark, and a minimum precision of 1/100 of a foot. Care should be taken to maintain riser pipe verticality.

3.2.2.3.2 Hydraulic Settlement Systems

Hydraulic settlement systems consist of a flat settlement plate, a settlement cell, liquid-filled tubing, a liquid reservoir, and signal cables connected to a remote readout system that instantly and automatically measures and records subsurface settlement, allowing for real-time readings viewable from a remote location. The settlement cell contains a pressure transducer to measure the pressure created by the column of liquid in the tubing from the hydraulic head difference between the transducer and liquid reservoir. The systems can be installed at existing ground or in a borehole anchored to a fixed datum below the compressible layer. Typically, the readout units are located outside of the construction area and the buried settlement system does not require vertical rod extensions, minimizing the potential for equipment damage during construction. Elevation surveys of the hydraulic settlement system should be conducted prior to fill placement to obtain baseline elevations. Care should be taken during installation as the system is sensitive to temperature, system de-airing, and installation methods. Barometric pressure at the site should be recorded to correct for atmospheric pressure.

3.2.2.4 ShapeArray

A ShapeArray (SAA) is a linear sensor system composed of rigid, stainless steel, sensorized segments connected by flexible joints that bend but not twist. The segments are manufactured at fixed lengths and are equipped with three Micro-Electro-Mechanical Systems (MEMS) tilt sensors and microprocessors to calculate the positional displacement of each segment. Displacements are calculated and automatically collected at regular intervals and are compared to the initial baseline displacement, obtained during installation, to determine direction and magnitude of deformation. A SAA can be oriented vertically, horizontally, or in an arc and be installed in a new or existing casing, concrete slabs, or tunnel linings. If installed in a casing, the segments create a zigzag pattern and a casing cap is installed at the top of the casing to hold the SAA in compression and keeps the joints in firm contact with the casing, the surface in which deflection is measured from. SAAs provide accurate, reliable, precise, and repeatable measurements of deformation in the subsurface and earth retention systems, settlement of foundations and structures, and convergence in underground openings. Software for analyzing and interpreting the data in real-time is available for use.

3.2.3 *Laboratory Testing Methods*

This section provides a general overview of laboratory testing methods. This is not an exhaustive list of available methods but an overview of common methods to guide the Engineer in developing the geotechnical laboratory testing program. Generally, a laboratory testing program will need to include testing for moisture content, classification (Atterberg limits and grain size distribution), organic content, and corrosion potential. Additionally, and depending on the soil types and conditions encountered, testing should also include consolidation and triaxial strength. The Engineer should recognize the project requirements to optimize laboratory testing, particularly soil strengths and consolidation testing.

3.2.3.1 Soil Samples

Laboratory soil testing should include visual classification and index property testing to provide general material consistency information and performance (quantitative) testing to provide permeability, strength, and consolidation parameters used for engineering analyses for design and constructability assessment. Classification tests are performed on disturbed (Geoprobe® tube and SPT) or intact samples. Laboratory testing procedures should be performed in accordance with ASTM or AASHTO standards. It should be noted that sample quality and testing procedures can affect test results; therefore, quality assurance is an important component and should always be maintained during sampling and testing.

3.2.3.1.1 Classification Testing

Soil classification (index property) testing includes moisture content, unit weight, Atterberg limits, grain size distribution (or material passing a U.S. Standard No. 200 mesh sieve, -200), hydrometer, specific gravity, and organic content tests.

3.2.3.1.1.1 Moisture content

Moisture content (MC) testing is used to determine the ratio of the mass of water to the dry mass of soil/material and expressed as a percentage. MC is related to a variety of soil properties, such as void ratio (e) and unit weight (γ). Tests can be conducted using: (1) the oven drying method, (2) the microwave oven method, or (3) the field stove or blowtorch method; see AASHTO T265. For organic soils, approximately 140° F (60° C) is sometimes recommended as a reduced drying temperature.

3.2.3.1.1.2 Unit Weight (γ)

Unit weight (γ) testing is a simple method used to determine the ratio of weight to volume of the soil sample. Dry unit weight can be determined from total unit weight, or vice versa, if water content and specific gravity measurements are obtained for the sample. Unit weight is typically measured on intact samples of soil and rock and is either measured or approximated as part of triaxial compression tests.

3.2.3.1.1.3 Atterberg Limits

Atterberg limit tests determine the liquid limit, shrinkage limit, and plasticity of fine-grained soils and organics as a function of water content. For classification purposes, soil consistency is typically determined by the Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI). The LL is the moisture content at which the soil is between a liquid and plastic state. The PL is the moisture content at which the soil is between a plastic and semi-solid state. The PI is the difference between the LL and PL. These indices can be used to characterize physical properties and behavior of soils, such as strength, permeability, compressibility, and shrink/swell potential. The LL can be determined in accordance with AASHTO T89. The PL and PI can be determined in accordance with AASHTO T90.

3.2.3.1.1.4 Grain Size Distribution

Grain size distribution tests are used to determine the percentage of various particle size distribution of the soil in order to texturally classify. Textural classification of the soil can be useful in evaluating engineering characteristics. Sieve analysis is performed on soil samples in accordance with AASHTO T88 to identify coarse-grained soils (sands and gravels) and fine-grained soils (silt and clay materials passing the No. 200 sieve [0.075 mm (0.00029 inches) or smaller]). To identify fine-grained soils, a wash sieve or hydrometer analysis should be performed in accordance with ASTM D 1140. The results of this test also provide other parameters, such as effective diameter (D_{10}) and coefficient of uniformity (C_u).

Once the grain size distribution of the soils has been determined, the soils are classified using the Unified Soil Classification System (USCS) in accordance with ASTM D 2487 or the AASHTO soil classification system (AASHTO T145), see Section 3.3.7.

3.2.3.1.1.5 Specific Gravity (G_s)

Specific gravity (G_s) testing determines the ratio of the weight in air of a given volume of solid particles to an equivalent weight in air of an equal volume of distilled water. Specific gravity measurements are required to estimate void ratio and unit weight. ASSHTO T100 can be used to determined specific gravity of sands, silts, clays, and peats (organics) that pass through the No. 4 (0.187 inch) sieve. Specific gravity tests are typically performed in conjunction with consolidation tests. For specific gravity of material larger than the No. 4 sieve, testing should be performed in accordance with AASHTO T85.

3.2.3.1.1.6 Organic Content

The amount of organic content can affect the soil characteristics, such as consolidation, strength, permeability, and stabilization. The organic content percentage of the total sample mass is evaluated in accordance with AASHTO T267 or T194, as appropriate for soils that contain organic material.

3.2.3.1.2 Permeability Testing

Permeability (hydraulic conductivity) testing determines the flow rate of water through soils, or the coefficient of permeability (*k*), a major factor in material selection for construction projects as it can influence seepage, drainage, dewatering, and settlement. The test is performed on intact or remolded soil samples and measures the time required for a measured volume of water to flow through the soil using a rigid-wall or flexible-wall permeameter cell. Permeability tests are highly sensitive to air, temperature, and viscosity. Care must be taken prior to testing to ensure sample is fully saturated and during testing to minimize the introduction of air or gases into the voids and water. Permeability testing is conducted in accordance with AASHTO T215 or ASTM D5084, depending on the fines content of the soil. The testing procedures are designed for specific hydraulic gradient conditions; therefore, the Engineer should select appropriate test for project.

3.2.3.1.3 Strength Testing

Common soil strength tests include unconfined compression, triaxial compression, and direct shear.

3.2.3.1.3.1 Unconfined Compression

Unconfined compression (UC) strength tests are quick and inexpensive tests used to estimate an undrained shear strength (S_u) of cohesive soils or naturally- or artificially-cemented soils. For cohesive soils, the undrained shear strength (or cohesion) is estimated as approximately one-half of the unconfined compressive strength. Stress-strain curves and failure modes observed from the tests can be used to obtain an index value of the soil properties. The reliability of the results decreases with increasing sample depth due to increase in swelling potential of deeper samples. The test should not be performed on granular soils, dry soils, silts, peats (organics), or fissured or varved material as the results may underestimate the shear strength of the soil. Due to lack of control over pore pressure, the test is not recommended for compressible clay soils subjected to embankment or structural foundation loads. UC strength tests should be performed in accordance with AASHTO T208. Care should be taken to ensure quality of test specimen as to not affect the strength results of the test.

Tests are typically performed in the laboratory but can be performed in the field using portable equipment for rapid measurements of undrained shear strength.

3.2.3.1.3.2 Triaxial Compression

Triaxial compression tests are versatile tests performed on intact or remolded soil samples sealed within a rubber membrane, placed in a cell, and subject to fluid pressure to determine stress-strain properties and strength characteristics. The effects of lateral confinement, porewater pressure, drainage, and consolidation on soil behavior can be observed through the triaxial test. Typically, three test points for each test sample are performed, and results are plotted as Mohr's circles of stress on a shear vs. stress or q-p plot with stress paths visible. Five types of triaxial tests can be performed:

- Undrained Unconsolidated (UU),
- Consolidated Drained (CD),
- Consolidated Undrained (CU) without pore pressure measurement,
- CU with pore pressure measurements, or
- Cyclic Triaxial Loading Tests (CTX).

Three of these tests (UU, CD, and CU with pore pressure measurements) are more commonly used, and therefore, will be discussed subsequently.

The UU test is typically performed on cohesive soils to determine undrained shear strength. UU test results can be used to calculate embankment stability during quick-load (i.e., short-term) conditions. Test results are dependent on the degree of saturation (S) of the specimen. The test is not applicable for cohesionless/granular soils. During testing, drainage and consolidation does not occur as the confining or shear stress is applied to the specimen. Test procedures are performed in accordance with AASHTO T296.

The CU test is commonly performed on cohesive soils or cohesionless soils with some cohesion from partial drainage or remolded cohesionless soils to determine undrained shear strength, effective stress friction angle, and effective cohesion. CU test results can be used to simulate quick-load conditions after long-term stability in cohesive soils. In cohesionless soils, CU test results can be used to evaluate stress-strain properties as a function of effective confining stress. During testing, the sample is consolidated as the confining stress is applied but drainage is not allowed during shearing of sample. Test procedures are performed in accordance with AASHTO T297.

The CD test is typically performed on intact or remolded sands, clays, and silts to determine the effective stress parameters to calculate long-term (drained) stability of embankment. The specimen is consolidated to a specified pressure, followed by shearing at a slow rate to allow for drainage and prevent pore pressure buildup, thus modeling long-term soil strength properties. Effective friction angle and effective cohesion are obtained from the test. Test procedures are performed in accordance with ASTM D 7181.

3.2.3.1.3.3 Direct Shear

Direct shear testing is a consolidated-drained test typically performed on intact or recompacted cohesionless soils to determine shear strength parameters along a pre-defined, horizontal shear plane. The test requires a minimum of three tests performed with varying normal pressures to determine the effective stress friction angle and effective cohesion (ϕ' and c') of the material for foundation stability calculations. Effective residual strength parameters (c_r' and ϕ_r') can be determined from repeated cycles of shearing along the same direction and used for landslide or slope stability calculations. Tests should be performed in accordance with AASHTO T236. In accordance with ASTM D 3080, the consolidated-drained direct shear test can also be performed on fine-grained soils to determine primary consolidation parameters.

3.2.3.1.4 Consolidation Testing

The one-dimensional consolidation tests are a reliable method and are performed in the oedometer (or one-dimensional consolidometer) to determine consolidation and settlement behaviors of cohesive/semi-cohesive soils under incremental loading. Tests are performed on intact or remolded samples to determine primary consolidation and secondary compression parameters for settlement estimates. Primary consolidation parameters include rate of consolidation, preconsolidation pressure (σ_p'), overconsolidation ratio (OCR), compression, recompression, swell indices (C_c, C_r, and C_s), coefficient of consolidation (c_v), stiffness, and permeability (k). Secondary compression, also referred to as long-term creep, follows primary consolidation in cohesive soils and high organic material (organic content >50%) and is expressed as a creep rate (C_α). Consolidation tests should be performed in accordance with AAHSTO T216. Test results are typically presented as graphs (void ratio vs. log stress; strain vs. log stress) and parameters required for primary and secondary consolidation and settlement calculations are derived from the graphs using the Casagrande or Taylor Method.

3.2.3.1.5 Shrink/Swell Potential Testing

Shrink/swell potential tests are performed on cohesive soils and highly weathered/decomposed chalk to determine the potential for volume changes or loss of strength due to changes in moisture content. The tests induce hydrocompression or hydroswell and are typically performed in an odometer. Shrink/swell potential tests can help assess the suitability of the soil in construction of foundations, roads, embankments, and dams.

The shrinkage, or collapse, potential test evaluates the decrease in volume of the material as a result moisture content change. An index typically obtained from this test (and that be obtained from the Atterberg limit test) is the shrinkage limit (SL), which is the moisture content at which the minimum volume of the fully saturated (degree of saturation is 100 percent) soil is obtained. At the SL, further reduction in moisture content will not result in a volume reduction. Shrinkage potential tests should be performed in accordance with ASTM D 4943.

The swell potential test evaluates the increase in volume of the soil, typically in clay, as a result of an increase in moisture content and is dependent on the mineralogical composition of the clay. Montmorillonite (smectite) clay exhibits a high potential for swelling, illite exhibits moderate to extremely low potential for swelling, and kaolinite exhibits extremely low potential for swelling. Swell potential tests should be performed in accordance with AASHTO T258.

3.2.3.2 Rock

Laboratory tests are typically conducted on rock core samples to determine strength, stiffness, and durability properties to evaluate the rock mass and develop parameters for design of rock fills, cut slopes, and rock-bearing foundations through empirical relationships. Common strength, stiffness, and durability tests include uniaxial compression, elastic modulus, and slake durability, respectively. Laboratory testing procedures should be performed in accordance with the most recent ASTM or AASHTO standard available for the test. It should be noted that sample quality and testing procedures can affect test results; therefore, quality assurance is an important component and should always be maintained during sampling and testing. Tests other than those noted below may be performed at the direction of the Engineer. Details on these and other laboratory rock test procedures can be found in FHWA-HI-97-021 (1997).

3.2.3.2.1 Uniaxial Compressive Strength Test

The uniaxial compression test is a simple and quick test performed on rock core samples to measure uniaxial compressive strength. A rock core specimen is placed in a properly-sized loading frame and loaded axially until peak load and failure occur to directly determine rock strength. Moisture content and physical characteristics of the rock core should be recorded (via pictures or drawings) prior to and after testing as they can influence the test results. Uniaxial compression test procedures should be performed in accordance with ASTM D 7012, Method C. Rock core specimens generally have a length to diameter (L/D) ratio of 2.0 to 2.5 and should be prepared in accordance with ASTM D 4543 for testing.

Strain gages can be used to measure stress and strain data during testing. A stress vs. strain graph can be produced and used to determine compressive strength values during testing.

3.2.3.2.2 Elastic Modulus Test

The elastic moduli test is a simple and reasonably reliable test performed on intact rock cores to determine the elastic (Young's) modulus, Poisson's ratio, and uniaxial compressive strength. A rock core specimen is placed in properly-sized loading frame or loading chamber and loaded axially until peak load and failure. Axial (or vertical) and lateral (or diametral) strain as a function of stress is recorded using a strain gauge or LVDT to produce stress vs. strain curves used to determine average, secant, and tangent Young's moduli. Elastic moduli test procedures should be performed in accordance with ASTM D 7012, Method D. Rock core specimens generally have a L/D ratio of 2.0 to 2.5 and should be prepared in accordance with ASTM D 4543 for testing.

3.2.3.2.3 Slake Durability

The slake durability test is a test performed to estimate the potential for degradation and disintegration of shale or other weak or soft rocks to evaluate material competency over the service life of the project. Rock specimens are placed in a drum and subjected to a minimum of 2 cycles (10 minutes at 20 revolutions per minute) of wetting and drying to mimic weathering processes in the environment. The shape and size of the remaining specimens are recorded, and the Slake Durability Index (SDI) is calculated. A high SDI indicates low susceptibility to degradation during service life, and a low SDI indicates highly degradable rock that can potentially revert to soil during its service life. Soil classification testing is recommended for material with low SDI. Slake durability test procedures should be performed in accordance with ASTM D 4644.

3.2.3.3 Corrosion Potential

Corrosion potential tests are performed in the laboratory or in the field to determine the aggressiveness and corrosivity classification of the soil and water. At a minimum, testing should include pH, electrical resistivity, chloride content, and sulfate content testing. Electrical resistivity and pH can affect corrosion potential of steel elements and geosynthetic materials. Chloride and sulfate content can affect the corrosion potential of concrete elements. Test methods are noted below, though other suitable methods can be employed.

- The pH of soils should be determined in accordance with AASHTO T289. The pH of water should be determined in accordance with ASTM D 1293.
- Electrical resistivity of soils should be determined in accordance with AASHTO T288. Electrical resistivity of water should be determined in accordance with ASTM D 1125.
- Chloride content in soils should be determined in accordance with AASHTO T291. The chloride content in water should be determined in accordance with ASTM D 512 or ASTM D 4327.
- Sulfate content in soils should be determined in accordance with AASHTO T290. The sulfate content in water should be determined in accordance with ASTM D 516 or ASTM D 4327.

3.3 Field Exploration Procedures and Considerations

Guidance and requirements are provided in the following subsections for:

- Section 3.3.1 – Property access and right-of-entry,
- Section 3.3.2 – Stormwater discharge permits,
- Section 3.3.3 – Boring location layout and survey,
- Section 3.3.4 – Utility locates and clearance,
- Section 3.3.5 – Traffic control,
- Section 3.3.6 – Field logging procedures,
- Section 3.3.7 – Soil and rock classification,
- Section 3.3.8 – Water table readings,
- Section 3.3.9 – Grouting boreholes,
- Section 3.3.10 – Transporting and storing samples, and
- Section 3.3.11 – Sample retention policy.

3.3.1 *Property Access and Right-of-Entry*

Right-of-entry for fieldwork will be obtained for properties where access may be required during the geotechnical exploration, including for field reconnaissance, equipment access, drilling/sampling activities, erosion control measures, etc. The Engineer will submit a map, list of properties, and the nature of the work for each property (e.g., clearing, grading, drilling, etc.) where access is required to the ROW Office. Fieldwork will not proceed until right-of-entry for impacted properties is obtained.

3.3.2 *Stormwater Discharge Permits*

As part of the Geotechnical Exploration Plan, the Engineer will determine if an ADEM National Pollutant Discharge Elimination System (NPDES) Permit for stormwater discharges and/or U.S Army Corps of Engineers Permit are required for the fieldwork. The Engineer will estimate the area of soil disturbance required to perform the work. Should the sum of soil disturbance areas equal one or more acres, a stormwater permit per ADEM regulations will be required; although, a stormwater permit may be required regardless of soil disturbance area in some cases. For stormwater permit coverage, the Pre-Construction Investigation Activities Environmental Permits Checklist (Form A-3 in Appendix A) will be completed for the project. The Engineer will also prepare pertinent documentation and follow Priority Construction Site regulations if the project area is considered as such by ADEM. The Engineer will determine if the work activity impacts defined wetlands, Waters of the U.S., or a stream crossing(s). Should the work activity impact any of these sites, a Corps of Engineers Permit will be required.

Upon approval of the Geotechnical Exploration Plan and if requested by the State Geotechnical Engineer, the Engineer will prepare pertinent applications and documentation for a NPDES Stormwater Permit and a Corps of Engineers Permit, as applicable, and submit that information along with the correct fees to the appropriate departments. Once approval of the permits is received, fieldwork can commence. It is the responsibility of the Engineer to perform site inspections and documentation required to maintain the permits in good standing and to terminate the permits, as applicable, once the fieldwork is complete and the soil disturbance has been reclaimed.

3.3.3 *Boring Location Layout and Survey*

Proposed boring and testing locations will be surveyed/staked in the field by ALDOT survey crews wherever possible. A Survey Request Form (see Table A-2 in Appendix A) will be submitted to the Geotechnical Division by the Engineer upon approval of the Geotechnical Exploration Plan. Where locations cannot be surveyed (such as borings over water), borings will be located as close as practical at the time of drilling using handheld GPS and, if available, distances from reference structures, such as existing bridge piers.

Offset boring locations from staked locations (station, offset, and elevation) will be measured/estimated by field personnel. Offsets greater than 10 feet from staked locations (or less, if specified in the Boring Assignment Sheet) must be approved by the Engineer before drilling. Additionally, as-drilled locations will be staked by the field personnel after drilling and a Survey Request Form (see Table A-2 in Appendix A) for the offset locations will be submitted to the Geotechnical Division upon completion of drilling.

3.3.4 *Utility Locates and Clearance*

Prior to commencing ground disturbance activities, utilities will be located and marked in the vicinity of work areas. At a minimum, an Alabama 811 (OneCall) ticket will be obtained for the working areas. Additionally, a GPR/EM (or other locate methods, as necessary) utility locate will be performed if:

1. There are proposed or potential ground disturbance work areas are outside of the ROW,
2. There is a utility (or utilities) not in the Alabama 811 network that may be present at the site,
3. There is uncertainty regarding the presence of utilities not marked as part of the Alabama 811 utility locate, or
4. At the discretion of the Engineer.

Ground disturbance will be suitably offset from overhead power lines and from underground utility markings. At a minimum, ground disturbance zones must be outside of the utility locate marking tolerance zones as provided by Alabama 811. Equipment and personnel must maintain safe working distances from overhead power lines.

3.3.5 *Traffic Control*

Traffic control will be employed when fieldwork activities will be on roadways, shoulders, or medians or when work will require equipment crossing or unloading on roadways. Prior to beginning such fieldwork, a Traffic Control Plan will be completed in accordance with the *Manual on Uniform Traffic Control Devices for Streets and Highways* (FHWA, current edition). The Traffic Control Plan will be submitted to the Local District Office for review and approval. The Traffic Control Plan may also require approval from the impacted municipality. During the performance of fieldwork, traffic control measures will be implemented in accordance with the Traffic Control Plan.

3.3.6 *Field Logging Procedures*

A field log is a record prepared by the rig geologist/engineer during subsurface explorations of soil and rock to document procedures used, test data, descriptions of materials and depths where encountered, groundwater conditions, and other information. The Engineer will ensure that accurate and professional boring logs are maintained, accurately depicting soil conditions encountered during drilling. Borings will be performed in general accordance with *AASHTO T-306: Progressing Auger Borings for Geotechnical Explorations* and coring operations will generally conform to AASHTO T-225.

The preparation of field logs provides documentation of field exploration procedures and findings for geotechnical, geologic, hydrogeologic, and other explorations of subsurface conditions. The recorded information in a field log will depend on the specific purpose of the site exploration. Items such as the following should be noted in the field logs:

- Project identification number or name.
- Name of rig geologist/engineer(s) completing the logs.
- Boring information:
    - Boring number/designation;
    - Boring location, including staked location information and measured offset;
    - Drill rig number and model;
    - SPT hammer type and results of the most recent SPT hammer efficiency calibration;
    - Methods of drilling and sampling employed;
    - Name of driller; and
    - Date of start and completion of boring.
- Drilling and sampling information:
    - Field number of each sample; sample type; sample depth;
    - SPT blow-counts for each six-inch interval;
    - Results of field testing, such as pocket penetrometer values or hand Torvane readings;
    - Visual identification (see Section 3.3.6.1) and field classification (see Section 3.3.7) of each layer encountered and sample collected;
    - Depth at which obstacles were encountered in advancing the boring;
    - Depth to which casing was driven;

  - Length of each run for rock core and length of core recovered;
  - Record of changes in the type of cuttings flushed to surface while drilling;
  - Depth where drilling mud or coring water circulation was lost;
  - Changes occurring in rate of advance of bit;
  - Unexpected conditions encountered during drilling and sampling; and
  - Reason for abandoning boring in the event specified depth was not reached.
- Water table readings.
- Relevant comments from the driller.

3.3.6.1 *Visual Identification of Soils*

Visual identification, or description, should be performed by the rig engineer/geologist as samples are collected, with relevant notes and descriptions provided on the boring logs for each sample collected. Visual identification of soil should be performed in the field in accordance with AASHTO M145. At a minimum, the following soil visual descriptions should be included in the field logs:

- Composition and constituents (e.g., clay, silt, sand, silty sand, etc.);
- Soil classification (USCS classification system or, for Soil Survey borings, the AASHTO classification system; see Section 3.3.7.1);
- Inclusions of non-soil components, such as root fragments, organic matter, shells, brick, slag, wood fragments, etc.;
- Gradation (e.g., fine, fine to medium, coarse, etc.);
- Consistency or density following Table 3-2 in Section 3.2.1.1.2.1;
- Color (following the Munsell Color System); and
- Qualitative moisture content (e.g., dry, moist, wet).

3.3.7 *Soil and Rock Classification*

The process of classification groups soil and rock materials with similar characteristics and engineering properties into a category. Soil and rock samples collected during field explorations should be classified in the field by the rig engineer/geologist.

Soil will be classified according to the USCS. AASHTO Soil Classification System is required for Soil Survey borings. Procedures for field classification are provided in ASTM D 2488. Soil sample classification should be verified on representative samples with laboratory testing.

Rock classification is divided into two types: (1) intact rock and (2) rock mass. Intact rock is characterized in terms of geologic origin, mineral composition, and texture. Rock mass is characterized by structural features, such as discontinuities, joints, and fractures, and classified by the Rock Mass Classification Systems (RMCS) and RQD.

3.3.7.1 Soil Classification

3.3.7.1.1 Unified Soil Classification System

The Unified Soil Classification System (USCS) is the most commonly used soil system and classifies the soils into one of 27 groups based on grain size distribution, plasticity, and organic content under 3 major categories (coarse-grained, fine-grained, and organic soils). The groups are designated with 2 or 4 letters called Group Symbols. The first letter represents the primary constituent based on percentage passing the No. 200 sieve (i.e., "C" for clay, "M" for silt, "S" for sand, "G" for gravel, and "O" for organics). The second letter represents the secondary description based on grain size distribution and/or plasticity. Four letter groups are used to classify coarse-grained soils with silt or clay content (such as SP-SM) and for silty clays (CL-ML). Procedures and guidelines for the USCS classification system are provided in AASHTO M145.

3.3.7.1.2 AASHTO Soil Classification System

The AASHTO soil classification system is primarily used for determining soil suitability for highway and pavement construction. The system classifies the soils into eight major categories (A-1 through A-8) and provides a Group Index (GI) value (0 through 20) based on soil physical properties. Groups A-1 through A-7 are classified based on grain size distribution and plasticity. Group A-8 is used to classify organic soils. The GI value indicates soil performance (0 indicating good soil; 20 indicating very poor soil) in highway and roadway construction. Procedures and guidelines for the AASHTO classification system are provided in AASHTO M145.

3.3.7.2 Rock Classification

3.3.7.2.1 Rock Mass Classification System

There are eight major rock mass classification systems used to quantity and assess the conditions of rock masses. The Rock Mass Rating (RMR) System and Geological Strength Index (GSI) System are the more commonly used systems to evaluate rock mass conditions, discontinuities, and strength.

The RMR system, or Geomechanics Classification, is used to classify and evaluate rock property for a variety of applications such as determining rock slope stability and rippability and also rock foundation suitability. The RMR value is based on five parameters:

1. Uniaxial compressive strength (ASTM D 7012C or ASTM D 5731),
2. RQD (ASTM D 6032),
3. Discontinuity spacing,
4. Discontinuity condition, and
5. Groundwater conditions.

A sixth parameter, discontinuity orientation, can be used to adjust the final rating for special project types (tunnels, foundations, and slopes).

Each parameter is assigned a rating value. The summation of the rating value, ranging between 0 (very poor) to 100 (excellent), is assigned to the rock mass. Standard guidelines and procedures for the system are provided in ASTM STP 984 and ASTM D 5878.

The GSI system is used to determine the rock mass strength and stiffness based on geologic conditions. The GSI value can be calculated from the RMR. If the RMR system is utilized, GSI can only be calculated if the RMR value is greater than 25. To determine the GSI value in the field, a GSI value chart, shown below in Figure 3-1, is typically used.

3.3.7.2.2 Rock Quality Designation

RQD is a quantitative field measurement of fractures and joints in a rock mass retrieved during coring. RQD is expressed as the percentage of the summation of intact or sound core greater than four inches over the total length of core. A core length can be defined as the length equal to:

- The core run (typically equal to the core barrel length);
- A rock formation, rock type, or layer within a core run; or
- A specific zone of interest or concern.

RQD values indicate rock quality. An RQD of 75 percent or higher indicates good to excellent (high quality) rock mass, and an RQD of 50 percent or lower indicates poor to very poor (low quality) rock. Zones with low RQD may warrant further exploration and require careful design considerations. It should be noted that mechanical fracture zones should not be considered as natural breaks and not impact the intact length of the rock core. Procedures and guidelines for RQD are provided in ASTM D 6032.

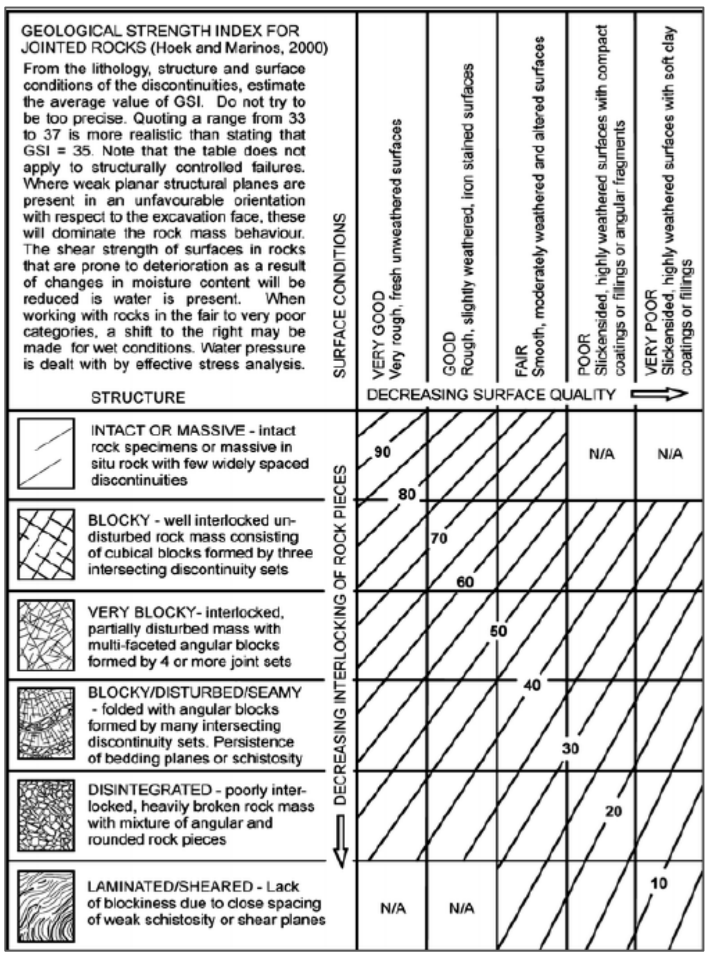


Figure 3-1: Chart for Estimating the Geologic Strength Index (GSI) in the Field
(from *Practical Estimates of Rock Mass Strength;* Hoek, E., and Brown, E.T., 1998)

3.3.8 *Water Table Readings*

Water table information is critical to most geotechnical explorations. This includes surface water, groundwater, perched water tables, and artesian/confined aquifer conditions. Water table readings in boreholes should be taken as follows:

- At time of drilling: During drilling and sampling, the rig geologist should be alert for conditions that may indicate the groundwater table, such as increasing moisture content in samples. The rig geologist should also be alert for conditions that may indicate perched water table or confined aquifers, such as persistent layers of lower-permeability soils.
- After drilling: Measure depth to groundwater in the borehole immediately after drilling (if the drilling method allows) and after 24 hours, if possible. If borehole collapse occurs, the depth of borehole collapse and time of measurement (as time after completion of drilling) should be noted.

Selected borings/boreholes should have observation wells or piezometers installed to obtain water table readings over time. Where perched water tables are encountered or suspected, nested wells/piezometers should be considered to check water table elevations in different strata. Wells/piezometer installations should be logged (see Section 3.2.2.1), and a log of water table measurements should be kept, which will include the date of the measurement and the water table depth below top of casing. If artesian conditions are encountered or expected, a note should be placed on the boring logs noting the layer where the artesian conditions were encountered along with an estimate of the pressure head and/or rate of flow if free flowing.

3.3.9 *Grouting Boreholes*

Boreholes will be backfilled by grouting if the borehole is greater than 25 feet below the ground surface or if the borehole penetrates a water bearing stratum. Similarly, subsurface equipment (observation wells, piezometers, inclinometers, etc.) will be abandoned by grouting. Grouting is not required for boreholes drilled in planned roadbed construction areas where the natural overburden will be removed to within 25 feet of the bottom of the borehole.

Backfilling procedures for boreholes will be noted in the Geotechnical Report and on the boring logs. If subsurface equipment is abandoned after issuing the Geotechnical Report, a letter documenting the date and method of abandonment will be provided to the Geotechnical Division by the Engineer.

Grout will consist of neat cement, cement grout, cement bentonite mixture (5% to 8% bentonite), or bentonite. Bentonite pellets may be added under free-fall conditions for depths not exceeding 25 feet, provided that pellets are placed in layers not more than 5 feet deep and tamped into placed after addition of each layer. Granulated or palletized bentonite may be placed to greater depths if introduced through a tremie pipe. Free-fall addition of other types of grout from the surface is prohibited.

3.3.10 *Transporting and Storing Samples*

Sample storage and transportation will be in accordance with ASTM D 4220 *Standard Practices for Preserving and Transporting Soil Samples* and ASTM D 5079 *Standard Practices for Preserving and Transporting Rock Core Samples.* Special care should be taken to store and transport soil and rock samples. Intact samples (Shelby tube samples) must be transported in an upright position with the original vertical orientation. Samples should not be subject to freezing temperatures. Rock core that will be tested should be wrapped in plastic wrap to retain its moisture content.

3.3.11 *Sample Retention Policy*

Soil samples will be retained until the associated report(s) are finalized. Rock core samples will be kept through construction of the project and will be stored at the ALDOT Drill Crew Warehouse.

4.0 Project Types and Scope of Work Guidance

For typical project types, guidance on field exploration scope, analysis methods, and reporting requirements are provided in the following sections. Additionally, an overview of Load and Resistance Factor Design (LRFD) design methodology is provided in Section 4.1.

- Section 4.2 – Bridge Foundations
- Section 4.3 – Bridge Culverts
- Section 4.4 – Retaining Walls
- Section 4.5 – Slope Studies
- Section 4.6 – Landslide Studies
- Section 4.7 – Soil Survey
- Section 4.8 – Sign, Lighting, and Signal Pole Foundations
- Section 4.9 – Sound Barrier Walls
- Section 4.10 – Other Geotechnical Considerations
    - Section 4.10.1 – Soft Soils
    - Section 4.10.2 – Ground Improvement
    - Section 4.10.3 – Sinkholes
    - Section 4.10.4 – Rockfall
    - Section 4.10.5 – Mine Studies
    - Section 4.10.6 – Erosion Control

4.1 Current Practice and Overview of Load and Resistance Factor Design

4.1.1 *Current ALDOT Geotechnical Design Practice*

Foundation bearing resistances and settlement should be calculated using LRFD methods as presented in *AASHTO LRFD Bridge Design Specifications* and relevant FHWA manuals. This includes foundations (bridges, bridge culverts, and retaining walls), ground anchors for soil nail walls and tiebacks, ground improvement, etc.

Calculation methods for foundation nominal bearing resistance (i.e., "ultimate bearing capacity" under the previous allowable stress design [ASD] methods), settlement, and global slope stability ("overall stability" of foundations) remain unchanged from recent practice. Following the intent of LRFD, the Engineer should consider average material properties in engineering design calculations and account for uncertainty and/or variability in the selection of resistance factors.

The Engineer should continue to report factors of safety for global slope stability. Unless otherwise indicated, the minimum required factor of safety is the inverse of the resistance factor as provided in *AASHTO LRFD Bridge Design Specifications*.

4.1.2 *Overview of LRFD*

The concepts behind LRFD are presented in detail in *AASHTO LRFD Bridge Design Specifications* and various FHWA geotechnical publications, including those referenced in Section 1.2. In general, the intent of LRFD is to account for uncertainties to provide a consistent level of risk and reliability throughout the structure. Previously under ASD, uncertainty was combined into the factor of safety. LRFD separately accounts for uncertainty in loads and uncertainty in the available geotechnical resistance (including uncertainty in the calculation methods used for geotechnical resistance). The basic equation for the LRFD method is:

$$\sum \eta_i \gamma_i Q_i \leq \sum \varphi_i R_i = R_r$$

where: η_i = a load modifier to account for ductility, redundancy, and operational importance
γ_i = load factor
Q_i = force effect (service level load)
φ_i = resistance factor for resistance component i
R_i = nominal value of resistance component i
R_r = factored resistance

The three limit states that must be considered from a geotechnical standpoint are:

- Service Limit State: Loads that must be considered in evaluating deformation/settlement of foundations.
- Strength Limit State: Loads that must be considered in evaluating required geotechnical resistance for foundations.
- Extreme Event Limit State: Loads that must be considered in conjunction with associated change in ground conditions, such loss of support due to scour or liquefaction, in evaluating required geotechnical resistance for foundations.

Based on applicable limit states, the foundation system should have a factored resistance greater than or equal to the factored loads.

4.2 Bridge Foundations

4.2.1 *Overview*

Bridge foundation studies will generally include the following steps:

1) The Engineer should review project and site information (see Section 3.1 and Section 4.2.2).
2) The Engineer should develop a Geotechnical Exploration Plan that is scoped to provide data to evaluate the foundation alternatives under consideration (see Section 3.1.2 for general requirements and Section 4.2.3 for additional considerations for bridge foundation studies).
3) Perform the geotechnical exploration upon approval of the Geotechnical Exploration Plan by the Geotechnical Division.
4) The Engineer should perform analyses and develop recommendations for the foundation type(s) under consideration (Section 4.2.4).
5) The Engineer should issue a Bridge Foundation Report (see Section 4.11 for general report requirements with other requirements noted in the following subsections).

Geotechnical analyses should be performed in accordance with the methods presented in the current edition of *AASHTO LRFD Bridge Design Specifications* or relevant FHWA manuals as referenced in the following subsections.

Procedural Guidelines for County Projects (issued on October 31, 2017) can be found in the Design Section of the Local Transportation Bureau website at https://www.dot.state.al.us/publications.html.

4.2.2 *Provided Information*

The Structural Engineer will provide the following to the Engineer:

1) A bridge plan and profile drawing;
2) Preliminary foundation types and sizes; and
3) The design loads (service, strength, and extreme event limit states) for the preliminary foundation types and sizes.

After the geotechnical exploration is complete, the Engineer should submit a "Request for Scour Analysis" memo to the Structural Engineer, which should also include the preliminary LPILE parameters recommended by the Engineer (see Section 4.2.5.1). The Structural Engineer will then provide scour depths at the bridge foundations for both the design flood scour and the check flood scour cases. Separately, the Structural Engineer may revise the foundation types, sizes, and/or loads based on the scour data and the recommended LPILE parameters.

4.2.3 *Geotechnical Exploration Scope*

In evaluating bridge foundations, the Engineer will consider the feasibility of shallow foundations (spread footings) and deep foundations (pile footings, pile bents, drilled shafts, and micropiles) for support, regardless of the preliminary foundation type identified by the Structural Engineer on the project plans. The Engineer may eliminate one or more foundation types from consideration based on prior experience or experience with similar structures and project specific information gleaned during the site review (see Section 3.1.1).

For the foundation types considered potentially viable by the Engineer, data collection should be scoped in the Geotechnical Exploration Plan (see Section 3.1.2) such that the alternatives can be evaluated. The Engineer must review/understand the data requirements for the particular project, site, and foundations type(s) under consideration, and select appropriate field exploration and laboratory testing methods to obtain subsurface data and develop and establish the required geotechnical design parameters (see Section 3.2).

4.2.3.1 Boring Layout and Depth

Guidance is provided in Table 3-1. Borings should be advanced through unsuitable materials (i.e., organic soils, peat, muck, humus, liquefiable soils, etc.) and suitably far into competent material for foundation support. The Engineer will estimate the likely competent material type (e.g., soil type and minimum SPT N-values or bedrock core recovery/RQD requirements) and establish minimum SPT/rock core drilling penetrations into such competent material as part of the Geotechnical Exploration Plan.

4.2.3.2 Additional Field/Lab Data and Exploration Considerations

The Engineer should also scope intact sampling, in-situ testing, and/or laboratory testing to aid in developing geotechnical parameters. For bridge foundation projects:

- Field soil classifications should be verified with laboratory classification testing.
- Field and/or laboratory testing should be performed to support evaluation of foundation settlements and resistances for the foundation alternatives under considerations. This will generally require development

of soil design parameters comprising moisture contents, unit weights, consolidation/compressibility parameters, undrained shear strengths, effective stress friction angle, rock compressive strengths, etc. Section 3.2 gives an overview of more commonly employed field and laboratory testing methods used to aid in development of these properties.

- Representative samples from the near-surface soils, groundwater, and surface water should be tested for corrosion potential indicators, as noted in Section 3.2.3.3, for the purpose of determining the corrosion potential and appropriate pile or concrete type (steel or concrete piles or steel encased by concrete).
- Seismic site class must be determined according to Section 3.10.3.1 of *AASHTO LRFD Bridge Design Specifications*.

Additionally, data should be developed to address the following considerations, as applicable, along with other project or site-specific data needs identified by the Engineer:

- Data for scour analysis: Where foundations may be exposed to scour, laboratory tests will be performed so that median grain size (D_{50}) information can be provided to the Structural Engineer for soil strata that may be exposed to scour (see Section 4.2.5.1).
- Abutment slope and approach embankment side slopes: Unless specified otherwise by the State Geotechnical Engineer, the Engineer will evaluate spill through abutment slopes and approach embankment side slopes within 50 feet of the bridge abutments. Data should be collected to evaluate slope stability of the proposed slopes and to provide recommendations for suitably stable slopes (see Section 4.5 for guidance on evaluating slopes).
- Downdrag/drag loading: Placement of approach embankments and structural fill material may result in negative skin friction (drag loads) on the surface of the piles supporting the abutment foundations. These drag loads have the potential to increase settlement of vertical friction piles or to overstress piles bearing on rock. To analyze drag loads and potential mitigation measures, compressibility/consolidation data (to analyze settlement vs depth) must be collected as part of the exploration. Note, settlement estimates due to downdrag may be additive to pile settlements due to structural loads depending on construction sequencing and the time-rate of settlement due to embankment placement.
- Expansive/collapsible soils: For spread footings, efforts should be made to identify the presence of expansive or collapsible soils in the vicinity of the footings.

4.2.4 *Perform the Geotechnical Exploration*

Once the Geotechnical Division has approved the Geotechnical Exploration Plan, the Engineer will proceed with the geotechnical exploration.

If soft soils or sinkholes/karst features that may impact the project are encountered during the course of the geotechnical exploration, the scope should be modified accordingly (see Section 4.10 for guidance) with approval of the State Geotechnical Engineer.

4.2.5 *Interpretation of Data and Development of Recommendations*

Upon completion of the geotechnical field exploration and laboratory testing activities, the Engineer will interpret the data, analyze the foundation alternatives, and develop recommendations. The Engineer will:

- Prioritize the submittal of a "Request for Scour Analysis" memo (which includes recommended LPILE parameters) to the Structural Engineer (see Section 4.2.5.1).

- Delineate the project into Geotechnical Sites and develop subsurface stratigraphy and relevant geotechnical parameters for the subsurface strata (see Section 4.2.5.2).
- Analyze the foundation alternatives under consideration as generally outlined in the following sections and according to procedures and guidance in *AASHTO LRFD Bridge Design Specifications*.
    - Section 4.2.5.3.1 – Driven piles
    - Section 4.2.5.3.2 – Drilled shafts
    - Section 4.2.5.3.3 – Micropiles
    - Section 4.2.5.3.4 – Spread footings

4.2.5.1 Scour Analysis Request and Recommended LPILE Parameters

To evaluate the effects of bridge scour on the foundations, the Engineer must submit a request to the Structural Engineer to perform a scour analysis. The request will be submitted as a memo containing the following data:

- Soil stratigraphy at each bridge substructure, which should include layer depths/elevations, material type, and the median particle size, D_{50}, for each layer as interpreted from laboratory test data;
- Digital copies (PDF) of preliminary/draft graphical boring logs; and
- Digital copies (PDF) of laboratory grain size distribution test reports.

If the foundations are to be embedded in rock, the Engineer, after consultation with the State Geotechnical Engineer, may assume no formal scour analysis is required and set the scour limit as the top of the rock. This assumption and its basis will be included in the report.

Additionally, the Engineer will provide recommended LPILE input parameters for lateral analysis of the bridge foundations. LPILE (Ensoft, Inc.) is a commonly used program by the Structural Engineer to analyze lateral deflections for deep foundations. Guidance on selecting LPILE parameters is provided in Table 4-1 below; however, the Engineer should select values appropriate for the project/site based on their understanding of the site and subsurface data.

The Structural Engineer will then provide both design flood and check flood scour depths/elevations for the bridge foundations. Separately, the Structural Engineer may revise the foundation types, sizes, and/or loads based on the scour analysis and the provided LPILE parameters.

Table 4-1: Guidance on Selecting LPILE Parameters

Material Type	Relative Density		Undrained Shear Strength, s_u (psf)	K_s (static; pci)	K_c (cyclic; pci))	SPT N-value (bpf)	E_{50}
Clay	Soft		250 – 500	30	--	≤4	0.2
	Medium		500 – 1,000	100	--	4 – 8	0.01
	Stiff		1,000 – 2,000	500	200	8 – 15	0.007
	V. Stiff		2,000 – 3,750	1,000	400	15 – 30	0.005
	Hard		3,750 – 6,250	2,000	800	30 – 50	0.004
	50+ bpf		6,250 – 8,000	2,000	800	50 – 64	0.004
Sand	Above Water Table	Loose	--	25	25	4 – 10	--
		Med.	--	90	90	10 – 30	--
		Dense	--	225	225	30 – 50	--
	Below Water Table	Loose	--	20	20	4 – 10	--
		Med.	--	60	60	10 – 30	--
		Dense	--	125	125	30 – 50	--

Notes: (1) For rock, use rock mass modulus, E_m, for k values and $E_{50}<0.001$.

4.2.5.2 Delineation of Geotechnical Sites and Subsurface Stratigraphy

Upon review of historical data (e.g., pile driving records or load test data) and data developed during the geotechnical exploration, the Engineer will divide the project into discrete “sites” (referred to as Geotechnical Sites). Delineation of Geotechnical Sites is of particular importance where load testing and the associated higher resistance factors may be used.

Each Geotechnical Site will be defined as an area with similar geology, subsurface stratification, groundwater conditions, soil and/or rock properties, scour depths, and foundation type, sizes, and loads. Note that a Geotechnical Site may range from encompassing the entire project for highly uniform subsurface conditions to a single foundation element for highly variable conditions. The Engineer will provide justification for delineation of Geotechnical Sites.

An idealized soil profile will be developed for each Geotechnical Site showing subsurface stratigraphy and design material properties for each soil and rock stratum. These idealized profiles and material properties will be presented in the Bridge Foundation Report.

See Appendix C for example figures for delineation of Geotechnical Sites and subsurface stratigraphy.

Guidance on developing material properties can be found in the following:

- *AASHTO LRFD Bridge Design Specifications*,
- *GEC 5 – Geotechnical Site Characterization (FHWA-NHI-16-072)*, and
- Other FHWA manuals for specific foundation types.

4.2.5.3 Evaluation of Foundation Alternatives

Guidelines are provided in the following subsections for specific foundation types. Additionally, there are several general considerations that may apply to a project:

- Lateral loading: Lateral loading will be addressed by the Structural Engineer. The Structural Engineer requires recommended LPILE parameters from the Engineer as noted in Section 4.2.5.1.
- Settlement: Settlement is calculated in the service limit state using a resistance factor of $\varphi=1.0$ (per 10.5.5.1 of *AASHTO LRFD Bridge Design Specifications*). Transient loads may be omitted from settlement analyses for foundations bearing on or in cohesive soil deposits that are subject to time-dependent consolidation settlements (per Section 10.5.2.2 of *AASHTO LRFD Bridge Design Specifications*). Settlement criteria for bridge foundations is provided in Table 4-2 below.

Table 4-2: Settlement Criteria for Bridge Foundations

Total Settlement	Differential Settlement over 100 feet within Pier or Abutment, and Differential Settlement Between Piers	Action
$\Delta H \leq 1$ in	$\Delta H_{100} \leq 0.75$ in	Design and construct
1 in < $\Delta H \leq 4$ in	0.75 in < $\Delta H_{100} \leq 3$ in	Ensure structure can tolerate settlement
$\Delta H > 4$ in	$\Delta H_{100} > 3$ in	Obtain approval prior to proceeding with design and construction (1)

Notes: (1) Approval of ALDOT State Geotechnical Engineer and ALDOT Bridge Design Engineer required.

- Scour: The Engineer will consider the effects of the design flood at the strength limit state, and the effects of the check flood (and other hydraulic events) at the extreme event limit state. The Engineer will report this estimated side resistance lost due to scour in the Bridge Foundation Report.
- Downdrag/drag loading: Placement of approach embankments may result in long term settlement that can impose drag loading on abutment foundations. Drag loads develop when downward settlement of soil around deep foundation elements imposes additional load. Downdrag should be evaluated in the strength and extreme event limit states. See applicable sections of *AASHTO LRFD Bridge Design Specifications* and related FHWA publications for guidance on analyzing the effects of downdrag and drag loading on deep foundations.

  It will typically be preferable to mitigate conditions that may lead to downdrag. Generally, this will involve driving abutment piles after most of the settlement imposed by approach embankment fill has occurred. Settlement can be accelerated using wick drains and/or surcharge loading. Alternatively, piles may be encased in an effective bond breaker, such as Yellow Jackets®, etc. Battered piles should be avoided where downdrag is expected.
- Liquefaction: Potential for liquefaction should be evaluated following the guidance in Section 10.5.4.2 of *AASHTO LRFD Bridge Design Specifications*. A site-specific seismic hazard analysis can be performed as an option at the request of the Structural Engineer.

4.2.5.3.1 Driven Piles

4.2.5.3.1.1 General

The Engineer will evaluate driven pile geotechnical resistance and settlement and provide:

- Estimated tip elevations and resistances vs depth curves,
- Potential pile group settlement for the tip elevations provided, and
- Recommendations regarding load testing, construction, and inspection.

4.2.5.3.1.2 General LRFD Analysis Methodology

The following provides general guidance on the steps required to estimate pile tip elevations and settlement. This process should be performed for each Geotechnical Site. This section provides an overview of steps that the Engineer may consider in order to provide driven pile recommendations. Calculation methods are well documented in both *AASHTO LRFD Bridge Design Specifications* and *GEC 12 – Design and Construction of Driven Pile Foundations – Volumes I and II* (FHWA Publication Nos. FHWA-NHI-16-009 and FHWA-NHI-16-010), and the Engineer will select suitable methods from either of these references.

1) Calculate nominal axial bearing resistance: The Engineer will develop axial nominal bearing resistance curves as a function of tip elevation, including consideration of single piles and pile groups. Depending on provided loads and scour information, nominal axial compression resistance cases should include, but are not limited to, consideration of scour (no scour, design flood scour, and check flood scour cases) and drag load.
2) Evaluate factored resistance: For each of the nominal resistance cases under consideration (as noted in Item 1 above), the Engineer will develop factored resistance curves by applying resistance factors, φ, as a function of the Nominal Resistance Verification Program (NRVP) as outlined below in Section 4.2.5.3.1.3. Additionally, resistance factors should be in accordance with:
    - Table 10.5.5.2.3.-1 of *AASHTO LRFD Bridge Design Specifications* (strength limit state) or
    - Tables 7-1 and 7-2 of FHWA-NHI-16-009 (strength limit state) and
    - For the Extreme Event Limit State, $\varphi_{compression}$=1.0 and φ_{uplift}=0.8 in accordance with Section 10.5.5.2 of *AASHTO LRFD Bridge Design Specifications*.

    Note that for small pile groups (less than 5 piles), the resistance factors should be reduced by 20 percent (per Section C10.5.5.2.3 of *AASHTO LRFD Bridge Design Specifications)*.
3) Select NRVP and estimate pile tip elevations: The Engineer will recommend one or more suitable NRVPs considering the pile resistance mobilization mechanism, project size (number of piles), and expected access conditions for static load testing. When providing recommendations for NRVP-A sites, the Engineer will also provide recommendations NRVP-C in the event static load tests cannot be performed at the project site. For each NRVP recommended, the Engineer will provide estimated pile tip elevations such that, for cases considered, the factored resistance is greater than or equal to the factored load (strength and extreme event limit states) and provide factored pile bearing resistance curves as a function of tip elevation.
4) Estimate potential settlement: Based on the estimated pile tip elevation, potential settlement should be estimated using the service limit state loads for both the no scour and design flood scour profiles.

4.2.5.3.1.3 Selection of Nominal Resistance Verification Program

This section provides guidance to the Engineer in selecting NRVP. The Engineer will select what they consider to be the more appropriate NRVP for each Geotechnical Site based on pile resistance mobilization mechanism, project size (number of piles), and expected access conditions for static load testing. The purpose of the NRVP is to verify design assumptions during construction of a project. With a more rigorous field verification program, a higher resistance factor (φ) can be applied. The NRVPs listed in Table 4-3 below are presented in decreasing order relative to the quality and quantity of field verification testing.

Table 4-3: Nominal Resistance Verification Programs

Nominal Resistance Verification Program (NRVP)	Minimum Testing Requirements	Resistance Factor, φ	Resistance Factor for Small Pile Groups (<5 piles/group)
NRVP-A	Driving criteria established by successful static load test of at least one pile per Geotechnical Site, quality control by dynamic testing of at least two production piles per Geotechnical Site, but no less than 2 percent of the production piles.	0.80	0.64
NRVP-B	Driving criteria established by successful static load test of at least one pile per Geotechnical Site without dynamic testing.	0.75	0.60
NRVP-C	Driving criteria established by dynamic testing conducted on 100 percent of production piles	0.75	0.60
NRVP-D	Driving criteria established by dynamic load test, quality control by dynamic testing of at least two production piles per Geotechnical Site, but no less than 2 percent of production piles.	0.65	0.52
NRVP-E	Driving criteria established by wave equation analysis, without pile dynamic measurements but with field confirmation of hammer performance by driving to refusal.	0.50	0.40

4.2.5.3.1.3.1 NRVP-A

NRVP-A is generally applicable for Geotechnical Sites where driven piles will develop resistance through some combination of side and base resistance, where no rock was encountered during the geotechnical exploration (or the rock surface is very deep), and where there is no obvious bearing stratum for end-bearing piles. The following load testing requirements apply to NRVP-A:

1) Static load test of at least one pile per Geotechnical Site.
   a) The static test pile will be driven to the elevation shown in the plans or until the required nominal static bearing resistance is reached according to dynamic measurements with signal matching, whichever is deeper.
   b) A set check will be performed one hour after end of driving (EOD) to provide initial estimates of time-dependent changes in static pile bearing resistance.
   c) The pile will be statically load tested no sooner than 36 hours after EOD.

d) If the pile fails the static load test (does not achieve the required nominal static resistance), the State Geotechnical Engineer will be immediately notified, and the pile will be spliced (if necessary) and re-driven according to the requirements in Section 505 of the ALDOT *Standard Specifications for Highway Construction* or to the revised tip elevation recommendations provided by the State Geotechnical Engineer. This process will be repeated until the test pile can support the required nominal geotechnical resistance.

e) After the static test pile meets the required nominal geotechnical resistance, the pile will undergo a dynamic re-strike to obtain revised dynamic measurements that consider the effects of pile setup. If the load-settlement curve obtained from the static load test reaches the failure criteria as outlined in ALDOT Standard Specification 505 (assuming that this load is greater than the required nominal resistance), this dynamic restrike requirement may be waived.

f) Using the Wave Equation, the contractor will be provided a series of bearing curves based on hammer stroke and blow count and will drive the remaining piles in that Geotechnical Site to either the test pile tip elevation or the elevation to which the provided bearing curves show a resistance equal to the design factored load divided by the resistance factor, whichever is deeper.

2) Dynamic testing with signal matching using the Case Pile Wave Analysis Program (CAPWAP) should be performed during installation of at least two piles per Geotechnical Site and no less than two percent of production piles. The test piles should also undergo a re-strike a minimum of 36 hours after EOD. These dynamic measurements will be analyzed using signal matching software to provide an estimate of static bearing resistance. The estimated resistances at the EOD and re-strike should be equal to or greater than the design factored load divided by the resistance factor. Pile setup should be evaluated based on results of the re-strike. Results of the dynamic testing and the static test should be compared and demonstrate that the design required resistance is achieved.

4.2.5.3.1.3.2 NRVP-B

Geotechnical Sites where the data gained from dynamically monitoring piles during driving will not have a significant impact on final pile tip elevations will be classified as NRVP-B. This may be thought of as a "proof testing" program, where piles are driven to estimated tip elevations and then simply statically load tested to verify nominal geotechnical resistance. NRVP-B may be more applicable to small projects (1 or 2 span bridges) with a small number of lightly loaded piles, where the cost of doing comprehensive pile testing may outweigh the benefit of simply driving the piles deeper based on a lower resistance factor. The following requirements apply to NRVP-B:

- Static load test of at least one pile per Geotechnical Site (without dynamic testing) following the static load test guidance for NRVP-A.

4.2.5.3.1.3.3 NRVP-C

NRVP-C should be applied to Geotechnical Sites where static load testing a pile is logistically or financially prohibitive, such as a pile driven over water. The following requirements apply to NRVP-C:

- Dynamic testing with signal matching on 100 percent of production piles. Dynamic re-strike will be a minimum of 36 hours after EOD. These dynamic measurements will be analyzed using signal matching software to provide an estimate of static bearing resistance. This resistance must be equal to or greater than the provided factored load divided by the geotechnical resistance factor.

4.2.5.3.1.3.4 NRVP-D

NRVP-D should be applied to Geotechnical Sites where static load testing a pile is logistically or financially prohibitive, or where dynamic testing of 100 percent of production piles is not performed. The following requirements will apply to NRVP-D:

1) Driving criteria established by dynamic test pile with signal matching.
    a) The dynamic test pile will be driven to the elevation shown in the plans or until the required nominal static bearing resistance is reached according to dynamic measurements with signal matching, whichever is deeper.
    b) The pile will undergo a dynamic re-strike no sooner than 36 hours after EOD.
    c) Results of the dynamic re-strike will be analyzed with signal-matching software to estimate the nominal static bearing resistance of the pile.
    d) If the estimated nominal static resistance of the pile after the dynamic re-strike with signal matching is less than the required nominal geotechnical resistance using the geotechnical resistance factor for final design, the State Geotechnical Engineer will be immediately notified, and the pile will be spliced (if necessary) and re-driven according to the requirements in ALDOT Standard Specification 505 or to the revised tip elevation recommendations provided by the State Geotechnical Engineer. This process will be repeated until a passing pile is achieved.
    e) Using the Wave Equation, the contractor will be provided a series of bearing curves based on hammer stroke and blow count and will drive the remaining piles in that Geotechnical Site to either the test pile tip elevation or the elevation to which the provided bearing curves show a resistance equal to the design factored load divided by the final geotechnical resistance factor, whichever is deeper.
2) Dynamic testing with signal matching of at least two piles per Geotechnical Site but no less than two percent of production piles. Dynamic re-strike will be a minimum of 36 hours after EOD. These dynamic measurements will be analyzed using signal matching software to provide an estimate of static bearing resistance. This resistance must be equal to or greater than the provided factored load divided by the geotechnical resistance factor.

4.2.5.3.1.3.5 NRVP-E

NRVP-E should be applied to Geotechnical Sites in which the piles are expected to be driven to refusal into rock or other bearing stratum in which the Engineer feels confident the pile will reach refusal criteria as outlined in ALDOT Standard Specification 505. NRVP-E does not require field verification of nominal geotechnical resistance since, in these situations, the geotechnical nominal bearing resistance of the pile is assumed to approach the pile's structural resistance. If the pile is expected to be driven into a hard clay, very dense sand, or other competent material that may allow some pile penetration before refusal, a static analysis should be used to determine the depth of penetration into the bearing stratum. The following requirements apply to NRVP-E:

- Static analysis and/or engineering judgment to determine pile tip elevations.
- No field verification of geotechnical resistance.

4.2.5.3.1.4 Additional Considerations and Recommendations to be Provided

The Engineer will also provide recommendations regarding the following:

- Discussion of pile drivability, with particular attention paid to underlying soft soils and/or boulders, and the potential effects of pile driving on adjacent structures. If difficult driving is anticipated, this should be noted in the Bridge Foundation Report along with the need for pre-drilling or pile driving shoes.
- The Engineer should recommend the type of pile and reasons for the choice and/or exclusion of pile types. In highly corrosive areas (such as Mobile and Baldwin counties), the Engineer will evaluate the use of concrete piles.
- Comments and recommendations regarding pile relaxation, if anticipated.
- When driving piles subject to potential scour, the required pile driving resistance should consider compensating for the estimated pile side resistance above the scour depth, and pile setup effects should be considered in the evaluation. At least one pile per site condition should be dynamically tested and monitored for estimating the side resistance within the potential scour layers.

4.2.5.3.2 Drilled Shafts

4.2.5.3.2.1 General

The Engineer will evaluate drilled shaft bearing resistance and settlement and provide:

- Minimum (highest permissible) or target shaft tip elevations,
- Estimated potential shaft settlement for the tip elevations provided, and
- Recommendations regarding load testing, construction, and inspection.

4.2.5.3.2.2 General LRFD Analysis Methodology

The following provides general guidance on the steps required to develop shaft tip elevations and potential settlement estimates. This process should be performed for each Geotechnical Site. This is not prescriptive but rather an overview of steps that the Engineer may take to arrive at the required recommendations. Calculation methods are provided in both *AASHTO LRFD Bridge Design Specifications* and *GEC 10 – Drilled Shafts: Construction Procedures and LRFD Design Methods* (FHWA Publication No. FHWA-NHI-18-024). The Engineer will select suitable methods from either of these references.

1) Calculate axial nominal bearing resistance: The Engineer will develop axial nominal bearing resistance curves as a function of shaft tip elevation. Depending on provided loading and scour information, nominal axial compression/uplift resistance cases should include, but are not limited to, consideration of scour (no scour, design flood scour, and check flood scour cases) and drag load.
2) Apply resistance factors: For each of the nominal bearing resistance cases under consideration (as listed in Item 1 above), the Engineer will develop factored resistance curves by applying resistance factors, φ, to the nominal resistance components (side resistance and end bearing) as given in:
    - Table 10.5.5.2.4-1 of *AASHTO LRFD Bridge Design Specifications* (Strength Limit States), or
    - Table 8-4 of *GEC 10 – Drilled Shafts: Construction Procedures and LRFD Design Methods* (FHWA Publication No. FHWA-NHI-18-024).

   Note that per Section 10.5.5.2.4 of *AASHTO LRFD Bridge Design Specifications,* resistance factors will be reduced by 20 percent where a single shaft supports a bridge pier.

3) Select shaft tip elevations: The Engineer will select a tip elevation such that, for cases considered, the factored resistance is greater than or equal to the factored load (strength and extreme event limit states).
4) Estimate potential settlement: Based on the selected shaft tip elevation, potential settlement will be estimated using the service case limit state for both the no scour and design flood scour profiles.

4.2.5.3.2.3 Additional Considerations and Recommendations to be Provided

The following will also be considered by the Engineer, as applicable to the project:

- Rock bearing shafts: For rock-bearing shafts where resistance is achieved in base resistance alone (without considering rock socket side resistance), it is not necessary to develop resistance curves as noted above. Tip elevations will be governed by bedrock conditions that are suitable for shaft base resistance.
- Shaft load test: If load testing is under consideration for a project, the Engineer can recommend tip elevations based on the applicable higher resistance factors in compression and/or uplift. However, the Engineer should be aware of the comments provided in Section C10.5.5.2.4 of *AASHTO LRFD Bridge Design Specifications* regarding the use of higher resistance factors.

In addition to shaft tip elevations and potential settlement estimates, the Engineer will also provide recommendations regarding the following:

- Suggestions of probable methods of construction, whether the shaft may be cast using slurry, temporary or permanent casing, or in the dry will be included in the foundation recommendations. These suggestions will not be statements of fact and will contain caveats stating the suggestions are based upon interpreted site conditions.
- The Engineer will be prepared to evaluate and make recommendations on data collected from the cross-hole sonic logging report, if required by the wet method of construction.
- If coring is required to verify the cross-hole sonic logging results, the Engineer will be prepared to evaluate and make recommendations on the cores collected.
- The Engineer will make recommendations on the need for permanent casing and the depth for casing, if required.
- For rock bearing drilled shafts, a minimum socket of one shaft diameter into competent bedrock is required (regardless of the need for side resistance to achieve the required resistance).
- The Engineer will indicate whether probe/boring/core holes are required at the planned/recommended tip elevation to verify subsurface conditions. Specify which type of hole is needed, the number of holes required per shaft (if more than one), and a minimum depth of the hole(s). Provide justification if holes deeper than 10 feet below the tip elevations are recommended.

4.2.5.3.3 Micropiles

4.2.5.3.3.1 General

Micropiles should generally be considered as an alternative to rock bearing drilled shafts when: (1) bedrock conditions are less favorable for drilled shafts base resistance (e.g., karst bedrock or bedrock with joints/seams of weathered or compressible material), (2) where boulders or debris fill are present, or (3) when foundations must be constructed in low-headroom conditions or near vibration sensitive structures. A group of micropiles is typically required to replace a single shaft. Guidelines provided in this section apply to micropiles installed into bedrock only; although, soil-bearing micropiles could be evaluated following published guidance if the Engineer considered them to be a viable alternative.

Micropiles types are typically designed by a specialty contractor. Micropile resistances are verified through load testing, and tip elevations are determined in the field as a minimum penetration through continuous rock. Therefore, the Engineer will evaluate the need for micropiles and provide:

- Justification for the use of micropiles.
- Preliminary bond zone strength based on historical site load test data or presumptive values provided in:
    - Table C10.93.5.2-1 of *AASHTO LRFD Bridge Design Specifications, or*
    - Table 5-3 of *Micropile Design and Construction Reference Manual* (FHWA Publication No. FHWA-NHI-05-039).
- Estimated tip elevations or micropile lengths for quantity estimation using an appropriate resistance factor.
- Estimated potential micropile group settlement.
- Recommendations regarding load testing, construction, and inspection:
    - Load testing should conform to the requirements given in *Micropile Design and Construction Reference Manual* (FHWA Publication No. FHWA-NHI-05-039) and Plans and Specifications.
    - Verification load test: The Engineer should recommend at least one performance load test should be performed per Geotechnical Site before production micropile installation begins.
        - The verification load test is intended to show that the assumed bond zone strength and installation methods are suitable.
        - The test pile (not a production micropile) should be located as close as practical to the proposed micropile supported footing and installed using the same equipment and methods that will be used for the production micropiles.
    - Proof load testing: The Engineer should recommend that proof load testing be performed on at least one micropile per footing/group and on at least five percent of production micropiles.

4.2.5.3.4 <u>Spread Footings</u>

4.2.5.3.4.1 General

Where spread footings are indicated on the plans, or where spread footings may be a viable alternative to the indicated foundation type, the Engineer will evaluate to use of spread footings. Shallow footings are not to be used in soils with insufficient bearing resistance, in soils where potential settlement exceeds the tolerance of the supported structure, where potential differential settlement exceeds the tolerance of the structure, or where excessive scour or erosion could endanger the integrity of the foundation. Spread footings are not generally considered economical at depths greater than 10 feet.

If a shallow foundation is feasible, the Engineer will evaluate bearing resistance and settlement for spread footings and provide:

- Footing bearing elevation and bearing resistance,
- Estimated potential settlement, and
- Recommendations regarding load testing, construction, and inspection.

4.2.5.3.4.2 Overview of LRFD Analysis Methodology

The following provides general guidance on evaluating spread footings for bridge foundations. Calculation methods for calculating nominal bearing resistance and settlement are well documented in both *AASHTO LRFD Bridge Design Specifications* and *GEC 6 – Shallow Foundations* (FHWA Publication No. FHWA-SA-02-054), and the Engineer will select suitable methods from either of these references.

The Engineer will select footing size and bearing elevation such that the factored bearing resistance is greater than or equal to the factored loads. Resistance factors for the strength limit states are as provided in Table 10.5.5.2.2-1 of *AASHTO LRFD Bridge Design Specifications*. Resistance factors for the extreme event limit state will be taken as $\varphi=1.0$. Footings bearing on rock should take into consideration the presence, orientation, and condition of discontinuities.

Potential settlement will be estimated using the service limit state. Typically, settlement will govern for soil bearing footings. If settlement criteria are not provided by the Structural Engineer, the Engineer will estimate potential total and differential settlement for the footings at the size and bearing elevation determined from the bearing resistance calculation. Otherwise, the Engineer will also consider the footing size and bearing elevations to meet both the required bearing resistance and the provided settlement criteria.

4.2.5.3.4.3 Additional Recommendations to be Provided

Additionally, the Engineer will provide recommendations regarding the following:

- A detailed description of the founding material and specific inspection requirements, so that the project inspector can verify the material is as expected.
- The need for dewatering, sheeting, or shoring, and other construction considerations.

4.2.6 *Bridge Foundation Report*

The Engineer will provide the Geotechnical Division with a Bridge Foundation Report as outlined in Section 4.11, including information relevant to the specific foundation types as provided in the preceding sections and the following, as applicable:

1) A recommended foundation type and other foundation types that may be viable alternatives along with justifications. Recommendations should be summarized by substructure or Geotechnical Site (preferably in a table) and include the following, at a minimum:
    a. Provided loads (service, strength, and extreme event limit states);
    b. The recommended foundation type and size;
    c. LRFD resistance factors for the limit states considered;
    d. Foundation bearing elevations;
    e. Driven Piles: Provide the number of test piles, number of tests, required driving resistance, estimated tip elevations, and minimum tip elevations (see Table A-4 in Appendix A for an example Driven Pile Foundation Recommendation Table); and
    f. Drilled shafts: Provide the controlling limit state, recommended or minimum shaft tip elevation, and permanent casing elevation (see Table A-5 in Appendix A for an example Drilled Shaft Foundation Recommendation Table).

2) A summary of the Geotechnical Sites delineated for the project, including a site plan showing the Geotechnical Site locations. Geotechnical Site delineation may vary depending on the foundation alternatives provided.
3) The idealized subsurface profile for each Geotechnical Site, including relevant geotechnical properties interpreted for the strata (including unit weight, drained/undrained strength parameters, compressibility/consolidation parameters, and others as necessary for the foundations under consideration). The interpreted subsurface profiles should be shown graphically on the project profile drawings.
4) Recommended LPILE input parameters previously provided to the Structural Engineer.
5) Foundation resistance curves, tip elevations, potential settlement estimates, and other data as outlined in the preceding sections for the foundation type(s) recommended. The Engineer should note the software or calculation methods used in the design and/or calculations.
6) Load testing recommendations, including recommendations and requirements where they vary from the *Standard Specifications for Highway Construction*.
7) The Engineer will recommend drilled shaft concrete class (Class DS1, DS2, or DS3 concrete) and if the "special requirements for piles constructed in severe exposure conditions" given in the *Standard Specifications for Highway Construction* should be followed for prestress concrete piles based on the results of corrosion potential laboratory testing.
8) Recommendations on spill-through abutment slopes and approach embankment side slopes. If specific slopes, rip rap armoring thickness, rock buttress configuration, etc., are required to achieve the required global slope stability factor of safety (FS≥1.5), the Engineer should provide recommendations, plan notes, estimated quantities, and typical detail drawings suitable for inclusion in the project plans.
9) Evaluation of approach embankments by considering potential settlement of embankments due to consolidation of underlying soils, embankment stability, abutment rotation, horizontal component of vertical settlement of approach embankments, and drag loads on the abutment foundations due to settlement of the embankment and underlying soils. Recommended precautions that should be taken to reduce the possibility of damage to the structure caused by activities associated with the approach embankment. Where applicable, recommendations on ways to accelerate drainage/settlement by evaluating items such as wick drains, surcharging, etc.

4.3 Bridge Culverts

4.3.1 *Overview*

Bridge culverts are defined as culverts with a span length of at least 20 feet. The span length is defined as the widest culvert opening measured along the roadway centerline.

Bridge culvert foundation studies will generally proceed with the following steps:

1) The Engineer will review project and site information (see Section 3.1 and Section 4.3.2).
2) The Engineer will develop a Geotechnical Exploration Plan that is scoped to provide data to evaluate the proposed bridge culvert foundation (see Section 3.1.2 for general requirements and Section 4.3.3 for additional considerations for bridge culvert foundation studies).
3) Perform the geotechnical exploration upon approval of the Geotechnical Exploration Plan by the Geotechnical Division (see Section 4.3.4).

4) The Engineer will perform analyses and develop recommendations (Section 4.3.5).
5) The Engineer will issue a Bridge Culvert Foundation Report (see Section 4.11 for general report requirements, with specific requirements for bridge culvert foundations reports noted in Section 4.3.6).

4.3.2 *Provided Information*

The Engineer will be provided with the culvert location and available plan, profile, and cross section drawings.

If drainage sections are not available and transmitted at the time of the bridge culvert foundation study request, use the tables "Earth Slopes Horizontal to Vertical for Types of Terrain" found in Notes 106, 107, and/or 108 of the ALDOT Special Drawing No. GN-2 in conjunction with the applicable "typical roadway section" and planned roadway profile to establish the probable inlet and outlet locations of the culvert.

4.3.3 *Geotechnical Exploration Scope*

Data collection should be scoped in the Geotechnical Exploration Plan (see Section 3.1.2) such that bearing resistance and potential settlement can be evaluated, and recommendations provided. The Engineer must review/understand the data needs for the particular site and project and select appropriate field exploration and laboratory testing methods to develop the required geotechnical data (see Section 3.2).

The following can be considered as general guidelines for establishing the geotechnical exploration scope; however, the Engineer must consider project specific needs to scope the exploration. These include:

- Typically, culverts having a length of 300 feet or less should be drilled at mid-length and at each end. Culverts exceeding 300 feet in length should be drilled so that the distance between inlet and outlet borings is equally divided into approximately 150 linear foot increments.
- Borings should extend:
  - A minimum of 10 feet below the bearing elevation, and
  - To a depth below the invert of at least three times the width of the culvert or to SPT refusal, whichever is shallower.

In addition to SPT borings, the Engineer should scope intact sampling, in-situ testing, and/or laboratory testing to aid in developing necessary geotechnical parameters, such as:

- Field soil classifications should be verified with laboratory classification testing.
- Field and/or laboratory testing should be performed to characterize material parameters required to evaluate bearing resistance and potential settlement (see Section 3.2).

4.3.4 *Perform the Geotechnical Exploration*

Once the Geotechnical Division has approved the Geotechnical Exploration Plan, the Engineer will proceed with the geotechnical exploration.

If soft soils or sinkholes/karst features that may impact the project are encountered during the course of the geotechnical exploration, the scope should be modified accordingly (see Section 4.10 for guidance) with approval of the State Geotechnical Engineer.

4.3.5 *Interpretation of Data and Development of Recommendations*

Upon completion of the geotechnical field exploration and laboratory testing program, the Engineer will interpret the field and laboratory data, analyze bearing resistance and potential settlement, and develop recommendations. The Engineer will:

- Develop subsurface stratigraphy and relevant material parameters for the subsurface strata. At a minimum, these parameters should include unit weight, strength (undrained shear strength for cohesive soils, and effective stress friction angle for both cohesive and cohesionless soils), and consolidation/compressibility properties.
- Analyze bearing resistance and potential settlement according to procedures and guidance in *AASHTO LRFD Bridge Design Specifications* or applicable FHWA manuals. Table 10.5.5.2.2.1 in *AASHTO LRFD Bridge Design Specifications* provides recommended resistance factors for spread footing bearing resistance. The Engineer will report nominal and factored bearing resistance.
- Potential total and differential settlement criteria are provided in Table 4-4 below. Settlement should be considered excessive for potential total settlement greater than 2.5 inches or for potential differential settlement greater than 2 inches over 100 feet.

Table 4-4: Settlement Criteria for Bridge Culverts

Total Settlement	Differential Settlement over 100 feet	Action
$\Delta H \leq 1$ in	$\Delta H_{100} \leq 0.75$ in	Design and construct
1 in $< \Delta H \leq 2.5$ in	0.75 in $< \Delta H_{100} \leq 2$ in	Ensure structure can tolerate settlement
$\Delta H > 2.5$ in	$\Delta H_{100} > 2$ in	Obtain approval prior to proceeding with design and construction (1)

Notes: (1) Approval of ALDOT State Geotechnical Engineer and ALDOT Bridge Design Engineer required.

- Bearing pressures should be appropriately factored. If bearing resistance is inadequate or potential settlement is excessive, develop subgrade improvement recommendations, such as undercutting and replacement, aggregate piers (e.g., stone columns, vibro-replacement columns, Geopiers®, etc.), use of lightweight fill to reduce applied pressures, etc. See Section 4.10.1 - Soft Soils for additional guidance.
- For thicker embankment sections, the pressures on buried culverts can be quite high. Guidance should be provided to the structural engineers on the magnitude of pressures to be expected.

4.3.6 *Bridge Culvert Foundation Report*

The Engineer will provide the Geotechnical Division with a Bridge Culvert Foundation Report as outlined in Section 4.11 and including:

- Generalized subsurface profile, including recommended material parameters.
- Nominal and factored bearing resistance for the foundation soils, both with and without subgrade improvement, if required.
- Estimate potential total, differential, and time rate settlement along the culvert alignment.

- Recommendations for subgrade improvement(s) should the bearing resistance appear to be inadequate and/or excessive settlement is anticipated. Recommendations should include estimated quantities for the recommended subgrade improvement approach.

4.4 Retaining Walls

4.4.1 *Overview*

Retaining Wall Foundation Studies should generally include the following steps:

1) The Engineer will review project and site information (see Section 3.1 and Section 4.4.2).
2) The Engineer will develop a Geotechnical Exploration Plan that is scoped to provide data to evaluate the proposed wall (if the wall type is specified), and viable alternatives (see Section 3.1.2 for general requirements and Section 4.4.3 for additional considerations for retaining wall studies).
3) Perform the geotechnical exploration upon approval of the Geotechnical Exploration Plan by the Geotechnical Division (see Section 4.4.4).
4) The Engineer will perform analyses and develop recommendations (see Section 4.4.5).
5) The Engineer will issue a Retaining Wall Foundation Report (see Section 4.11 for general report requirements with specific requirements for retaining wall foundations noted in Section 4.4.6).

4.4.2 *Provided Information*

The Engineer will be provided with the plan, profile, and cross sections for the proposed wall location. The plans may or may not indicate the type of wall.

4.4.3 *Geotechnical Exploration Scope*

In evaluating retaining walls, the Engineer should consider the feasibility of different wall types, regardless of the wall type indicated on the plans. Often, the Engineer can select a more appropriate wall type(s) or eliminate others from consideration based on project requirements, information gleaned during the site review (see Section 3.1.1), and their experience. Generally, the following wall types should be considered for state projects; however, others may be appropriate depending on project or site-specific considerations:

- <u>Cast-in-place (CIP) cantilever wall:</u> Evaluation/design should be in accordance with the dimensions and details shown on ALDOT Standard Drawing RW 10-4.
- <u>Mechanically stabilized earth (MSE) wall</u>: Generally, more economical and more settlement tolerant than CIP walls for fill applications and may be feasible for cut applications if the temporary construction excavation required for an MSE wall is not excessive and does not impact adjacent structures/ROW limits.
- <u>Soil nail wall</u>: Generally, well suited for cut sections (top-down construction method) and should be evaluated as an alternative to wall requiring significant temporary excavations (bottom-up construction methods required for CIP or MSE walls). Soil nail walls are less favorable in cohesionless soils with very few fines (potential constructability issues), and soil nails may be subject to creep in highly plastic cohesive soils. Soil nail wall construction normally requires soil to be stable when cut to a height to five feet prior to installation of the nails. Additional information is provided in *GEC 7 – Soil Nail Walls* (FHWA Publication No. FHWA-NHI-14-007).
- <u>Soldier pile wall</u> (either non-gravity cantilever wall or anchored wall): Suitable as an alternative to soil nail wall in cut sections where soil nails may be subject to creep or where global stability requirements cannot

be met with a soil nail wall. Non-gravity cantilever walls are typically practical with exposed/freestanding/unsupported lengths up to approximately 15 feet, depending on site conditions. Otherwise, anchors (soil or rock tiebacks) are required for taller soldier pile walls.

For the wall type(s) considered by the Engineer to be potentially viable, data collection should be scoped in the Geotechnical Exploration Plan (see Section 3.1.2) such that the alternatives can be evaluated, and recommendations provided. The Engineer must review/understand the data needs for the particular site, project requirements, and wall type(s) under consideration, and select appropriate field and laboratory testing methods to develop the required geotechnical data (see Section 3.2).

4.4.3.1 Boring Layout and Depth

Typically, borings should be placed approximately every 100 feet along the wall alignment; although, spacing could vary between 50 and 200 feet depending on the anticipated subsurface soil and rock consistency. At least three borings should be performed regardless of wall length. Borings should be advanced through soft and/or highly compressible soils and to the shallower of:

- A depth of twice the wall height below the wall bearing elevation;
- A depth below the bottom of the wall where stress increase becomes less than 10 percent of existing effective overburden stress;
- To SPT refusal or bedrock, unless the Engineer selects different criteria based on preliminary analysis.

Additional borings may be warranted at the discretion of the Engineer, as follows:

- To aid in calculating potential differential settlement between the front and back of the wall, additional borings every 100 feet along the wall alignment should be considered at the back of CIP wall footings and MSE wall reinforced zones, particularly where the wall will be placed on sloping terrain. CIP wall footings widths can be assumed based on the wall height and ALDOT Standard Drawing RW 10-4, and MSE wall reinforced zone widths can be assumed as 0.7 times the wall height ($L_{min} = 0.7 \cdot H$) for purposes of boring layout.
- Within the approximate anchor bond stratum for soil nails or tiebacks at a spacing of 100 feet along the wall alignment.
- Uphill and downhill of the wall as necessary to analyze global stability of the wall and associated temporary cut slopes.

4.4.3.2 Additional Field/Lab Data and Exploration Considerations

In addition to SPT/rock core borings as noted above, the Engineer should scope intact sampling, in-situ testing, and/or laboratory testing to aid in developing necessary geotechnical parameters.

- Field soil classifications should be verified with laboratory classification testing.
- Field and/or laboratory testing should be performed to support evaluation of global slope stability, external wall stability modes, potential settlement, and preliminary soil nail/tieback resistance. This will generally require material unit weights, consolidation/compressibility parameters, soil shear strengths, and rock compressive strengths. Section 3.2 gives an overview of more commonly employed field and laboratory testing methods used to aid in development of these properties.

- Representative samples from the near-surface soils, potential wall retained/reinforced zone soil (if available), groundwater, and surface water will be tested for corrosion potential indicators, as noted in Section 3.2.3.3, to determine the soil and water chemical properties.

If the geology or site vicinity is known to have expansive soils (i.e., shrinking/swelling) that may impact the proposed wall or if potentially expansive soils are encountered (as indicated by the soils liquidity indices), the Engineer should scope additional field/laboratory testing to evaluate the shrink/swell potential and to determine the approximate vertical and lateral extents of such soils along the wall alignment.

4.4.4 *Perform the Geotechnical Exploration*

Once the Geotechnical Division has approved the Geotechnical Exploration Plan, the Engineer will proceed with the geotechnical exploration.

If soft soils or sinkholes/karst features that may impact the project are encountered during the course of the geotechnical exploration, the scope should be modified accordingly (see Section 4.10 for guidance) with approval of the State Geotechnical Engineer.

4.4.5 *Interpretation of Data and Development of Recommendations*

Design of the actual wall elements will be the responsibility of the Wall Designer. However, the Engineer must perform analysis to evaluate the feasibility of the wall and establish minimum design requirements. Calculations and analyses should be in accordance with methods presented in *AASHTO LRFD Bridge Design Specifications* or applicable FHWA publications as noted below, and should include the following as applicable for the wall type(s) under considerations:

- Global slope stability (for both short-term [undrained, total stress] and long-term [drained, effective stress] cases):
  - Wall global stability analysis: A minimum global stability factor of safety of 1.5 is required. If the geotechnical parameters are well-defined (based on in-situ and/or laboratory testing) and the slope does not support structural elements (other than the subject wall) or critical utilities, then a minimum global stability factor of safety of 1.3 may be used.
  - Temporary excavation stability: A minimum factor of safety of 1.2 is required for temporary conditions (i.e., during construction activities).
- Wall external stability: Load combinations/factors are provided in Tables 3.4.1-1 and 3.4.1-2 and resistance factors are provided in Table 11.5.7-1 and Section 11.5.8 of *AASHTO LRFD Bridge Design Specifications*.
  - Bearing resistance: Bearing resistance and wall bearing pressures should be appropriately factored. If conditions are such that the bearing resistance of the foundation soils is of concern, the Engineer will evaluate and provide recommendations on ground improvement and/or structural support alternatives, such as:
    - Undercutting and replacement,
    - Aggregate piers (e.g., stone columns, vibro-replacement columns, Geopiers®, etc.),
    - The use of deep foundations for support of the wall footings, or
    - The use of lightweight fill to reduce bearing pressures.

See *Ground Modification Methods Reference Manual, Volumes I and II,* FHWA Publication Nos. FHWA-NHI-16-027 and FHWA-NHI-16-028 for guidance on evaluating ground improvement methods.

- Wall sliding: Calculated along the base of the wall or along a weak layer near the base of the wall. Driving forces should be appropriately selected and factored.
- Wall eccentricity (overturning): Eccentricity, *e*, is the distance between the foundation load resulting from the summation of moments about the toe of the wall and the center of mass of the wall. Eccentricity must be within the middle 2/3 of base width for soil bearing walls and within the middle nine-tenths for rock bearing walls (see Sections 11.6.3.3 and 11.6.5.1 of *AASHTO LRFD Bridge Design Specifications* for additional requirements for seismic loading).

- Settlement: For retaining walls supporting new fill, settlement analyses should be performed, which should include potential total settlement, time-rate of settlement, potential differential settlement along the face of the wall, and potential differential settlement from front-to-back of the wall footing/reinforced zone. If potential settlement is excessive (magnitude, time-rate, and/or differential settlement, as given in the following tables), the Engineer will evaluate and develop recommendations to mitigate the settlement concerns, such as:
    - Ground improvement options as noted above for bearing resistance,
    - Preloading with or without wick drains,
    - Use of lightweight fill, or
    - Use of temporary wall facing for MSE wall until settlement is substantially complete.

Tables 4-5 through 4-7 below present settlement criteria for various wall and facing types.

Table 4-5: Settlement Criteria for Reinforced Concrete Walls, Non-Gravity Cantilever Walls, Anchored/Braced Walls, and MSE Walls with Full Height Precast Concrete Panels

Total Settlement	Differential Settlement over 100 feet	Action
$\Delta H \leq 1$ in	$\Delta H_{100} \leq 0.75$ in	Design and construct
1 in $< \Delta H \leq 2.5$ in	0.75 in $< \Delta H_{100} \leq 2$ in	Ensure structure can tolerate settlement
$\Delta H > 2.5$ in	$\Delta H_{100} > 2$ in	Obtain approval prior to proceeding with design and construction (1)

Notes: (1) Approval of ALDOT State Geotechnical Engineer and ALDOT Bridge Design Engineer required.

Table 4-6: Settlement Criteria for MSE Walls with Modular (Segmental) Block Facings, Prefabricated Modular Walls, and Rock Walls

Total Settlement	Differential Settlement over 100 feet	Action
$\Delta H \leq 2$ in	$\Delta H_{100} \leq 1.5$ in	Design and construct
2 in $< \Delta H \leq 4$ in	1.5 in $< \Delta H_{100} \leq 3$ in	Ensure structure can tolerate settlement
$\Delta H > 4$ in	$\Delta H_{100} > 3$ in	Obtain approval prior to proceeding with design and construction (1)

Notes: (1) Approval of ALDOT State Geotechnical Engineer and ALDOT Bridge Design Engineer required.

Table 4-7: Settlement Criteria for MSE Walls with Flexible Facings and Reinforced Slopes, and Walls in which the Structural Facing is Installed as a Second Construction Stage after the Wall Settlement is Complete

Total Settlement	Differential Settlement over 50 feet	Action
$\Delta H \leq 4$ in	$\Delta H_{50} \leq 3$ in	Design and construct
4 in $< \Delta H \leq 12$ in	3 in $< \Delta H_{50} \leq 9$ in	Ensure structure can tolerate settlement
$\Delta H > 12$ in	$\Delta H_{50} > 9$ in	Obtain approval prior to proceeding with design and construction (1)

Notes: (1) Approval of ALDOT State Geotechnical Engineer and ALDOT Bridge Design Engineer required.

- For anchors (soil nails, tieback, etc.), the Engineer will estimate a preliminary nominal bond zone stress for use in sizing bond zones by the Wall Designer. Appropriate recommendations should be developed for the verification of the bond zone strength during construction, including creep and/or extended creep testing for soils with a Plasticity Index (PI) > 20 (see Section 8.3.4 of *Recommendations for Prestressed Rock and Soil Anchors;* Post-Tensioning Institute, 2014).
- Should site characteristics indicate the use of retaining structures not addressed in the *Standard Specifications for Highway Construction* (such as rock bolts, tiebacks, gabions, shotcrete surfacing, etc.), the Engineer will be required to write and/or review pertinent special provisions. Special attention should be paid to the location of the ROW line as it pertains to rock bolts, soil nails, and tiebacks.
- References for wall analysis procedures/methods:
    - *AASHTO LRFD Bridge Design Specifications* – wall types listed in Section 4.4.3.
    - *GEC 4 – Ground Anchors and Anchored Systems;* FHWA-IF-99-015.
    - *GEC 7 – Soil Nail Walls;* FHWA Publication No. FHWA-NHI-14-007.
    - *GEC 11 – Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Volumes I and II;* FHWA Publication Nos. FHWA-NHI-10-024 and FHWA-NHI-10-025.

4.4.6 *Retaining Wall Foundation Report*

The Engineer will provide the Geotechnical Division with a Retaining Wall Foundation Report as outlined in Section 4.11 and including:

- Recommendations on suitable retaining wall type(s) along with justifications. Specific wall requirements (such as minimum MSE wall reinforced zone backfill material [if deviating from the standard specifications] and width [both overall minimum width and minimum width as a function of wall height], soil nail length, minimum embedment/tip elevation for soldier piles, etc.) should be noted.
- A drawing, suitable for inclusion in the plans, with the “wall envelope” shown. The wall envelope includes:
    - A profile view of the wall with the top of wall, bottom of wall (foundation elevation), final grade in front of wall, bottom of footing, and begin and end stations clearly indicated.
    - Minimum wall/facing embedment.
    - The factored bearing resistance along the length of the wall.

  - The limits (length and depth) of undercut and replacement or other soil remediation that may be necessary beneath the wall.

- Cross section drawing(s) showing the conceptual wall layout and the subsurface stratification along with a table of relevant geotechnical properties for the strata (including unit weight, drained and undrained strength parameters, and consolidation and compressibility parameters).
- Results of global stability analysis. For MSE walls, the report should note that compound stability (failure through retained soils, wall reinforced zone, and foundation soils) should be analyzed by the Wall Designer or the wall design provided to the Engineer to evaluate the compound stability based on the provided material parameters and subsurface stratification.
- Estimated parameters required for calculation of lateral earth pressures (pressure coefficients, unit weights, friction angles, etc.).
- Estimated potential settlements. These should include potential total settlement, potential differential settlement along the face of the wall, and potential differential settlement from the front-to-back of the wall footing or reinforced zone.
- For soil nail walls or tieback walls:
  - The preliminary anchor bond resistance for use by the Wall Designer in preliminary sizing of the anchors.
  - The minimum recommended load testing program (verification and proof testing) requirements to verify the actual bond zone resistance available.
- Recommendations, material requirements, and estimated quantities for the ground improvement alternative(s) recommended, if required.
- Estimated wall quantities (as square footage of the wall face).
- Note special, site-specific drainage details if required.
- Comments on site preparation recommendations such as dewatering, sheeting, shoring, etc., as required.

4.5 Slope Studies

4.5.1 *Overview*

Slope studies will generally be performed for rock cut slopes and for soil cut/fill slopes that are either: (1) proposed as steeper than 3H:1V or (2) have heights greater than 10 feet and slopes at 3H:1V or flatter.

Slope studies will generally proceed with the following steps:

1) The Engineer will review project and site information (see Section 3.1 and Section 4.5.2). As part of the site review, the Engineer should check if the slopes are in known areas/geologies that are prone to landslides/slope failures (see Section 2.2.2 for general information).
2) The Engineer will develop a Geotechnical Exploration Plan that is scoped to provide data to evaluate the proposed slopes (see Section 3.1.2 for general exploration plan requirements and Section 4.5.3 for additional considerations for slope studies).
3) Perform the geotechnical exploration (see Section 4.5.4).
4) The Engineer will perform analyses and develop recommendations for the slopes (see Section 4.5.5).

5) The Engineer will issue a Slope Study Report (see Section 4.11 for general report requirements, with specific requirements for slope studies noted in Section 4.5.6).

4.5.2 *Provided Information*

The Engineer will be provided with the general roadway alignment and plan, profile, and cross section drawings.

If the provided drawings indicate soil cut/fill slopes steeper the 3H:1V, the Engineer should inquire if there are project or site requirements that preclude using 3H:1V or flatter slopes, which are typically preferred.

4.5.3 *Geotechnical Exploration Scope*

The Engineer will review the provided drawings and tabulate information on each cut and fill slope and provide this information as part of the Geotechnical Exploration Plan (see Section 3.1.2). Information tabulated should include station range, offset (left or right of centerline for cut slopes), maximum cut/fill height, and proposed slope.

The Engineer must review/understand the data needs for the project and select appropriate field and laboratory testing methods to develop the required geotechnical data (see Section 3.2). Data collection should be scoped in the Geotechnical Exploration Plan such that the Engineer can analyze slope stability for cut/fill slopes and potential settlement for fill slopes/embankment.

4.5.3.1 Boring Layout and Depth

The following can be considered general guidelines for establishing boring layout and depths; however, the Engineer will need to scope the exploration to meet the project needs:

- At least two borings regardless of slope length.
- At least one boring every 200 feet along centerline.
- For fill slopes/embankments, centerline borings should extend to twice the proposed fill height or to SPT refusal, whichever is shallower, but generally no more than 10 feet into material with SPT N-values of >50 blows per foot. A limited amount of coring should be completed to ensure SPT refusal occurred on continuous bedrock and not a boulder. If boulders are encountered, boring/coring should extend until true competent material is reached.
- Borings should be performed along the left and right ditch lines, extending at least three feet below the proposed ditch line elevation to establish geologic cross sections for analysis.
    - In cut sections, the borings should extend to competent material not normally subject to slope stability issues. Typically, in cohesive soils, the borings should extend into material with a SPT N-value exceeding 16 bpf and in granular soils exceeding 10 bpf.
- For large cut slopes (over 30 feet high), additional borings should be performed at approximately the quarter points of the cut or where considered more effective, extended following the boring depth guidance above in order to establish geologic cross sections for analysis.
- If the site review indicates the presence of significant areas of instability or historic landslides, a more detailed exploratory program may be recommended.

4.5.3.2 Additional Field/Lab Data and Exploration Considerations

In addition to SPT/rock core borings as noted above, the Engineer should scope intact sampling, in-situ testing, and/or laboratory testing to aid in developing relevant geotechnical parameters. These include:

- Field soil classifications should be verified with laboratory classification testing.
- Soil strengths for slope stability analysis.
- Compressibility/consolidation data for soils below proposed fill slopes/embankments. Such data is particularly important for fill slopes near proposed bridge abutments or existing structures.

For rock cut slopes, assessment of rock cores is ideally combined with geologic reconnaissance of existing rock cuts or outcrops. The Engineer should consider the use of laboratory testing of rock core samples to supplement geologic reconnaissance data.

4.5.4 *Perform the Geotechnical Exploration*

Once the Geotechnical Division has approved the Geotechnical Exploration Plan, the Engineer will proceed with the geotechnical exploration.

The Engineer will perform a geologic reconnaissance of existing rock cuts or bedrock outcrops where rock cut slopes are proposed. For roadway widening projects, the Engineer can make direct observations of the rock mass to be excavated into. However, for new alignment projects, the Engineer may have to rely on nearby cuts in the same geologic formation(s) and/or surface outcrops of the formation(s). At a minimum, the geologic reconnaissance should gather observations/data on:

- Rock cut slope, cut height, and orientation;
- Rock type;
- Discontinuity type (bedding plane, joints, fractures), orientation (dip and dip direction or strike and dip), and frequency;
- Weathering condition of the slope (e.g., differential weathering along shale seams);
- Evidence of historical large-scale failures (e.g., large wedges of rock missing from the cut slope); and
- Observations of rockfall debris.

4.5.5 *Interpretation of Data and Development of Recommendations*

Upon completion of the geotechnical field exploration and laboratory testing activities, the Engineer will interpret the data, perform analyses, and develop recommendations.

4.5.5.1 Soil Slopes

The Engineer will:

- Develop subsurface stratigraphy and relevant geotechnical parameters for the subsurface strata. At a minimum, this should include:
    - Moisture content;
    - Unit weight;
    - Soil strengths for long-term, drained conditions and short-term, undrained conditions;

  - Compression/consolidation parameters for fill slopes/embankments; and
  - Water table conditions.

  Guidelines for developing geotechnical parameters can be found in the following:

  - *AASHTO LRFD Bridge Design Specifications* (current edition), or
  - *GEC 5 – Geotechnical Site Characterization* (FHWA Publication No. FHWA-NHI-16-072): https://www.fhwa.dot.gov/engineering/geotech/pubs/nhi16072.pdf.

- Perform slope stability analysis for both short-term (undrained, total stress) and long-term (drained, effective stress) cases. Methods should be in accordance with guidance in *AASHTO LRFD Bridge Design Specifications* with a minimum factor of safety of:
  - FS≥1.2 for temporary construction cut slopes where a slope failure will not impact existing structures or roadways.
  - FS≥1.3 where structures will not be impacted by a global stability failure and soil properties are based on field and laboratory data, or
  - FS≥1.5 where structures or critical utilities may be impacted by a global stability failure or where soil properties are not well defined.
  - FS≥1.0 for the seismic case (pseudo-static analysis with a horizontal acceleration equal to the peak ground acceleration [PGA]).
  - FS≥1.2 for a rapid drawdown case (such as where flood waters may drop rapidly, leaving elevated pore pressures in spill-through abutment slopes).
- Perform potential settlement analysis for fill slopes/embankments. Methods should be in accordance with guidance in *AASHTO LRFD Bridge Design Specifications*.

4.5.5.2 Rock Cut Slopes

Rock slopes should be designed to be as steep as possible while remaining stable with the large-scale rock slope failure modes (wedge, planar, toppling, and circular failures) either not kinematically or kinematically possible with an appropriate factor of safety. Additionally, appropriate rockfall hazard mitigation measures and/or a suitable rockfall catchment area configuration should be recommended.

Rock cut slope stability should be evaluated in accordance with methods presented in:

- *Rock Slopes Reference Manual,* FHWA Publication No. FHWA-HI-99-007 (or other applicable FHWA manuals), or
- Transportation Research Board (TRB) Special Report 247: *Landslides Investigation and Mitigation*.

See Section 4.10.4 for guidance on evaluating rock slopes for rockfall hazard.

4.5.6 *Slope Study Report*

The Engineer will provide the Geotechnical Division with a Slope Study Report as outlined in Section 4.11 and including:

- A description of the geology of the area together with known/reported naturally occurring slope stability issues.
- A discussion of the condition of observed existing natural, cut, or fill slopes in the general vicinity of the proposed project to include their slope and indications of instability, either naturally occurring or induced by grading, drainage, etc.
- Borings will be plotted on cross sections to provide site soil profiles through the cuts and fills.
- Generalized subsurface profile (borings and CPTs), including recommended material parameters.
- A table providing recommendations by station and offset throughout the length of the project with references to the figures associated with each station. Slope recommendations should be for the steepest slope that can be utilized and meet the required slope stability factor of safety and ALDOT guidelines.
- Typical geometry for each slope cross-section configuration, e.g. rock cuts and soil cuts.
- Profiles along each cut section as viewed from the centerline that graphically indicate the following:
  - Approximate top of cut,
  - Ditch line,
  - Zone of weathered rock and/or soil with slope recommendation,
  - Approximate top of rock cut,
  - Zone of rock cut with slope recommendations, and
  - Bench locations.

  These profiles should cover each cut section from station to station and indicate the stations covered and approximate elevations. If cuts are located on both sides of the roadway, then two profiles, one right and one left of centerline, will be required.
- Summary of the results of slope stability analyses, including the software used, graphical outputs for critical cases, and a tabulated summary of cases considered (including resulting factor of safety).
- Recommendations on the slope angle to use in fill areas, potential total settlement as a function of fill height, and time-rate of settlement.
- Recommendations on the need for blasting for rock cuts, if necessary, and an evaluation of the potential effects of blast induced vibrations on adjacent structures.
- Recommendations on the need for rockfall hazard mitigation measures (such as rock fall catch ditch, wire mesh slope protection, shotcrete, rock bolts, etc.), if required.
- Recommendations on the need for protection of erodible beds/seams within a rock slope to prevent differential erosion/weathering.
- If the soil or rock materials encountered in the cuts are not suitable for reuse on the project, a plan note should be included detailing the station-to-station location and elevations of the unsuitable material.

- If slides, slumps, or faults are noted during the exploration, these should be discussed along with measures to address problems in the future.
- Notes of evidence of springs and excessively wet areas, along with a general note regarding the surface and subsurface drainage observed.
- Maps indicating the location of caves/sinkholes/karst features encountered on the project and recommendations to reduce impacts from said features.

4.6 Landslide Studies

4.6.1 *Overview*

Landslide studies are performed for both naturally-occurring landslides and slope failures through cut or fill slopes, and the term "landslide" as used herein refers to either type of slope movement. Landslide studies will be performed when such slope movements impact or have the potential to impact the roadway or associated structures. Landslide studies will generally proceed with the following steps:

1) The Engineer will review project and site information (see Section 3.1 and Section 4.6.2).
2) The Engineer will develop a Geotechnical Exploration Plan that is scoped to provide data to evaluate the landslide (see Section 3.1.2 for general requirements and Section 4.6.3 for additional considerations for landslide studies).
3) Perform the geotechnical exploration upon approval of the Geotechnical Exploration Plan by the Geotechnical Division.
4) The Engineer will perform analyses and develop remediation recommendations (Section 4.6.4).
5) The Engineer will issue a Landslide Study Report (see Section 4.11 for general report requirements, with specific requirements in Section 4.6.5).

TRB Special Report 247: *Landslides Investigation and Mitigation* or other suitable references should be reviewed for Landslide Studies.

4.6.2 *Provided Information*

For a landslide remediation request, the Engineer will receive a location and vicinity map along with cross section of the landslide, if available at the time of the request.

The Engineer will contact the ALDOT District Engineer and/or County Engineer to ascertain the maintenance history of the slide area to include previous corrective measures taken. The Engineer should also review information in *Evaluation of Landslides Along Alabama Highways* (Auburn University, 2019) and other information provided by the Geotechnical Division from the ALDOT Landslide Database.

4.6.3 *Geotechnical Exploration Scope*

Data collection should be scoped in the Geotechnical Exploration Plan (see Section 3.1.2) such that the Engineer can evaluate and recommend a remediation/stabilization approach for the landslide. Landslides can vary widely in size and complexity; therefore, the scope of geotechnical explorations can vary widely. Unlike other geotechnical projects, no specific guidelines on the scope of the geotechnical exploration are provided in this manual.

The Engineer must review/understand the data needs for the particular site and select appropriate field instrumentation and field/laboratory testing methods to develop the required geotechnical data (see Section 3.2). The Engineer should consider the following when scoping a landslide study:

- Borings should be located to develop geologic cross sections in the vicinity of the landslide.
- If accessible and safe, SPT borings should also be advanced through the failure plane, with inclinometer(s) installed in the boreholes. If slope movement is ongoing, inclinometers will indicate the depth of the failure plane(s) at the inclinometer location.
- Strength of soils along the failure plane should be evaluated. For laboratory strength testing, the Engineer should either scope intact sampling of potential failure plane materials or should plan for follow-up sampling after the failure plane depth is identified from inclinometer data.
- Water table conditions are a frequent contributor to landslides. If thin, continuous layers of either water bearing or water restrictive soils (such as encountered in Coastal Plain formations in Alabama) are anticipated or encountered, the Engineer should consider CPT to detect such layers, in conjunction with nested piezometers to aid in detecting perched water tables.

4.6.4 *Interpretation of Data and Development of Recommendations*

Upon completion of the geotechnical field exploration and laboratory testing activities, the Engineer will interpret the data, perform slope stability analyses, and develop remedial/stabilization recommendations. The Engineer will:

- Develop subsurface stratigraphy and relevant geotechnical parameters for the subsurface strata. At a minimum, this should include unit weight, strength (undrained shear strength for cohesive soils and effective stress friction angle for both cohesive and cohesionless soils), and water table conditions (groundwater table, perched water table, water bearing seams, etc.).
- Analyze conditions that may have led to the landslide. The Engineer will develop a slope stability model using the ground surface and water table conditions that may have immediately preceded the landslide (i.e., by calibrating the slope stability model to a factor of safety of approximately 0.95-1.0). Knowing the depth of the failure plane (by inclinometer monitoring), understanding the likely strength of material along the failure plane (either by laboratory testing of intact samples and/or in-situ testing), and understanding the potential perched water table conditions, the Engineer can make a reasonable model of conditions that led to the landslide.
- Using the "calibrated" slope stability model noted above, the Engineer will evaluate potential remediation/stabilization approaches that will suitably stabilize the slope (global stability at least 1.3 or at least 1.5 if a structure or critical utility is involved), such as:
    - Rock buttress,
    - Regrade to a suitably stable slope,
    - Surface drainage,
    - Subsurface drainage-interceptor,
    - Drain trenches,
    - Horizontal drains, or
    - Mechanically stabilized retaining structures, which are normally constructed using top-down construction, such as tied back walls or soil nail reinforced slopes.

Additionally, construction sequence and slope stability during construction should be addressed.

4.6.5 *Landslide Study Report*

The Engineer will provide the Geotechnical Division with a Landslide Study Report as outlined in Section 4.11 and including:

- Site plan indicating ground surface cracking, head scarp location, toe bulge area, and general location of the landslide, along with springs noted along the face of the landslide.
- Detailed narrative history of the slide area, including movement history, maintenance work, and previous corrective measures.
- Cross section(s) used for analysis showing the subsurface stratigraphy, estimated failure plane, and assumed, measured, or back-calculated soil strength parameters.
- Proposed remediation/stabilization approach with a typical cross section showing dimensions and calculated slope stability factor of safety.
- Recommended construction sequence and associated construction slope stability factor of safety.

If special construction techniques or material are recommended, the Engineer will write and/or review special provisions related to the special construction techniques or materials. The Engineer will also indicate what types of materials to use for regular construction techniques and gradation and/or compaction requirements, per the *Standard Specifications for Highway Construction*.

The Engineer will separately provide the Geotechnical Division with information required for input into the Landslide Database, see attached Form A-6.

4.7 Soil Surveys

4.7.1 *Overview*

Soil Surveys are performed to determine the type of materials that will be encountered along a project alignment such that prospective bidders on the project will understand what is involved as it pertains to earthwork on the project.

Soil Surveys will generally proceed with the following steps:

1) The Engineer will review project and site information (see Section 3.1 and as noted in the following subsections). The Engineer will coordinate the field reconnaissance portion of the site review with the Area Materials Engineer (AME). If possible, the AME will participate in the field reconnaissance.
2) The Engineer will develop a Geotechnical Exploration Plan that is scoped to provide the required information based on the project type (see the following sections for guidance for specific project types and see Section 3.1.2 for general Geotechnical Exploration Plan requirements).
3) Perform the geotechnical exploration upon approval of the Geotechnical Exploration Plan by the Geotechnical Division and the AME. Changes to the exploration scope based on conditions encountered must be done with coordination of the AME and others as noted in the following sections.
4) The Engineer will develop recommendations and issue a Soil Survey Report (see Section 4.11 for general Geotechnical Report requirements, with specific requirements provided in the following sections for specific Soil Survey project types).

4.7.2 *Contractual Comments*

ALDOT will provide the Consultant with in-house amendments and policies concerning the application of the current AASHTO Guide for Design of Pavement Structures.

The Engineer will provide the State Materials and Tests Engineer with a boring layout on plan/profile sheets. The boring layout will include the proposed boring locations and depths and be submitted as part of the proposal/Geotechnical Exploration Plan.

The AME will have final approval of boring locations. Boring locations that are deleted during field operations will have a written explanation of rationale behind the deletion and will be presented in a table in the final report. Before additional borings are performed, approval for the additional borings will be obtained in writing from the AME.

The boring logs will be submitted to the State Materials and Tests Engineer for approval of completeness before being submitted to the AME.

The submission of boring logs for approval will be on an 11x17 sheet of regular paper. Boring log sheets will contain a space for the AME signature.

The Engineer will provide the AME and the State Materials and Tests Engineer with a copy of Resilient Modulus (M_R) laboratory tests results and will discuss with the State Materials and Tests Engineer the selection of the design M_R value.

When requested by ALDOT, the Engineer will deliver the soil samples specified by the AME to the Bureau of Materials and Tests or to the Area Materials Laboratory for testing by an ALDOT Laboratory.

4.7.3 *General Information*

Prior to beginning work on the project, the Engineer will review ALDOT procedures and inspect the project site. If possible, the project site inspection team will include the AME.

The Engineer will request and obtain a copy of the Findings of No Significant Impact (FONSI) and/or Environmental Assessment (EA) or reevaluation documents for the project, either from the Bureau of Materials & Tests, Environmental Services Division or from the Design Bureau, Environmental Technical Section. The Engineer will review these documents to determine if wetlands have been delineated. If so, the Geotechnical Exploration Plan should include probing and/or boring in these areas to determine depth of soft soils/unsuitable materials or muck. This work needs to be done as a part of the project field work.

The Engineer will include, with the Soil Survey, a written description of the soft soil areas noted. Soft soils are cohesive soils (i.e., silts and clays) that have low SPT N-values ($N_{60} \leq 6$). Soft soils can cause long-term settlement under load and post-construction instability of structures. The Engineer will delineate the extent of soft soils areas based on site observations and probing/drilling. Rainfall conditions (drought, wet, etc.) occurring at the time of the survey will be noted. A table listing soft soil areas will be included in the Soil Survey Report. The soft soils table will include, but will not be limited to, station numbers, anticipated depth of excavation or other method of remediation, and, if removal is recommended, the estimated quantity in cubic yards to be removed. In the event no soft soils are located within the project limits, the Engineer will submit a written statement outlining the level of effort utilized in attempting to locate soft soil areas.

The Engineer will assess the need for nuclear testing devices, gyratory compactors, inertial profilers, materials remixing devices, and soils/structures laboratories.

4.7.4 *New Location (Grade and Drain Projects)*

The Engineer will provide the geo-hydraulic settings for the project, which will include, at a minimum:

- A description of the geology (with the geological maps included) of the project vicinity,
- A description of the general topography of the project and identify such pertinent features as wet areas, rock outcrops, potential slide areas, and underwater embankment requirements,
- An investigation of the possibility of sinkholes in areas that are prone to sinkholes,
- An investigation of the drainage requirements, and
- Recommendations for the use of culverts, underdrain pipe, cross-drain pipe and pavement edge drains provided.

If recommendations are made for the use of pavement edge drains, a Permeable Asphalt Treated Base (PATB) layer, normally 4 inches (100 mm) thick, will be incorporated in the pavement structural design. There must be a minimum of 6 inches (150 mm) of asphalt cover over the PATB, which renders its applicability to mostly high traffic volume designs. Concrete roadway pipe will be used on roads that comprise the State Highway System. Soil and water testing will not be required for concrete pipe. If underdrain pipe is recommended, a sketch showing the elevation where the underdrain will be placed in the field and a typical section will be included in the report. The Designer of Record will size the pipe and develop the plan sheets to be included in the final plan assembly.

A Soil Survey will be conducted along the centerline of the project. The intent of the borings is to determine the type of materials that will be encountered along the project, such that prospective bidders on the project will understand what is involved as pertains to earthwork on the project. A site inspection to include walking the alignment from one end to the other and from right-of-way to right-of-way to look for areas that may be wet and/or may contain soft soils will be performed as part of the Soil Survey. The AME will accompany the Engineer during the site inspection to ensure that potential areas of concern are explored. The Bureau of Materials and Tests Chief Geologist will be available, upon written request by the AME, to participate in the site inspection.

Areas of wet and/or soft soils identified during the site walk through will require probing during the Soil Survey to determine the depth of soft soils. If the Soil Survey is completed during a drought, look for areas with vegetation that would indicate the area would be wet or soft during wetter periods. These areas will be delineated, and a note added to the plans stating that removal of unsuitable materials may be required in these areas. Additionally, if the survey is done during a drought, the report should clearly state this condition and should also state that conditions may change significantly with the resumption of normal weather functions. If water bodies (ponds, streams, etc.) crossing the alignment are located on the project, provide recommendations for drainage from the roadway alignment/embankment (drain, backfill, etc.) and details where the water is to be drained. A typical cross-section along the centerline of the project should be included, along with Pay Items.

Borings will be performed in fill areas every 300 feet (90 meters [m]) along centerline. The boring will extend to 1.5 times the proposed fill height or to SPT refusal, whichever is shallower, but no more than 10 feet (3 m) into competent material (with competent material defined as having SPT N-values of $N \geq 15$ for purposes of a Soil Survey). If uniform conditions are encountered while drilling every 300 feet (90 m), then the boring interval may be extended to 500 feet (150 m). Additional borings will be performed if there is a noticeable change in the soil between borings.

Borings will be performed in cut areas every 200 feet (60 m) along and on the centerline and extend approximately 3 feet (1 m) below the ditch line. For every third boring along the centerline, advance a boring in the left and right ditch lines, extending these borings approximately 3 feet (1 m) below the ditch line. Additional borings will be performed if there is a noticeable change in the soil between borings.

SPT borings will be performed in overburden soils at each boring location in both cut and fill areas for every 5 feet (1.5 m) of boring depth. SPT borings will be performed in accordance with AASHTO T 206.

The Engineer will determine shrinkage and/or swell values (i.e., between excavated volume and placed/compacted volume) for the project in general accordance with ALDOT Guidelines for Operation 3-11.

If water is encountered during the boring operations, determine and report the elevation of the water table. Standpipe piezometers will be installed at regular intervals along the alignment so that the contractor can determine the depth to ground water, particularly in areas where soft soils may be present.

If conditions are encountered during the Soil Survey explorations that indicate a slope stability problem could occur in either the back slopes or the front slopes (especially soft soils in the front slope foundation soils), notify the AME and the State Materials and Tests Engineer in writing so that consideration may be given to performing a slope study. Generally, if slopes are steeper than 3H:1V or if slopes have to be engineered for stability or drainage, a slope study will be prepared in accordance with Section 4.5.

If conditions exist on the project that indicate that it would not be appropriate to adhere to the earth slope criteria as outlined in GN-2, Notes 106, 107, and 108 of the ALDOT Standard Drawings, notify the AME and the State Materials and Tests Engineer in writing.

The Engineer will conduct a topsoil survey and provide topsoil depths, location, estimated quantities, and test results. The Engineer will also quantify the area within the construction limits of the project, measure topsoil depths utilizing methods directed by the AME, and estimate the average topsoil depth within construction limits. If topsoil depths vary considerably across the project, the area within the construction limits may need to be broken into sections. The Engineer will then calculate the estimated available topsoil and determine the number of samples required (one sample will be required for every 5,000 cubic yards of available topsoil), obtain samples from representative locations across the site (minimum of 25 pounds typically needed for analysis), and test samples in accordance with ASTM 5268 and ASTM D 2974 (Method C). Topsoil must meet the requirements of Section 650 of the Standard Specifications for Highway Construction.

Where unsuitable material, soft soils, and/or muck are encountered, notify the AME in writing. Upon approval from the AME, take soundings (or boring and samplings as directed in writing by the AME) along the centerline and right and left of the centerline out to the limits of construction to determine the depth and extent of the unsuitable material, soft soils, and/or muck. Undisturbed samples for consolidation testing may be required to perform settlement calculations. The Engineer will make recommendations, including estimated quantities and Pay Items, for removal and/or treatment of unsuitable material, soft soils, and/or muck encountered. If unusual circumstances are encountered, such as high groundwater table or the potential for differential settlement beneath embankments or structures, then further explorations may be needed. The Engineer will immediately notify the AME and State Materials and Tests Engineer in writing for further guidance on the need for additional explorations. The AME will notify the Engineer and the Assistant Geotechnical Engineer (AGE) - Consultant Management in writing on the need for more work.

The Engineer will identify areas where the subgrade may require stabilization or removal and replacement and recommend the remediation. If lime stabilization is recommended, the Engineer will recommend the percentage of lime based on laboratory soil testing results, and the results will be included in the Soil Survey Report. The final percent of lime before construction will be determined and reported in accordance with ALDOT-292 by ALDOT laboratory. If ground improvement methods or geosynthetic stabilization is recommended, a sketch of the locations, including an elevation view, plan view, typical sections, Pay Items, and estimated quantities, will be provided.

In areas where remediation is required, the recommendations will also include a long-term settlement analysis for cost comparison between removal and/or treatment during construction and pavement leveling during future maintenance activities.

The Engineer will delineate areas of high groundwater. If groundwater should be drained to stabilize the roadbed, a sketch of the underdrain will be included with the elevation to set the drains included in the sketch. If more than one area exists, include a table of areas showing high groundwater where underdrain will be installed; identify, by using boring logs, the locations where underdrains will be needed or required to remove groundwater from subgrade; and detail the elevation to set the drains (also to be included in the sketch). A drawing of the required underdrain system will be provided by the AME or the Engineer. This drawing should include the recommended locations of the underdrain systems, pipe sizes for main lines and lateral lines, and other needed information pertaining to the placement of the underdrain system.

If a stream crossing is encountered that will possibly require construction of a culvert during the course of obtaining the soil borings necessary to complete a Soil Survey, borings appropriate for the subsequent design of the culvert foundation will be obtained as outlined below. Use the tables “Earth Slopes Horizontal to Vertical for Types of Terrain”, found in Notes 106, 107 and/or 108 of GN-2 in conjunction with the applicable “typical roadway section” and planned roadway profile to establish the probable inlet and outlet locations of the culvert.

Note: For large stream crossings that are longer than 20 feet (6 m) along the centerline, where the use of a bridge culvert will be likely, refer to Section 4.3 for exploration, engineering analysis, and report development.

For non-bridge culverts, regardless of fill height, where soft soils can reasonably be expected, the following drill patterns will be used:

- Culverts having a length of 300 feet (90 m) or less will be drilled at mid-length and at each end.
- Culverts exceeding 300 feet (90 m) in length will be drilled so that the distance between inlet and outlet borings is equally divided into approximately 150 linear foot (45 m) increments.

Borings will extend to a depth equal to 1.5 times the expected fill height or 5 vertical feet (1.5 m) into competent rock, whichever occurs first. SPT borings will be conducted in accordance with AASHTO T 206 every 5 feet (1.5 m) for the full depth of the boring at each boring location. Undisturbed samples will be obtained in accordance with AASHTO T 207, which should be recovered and tested to permit calculation of the soil bearing resistance and potential settlement of the assumed culvert(s) as outlined in *AASHTO LRFD Bridge Design Specification* or *Soils and Foundations* (FHWA Publication Nos. FHWA-NHI-06-088 and FHWA-NHI-06-089). The above process will also be used for performing foundation explorations for large diameter pipes normally greater than 48 inches (1.2 m) that will have fills in excess of 60 feet (18 m) placed above them. Consideration should be given to investigating cross drains in areas of concern as well.

A brief foundation report will be prepared for each investigated culvert and/or pipe location. The report will include the expected site soil profile(s), the results of soil bearing resistance calculations, and the potential settlement. The report should also include recommendations for subgrade improvement(s) should the available bearing resistance be inadequate and/or the potential differential settlement exceeds 2 inches over 100 feet. The foundation recommendations for a non-bridge culvert will be developed as a standalone document that can also be included in the materials report.

Rock coring will be performed in accordance with AASHTO T 225. Rock cores will be stored at the Materials & Tests Bureau Drill Crew Warehouse.

AASHTO soil classifications will be shown on boring logs and boring fence diagrams placed on profiles and cross sections.

The Engineer will sample each soil strata encountered and furnish a soil analysis and AASHTO soil classification. AASHTO soil classifications, PIs, and moisture content results will be placed on the boring logs and in the soil classification summary sheet as shown in Appendix A (Table A-9). Laboratory M_R samples will be collected from each non-structure boring in the cut areas along centerline and designated borrow sources. Along the centerline, a laboratory M_R test will be conducted on a composite sample of soil collected from the boring above the proposed finished subgrade elevation and on a sample of soil collected from within the top 1 foot (0.30 m) below the proposed finished subgrade elevation. In large cuts where the borings indicate that uniform conditions exist in the cut, collect M_R samples at approximate one-quarter points along the centerline of the cut. M_R sampling and testing will be conducted according to AASHTO T 307.

The Engineer will supply the AME and the State Materials and Tests Engineer with a copy of M_R tests results and select the design M_R value as per Table A-9 in Appendix A and the following discussion.

- The design M_R for soils classified as A-1, A-3, A-2-4, and A-2-5 will be the average of the M_R values generated at a confining pressure of 4 pounds per square inches (psi) (0.03 MPa).
- For other AASHTO soil classes, the design M_R will be the average of the M_R values generated at a confining pressure of 2 psi (0.015 MPa).
    - Samples of soils that fall within the A-6 and A-7 groups will be remolded to a moisture content on the wet side of optimum. Moisture content on the wet side of optimum will be determined at 96 percent of the proctor density.
    - Other soils will be remolded at optimum moisture content.
- The M_R values generated at both confining pressures will be averaged to determine the design M_R value.

Upon computing the average M_R value, if there are values in the M_R data set that exceed the mean value by ±2 standard deviations, these values will be discarded and the remaining values will be re-averaged to determine the final average M_R value to be used in the pavement structural design analysis. The resulting design M_R value will be in units of psi (MPa) and it will be rounded to the nearest 100 psi (0.1 MPa). See attached "Design Resilient Modulus by AASHTO Soil Classification" (Table A-9 in Appendix A) for guidance. The subgrade M_R value is the required input in the DARWin™ AASHTOWare and will be determined in accordance with AASHTO T 307 using the testing criteria as noted above. Conversion to an M_R value from other types of subgrade strength tests will not be allowed. The pavement design will be completed in accordance with the *AASHTO Guide for Design of Pavement Structures* (1993).

The Engineer will review the soil profile data in the cut areas to determine if the material is suitable for use as improved roadbed. Cut areas where there is suitable material for improved roadbed will be identified in a table in the materials report. The table will include the station where the material is located and the approximate amount of material available for the Improved roadbed layers. The report should also state that this material is to be stockpiled for later use in the improved roadbed layers. If there is not sufficient material to build the improved roadbed layers, then a quantity of borrow will be specified to complete the improved roadbed layers. If there is not suitable material on the project for improved roadbed, then borrow will be specified for the entire amount of improved roadbed material required. A recommendation should be made as to the availability of suitable material for borrow before final borrow is set up. If there is not locally available material, then specify roadbed processing of the subgrade, which would be modified roadbed (no structural number [SN] value), lime stabilization, or stabilization with aggregate.

The Engineer will determine if a borrow pit location is to be included in the plan assembly for the project.

4.7.5 *Base and Pave Projects*

The Engineer will collect samples at the finished subgrade for laboratory M_R testing. The sampling frequency will be one sample per one-half mile (0.8 kilometer) per roadway or at each soil change per roadway. M_R sampling and testing will be performed in accordance with AASHTO T 307.

Items listed in Section 4.7.4 will be completed prior to base and pave type projects.

The Engineer will determine the availability of suitable local materials for use in subbase and base construction on the project and make relative cost comparisons to optimize the pavement structural design.

The Engineer will make recommendations for the pavement structural design for detours, cross-overs, and tie-ins to adjoining roads along the project.

4.7.6 *Bridge Replacement and Short Widening Projects*

The existing pavement structure should be cored to determine layer type, thickness, and condition.

If the pavement is performing satisfactorily, the Engineer will make recommendations for the pavement structural design based on the ALDOT "equivalent build-up" method. The structural number for in-place pavement is determined by the following equation:

$$SN_{in} = d_1 a_1 m_1 + d_2 a_2 m_2 + d_3 a_3 m_3 + \ldots + d_i a_i m_i$$

where: SN_{in} = structural number (SN) of in-place pavement
a = layer structural coefficient
d = layer thickness in inches
m = layer drainage coefficient

If the pavement is not performing satisfactorily, the pavement structural design will be determined from soil strength and applicable traffic data according to the procedures outlined in the current AASHTO Guide for Design of Pavement Structures as amended by ALDOT and using the DARWin™ AASHTOWare.

For bridge foundation design requirements, see Section 4.2 for drilling, sampling, and reporting requirements pursuant to completion of a bridge foundation exploration.

4.7.7 *Long Widening Projects*

In general, long widening projects will be treated as new location projects and will be investigated as required by Section 4.7.4.

Boring spacing may be adjusted according to historical soils data for the existing alignment. The AME will be contacted at the beginning of long widening projects to determine appropriate boring spacing.

4.7.8 *Grade, Drain, Base, and Pave Projects*

All items listed in the Section 4.7.4 and Section 4.7.5 will be required for a grade, drain, base, and pave projects.

Note 104 of GN-2 will be included on the typical section sheets.

4.7.9 *Soil Survey Report*

In addition to items noted above for specific project types and the general guidance provided in Section 4.11, Soil Survey Reports should include, but not be limited to, the following:

- A description of the project;
- Project location map;
- Geologic, topographic, and hydraulic maps of the vicinity of the project with begin/end project station numbers indicated;
- Description of the geology, topography, and water resources in the vicinity of the project;
- Boring location maps;
- Site soil profiles;
- Cross sections with borings;
- Soils classification summary;
- M_R calculations summary;
- Roadway pipe recommendations;
- Shrink and/or swell values;
- Topsoil testing table with depths and location;
- Location of unsuitable materials, soft soils, and/or muck and recommendations for treatment;
- Soft soil delineation tables;
- Table indicating high ground water level by station number;
- Location of rock; and
- Locations of areas that may require subgrade stabilization and recommended method of stabilization.

After the sampling, testing, and engineering analysis has been completed, the report will be submitted for review in accordance with the review process shown in Figure 4-1.

The AGE will approve the Soil Survey Report and distribute copies as follows:

- Construction: 1 copy.
- Materials & Testing (M&T) Project File: 1 copy.
- State Materials and Tests Engineer: electronic copy.

Note: On Interstate Maintenance (IM) Projects, an electronic copy will be furnished to the Maintenance Bureau.

Note: Soil Survey and Partial Materials Reports will be locally approved by the AME under a separate cover letter. Locally approved Soil Survey and Partial Materials Reports will be retained until remaining engineering data has been prepared by the AME and submitted as a complete Materials Report. Only complete Materials Reports will be approved and distributed by the SME.

Note: Soil Survey Reports prepared in-house, by ALDOT personnel, will have two hard copies of the report and an electronic copy submitted for final approval and distribution.

Note: Formatting issues or changes requested in the comments log by the AME or the State Materials and Tests Engineer that were not addressed in the final report by the Consultant will be changed at the Consultant's expense.

Note: In conjunction with the first draft submittal, two copies of a Soil Test Data Report will be furnished for review. The Soil Test Data Report will include applicable soil test data (e.g., M_R data, sieve and Atterberg data, topsoil data, etc.), as well as the Soil Classification Summary, the M_R Calculations Summary, and other applicable tables used in the Soil Survey.

Note: For projects to be let to contract utilizing English units of measurement, the Soil Survey Report and the supporting documentations and test results will be in English units of measurement. For projects to be let to contract utilizing SI units of measurement, the Soil Survey Report and supporting documentations and test results will be in SI units of measurement. Dual units of measurement are not allowed.

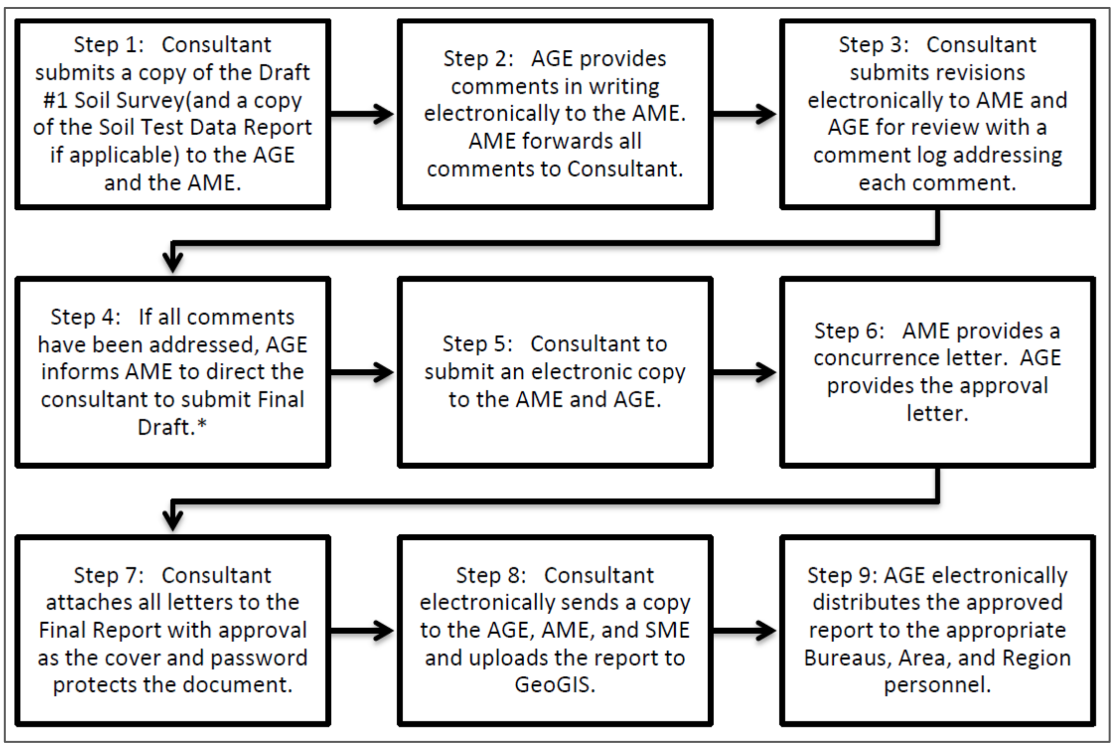


Figure 4-1: Soil Survey Approval Process – Consultant written, hired through Geotechnical Division

4.8 Sign, Lighting, and Signal Pole Foundations

The following guidelines will be adhered to during the design of traffic control devices, highway lighting, and overhead sign structures. Typically, the request for foundation investigations for traffic control devices, highway lighting, and overhead sign structures are made at the conclusion of the PS&E plan review by the lead designer. If a request for drilling assistance is sent to the Geotechnical Division, the request will include three copies of the plan sheets, including the title sheet, traffic signal, lighting or sign layout sheets, and the sheets that link structure numbers to the corresponding stations and offset locations. A cover letter will accompany the plan set that indicates the project number, charge number, letting date for the project, and instructions on whether assistance is needed with drilling and/or with preparation of the foundation report. A copy of this request letter will be forwarded to the Traffic Design Section of the Design Bureau for tracking purposes.

Drilling will be performed with the intent of providing boring logs and LPILE parameters to the contractors and designers to aid in the selection of the appropriate foundation type and size. The number of borings to be performed and location will be determined in accordance with the following:

- Generally, SPT borings should be performed as follows:
  - Metal ITS poles, high mast light structure, traffic signals, and lighting poles 70 feet or greater in height: One boring per structure.
  - Overhead signs and cantilever signs: One boring per foundation.
  - Lighting poles less than 70 feet in height: Number of borings demined by the State Geotechnical Engineer.
  - Borings should be drilled as close as possible to the structure locations.
  - In cases where the geology across the site is uniform, the number of borings drilled as given above may be reduced at the discretion of the State Geotechnical Engineer.
- Boring depths:
  - For lighting poles less than 70 feet in height, metals poles for ITS devices, and traffic signal strain poles, borings will be advanced a minimum of 20 feet deep.
  - For other structures, borings will be advanced a minimum of 30 feet deep.
  - Depths provided above are minimum depths. Generally, borings should not be terminated in loose, soft, or otherwise unsuitable soils. When auger refusal is encountered prior to reaching the recommended minimum depths, a minimum of 10 feet of competent rock (i.e. recovery greater than 75%) will be cored.

In cases where structures are to be placed in fill sections greater than 3 feet deep, a plan note will be developed indicating that the required soils data will be gathered after the fill placement is complete.

In addition to the boring logs, the Engineer will provide an LPILE parameters table for each boring, which will be placed on a full-size (22″x34″) sheet for inclusion in the final plan assembly; see Section 4.2.5.1 for guidance on LPILE parameters.

Recommendations provided by the Engineer will include subsurface stratigraphy along with relevant properties (water table conditions and material type, unit weight, strength, and recommended LPILE parameters). Where multiple foundations have similar subsurface conditions, the foundations can be grouped into a Geotechnical Site with a generalized subsurface profile presented for foundations within that Geotechnical Site.

4.9 Sound Barrier Walls

(pending)

4.10 Other Geotechnical Considerations

4.10.1 *Soft Soils*

If during the course of an exploration the Engineer encounters soft soils or unsuitable materials, such as muck, organic soils, peat, humus, stumps, roots, buried logs or other landfill material, old piling/buried structures, old paving, drains, and other objectional matter that may impact the project, further exploration and analysis of the area should be performed.

Where such soft soils or unsuitable materials are encountered, soundings or borings will be taken along the centerline and right and left of centerline out to the limits of construction to determine the depth and extent of the soft soils. Appropriate in-situ and/or laboratory testing should be performed to characterize the material properties of the soft soil, including moisture content, unit weight, shear strength, consolidation parameters, and organic content.

Recommendations will be made for removal and/or improvement of unsuitable material and/or muck encountered. See *Ground Modification Methods Reference Manual, Volumes I and II,* FHWA Publication Nos. FHWA-NHI-16-027 and FHWA-NHI-16-028 for guidance on evaluating ground improvement methods.

- For soft soils below shallow foundations (such as for retaining walls or bridge culverts), the Engineer should consider alternatives such as:
  - Undercutting and replacement;
  - Aggregate piers (e.g., stone columns, vibro-replacement columns, Geopiers®, etc.);
  - Preloading with or without wick drains;
  - The use of lightweight fill to reduce bearing pressures and potential settlement; and
  - The use of deep foundations to support footings if removal or improvement are not practical.
- For soft soils of significant depth and/or lateral extent beneath proposed embankment, the stability and potential settlement of the soft soils should be carefully evaluated if removal or improvement using methods such as those noted above are not practical. The factor of safety for stability of the embankment must be equal to or greater than 1.3 or greater than 1.5 if a structure is involved, as noted in Section 4.5. The Engineer may also consider:
  - Change of alignment,
  - Change of grade,
  - Geosynthetic reinforcement of the embankment fill, or
  - Staged/sequenced construction.

The Engineer will also provide cost comparisons and advantages and disadvantages for the various alternatives given and recommend a preferred alternative for the site, considering anticipated cost, timeliness, and safety.

The Engineer should provide the following in the associated Geotechnical Report:

- An estimate of the delay time for the contractor due to settlement.
- If instrumentation will be required to monitor the fill stability and/or settlement, the specific location of the device(s) should be indicated in the report.
- If excavation and replacement is the recommended option for remediating the soft soils, the Engineer will provide full-size (22”x34”) PDF drawings suitable for inclusion in the project plans indicating the vertical and lateral limits of the excavation.
- If a surcharge program is recommended, the Engineer will provide full-size (22”x34”) PDF plans and cross section drawings suitable for inclusion in the project plans showing the treatment plan along with a recommendation of where to dispose of the surcharge material once settlement is complete.

The Engineer will also be responsible for developing and/or reviewing special provisions which are required for the instrumentation.

4.10.2 *Ground Improvement*

Ground improvement (GI) should be considered: (1) when potential settlements are excessive for footings, walls, embankment, etc., or (2) where the in-place soils do not provide adequate bearing resistance for shallow foundations (bridge, walls, culverts, etc.). GI may be used to strengthen and reduce compressibility of subsurface soils and is often more economical than considering deep foundation to support footings. Many forms of GI are available and may be considered, and most are designed and installed by specialty contractors. The following section presents more commonly used GI techniques to increase bearing resistance and decrease potential settlement.

The Engineer should refer to *GEC 13 – Ground Modification Methods Reference Manual, Volumes I and II* (FHWA Publication Nos. FHWA-NHI-16-027 and FHWA-NHI-16-028), unless otherwise noted, for guidance on evaluating GI methods:

- Undercutting and replacement;
- Aggregate piers (e.g., stone columns, vibro-replacement columns, Geopiers®, etc.);
- Rigid inclusions;
- Preloading with/without wick drains;
- Impact densification; or
- Soil mixing (wet or dry).

4.10.2.1 Undercutting and Replacement

Undercutting/over-excavation is used for removing unsuitable materials (i.e., peat, muck, organic clays, etc.) that exist to limited depths below footing bearing elevations. Undercutting excavations are backfilled with compacted structural fill material. If undercutting and replacement is not feasible for environmental, technical, or economic reasons, other forms of GI techniques should be considered. Stability of undercutting depths greater than 5 feet should be verified by in accordance with Section 4.5. The bottom of undercutting excavations should be checked to ensure that unsuitable materials have been removed and that firm soils are present.

4.10.2.2 Aggregate Piers

Aggregate piers are typically installed within foundation bearing areas and into underlying competent bearing soil. Aggregate columns are typically 2 to 4 feet in diameter and consist of compacted crushed stone. The aggregate piers serve as "stiffening elements" and help transfer shallow foundation/slab loads to underlying bearing soils, both improving bearing resistance and decreasing potential settlement. Refer to *Design and Construction of Stone Columns* (FHWA Publication No. FHWA-RD-83-027) for additional guidance.

4.10.2.3 Rigid Inclusions

Rigid inclusions (RIs) are soil stiffening elements used to reduce shallow foundation settlements in compressible soils. RIs are typically installed by specialty contractors. RIs differ from foundation piles in three major ways:

1. RIs are not connected to a pile cap; instead, a load transfer platform (LTP) consisting of several feet of compacted sand or gravel is used to transfer shallow foundation loads to the RIs.
2. The grout or concrete used in RI construction is usually of relatively low strength (1,500 to 2,000 psi) compared to that used in foundation piles.
3. The RI elements are not normally reinforced.

There are two basic types of RIs: (1) those that resembles a drilled displacement pile and (2) those that involve the vibratory installation of a closed-end steel pipe or mandrel into the ground, followed by filling of the pipe with grout as it is extracted to the design cut-off elevation.

- The drilled displacement pile type of RI is called Controlled Modulus Column® (CMC). The columns are typically 12 to 18 inches in diameter and would typically be installed into firm to dense sands. Due to the special tooling used, lateral soil displacement occurs during installation, and very little spoils are created. The CMCs share load support with the existing soil. Following CMC installation, a LTP would be constructed to transfer the foundation loads to the RIs and existing soils. This LTP is typically 3 feet thick and consists of dense aggregate or recycled concrete aggregate.
- The vibratory mandrel method of RI installation typically consists of vibrating a 16-inch diameter, closed-end steel pipe or mandrel into the ground using a vibratory hammer. The pipe acts as a displacement pile during its installation. Following vibratory installation, 4- to 6-inch slump grout is pumped into the pipe as the pipe is extracted, and a grout column is created. The LTP is then constructed over the top of the RIs. The LTP typically consists of several feet of either compacted sand or gravel, depending on the application.

4.10.2.4 Preloading (with or without Wick Drains)

An earthen preload can be used prior to construction to facilitate consolidation by accelerating drainage and dissipation of excess pore water pressures in soft soils, sludge, mine tailings, dredge fill, etc., and to minimize post construction settlement. Preloading requires time and can be combined with prefabricated vertical drains (PVDs) or wick drains. Vertical drains are used to accelerate this consolidation and strength gain process utilizing single state or multi-stage construction by shortening the drainage path. Preloading can be performed in stages as project needs require.

Vertical drains can be installed typically to a maximum depth of 80 feet to accelerate consolidation. Vertical drain spacing also impacts the hold times required for preload and staged loading with typical spacings of 3 to 5 feet in a triangular pattern. Vertical drains should be used in conjunction with horizontal drains at the surface (base of surcharge) to allow for the lateral evacuation of pore water. The design and installation of vertical drains is performed by a specialty contractor. A plan drawing showing the approximate extents of the earth preload must be provided.

4.10.2.5 Impact Densification

Impact densification uses energy from a falling heavy weight dropped onto the ground surface to improve granular and fill soils. Such methods have been used to improve roadways subgrades in Alabama to cross strip mine spoils, old landfills, etc. Densification is typically performed in a grid pattern that is determined by the subsurface conditions and foundation loads. Refer to *GEC 1 – Dynamic Compaction* (FHWA Publication No. FHWA-SA-95-037) for additional guidance.

4.10.2.6 Soil Mixing (Wet or Dry)

The soil mixing process involves the use of special mixing tools or paddles that are rotated into the ground through weak soil layer(s) and into underlying competent soil. The soil is premixed during penetration until the mixing tool reaches the termination depth. Dry binding agents (cement, fly ash, lime, gypsum, slag, etc.) are injected into the soil and mixed in-place, creating a dry soil mix column. Within a few hours after mixing, the treatment area is preloaded with 2 to 3 feet of soil to provide confinement during curing and to remove entrapped air. After curing is complete, the treated soil (sometimes called “soilcrete”) is stronger and stiffer than before treatment. Refer to *Deep Mixing for Embankment and Foundation Support* (FHWA Publication No. FHWA-HRT-13-046) for additional information.

4.10.3 *Sinkholes*

As noted in Section 2.2.1, sinkholes can occur anywhere that carbonate rock formations (e.g., limestone and dolostone) underlie the site. However, some formations are known to produce more sinkholes than others. The geotechnical exploration for projects overlying carbonate rock should include a detailed review of topographic maps and aerial photos for indications of past sinkhole activity. Liniments connecting these surface indications can be used to locate areas that are particularly susceptible to future subsidence.

Sinkholes in Alabama are typically cover-collapse type sinkholes, which typically occur suddenly (and sometimes fatally) without surface indications of incipient dropouts. Sinkholes naturally develop slowly; however, manmade conditions, particularly groundwater fluctuations due to pumping, can greatly accelerate the formation and surface expression of sinkholes. Projects with excavation into the natural soils may also encounter incipient sinkholes.

- For pre-existing open sinkholes, sinkholes exposed during excavation, and historic sinkholes that have been filled in an uncontrolled manner, the Engineer should recommend remediation in accordance with ALDOT Special Drawing LS-224: Special Lime Sink Treatment. Sinkhole remediation should be performed with the intent of allowing water (but not soil) to continue to flow into the bedrock at the location of the sinkholes. This is accomplished by constructing an “inverted filter” in the overburden above the sinkhole throat with coarser stone at the top of bedrock, grading upward to finer stone, and capped with low permeability soil at the ground surface.
- For areas particularly prone to sinkholes (e.g. the Roberts Field area [I-59] in Birmingham), the roadway is best stabilized across the troublesome zone with an at-grade bridge supported on deep foundations.
- To locate incipient sinkholes, geophysical electrical resistivity methods (see Section 3.2.1.4) have been used with some success. At specific locations of suspected incipient sinkholes, geophysics should be combined with drilling to check for voids in the soil overburden.
- The Geotechnical Report should recommend specific methods for locating and/or treating known or suspected sinkhole locations.

4.10.4 *Rockfall*

Rockfall, as it is discussed in this section, refers to blocks of rock that fall from the face of rock slopes over time due to weathering of more erodible seams, freeze/thaw, vegetation growth, etc. Such rockfall can pose a hazard to roadway users. Large scale rock slope failures, such as those due to rock mass properties, water table conditions, unfavorable cut slopes/orientations, etc., should be separately analyzed as part of the slope study (see Section 4.5).

For rockfall, the Engineer should take one of the following approaches:

- Recommended bench widths and rockfall catchment area widths and configurations in accordance with *Rockfall Catchment Area Design Guide, Final Report SPR-3(032);* Oregon Department of Transportation: https://www.oregon.gov/ODOT/Programs/ResearchDocuments/RockfallReportEng.pdf.
- Perform a site-specific rockfall hazard analysis and recommend rockfall hazard mitigation measures:
    - Analyze the rockfall hazard using the program *Colorado Rock-Fall Simulation Program (CRSP):* https://coloradogeologicalsurvey.org/publications/colorado-rockfall-simulation-program/.

        - The Engineer will estimate the required input parameters (including slope configuration, surface roughness, and falling rock size range and shape).
        - The Engineer will report their assumptions and give an approximate percentage of falling rock that will fall/travel outside of the catchment area (i.e., onto paved shoulders or roadways where it may pose a hazard).
    - Recommendations for reducing the hazard, if necessary, should be provided. Such recommendations may include:
        - Alternative slope, bench, and catchment area configurations,
        - Rockfall netting,
        - Selective rock bolting of larger blocks, or
        - Shotcrete and rock bolt cover, particularly for beds/seams of more erodible rock (such as shale) that may differentially weather, causing a progressive increase in rockfall or slope failure hazard as erosion progresses.

Additional information on rockfall hazard mitigation is provided in *Rock Slopes Reference Manual* (FHWA Publication No. FHWA-HI-99-007).

4.10.5 *Mine Studies*

To determine if a mining operation may impact the project area, the Engineer or State Geologist will conduct the following activities:

- Evaluate the need for study: Make an initial evaluation of the study area by reviewing available topographic maps and regional geology to determine the possibility of underground or strip mines.
- Historical research: After confirming the need to complete a study, conduct a detailed literature search to assess possible locations, type, and extent of mining activities.
- Agency search: Visit the Mine Safety and Inspection Office, the Surface Mining Commission, the Alabama Geological Survey, or other government offices or commercial facilities in order to find detailed maps and records of mining activity.
- Field reconnaissance: Walk the project area to evaluate potential impact of the suspected mining activity on the construction project.
- Drilling confirmation: If it appears from the available data that underground mines could be present and that they are shallow enough to potentially impact the project, an attempt to locate the suspected mines and/or minable mineral seams by using drilling methods, soil profile drilling, or additional drilling should be planned.
- Drilling considerations: It should be recognized that failure to encounter an open mine during drilling does not necessarily suggest that mining has not taken place, as the roof over certain ore measures (particularly in the Red Mountain formation) may have completely collapsed after mining. A series of borings, preferably in an "L" pattern, and spaced at less than the projected haulage way or room widths should be drilled, and the results should be compared. Ore seams encountered in some borings but not in others at similar depths suggests mining has taken place.
- Subsidence over time, or lack thereof: It should also be recognized that the passage of months or even years without the evidence of surface collapse over underground mines does not necessarily suggest that

future surface collapse will not occur. Early mining activity used timbers to support the roof, and these supports will decay with time, particularly with varying groundwater levels.

- Long-wall mining subsidence: With the more modern "long-wall" underground mining methods, the mining activity is designed so that the mine roof comes down behind the mining machinery and much of the subsidence propagates to the surface, often unevenly, no matter the depth of the mining. This potential subsidence can sometimes be measured in feet. Since, in Alabama, most of the mineral rights have long since been separated from the surface rights, control of this type of future subsidence may be difficult without cooperation with the mining companies.
- Surface mining: Surface mining activities should also be identified. Open pits above the groundwater table can be backfilled with compacted embankment material. Shallow mining activities filled with loose mine spoils are best treated by undercutting and recompacting the unconsolidated mine spoils.
- Strip mine remediation: For deeper strip mine spoils up to 100 feet deep or more and usually associated with coal extraction, ground modification methods, such as impact densification as outlined above, have been used to stabilize the upper 30 feet or so before roadway construction.
- High groundwater conditions: For surface mining activities where the groundwater table is within the unconsolidated material, special construction methods, such as the use of well points, wick drains, etc., may be warranted to stabilize the subgrade.
- Present findings: Submit a written report of findings to include the mine location, type of mine, material mined, approximate mine depths, approximate mine alignment in relation to the proposed roadway layout, potential impact on the road and construction, recommended method of treatment, if needed, and other information that the Engineer/Geologist deems appropriate.

4.10.6 *Erosion Control*

In cases where there is a significant risk of loss-of-ground or other deleterious effects, aspects of scour or erosion control should be incorporated into the recommendations provided in the Geotechnical Report. This may include:

- Geosynthetic separators, filters, or drains;
- Swales, pipes, or flumes;
- Impervious barriers or drainage and filter systems;
- Geocells or other cellular soil confinement systems, gabion baskets or revetment mattresses, bio stabilization, or facing with geotextiles or other semi-permanent or permanent erosion control products; or
- Recommendations to lower or control of groundwater to reduce seepage and potential instabilities at slope faces.

4.11 Geotechnical Report Requirements

The final Geotechnical Report provided by the Engineer will include the following items as a minimum, and including project-specific items noted in the relevant project-specific sections of this manual:

- A brief description of the purpose of the exploration and the field work performed to include such information/data as when and who performed the work, site conditions at time work was performed, scope, and scope changes during fieldwork (variances from the Geotechnical Exploration Plan).

- Relevant observations and findings from the site review/field reconnaissance (see Section 3.1.1), which should include observed or mapped landslide/slope failures, sinkholes, etc., in the vicinity that may impact the project.
- Site and subsurface conditions:
    - A geologic and physiographic description of the site to include location, geologic, and topographic maps.
    - Brief description of in-place structures to include foundation type, design load, pile tips, etc.; the structure’s performance; and a copy of boring logs and pile driving records found during the historical search.
    - A general description of subsurface soil, rock, and groundwater conditions encountered.
    - Boring logs (as outlined below), boring location plan, field test data, summary of laboratory test data, and graphical and/or tabular information obtained through field soft soil soundings, if performed.
    - Interpreted subsurface stratigraphy/profile and material properties.
    - Summary and specific comments regarding groundwater conditions at the site, to include artesian conditions, if present, and impacts the groundwater conditions may have on the project.
- Engineering analysis and recommendations:
    - A listing of critical assumptions used in developing the recommendations.
    - References, methods, and software used for engineering analysis.
    - Specific engineering recommendations for design (as noted in the project-specific sections of this manual).
    - The report should include a separate section for “Recommended Plan Notes,” which should include requirements and/or recommendations that the Engineer sees a benefit to including in the project plans. Such recommended plan notes should be worded as requirements/specifications (i.e., using the word “shall”).
    - The report should include a separate section for “Estimated Construction Quantities,” which should include estimated quantities for Pay Items relevant to the foundations, ground improvement, etc., as recommended in the Geotechnical Report.
- Although the specific means and methods for the construction as well as the means of controlling unwanted soil/rock movements will remain in the purview of the Contractor, the Geotechnical Report should point to particular hazards noted and/or anticipated construction difficulties based on the geotechnical exploration. The Geotechnical Report should present such potential construction considerations, for example:
    - Potential groundwater impacts on construction, such as fluctuations in height of groundwater or surface water, water control in excavations using well points, pumping, tremie seals, etc.
    - Potential impacts to adjacent structures and damage that may result from excavation/fill, pile driving, blasting, drainage, etc.
    - Evaluation of pile driving to include difficulties or unusual conditions that may be encountered (i.e. hard driving), special precautions that may be required, special equipment, and pile driving sequence.

- Required excavations to include control of earth slopes, types of material to be encountered, and the need for a cofferdam or sheeting and shoring.
- Comments or recommendations on construction sequence for landslide remediations and other special construction situations.

- Boring Logs:
  - A 22"x34" size plan sheet/drawing border for use with boring logs and boring location plan drawings will be provided by the Geotechnical Division upon request.
  - The Engineer will prepare full-size (22"x34") PDF sheets depicting the boring location plan and boring logs for inclusion in the final plan assembly. These boring logs will be easily readable when produced at half-scale, quarter size, and 11"x17" plots. The appropriate font types and sizes as well as line weights for use in MicroStation are available on the ALDOT website under the Design Bureau.
  - The boring logs will indicate the measured water level in the boring; depth to caving if the hole caved in; and/or no water table encountered.
  - The boring logs will also indicate the make and model of the drill rig, method of drilling and/or coring, the type of hammer used for SPT sampling, and the calibrated SPT hammer efficiency.
  - A note should be placed on the boring log sheet indicating where rock cores (if collected) will be stored.
    - Cores will be taken to the Geotechnical Division Drill Crew Warehouse and stored for review by potential contractors.
    - The boxes will be labeled as noted in Section 3.2.1.2, which includes the top and four sides of the core boxes, and should be labeled with the following information, at a minimum:
      - Project identification number/name;
      - Borehole number/designation;
      - Boring location, including staked location information and measured offset;
      - Top and bottom depth of core stored in box;
      - Box number; and
      - Measured length of core recovery and RQD (labeled on the top side of box only).
  - All borings will be identified sequentially referencing station numbers, offsets, and bent/abutment/pier numbers as designated on the boring layout.
  - If soft soil "soundings" were obtained on the project, a graphical interpretation of this information should also be placed on full-size (22"x34") PDF sheets for inclusion in the final plan assembly.
  - The Engineer can propose a different boring log product for review at time of boring layout submittal as long as said proposal is accepted by the State Geotechnical Engineer, prior to initiation of the work.

The Engineer will submit a draft copy of the Geotechnical Report for review by the Geotechnical Division. Along with the draft, the Engineer will submit a completed copy of the FHWA document "Checklists and Guidelines for

Review of Geotechnical Reports and Preliminary Plans and Specifications." Once the Geotechnical Division has returned comments, the Engineer will finalize the report, submit a locked PDF to the Geotechnical Division, and upload a locked PDF to GeoGIS.

5.0 Information and Report Management

Once a Geotechnical Report is finalized (see Section 6.2 as it relates to consulting):

- Relevant files will be uploaded to GeoGIS (https://aldotgis.dot.state.al.us/geogis):
    - A secured PDF of the report with a Letter of Concurrence from the Geotechnical Division must be attached as the first page(s) of the PDF document. Physical copies will be submitted if requested by the Geotechnical Division.
    - A DIGGS (Data Interchange for Geotechnical and Geoenvironmental Specialists; http://diggsml.org) file with laboratory testing data will be uploaded.
    - For landslide studies, the Engineer will submit required information for inclusion in the State Landslide Database (see Form A-6 in Appendix A).
- A signed/sealed PDF of the full-size (22"x34") boring plan and boring logs will be submitted to the Geotechnical Division.
- Upon request by the Geotechnical Division, the Engineer will also provide physical copies, Microstation drawings, and gINT® files for the boring logs.

6.0 Geotechnical Consultant Information

6.1 ALDOT Organization

ALDOT is organized into five regions: North, West Central, East Central, Southwest, and Southeast. A map showing the counties included in each region, along with numerous other maps useful during geotechnical explorations, is available at: https://www.dot.state.al.us/maps.html and shown below. A Bureau and Region Directory is available at https://www.dot.state.al.us/pdf/StaffDirectory.pdf.

6.2 Geotechnical Consulting and Design Process

For projects where the Geotechnical Division contracts a Consultant to perform the work, the Consultant will act as the Engineer, as noted above in Section 1.1. The Engineer has primary responsibility for the scoping and execution of the geotechnical exploration, evaluating alternatives, performing analyses, developing recommendations, issuing reports, and providing other support as requested by the Geotechnical Division.

Projects should generally proceed in the following steps; although, variations may occur at the direction of the State/Assistant Geotechnical Engineer. Additional requirements for Soil Surveys are provided in Section 4.7.

1. Request for Cost (RFC): The Geotechnical Division will issue an RFC to the Consultant. The RFC will specify the project type and will be transmitted with relevant and available project information, plans, drawings, etc. The Consultant will submit an outline of the anticipated efforts and budget.
2. Request for Proposal (RFP): The Geotechnical Division will issue an RFP to the Consultant. Often, the RFC and RFP may be requested simultaneously. The Consultant will perform a site review (see Section 3.1.1) and develop a Geotechnical Exploration Plan (see Section 3.1.2). The proposal should include a detailed budget estimate along with the Geotechnical Exploration Plan. The Geotechnical Division may provide comments, request justification, or directions regarding the Geotechnical Exploration Plan, which the Consultant should address and resubmit.
3. Perform the Geotechnical Exploration: The Consultant will execute the Geotechnical Exploration Plan once the Consultant Purchase Order (CPO) is received. The Consultant will update the State/Assistant Geotechnical Engineer in a timely manner of significant deviations from the Geotechnical Exploration Plan. When additional exploration efforts are required due to unanticipated conditions, the Consultant must obtain approval from the State/Assistant Geotechnical Engineer for the additional efforts if they will cause the Consultant to exceed the provided budget estimate.
4. Issue a Draft Report: The Consultant will perform engineering work required to produce the Geotechnical Report, as outlined in this manual. The Geotechnical Division will review the report and provide comments or required revisions. Once the comments and revisions are addressed, the Consultant should submit a record of the changes to the State/Assistant Geotechnical Engineer for review before issuing the final report.
5. Issue a Final Report: Once the State/Assistant Geotechnical Engineer has approved revisions to the report, the Consultant will finalize the report. The report will be provided as a secured PDF document and uploaded to the GeoGIS site. A Letter of Concurrence from the Geotechnical Division will be attached as the first page(s) of the PDF document. Hard copies will be provided by the Consultant at the request of the State/Assistant Geotechnical Engineer.
6. Calculations: If requested by the State Geotechnical Engineer, the Engineer will also produce example calculations, preferably in Mathcad® (PTC) format.
7. Provide Design and Construction Support: As design and construction proceeds, the Consultant will provide additional support at the request of the Geotechnical Division. This may include assisting with the development of special provisions, reviewing design drawings, evaluating additional alternatives, etc. Unless stated otherwise in the RFC and/or RFP, the cost and effort for these follow-on services should not be included in the RFP for the geotechnical exploration.

7.0 Construction Submittals

The purpose of this section is to outline areas where the Engineer may be involved during the construction phase of an ALDOT project. The State/Assistant Geotechnical Engineer may request that the Engineer provide the following support:

- Review construction submittals required by specification and plans.
- Confirm compliance with geotechnical designs and recommendations.
- Develop and implement an instrumentation program to measure ground and structure movements, water table elevations, vibrations, or other conditions.
- Verify foundation resistance and integrity.
- Review conditions encountered during construction and revise recommendations, as appropriate.
- Provide clarification related to geotechnical recommendations and specifications.

7.1 Submittal Review

At the request of the State Geotechnical Engineer, the Engineer may be involved in the review of the Contractors' design and installation submittals for compliance with the requirements and intent of the plans and specifications during the initial stages of construction. On consultant-reviewed projects, the Engineer will return the response to the State Geotechnical Engineer with a cover letter containing appropriate comments concerning the reviewed item. Submittals will be accepted or rejected by the State Geotechnical Engineer, regardless of who reviewed the project (i.e., the Department or a consultant) and will be forwarded to the Construction Bureau for distribution to the Contractor. As required, rejected submittals will be resubmitted for review. Below list typical submittals received by the Geotechnical Division:

- Pile driving equipment data;
- Drilled shaft installation plan (DSIP);
- Drilled shaft integrity testing results;
- Retaining wall designs, details, and installation plan;
- Wick drain submittal;
- Temporary shoring;
- Micropile design, details, and installation; and
- Design and details of the proposed cable guiderail system.

7.2 Foundations

7.2.1 *Shallow Foundations*

A review by the Engineer is typically not required if shallow foundations are supported on the expected bearing stratum. An on-site review of the bearing stratum may be requested if the bearing stratum is questionable or if the top of the bearing level is variable.

7.2.2 *Deep Foundations*

7.2.2.1 Driven Piles

The Engineer may be involved during pile installation if piles do not achieve design requirements, require pile load testing, are driven deeper than anticipated (overdrive), encounter early refusal, or have pile driving resistances that are different than anticipated. In addition, guidance and recommendations regarding predrilling of shallow rock and protective pile tips may be requested from the Engineer.

7.2.2.1.1 Pile Testing

Depending on the resistance factor (φ) selected during design, (refer to Section 4.2.5.3.1.3), load testing may be required. The load testing may consist of either high-strain (Pile Driving Analyzer [PDA]) or static/rapid (Statnamic®) load tests.

7.2.2.1.1.1 Dynamic Pile Testing

The Engineer should resolve deficiencies in driving resistance determined from PDA testing and establish production pile driving criteria based on a pile for which the required driving resistance has been verified by PDA testing and signal matching analysis.

Acceptance of piles based on projected setup is not allowed. Where additional setup is anticipated and is necessary to meet the required driving resistance, conduct PDA testing and signal matching analysis after the setup has occurred.

7.2.2.1.1.2 Static Load Testing

The Engineer will use the information obtained from the load test to revise the bearing curves to provide a better estimation of bearing for a specific blow count. These revised curves will be submitted to the ALDOT Construction Bureau to be used for estimating the bearing of the production piles.

7.2.3 *Drilled Shafts*

The Engineer will review the Drilled Shaft Installation Plan (DSIP) and shaft constructability (e.g., dry vs. wet construction method). The Engineer may also be called upon to evaluate rock socket length at the request of the State Geotechnical Engineer. For projects where cross-hole sonic logging (CSL) or load testing is performed, the Engineer will review the test results, including the confirmation boring (probe hole and/or core sample), and participate in determining the acceptance of the shaft construction or design. When these tests are required, the Engineer may be requested to aid Construction Project personnel in monitoring the tests.

Appendix A

Example Forms and Tables

Table A-1: SPT/Rock Core Boring Assignment Table

Boring No.	Station	Offset	Min. Boring Depth or Min. Penetration into Bedrock	Additional Termination Criteria (Note 1)	Intact (Shelby Tube) Samples (Note 2)	Other Testing (Note 3)

Note 1: Termination criteria for soil should be established as a required penetration into a specified material (material type and minimum SPT N-values). Termination criteria for rock coring should be established as a minimum recovery and RQD for a specified length of rock core.
Note 2: Provide approximate quantities and collection criteria (such as material type and N-values).
Note 3: Provide testing that will be performed with support of the SPT drill rig/crew, such as portable (rig-mounted) CPT, vane shear testing, pressuremeter testing, etc.

Table A-2: Survey Request Form

Boring No.	Station	Offset	Northing	Easting	Elevation

Note 1: The Engineer is to complete the Boring No., Station, and Offset columns. Surveyor will return the Survey Request Table with Northing, Easting, and Elevation information.

Form A-3: Pre-Construction Investigation Activities Environmental Permits Checklist

Following page

6. Appendix A

Pre-Construction Investigation Activities
Environmental Permits Checklist

ALDOT Project Number: Date:

County:

1. ADEM NPDES STORMWATER PERMIT	
1.1 Will the soil disturbance with the project activity be 1 or more acres? (If no, skip to Section 2)	☐ Yes ☐ No
1.2 If Yes, Has NPDES stormwater permit coverage been requested and obtained from ADEM?	☐ Yes ☐ No ☐ N/A
1.3 List the permit number issued by ADEM for the project activity, if applicable.	
1.4 Will mulching be utilized as the method of clearing (above the soil line)?	☐ Yes ☐ No ☐ N/A
1.5 Is the project area considered to be a Priority Construction Site per ADEM Regulations?	☐ Yes ☐ No
1.6 Has CBMPP for Pre-Construction Investigation Activities template been completed for the project activity?	☐ Yes ☐ No
2. US ARMY COPRS OF ENGINEERS PERMIT	
2.1 Will work activity involve defined wetlands, waters of the US or a stream crossing? (If no, skip to Section 3)	☐ Yes ☐ No
2.2 If Yes, Has a Corps of Engineers permit been requested and obtained? If no, explain.	☐ Yes ☐ No ☐ N/A
2.3 List the permit(s) issued for the project activity, if applicable.	
3. OTHER ENVIRONMENTAL CONCERNS OR CONSIDERATIONS	
3.1 Has coordination with the Environmental Technical Section (ETS) been conducted for known resources in the area? If no, contact ETS to identify any impacts that may be in the area.	☐ Yes ☐ No
3.2	

Table A-4: Driven Pile Foundation Recommendations Table

Geotechnical Sites	**Site #1**		**Site #2**		
Substructure Location	**Abutment 1**	**Abutment 4**	**Bent 2**	**Bent 3**	**Bent 4**
Pile Type					
Controlling Limit State					
Factored Design Load (tons) (a)					
Unfactored Load Resistance Loss from Scout (tons) (b)					
Total Load for Controlling Limit State (tons) (a+b)					
NRVP Designation					
Geotechnical Resistance Factor (GRF)					
Estimated Scour Elevation at Controlling Limit State (feet)					
Estimated Pile Tip Elevation (feet)					
Minimum Pile Tip Elevation (feet)					
Required Driving Resistance (tons)					

Quantities for Required Driving Resistance Verification					
Substructure Location	**Abutment 1**	**Abutment 4**	**Bent 2**	**Bent 3**	**Bent 4**
Number of Test Piles (per each)					
Number of Static Loading Tests					
Number of Dynamic Loading Tests, Driving (per each)					
Number of Dynamic Loading Tests, Re-Strike (per each)					

Pile Points (per each)	
Pilot Holes (per linear foot)	

Note 1: Minimum Tip Elevations are controlled by Lateral Loading requirements as provided by the Bridge Designer.
Note 2: Piles will be driven to the minimum tip elevation and the Required Driving Resistance.
Note 3: Test Pile locations may be altered within a site to accommodate construction sequence with approval from State Geotechnical Engineer.
Note 4: ***Dynamic Loading Test, Driving*** will be applied to Test Piles, at the designated Substructure Location.
Note 5: ***Dynamic Loading Test, Re-Strike*** will be applied to Test and Production Piles at the respective Substructure Location.

Table A-5: Drilled Shaft Foundation Recommendations Table

Location	Station	No. of Shafts	Shaft Diameter	Load Information (tons per shaft)			Controlling Limit State	Recommended Shaft Tip Elevation	Minimum Shaft Tip Elevation	Permanent Casing Elevation
				Service	Strength	Extreme Event				
Abutment 1										
Bent 2										
Bent 3										

Form A-6: Landslide Reporting Form

Following 2 pages

Landslide Reporting Form Report or CPMS #:

Completed By: ______________________ Inspection Date: ___________

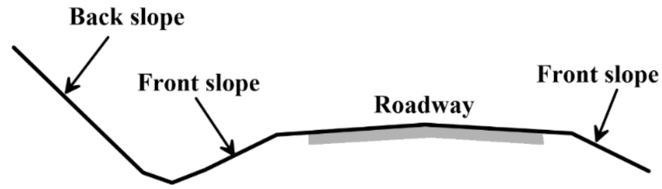
Location

Route Number: ______ Milepost: ____ Direction: ☐ North ☐ South ☐ East ☐ West

County: _____________ Latitude: _ _ . _ _ _ _ _ _° Longitude: _ _ . _ _ _ _ _ _°

Location along roadway:

☐ Front Slope ☐ Back Slope



Failure Description

Landslide Type: ☐ Rotational ☐ Translational ☐ Surface Erosion

See definitions on next page ☐ Fall ☐ Topple ☐ Flow

Slide Material:
Select best description

◯ Earth (predominantly sand and/or clay)

◯ Debris (20 – 80% of particles are larger than 1 inch)

◯ Rock (intact before movement occurred)

Length of slide (ft, parallel to road): ____________ Slope Ratio: ____ H : _____ V

When did the failure occur? Month: ________ Day: _____ Year: _________

Impacts: ☐ Shoulder Closed ☐ Lane Closed ☐ Road Closed ☐ No Traffic Impact

Additional Information

Nearby Structures: ☐ Bridge ☐ Retaining wall ☐ Culvert ☐ Buried Utilities

Other:

Vegetation on Slope: ☐ Grass ☐ Brush ☐ Trees ☐ None

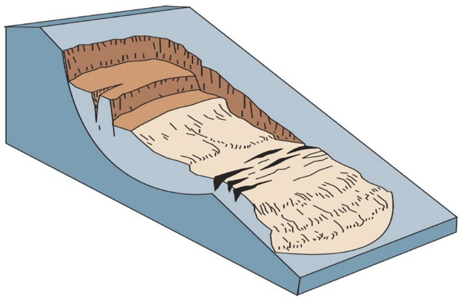
Are there any previously repaired slope failures within 500 feet? ◯ Yes ◯ No

Is there rutting from vehicle tires on nearby slopes? ◯ Yes ◯ No

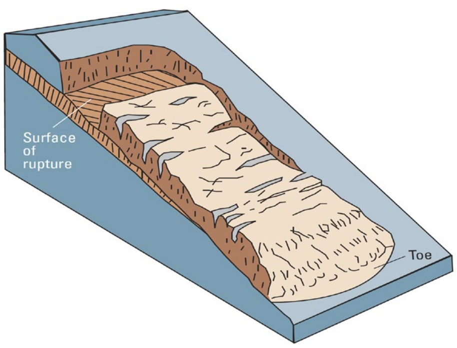
Comments:

Definitions for Landslide Types:

Rotational Slide: Failure occurs on a well-defined curved failure surface. Blocks of failed material can rotate and can at times be seen to tilt backwards towards the slope.

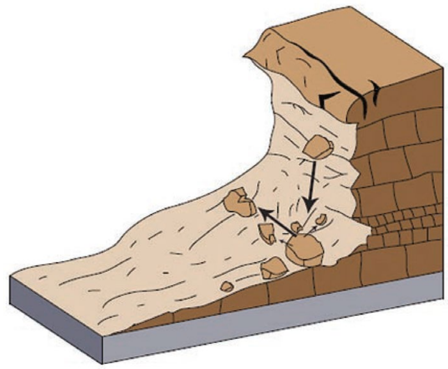


Translational Slide: The mass in a translational landslide moves out, or down and outward, along a relatively flat surface with little rotation or backward tilting.

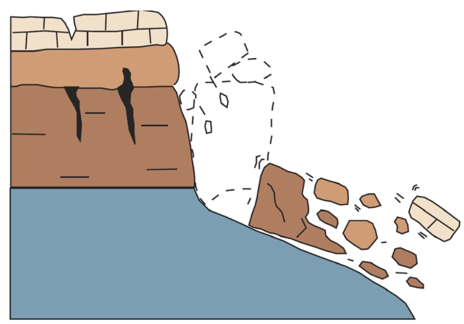


Surface Erosion: The upper few inches to foot of soil is eroded by moving water leaving an area of bare soil behind.

Fall: Pieces of rock or earth, or both, quickly detach from steep slopes or cliffs and collect near the base of the slope.



Topple: A mass of soil or rock rotates out from the intact material (tilting) around an axis (or point) near the base of the block.



Flow: Flows are landslides that involve the movement of material down a slope in the form of a fluid. The failed mass does not usually have a well-defined structure.

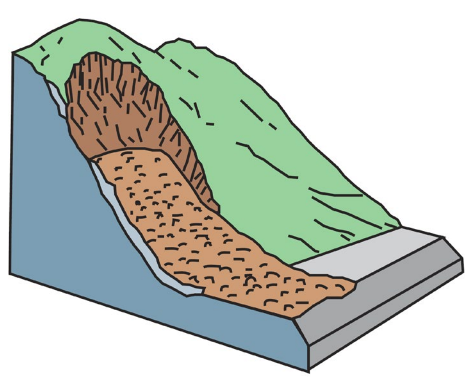


Table A-7: Topsoil Survey Summary Table

Sample No.	Sample Collected Date	Representative Beginning Station	Representative End Station	Average Topsoil Depth (feet)	Estimated Topsoil Volume (yd^3)	Deleterious Materials (7% max)	Organic Materials (2-20%)	Sand Content (10-90%)	Silt and Clay Content (10-90%)	pH (5-7)	Meets ALDOT Standard Specifications?

	Topsoil Test Results Table
	Alabama Department of Transportation
	Project No.:
	Project Name:
	County:

Notes:

Table A-8: Resilient Modulus Summary Table

Boring No.	Station	Depth	AASHTO Classification	Dry Density (pcf)	Moisture Content (%)	% Compaction	M_R Values at 4 psi Confining Pressures (psi)					Avg.
							Seq6	Seq7	Seq8	Seq9	Seq10	

Boring No.	Station	Depth	AASHTO Classification	Dry Density (pcf)	Moisture Content (%)	% Compaction	M_R Values at 2 psi Confining Pressures (psi)					Avg.
							Seq11	Seq12	Seq13	Seq14	Seq15	

Avg.	
Standard Dev. (stdev)	
(Avg. – 2*stdev)	
85th percentile	
90th percentile	
100th percentile	

	M_R Calculation Summary
	Alabama Department of Transportation
	Project No.:
	Project Name:
	County:

Notes:

Table A-9: Soil Classification Summary Table

Station and Offset	Boring No.	Sample ID	Depth (feet)	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	% Gravel	% Pass #200		D50 (mm)	USCS	AASHTO Classification
									% Silt (if hydrometer data available)	% Clay			

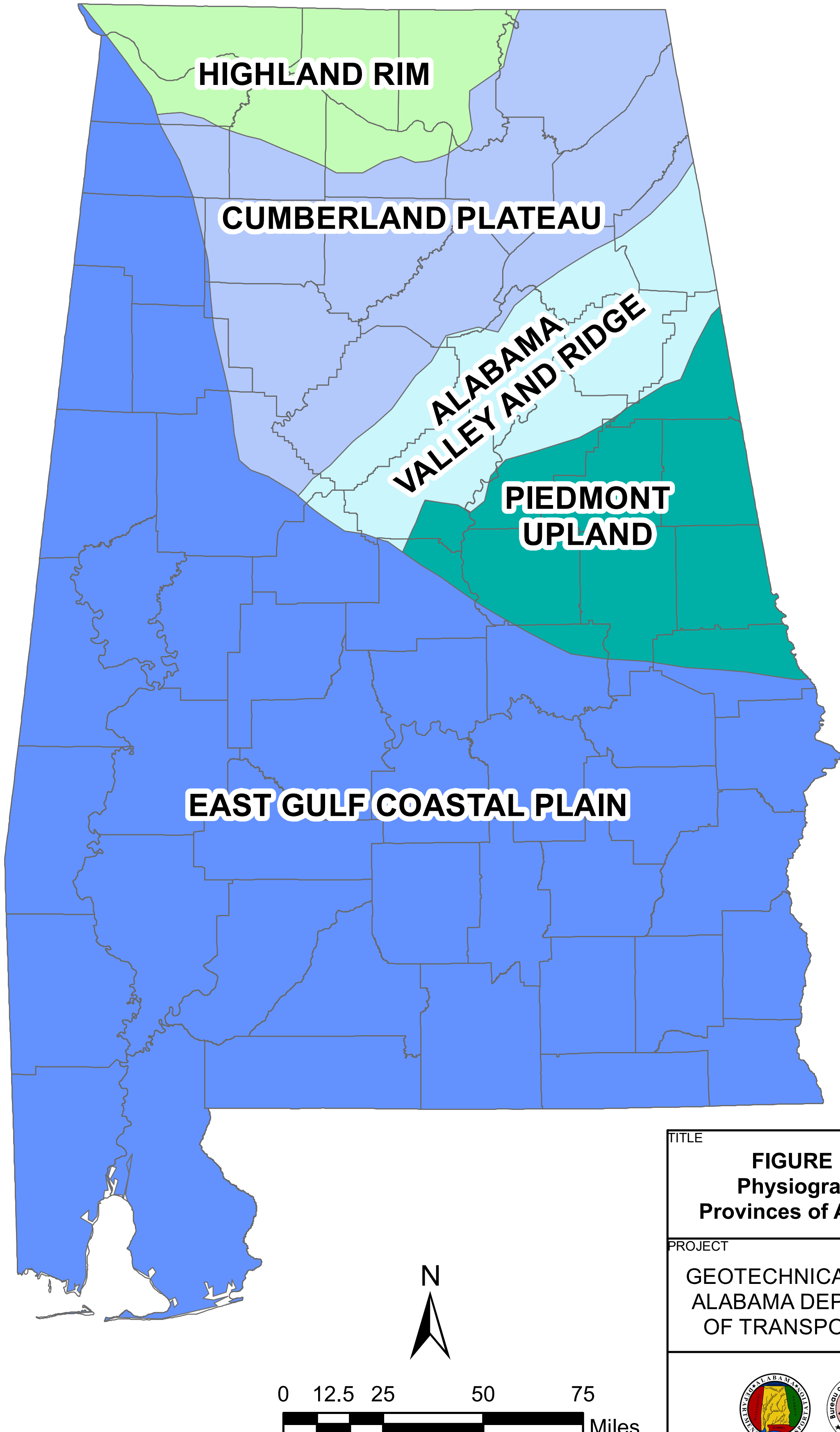
	Soil Classification Summary
	Alabama Department of Transportation
	Project No.:
	Project Name:
	County:

Table A-10: Design Resilient Modulus by AASHTO Soil Classification

AASHTO Group Classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7, A-7-5, A-7-6
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				
Confining Pressure for Design	4 psi	4 psi	4 psi	4 psi	4 psi	2 psi	2 psi	2 psi	2 psi	2 psi	2 psi
Sample Moisture Content	Optimum	Optimum	Optimum	Optimum	Optimum	Optimum	Optimum	Optimum	Optimum	Wet Side of Opt.	Wet Side of Opt.

Appendix B

Figures – Geology of Alabama

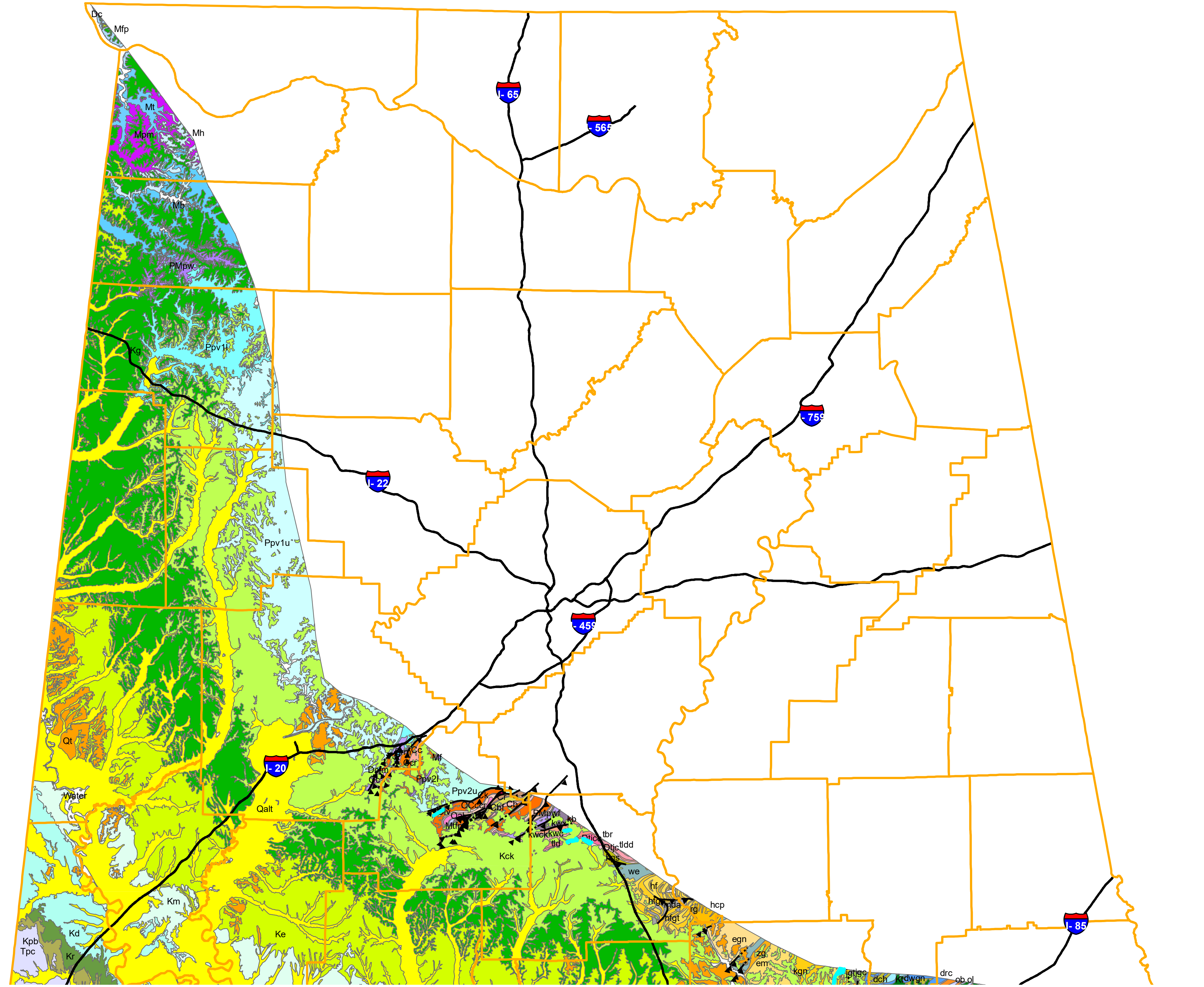


TITLE

FIGURE 2-1
Physiographic
Provinces of Alabama

PROJECT

GEOTECHNICAL MANUAL
ALABAMA DEPARTMENT
OF TRANSPORTATION



LEGEND

- Alabama County Boundary
- Concealed Fault
- Fault (sense of movement unknown)
- Thrust Fault
- Thrust Fault - Inferred
- Qalt — Alluvial, coastal, and low terrace deposits
- Oal — Athens Shale and Lenoir Limestone undifferentiated
- Mb — Bangor Limestone
- Cb — Bibb Dolomite
- ob — Bottle Granite
- bp — Brewer Phyllite
- Cbf — Brierfield Dolomite
- tbr — Butting Ram Sandstone
- dch — Camp Hill Granite Gneiss
- Dc — Chattanooga Shale
- Dcfm — Chattanooga Shale and Frog Mountain Sandstone undifferentiated
- OCcor — Chepultepec and Copper Ridge Dolomites undifferentiated
- Oc — Chickamauga Limestone
- Kck — Coker Formation
- Cc — Conasauga Formation
- Ccr — Copper Ridge Dolomite
- Kd — Demopolis Chalk
- egn — Elkahatchee Quartz Diorite Gneiss
- em — Emuckfaw Group undifferentiated in part
- Ke — Eutaw Formation
- Mf — Floyd Shale
- Mfp — Fort Payne Chert
- Kg — Gordo Formation
- Mh — Hartselle Sandstone
- hf — Higgins Ferry Group
- hfgt — Higgins Ferry Group garnet quartzite unit
- hfgr — Higgins Ferry Group graphitic unit
- Qt — High terrace deposits
- hgs — Hillabee Greenstone
- jgc — Jacksons Gap Group sericite and chlorite phyllite unit
- Djcc — Jemison Chert and Chulafinnee Schist undif., fossilliferous chert facies
- Djc — Jemison Chert and Chulafinnee Schist undifferentiated
- kwck — Kalona Quartzite Member of the Wash Creek Slate
- Ck — Ketona Dolomite
- kgn — Kowaliga Gneiss
- tld — Lay Dam Formation
- tldd — Lay Dam Formation diamictite unit
- ol — Loachapoka Schist
- Olv — Longview Limestone
- mda — Mitchell Dam Amphibolite
- Km — Mooreville Chalk
- On — Newala Limestone
- Onlv — Newala and Longview Limestones undifferentiated
- PMpw — Parkwood Formation
- PMpwf — Parkwood Formation and Floyd Shale undifferentiated
- hcp — Pinchoulee Gneiss
- Tpc — Porters Creek Formation
- Ppv1l — Pottsville Formation (lower part), Appalachian Plateaus
- Ppv2l — Pottsville Formation (lower part), Valley and Ridge
- Ppv1u — Pottsville Formation (upper part), Appalachian Plateaus
- Ppv2u — Pottsville Formation (upper part), Valley and Ridge
- Kpb — Prairie Bluff Chalk
- Mpm — Pride Mountain Formation
- Srm — Red Mountain Formation
- Kr — Ripley Formation
- rg — Rockford Granite
- Cr — Rome Formation
- drc — Ropes Creek Amphibolite
- ksc — Stumps Creek Formation
- jgt — Tallassee Metaquartzite
- Kt — Tuscaloosa Group undifferentiated
- Mt — Tuscumbia Limestone
- Mtfp — Tuscumbia Limestone and Fort Payne Chert undifferentiated
- kwc — Wash Creek Slate
- Water
- dwgn — Waverly Gneiss
- kwx — Waxahatchee Slate
- we — Wedowee Group undifferentiated
- zg — Zana Granite

N

0 25 50 Miles

PROJECT

GEOTECHNICAL MANUAL
ALABAMA DEPARTMENT
OF TRANSPORTATION

TITLE

FIGURE 2-2A
Geology of
the East Gulf Coastal Plain
of Alabama (North)

DRAWN BY

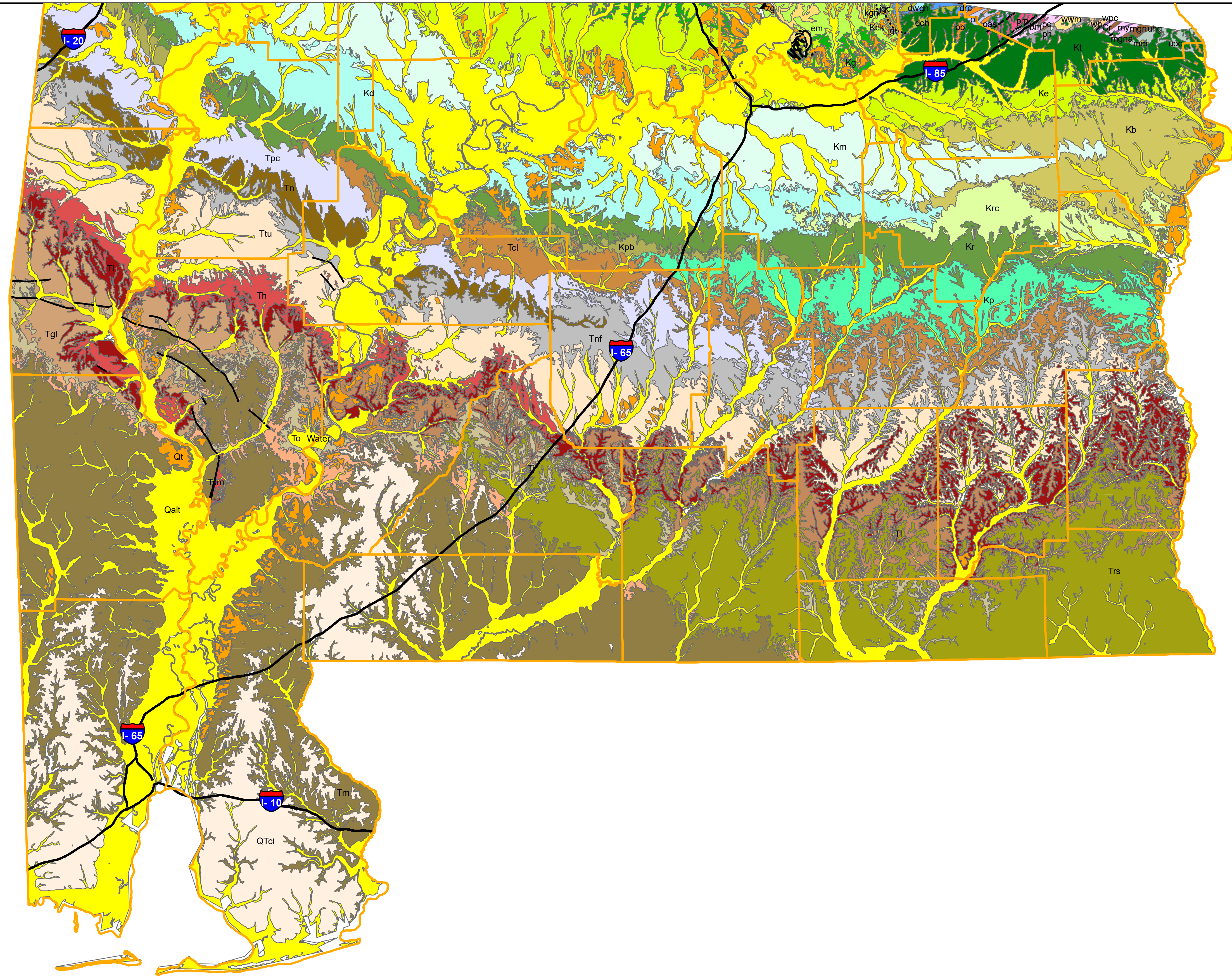
MMK 8/28/20

REVIEWED BY

LN 8/28/20

LEGEND

- Alabama County Boundary
- Concealed Fault
- Fault (sense of movement unknown)
- Normal Fault
- Qalt — Alluvial, coastal, and low terrace deposits
- oas — Auburn Gneiss schist unit
- bm — Blastomylonite and mylonite
- Kb — Blufftown Formation
- ob — Bottle Granite
- dch — Camp Hill Granite Gneiss
- pc — Chewacla Marble
- QTci — Citronelle Formation
- Tcl — Clayton Formation
- Kck — Coker Formation
- Krc — Cusseta Sand Member of the Ripley Formation
- Kd — Demopolis Chalk
- em — Emuckfaw Group undifferentiated in part
- Ke — Eutaw Formation
- Kg — Gordo Formation
- Tgl — Gosport Sand and Lisbon Formation undifferentiated in part
- wh — Halawaka Schist
- Th — Hatchetigbee Formation
- Qt — High terrace deposits
- ph — Hollis Quartzite
- uhg — Hospilika Granite
- Tj — Jackson Group undifferentiated
- jgc — Jacksons Gap Group sericite and chlorite phyllite unit
- kgn — Kowaliga Gneiss
- Tl — Lisbon Formation
- ol — Loachapoka Schist
- pm — Manchester Schist
- Tm — Miocene Series undifferentiated
- mm — Moffits Mill Schist
- Km — Mooreville Chalk
- mgn — Motts Gneiss
- mgna — Motts Gneiss amphibolite unit
- my — Mylonite and blastomylonite
- Tn — Naheola Formation
- Tnf — Nanafalia Formation
- To — Oligocene Series undifferentiated
- wpc — Phelps Creek Gneiss
- upc — Phenix City Gneiss
- Tpc — Porters Creek Formation
- Kpb — Prairie Bluff Chalk
- Kp — Providence Sand
- Trs — Residuum
- Kr — Ripley Formation
- Cr — Rome Formation
- drc — Ropes Creek Amphibolite
- Tsm — Salt Mountain Limestone
- Tt — Tallahatta Formation
- jgt — Tallassee Metaquartzite
- Ttu — Tuscahoma Sand
- Kt — Tuscaloosa Group undifferentiated
- Water
- dwgn — Waverly Gneiss
- we — Wedowee Group undifferentiated
- wwm — Whatley Mill Gneiss
- zg — Zana Granite



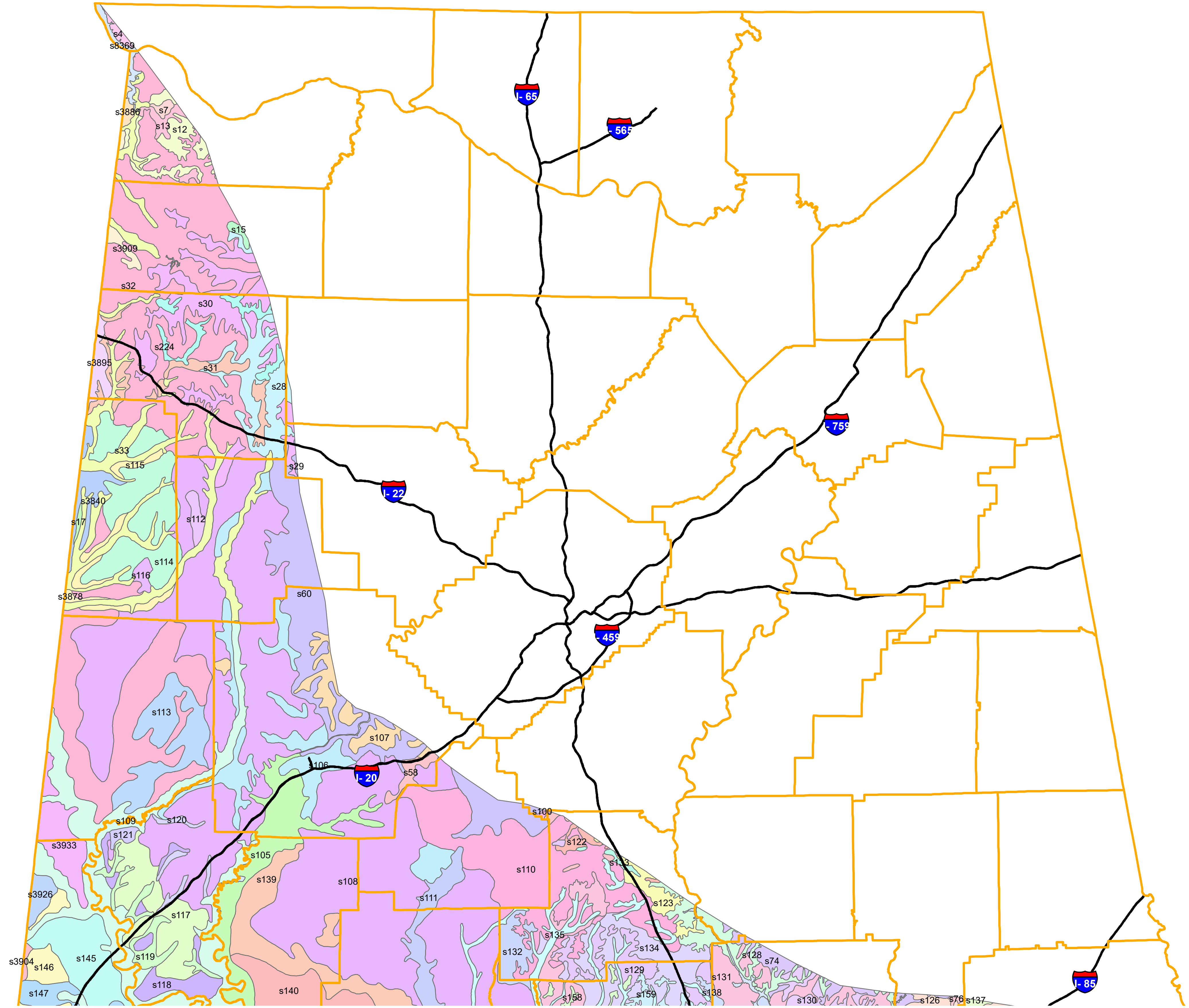
N

0 25 50 Miles

PROJECT	TITLE	DRAWN BY
GEOTECHNICAL MANUAL ALABAMA DEPARTMENT OF TRANSPORTATION	**FIGURE 2-2B Geology of the East Gulf Coastal Plain of Alabama (South)**	MMK 8/28/20
		REVIEWED BY LN 8/28/20

LEGEND

- Alabama County Boundary
- Fullerton-Emory-Decatur (s7)
- Ketona-Colbert-Chisca-Cap... (s13)
- Leaf-Cahaba-Annemaine-Al... (s109)
- Luverne-Halso-Conecuh (s132)
- Luverne-Lucedale-Bama (s158)
- Luverne-Lucedale-Jones-B... (s131)
- Mantachie-Ellisville-Cahaba... (s105)
- Marvyn-Luverne-Cowarts (s130)
- McQueen-Mantachie-Golds... (s138)
- Minvale-Fullerton-Bodine (s58)
- Myatt-Mantachie-Kinston-Iu... (s135)
- Nauvoo-Gorgas (s28)
- Nauvoo-Linker-Hartsells (s133)
- Nella-Nectar-Chisca (s12)
- Ochlockonee-Myatt-Falaya-... (s120)
- Pacolet-Cecil (s137)
- Remlap-Ketona-Decatur-Co... (s15)
- Riverview-Minter-Leeper-C... Bend-Cahaba-Annemaine (s111)
- Rock outcrop-Pikeville-Hector (s224)
- Saffell-Bodine (s4)
- Savannah-Ora-Faceville (s121)
- Smithdale-Luverne (s110)
- Smithdale-Luverne-Flomaton (s114)
- Smithdale-Luverne-Flomato... (s129)
- Smithdale-Maubila-Luverne (s108)
- Smithdale-Okeelala-Luverne (s3895)
- Smithdale-Ora-Luverne-Bo... (s112)
- Smithdale-Ora-Luverne-Iuk... (s116)
- Smithdale-Ora-Mantachie-L... (s115)
- Smithdale-Pikeville-Palmer... (s107)
- Smithdale-Sacul-Ora-Luverne (s113)
- Smithdale-Saffell-Providence (s3886)
- Smithdale-Savannah-Luver... (s32)
- Smithdale-Savannah-Luver... (s147)
- Smithdale-Savannah-McLa... (s3878)
- Smithdale-Savannah-Myatt-... (s118)
- Smithdale-Savannah-Ochlo... (s31)
- Smithdale-Savannah-Ora (s30)
- Smithdale-Savannah-Rusto... (s3909)
- Smithdale-Shatta-Bama (s106)
- Stough-Savannah-Ochlock... (s33)
- Subran-Smithdale-Lucedale... (s139)
- Sumter-Rock outcrop-Oktibbeha-Kipling-... (s3926)
- Sweatman-Smithdale (s3840)
- Tallapoosa-Madison-Louisb... (s128)
- Tatum-Pacolet (s123)
- Tatum-Tallapoosa-Louisbur... (s76)
- Townley-Nauvoo-Montevallo (s60)
- Townley-Sunlight-Quitman-... (s100)
- Trinity-Sumter-Leeper (s119)
- Troup-Blanton (s159)
- Troup-Saffell-Luverne (s134)
- Uchee-Troup-Marvyn-Luver... (s126)
- Vaiden-Okolona-Kipling (s3933)
- Vaiden-Sumter-Oktibbeha (s140)
- Vaiden-Sumter-Oktibbeha-... (s117)
- Vaiden-Sumter-Sucarnooch... (s145)
- Vance-Pacolet-Louisburg-C... (s74)
- Water (s8369)
- Waynesboro-Tupelo-Locust... (s17)
- Weogufka-Tatum-Talladega (s122)
- Whitwell-Spadra (s29)
- Wilcox-Falkner (s3904)
- Wilcox-Mayhew (s146)



N

0 25 50 Miles

PROJECT

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ALABAMA DEPARTMENT
OF TRANSPORTATION

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FIGURE 2-3A
Soils of
the East Gulf Coastal Plain
of Alabama (North)

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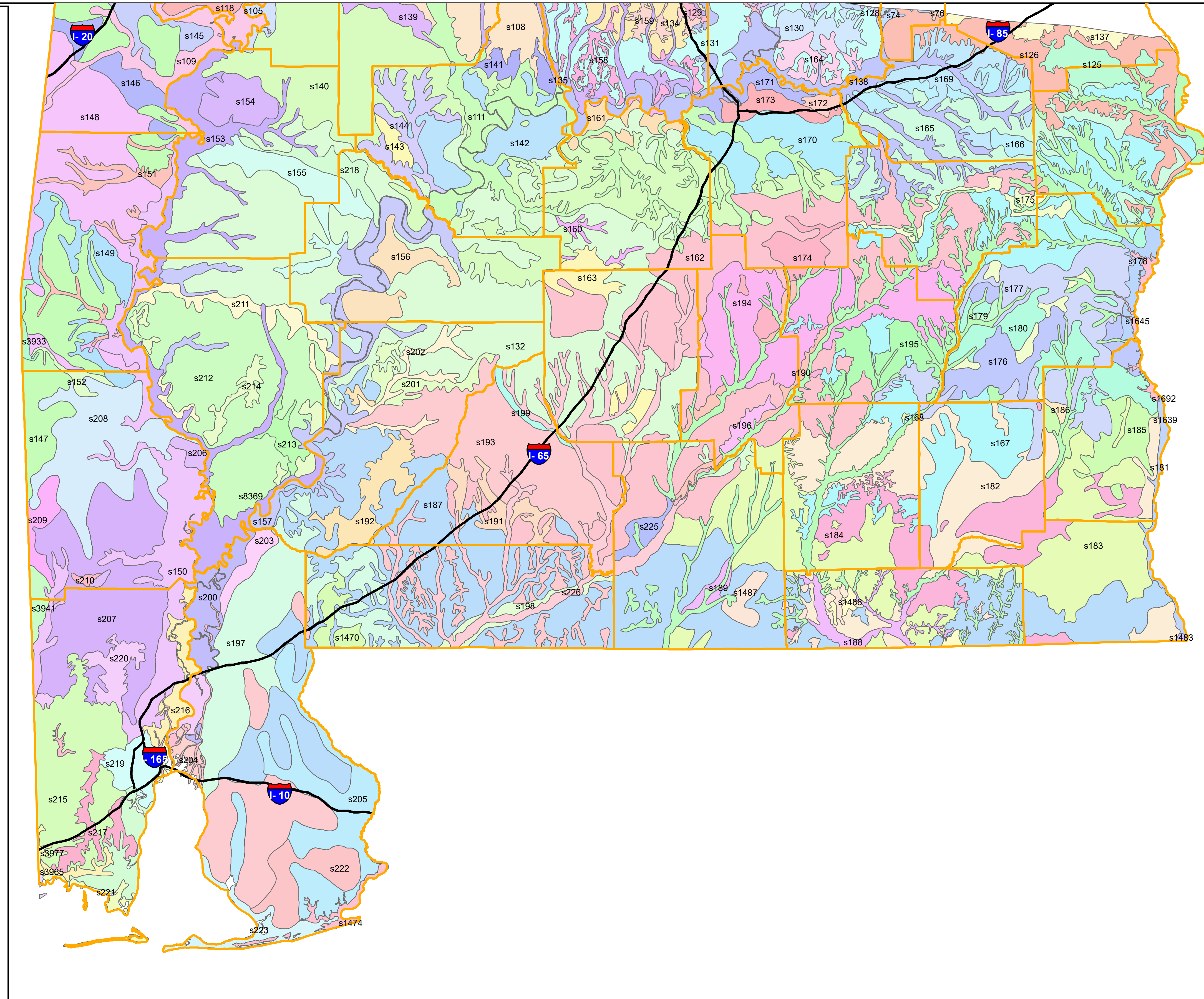


LEGEND

- Alabama County Boundary
- Fuquay-Arundel (s199)
- Fuquay-Florala-Dot... (s1487)
- Garcon-Eunola-Ca... (s1483)
- Handsboro (s3965)
- Harleston-Escambi... (s218)
- Kalmia-Craven-Cah... (s198)
- Kalmia-Eunola-Bigb... (s188)
- Lakeland-Kolomoki-... (s1639)
- Leaf-Cahaba-Anne... (s109)
- Levy-Johnston-Dor... (s216)
- Levy-Lafitte (s204)
- Levy-Leon-Lafitte-F... (s223)
- Luverne-Arundel (s201)
- Luverne-Cantuche-... (s211)
- Luverne-Halso-Con... (s132)
- Luverne-Lucedale-... (s158)
- Luverne-Lucedale-J... (s131)
- Malbis-Dothan-Cow... (s225)
- Malbis-Lucedale (s192)
- Malbis-Lucedale (s206)
- Malbis-Lucedale-Ba... (s156)
- Mantachie-Ellisville-... (s105)
- Mantachie-Lenoir-I... (s202)
- Mantachie-Lenoir-L... (s160)
- Marvyn-Luverne-Co... (s130)
- McLaurin-Heidel (s3941)
- McQueen-Cahaba-... (s172)
- McQueen-Houlka-B... (s161)
- McQueen-Mantachi... (s138)
- Myatt-Mantachie-Ki... (s135)
- Nankin (s1645)
- Ochlockonee-Megg... (s181)
- Orangeburg-Fuqua... (s183)
- Orangeburg-Luvern... (s162)
- Pacolet-Cecil (s137)
- Pamlico-Lafitte-Axis (s221)
- Pamlico-Leaf-Hyde-... (s3977)
- Pickney-Grady-Dor... (s189)
- Rains-Bonneau-Bet... (s143)
- Rains-Lucy-Fuquay... (s1488)
- Rains-Mantachie-Iu... (s168)
- Red Bay-Izagora-Iuka-C... (s173)
- Red Bay-Orangeburg-D... (s196)
- Red Bay-Orangeburg-Lu... (s186)
- Riverview-Minter-L... Bend-Cahaba-Anne... (s111)
- Roanoke-McQueen... (s171)
- Saucier-Notcher-M... (s217)
- Savannah-Mantach... (s151)
- Savannah-Quitman... (s142)
- Scranton-Poarch-P... (s222)
- Smithdale-Cantuch... (s149)
- Smithdale-Luverne-... (s129)
- Smithdale-Maubila-... (s108)
- Smithdale-Savanna... (s147)
- Smithdale-Savanna... (s118)
- Springhill-Cowarts (s194)
- Springhill-Orangeb... (s195)
- Subran-Smithdale-... (s139)
- Sumter-Oktibbeha-... (s165)
- Sumter-Prim-Oktibb... (s152)
- Sumter-Searcy-Okti... (s163)
- Tadlock-Mantachie-... (s141)
- Tallapoosa-Madiso... (s128)
- Tatum-Tallapoosa-L... (s76)
- Tifton-Plummer-Ora... (s1470)
- Troup-Blanton (s159)
- Troup-Dothan (s185)
- Troup-Dothan-Cow... (s166)
- Troup-Esto-Benndale (s220)
- Troup-Heidel-Bama (s215)
- Troup-Luverne (s176)
- Troup-Luverne (s177)
- Troup-Luverne-Bama (s164)
- Troup-Luverne-Con... (s167)
- Troup-Luverne-Cow... (s175)
- Troup-Luverne-Dot... (s169)
- Troup-Luverne-Esto... (s210)
- Troup-Maxton-Com... (s190)
- Troup-Orangeburg-... (s180)
- Troup-Orangeburg-... (s182)
- Troup-Rains-Plum... (s205)
- Troup-Red Bay-Orangeburg (s184)
- Troup-Red Bay-Orangeburg-Lu... (s179)
- Troup-Red Bay-Orangeburg-M... (s193)
- Troup-Saffell-Luverne (s134)
- Troup-Saffell-Orang... (s187)
- Troup-Smithdale-Bi... (s209)
- Troup-Smithdale-Lu... (s214)
- Troup-Smithdale-Lu... (s148)
- Troup-Smithdale-Lu... (s174)
- Troup-Smithdale-M... (s197)
- Troup-Smithdale-M... (s208)
- Troup-Smithdale-M... (s212)
- Troup-Smithton-Bibb (s207)
- Uchee-Troup-Cowa... (s125)
- Uchee-Troup-Marvy... (s126)
- Urban land-Smithton-John... (s219)
- Urbo-Levy-Iuka-Dor... (s200)
- Urbo-Mantachie-Iza... (s157)
- Urbo-Una (s153)
- Urbo-Una-Izagora-... (s150)
- Vaiden-Minter-Kipli... (s144)
- Vaiden-Okolona-Ki... (s3933)
- Vaiden-Sumter-Fac... (s154)
- Vaiden-Sumter-Lee... (s170)
- Vaiden-Sumter-Okti... (s140)
- Vaiden-Sumter-Suc... (s145)
- Vance-Pacolet-Loui... (s74)
- Wagram-Troup-Nor... (s1692)
- Wahee-Myatt-Leaf-... (s203)
- Wahee-Riverview-C... (s226)
- Water (s8369)
- Water-Urban land-Newhan-Corolla (s1474)
- Watsonia-Troup-S... (s213)
- Wilcox-Mayhew (s146)
- Wilcox-Vaiden-Luve... (s155)
- Yonges-Saffell-Ora... (s191)
- Yonges-Troup-Oran... (s178)

N

0 25 50 Miles

PROJECT

GEOTECHNICAL MANUAL
ALABAMA DEPARTMENT
OF TRANSPORTATION

TITLE

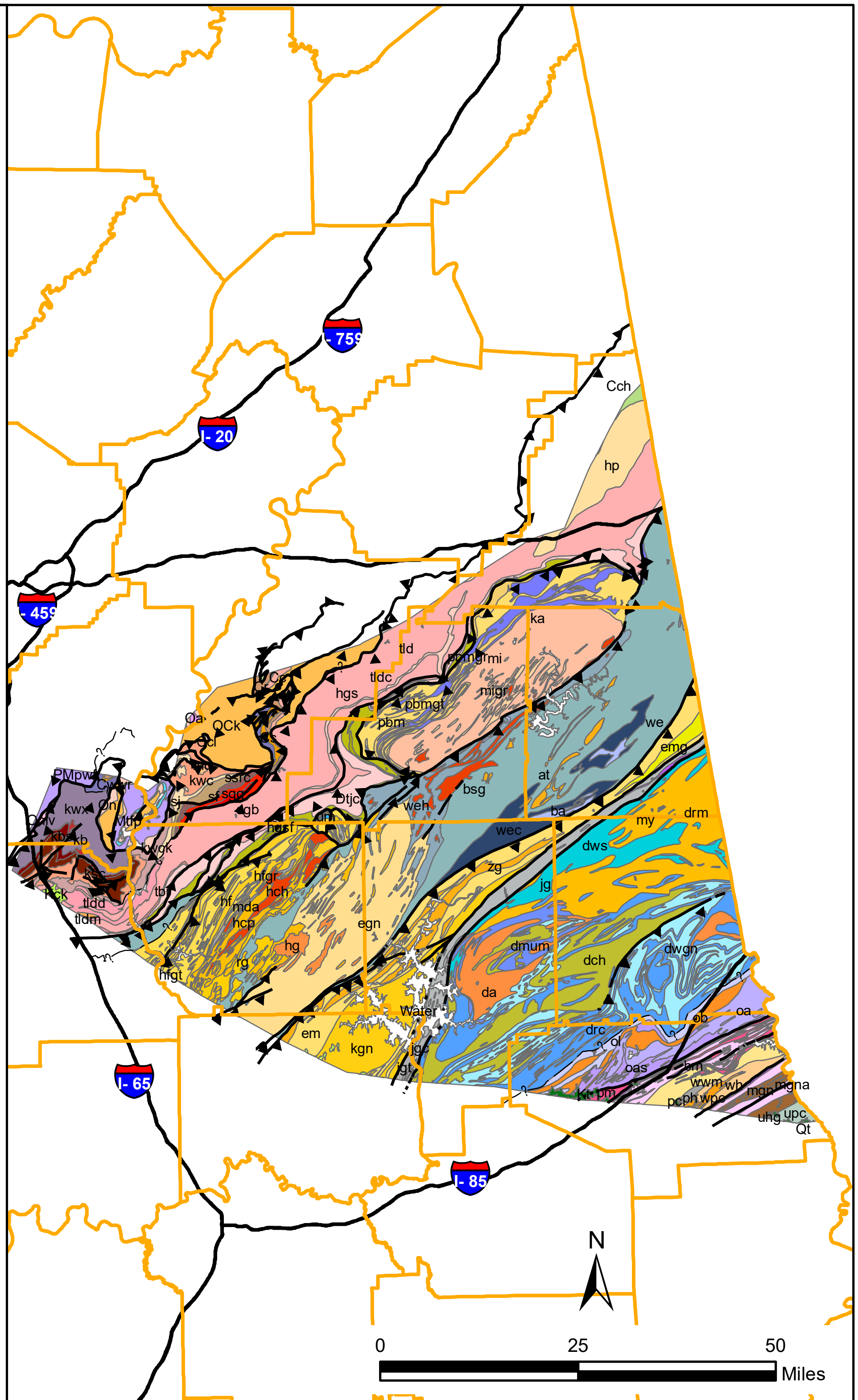
FIGURE 2-3B
Soils of
the East Gulf Coastal Plain
of Alabama (South)

DRAWN BY
MMK 8/28/20

REVIEWED BY
LN 8/28/20

LEGEND

- Alabama County Boundary
- Concealed Fault
- Fault (sense of movement unknown)
- Fault (sense of movement unknown) Inferred
- ? Nature of Contact Uncertain
- Normal Fault
- Normal Fault - Inferred
- Strike-Slip Fault
- Thrust Fault
- Thrust Fault - Inferred
- da Agricola Schist
- at Almond Trondhjemite
- Oa Athens Shale
- oa Auburn Gneiss
- oas Auburn Gneiss schist unit
- ba Beaverdam Amphibolite
- bm Blastomylonite and mylonite
- bsg Bluff Springs Granite
- ob Bottle Granite
- kb Brewer Phyllite
- tbr Butting Ram Sandstone
- dch Camp Hill Granite Gneiss
- tldc Cheaha Quartzite Member of the Lay Dam Formation
- pc Chewacla Marble
- Cch Chilhowee Group undifferentiated
- Kck Coker Formation
- Cc Conasauga Formation
- Ccl Conasauga Formation lower shale unit
- wec Cornhouse Schist
- egn Elkahatchee Quartz Diorite Gneiss
- em Emuckfaw Group undifferentiated in part
- sf Fayetteville Phyllite
- sgq Gantts Quarry Formation
- emg Glenloch Schist
- sgb Gooch Branch Chert
- weh Hackneyville Schist
- wh Halawaka Schist
- hch Hanover Schist
- hp Heflin Phyllite
- hf Higgins Ferry Group
- hfgt Higgins Ferry Group garnet quartzite unit
- hfgr Higgins Ferry Group graphitic unit
- Qt High terrace deposits
- hgs Hillabee Greenstone
- hgsf Hillabee Greenstone felsic unit
- hg Hissop Granite
- ph Hollis Quartzite
- uhg Hospilika Granite
- jg Jacksons Gap Group
- jgc Jacksons Gap Group sericite and chlorite phyllite unit
- Dtjc Jemison Chert and Chulafinnee Schist undifferentiated
- sj Jumbo Dolomite
- kwck Kalona Quartzite Member of the Wash Creek Slate
- ka Ketchepedrakee Amphibolite
- OCk Knox Group undifferentiated
- kgn Kowaliga Gneiss
- tld Lay Dam Formation
- tldd Lay Dam Formation diamictite unit
- Olo Little Oak Limestone
- ol Loachapoka Schist
- mi Mad Indian Group
- migr Mad Indian Group graphitic unit
- dmum Mafic and ultramafic rock
- pm Manchester Schist
- mu Metaclastic rocks of unknown affinity
- tldm Miller Mill Quartzite Member of Lay Dam Formation
- mda Mitchell Dam Amphibolite
- mgn Motts Gneiss
- mgna Motts Gneiss amphibolite unit
- my Mylonite and blastomylonite
- On Newala Limestone
- Onlv Newala and Longview Limestones undifferentiated
- PMpwf Parkwood Formation and Floyd Shale undifferentiated
- wpc Phelps Creek Gneiss
- upc Phenix City Gneiss
- hcp Pinchoulee Gneiss
- pbm Poe Bridge Mountain Group
- pbmgt Poe Bridge Mountain Group garnet quartzite unit
- pbmgr Poe Bridge Mountain Group graphitic unit
- drm Rock Mills Granite Gneiss
- rg Rockford Granite
- Cr Rome Formation
- drc Ropes Creek Amphibolite
- kbs Sawyer Limestone Member of the Brewer Phyllite
- Cs Shady Dolomite
- ssrc Shelvin Rock Church Formation
- ksc Stumps Creek Formation
- jgt Tallassee Metaquartzite
- Kt Tuscaloosa Group undifferentiated
- Mtfp Tuscumbia Limestone and Fort Payne Chert undifferentiated
- um Ultramafic rock
- dws Waresville Schist
- kwc Wash Creek Slate
- Water
- dwgn Waverly Gneiss
- kwx Waxahatchee Slate
- we Wedowee Group undifferentiated
- Cwwr Weisner and Wilson Ridge Formations undifferentiated
- wwm Whatley Mill Gneiss
- zg Zana Granite



TITLE

FIGURE 2-4
Geology of the Piedmont Upland of Alabama

PROJECT

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ALABAMA DEPARTMENT OF TRANSPORTATION

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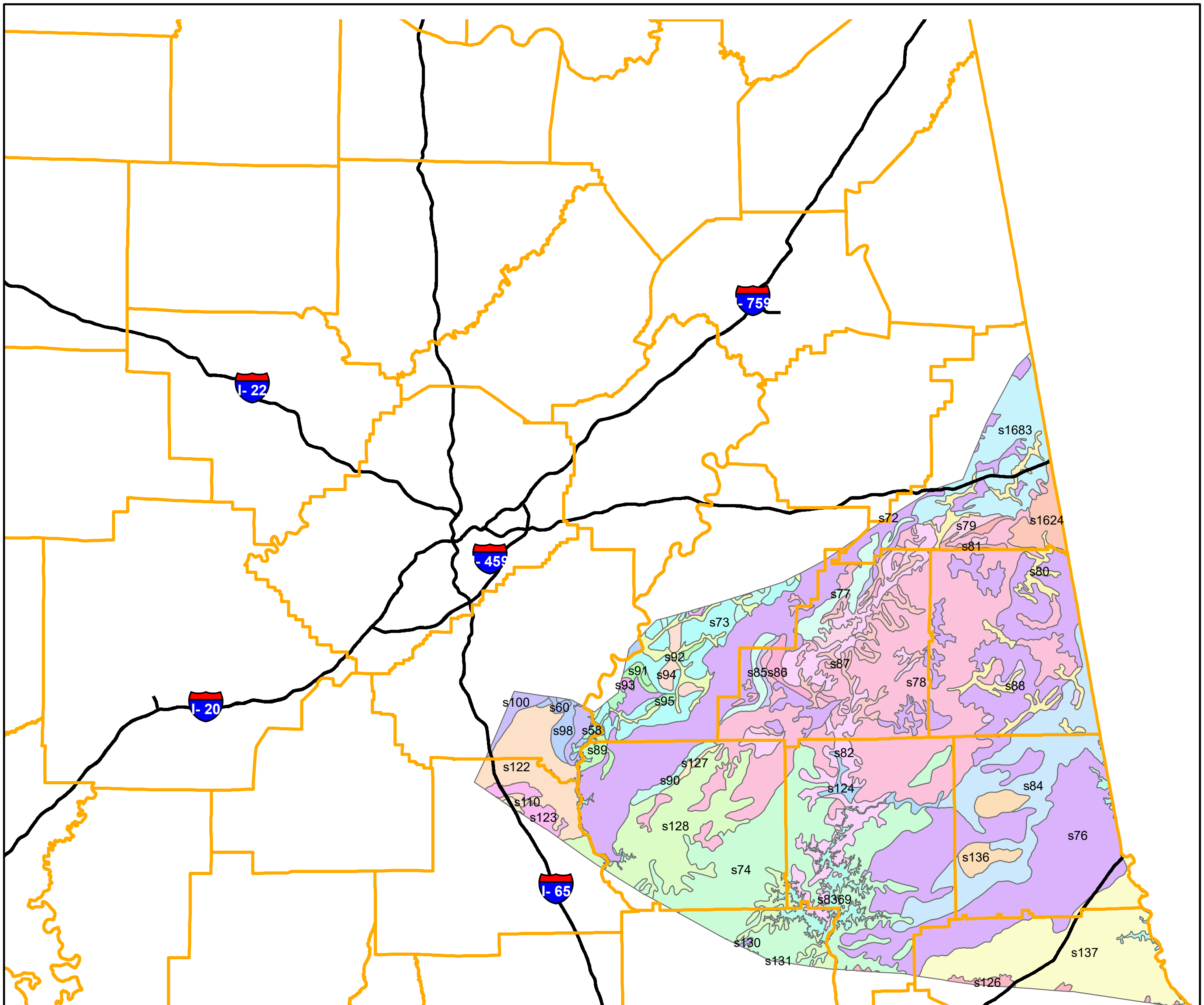
LN 8/28/20



LEGEND

- Alabama County Boundary
- Allen (s85)
- Allen (s91)
- Cecil-Appling (s136)
- Fullerton-Dewey-Decatur-Allen (s73)
- Hiwassee-Gwinnett (s81)
- Locust-Allen (s93)
- Luverne-Lucedale-Jones-Bos... (s131)
- Madison-Gwinnett-Cecil-Appl... (s84)
- Madison-Louisa-Davidson (s79)
- Marvyn-Luverne-Cowarts (s130)
- McQueen-Lobelville-Chewacl... (s92)
- Mecklenburg-Iredell-Bremo (s86)
- Minvale-Fullerton-Bodine (s58)
- Minvale-Locust-Bodine (s94)
- Nauvoo-Linker-Hartsells (s133)
- Pacolet-Cecil (s137)
- Pacolet-Madison-Davidson-C... (s1624)
- Quitman-Decatur-Allen (s98)
- Rock outcrop-Cheaha (s77)
- Smithdale-Luverne (s110)
- State-Roanoke-Riverview-Ch... (s87)
- Tallapoosa-Madison-Louisbur... (s82)
- Tallapoosa-Madison-Louisbur... (s128)
- Tatum-Madison-Louisa (s78)
- Tatum-Pacolet (s123)
- Tatum-Tallapoosa (s95)
- Tatum-Tallapoosa-Louisburg-... (s76)
- Tatum-Tallapoosa-State-Rive... (s1683)
- Toccoa-Riverview-Cartecay (s80)
- Townley-Nauvoo-Montevallo (s60)
- Townley-Sunlight-Quitman-N... (s100)
- Townley-Talladega-Sunlight-... (s89)
- Uchee-Troup-Marvyn-Luvern... (s126)
- Vance-Pacolet-Louisburg-Ce... (s74)
- Water (s8369)
- Wehadkee-Leesburg-Ketona-... (s72)
- Weogufka-Tatum-Iredell-Durh... (s90)
- Weogufka-Tatum-Madison-G... (s127)
- Weogufka-Tatum-Talladega (s122)
- Wickham-Roanoke-Congare... (s124)
- Wilkes-Wehadkee-Louisa-Da... (s88)

N

0 25 50 Miles

TITLE

FIGURE 2-5
Soils of the Piedmont Upland of Alabama

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ALABAMA DEPARTMENT OF TRANSPORTATION

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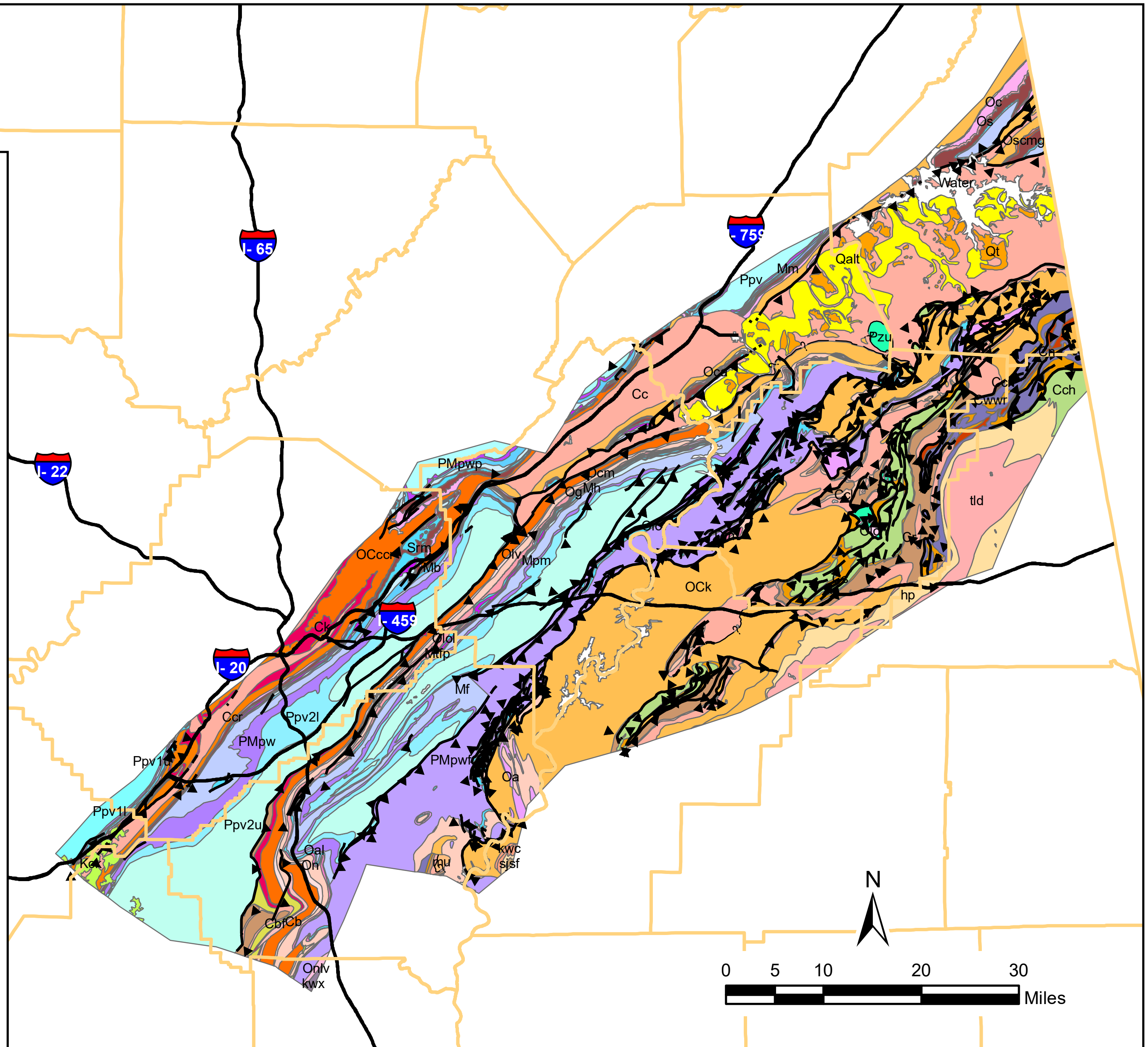
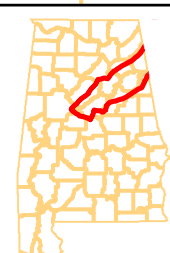


	Alabama County Boundary
	Concealed Fault
	Fault (sense of movement unknown)
?	Nature of Contact Uncertain
	Normal Fault
	Strike-Slip Fault
	Thrust Fault
	Thrust Fault - Inferred
Qalt	Alluvial, coastal, and low terrace deposits
Oa	Athens Shale
Oal	Athens Shale and Lenoir Limestone undifferentiated
Oca	Attalla Chert Conglomerate Member of the Chickamauga Limestone
Mb	Bangor Limestone
Cb	Bibb Dolomite
Cbf	Brierfield Dolomite
OCccr	Chepultepec and Copper Ridge Dolomites undifferentiated
Oc	Chickamauga Limestone
Cch	Chilhowee Group undiffentiated
Ccn	Cochran Formation
Kck	Coker Formation
Ocm	Colvin Mountain Sandstone
Cc	Conasauga Formation
Ccl	Conasauga Formation lower shale unit
Ccr	Copper Ridge Dolomite
sf	Fayetteville Phyllite
Mf	Floyd Shale
Dfm	Frog Mountain Sandstone
Og	Greensport Formation
Mh	Hartselle Sandstone
hp	Heflin Phyllite
Qt	High terrace deposits
sj	Jumbo Dolomite
Ck	Ketona Dolomite
OCk	Knox Group undifferentiated
tld	Lay Dam Formation
Olo	Little Oak Limestone
Olol	Little Oak and Lenoir Limestones undifferentiated
Olon	Little Oak and Newala Limestones undifferentiated
Olv	Longview Limestone
mu	Metaclastic rocks of unknown affinity
Mm	Monteagle Limestone
On	Newala Limestone
Onlv	Newala and Longview Limestones undifferentiated
Cn	Nichols Formation
Pzu	Paleozoic shale undifferentiated
PMpw	Parkwood Formation
PMpwf	Parkwood Formation and Floyd Shale undifferentiated
PMpwp	Parkwood and Pennington Formations undifferentiated
Ppv	Pottsville Formation
Ppv1l	Pottsville Formation (lower part), Appalachian Plateaus
Ppv2l	Pottsville Formation (lower part), Valley and Ridge
Ppv1u	Pottsville Formation (upper part), Appalachian Plateaus
Ppv2u	Pottsville Formation (upper part), Valley and Ridge
Mpm	Pride Mountain Formation
Srm	Red Mountain Formation
Cr	Rome Formation
Os	Sequatchie Formation
Oscmg	Sequatchie Formation, Colvin Mt. Sandstone, Greensport Formation undif.
Cs	Shady Dolomite
Mtfp	Tuscumbia Limestone and Fort Payne Chert undifferentiated
kwc	Wash Creek Slate
	Water
kwx	Waxahatchee Slate
Cwwr	Weisner and Wilson Ridge Formations undifferentiated



PROJECT

GEOTECHNICAL MANUAL
ALABAMA DEPARTMENT OF TRANSPORTATION

TITLE

FIGURE 2-6
Geology of the Alabama Valley and Ridge

DRAWN BY
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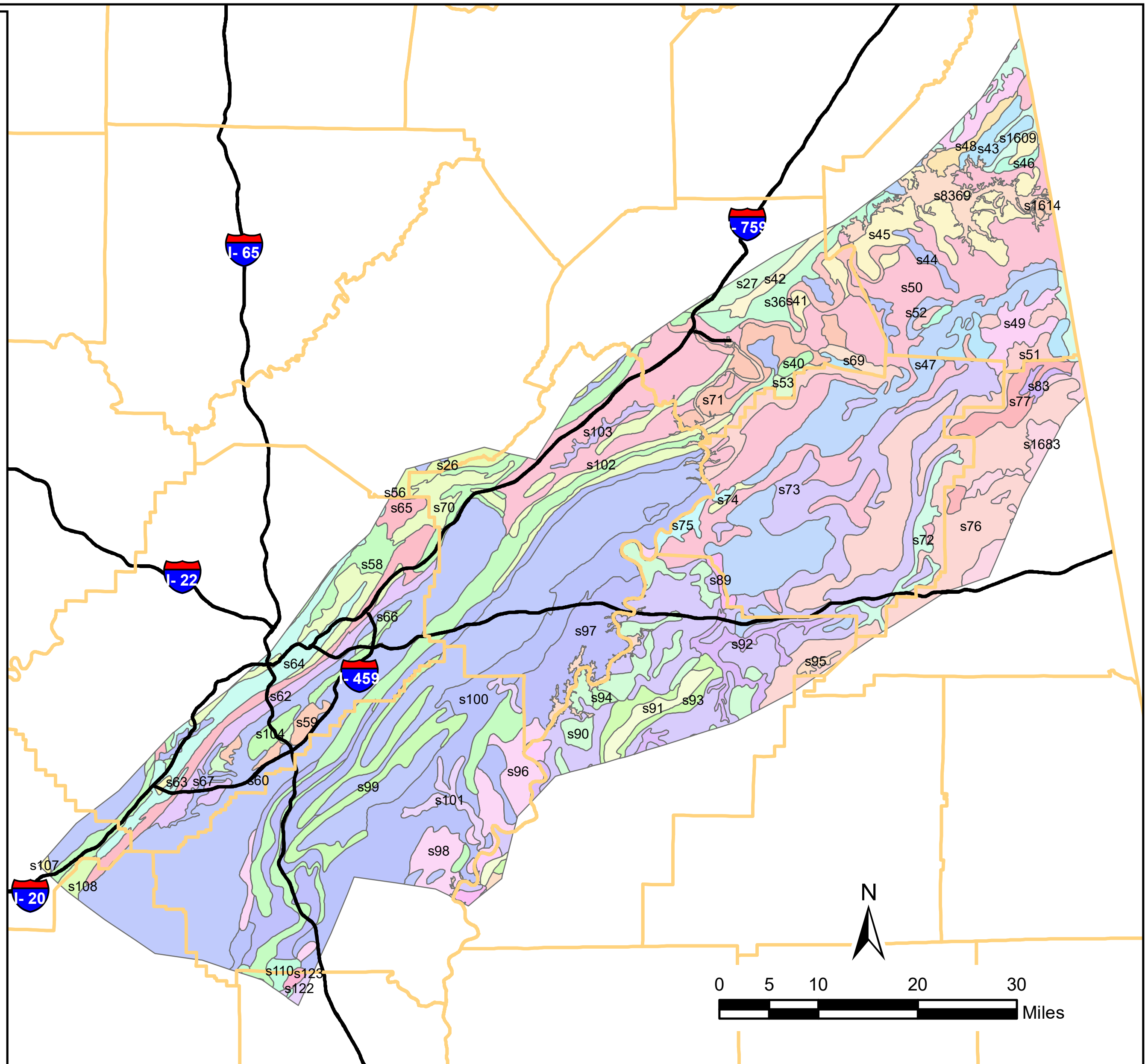
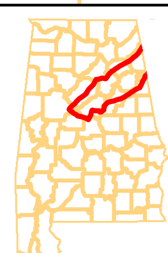
LN 8/28/20

LEGEND

- Alabama County Boundary
- Allen (s91)
- Dewey-Allen (s46)
- Fullerton-Bodine-Birmingham (s65)
- Fullerton-Dewey-Decatur-Allen (s73)
- Gaylesville-Ellisville-Cloudland-... (s44)
- Holston-Cane-Allen (s96)
- Holston-Cloudland-Cedarbluff (s71)
- Leesburg-Allen (s51)
- Leesburg-Gorgas-Allen (s66)
- Leesburg-Holston-Cloudland (s45)
- Locust-Allen (s93)
- McQueen-Lobelville-Chewacla-... (s92)
- Minvale-Bodine (s102)
- Minvale-Dewey-Bodine (s49)
- Minvale-Etowah-Dewey (s97)
- Minvale-Fullerton-Bodine (s40)
- Minvale-Fullerton-Bodine (s47)
- Minvale-Fullerton-Bodine (s58)
- Minvale-Fullerton-Decatur-Bodine (s36)
- Minvale-Locust-Bodine (s94)
- Montevallo-Firestone-Conasauga (s50)
- Montevallo-Firestone-Conasaug... (s52)
- Montevallo-Leesburg-Firestone (s48)
- Nella-Minvale-Fullerton-Dewey-... (s70)
- Quitman-Decatur-Allen (s98)
- Rock outcrop-Cheaha (s77)
- Rock outcrop-Hartsells-Gorgas-Allen (s56)
- Sequatchie-Holston (s75)
- Shack-Minvale-Fullerton-Bodine (s1609)
- Smithdale-Luverne (s110)
- Smithdale-Maubila-Luverne (s108)
- Smithdale-Pikeville-Palmerdale-... (s107)
- Stemley-Ennis-Chewacla (s43)
- Sterrett-Quitman-Choccolocco (s101)
- Sullivan-Etowah-Decatur (s63)
- Sullivan-State-Ketona-Decatur-B... (s67)
- Tallapoosa-Sylacauga-State-Riv... (s83)
- Tatum-Pacolet (s123)
- Tatum-Tallapoosa (s95)
- Tatum-Tallapoosa-Louisburg-Gw... (s76)
- Tatum-Tallapoosa-State-Rivervie... (s1683)
- Townley-Nauvoo-Hartsells-Gorgas (s27)
- Townley-Nauvoo-Montevallo (s60)
- Townley-Nauvoo-Montevallo-Alb... (s59)
- Townley-Nella-Hartsells-Gorgas (s26)
- Townley-Nella-Mountainburg-Ha... (s99)
- Townley-Rock outcrop-Hartsells-Gorgas (s42)
- Townley-Rock outcrop-Nella-Hartsells-Allen (s53)
- Townley-Stemley-Minvale-Bodine (s69)
- Townley-Sunlight-Quitman-Nauv... (s100)
- Townley-Talladega-Sunlight-Nect... (s89)
- Urban land-Nauvoo-Gorgas-Allen (s104)
- Urban land-Townley-Holston-Albertville (s62)
- Urban land-Tupelo-Decatur (s64)
- Vance-Pacolet-Louisburg-Cecil-... (s74)
- Water (s8369)
- Wax-Tanyard-Mooreville-Gayles... (s103)
- Wehadkee-Leesburg-Ketona-Ge... (s72)
- Weogufka-Tatum-Iredell-Durham... (s90)
- Weogufka-Tatum-Talladega (s122)
- Whitwell-Etowah-Chewacla (s1614)
- Wickham-McQueen-Choccolocc... (s41)





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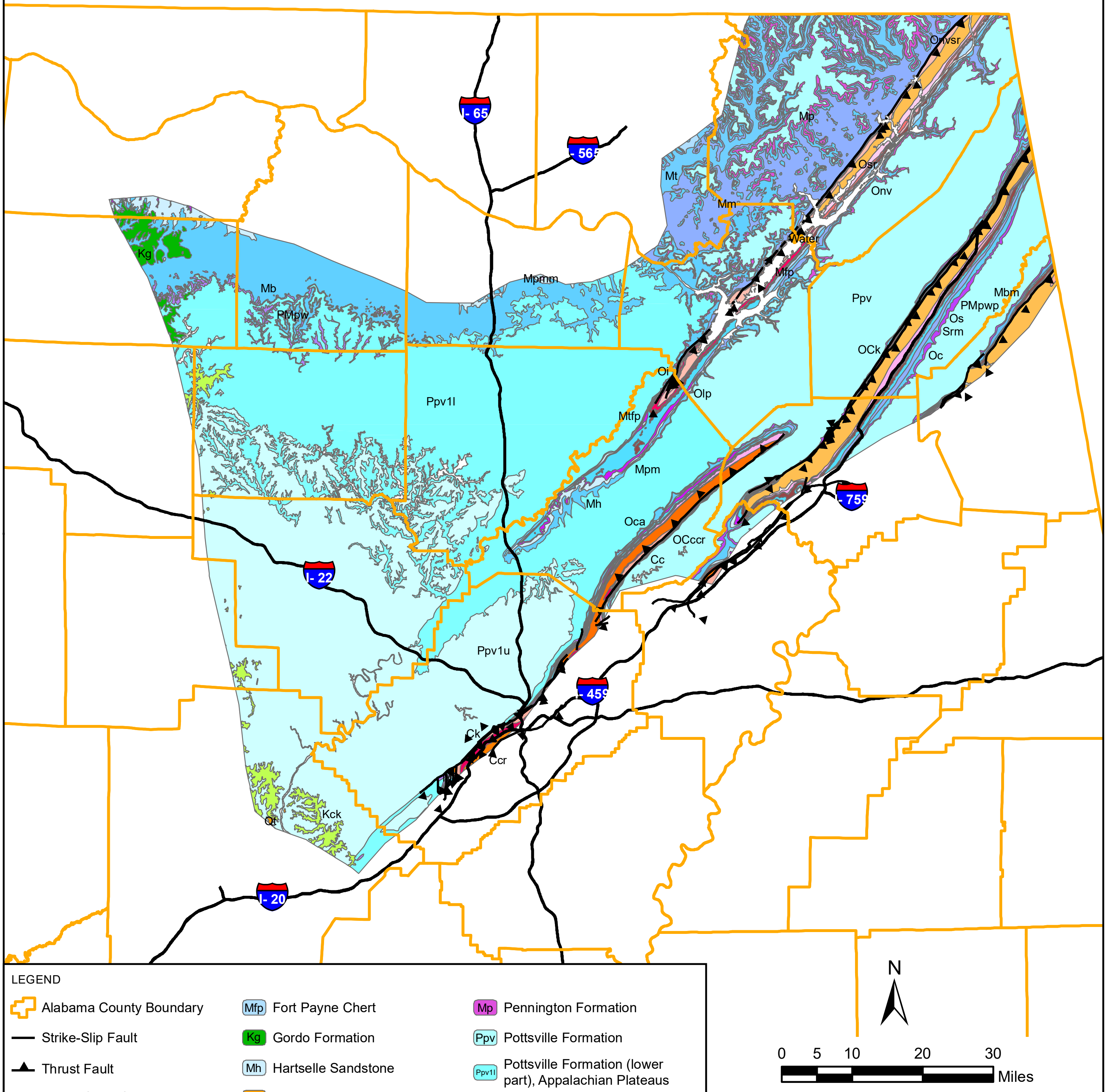
TITLE
FIGURE 2-7
Soils of the Alabama Valley and Ridge



LEGEND

	Alabama County Boundary
	Strike-Slip Fault
	Thrust Fault
Oca	Attalla Chert Conglomerate Member of the Chickamauga Limestone
Mb	Bangor Limestone
Mbm	Bangor and Monteagle Limestones undifferentiated
OCccr	Chepultepec and Copper Ridge Dolomites undifferentiated
Oc	Chickamauga Limestone
Kck	Coker Formation
Cc	Conasauga Formation
Ccr	Copper Ridge Dolomite
Mf	Floyd Shale
Mfp	Fort Payne Chert
Kg	Gordo Formation
Mh	Hartselle Sandstone
Qt	High terrace deposits
Oi	Inman Formation
Ck	Ketona Dolomite
OCk	Knox Group undifferentiated
Olp	Leipers Limestone
Mm	Monteagle Limestone
Onv	Nashville Group
Onvsr	Nashville and Stones River Groups undifferentiated
PMpw	Parkwood Formation
PMpwp	Parkwood and Pennington Formations undifferentiated
Mp	Pennington Formation
Ppv	Pottsville Formation
Ppv1l	Pottsville Formation (lower part), Appalachian Plateaus
Ppv1u	Pottsville Formation (upper part), Appalachian Plateaus
Mpm	Pride Mountain Formation
Mpmm	Pride Mountain Formation and Monteagle Limestone undifferentiated
Srm	Red Mountain Formation
Os	Sequatchie Formation
Osr	Stones River Group
Mt	Tuscumbia Limestone
Mtfp	Tuscumbia Limestone and Fort Payne Chert undifferentiated
	Water

TITLE

FIGURE 2-8
Geology of the Cumberland Plateau of Alabama

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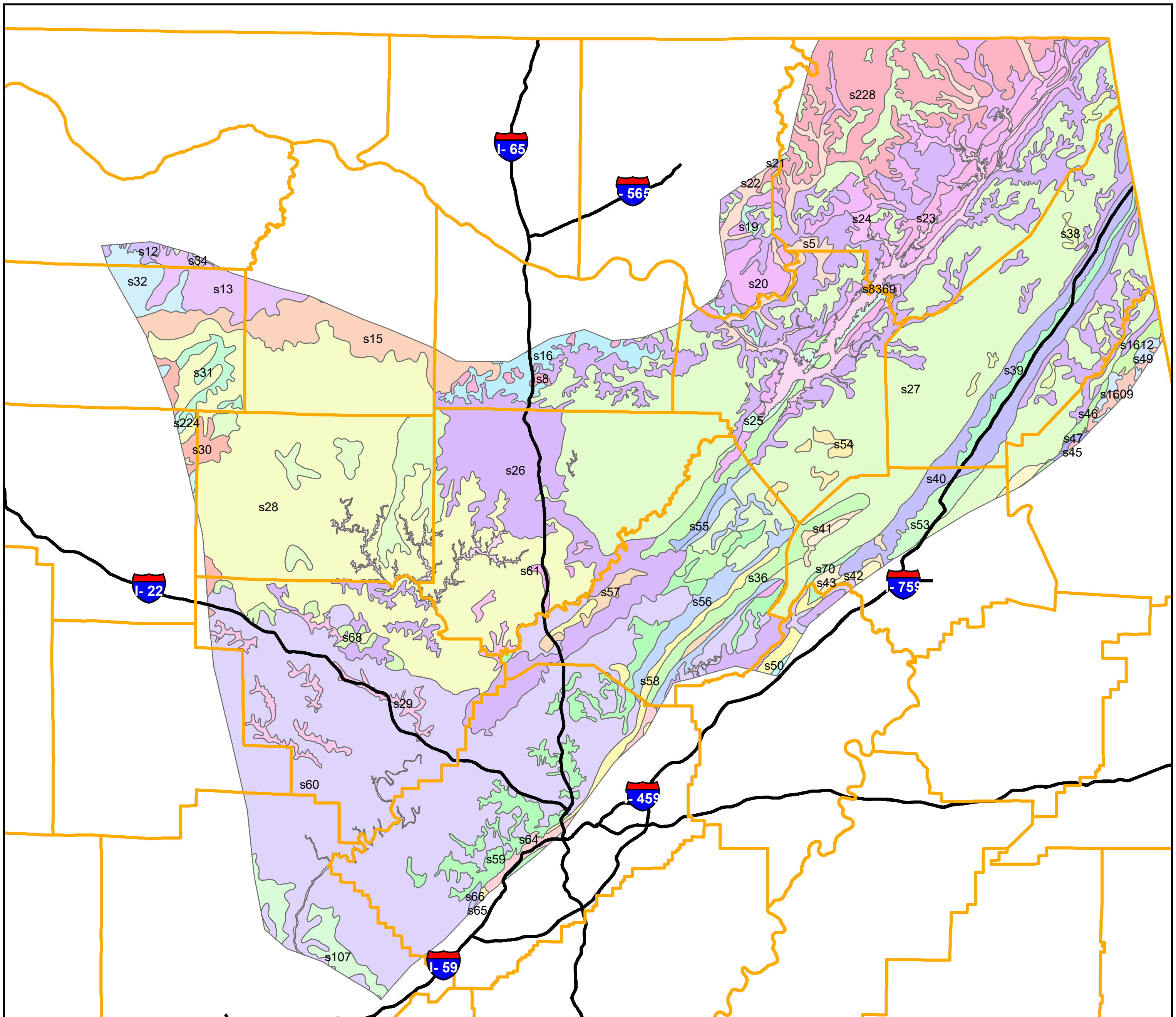

- Alabama County Boundary
- Dewey-Allen (s46)
- Etowah-Emory-Decatur (s8)
- Fullerton-Bodine-Birmingh... (s65)
- Fullerton-Decatur-Colbert-... (s21)
- Ketona-Colbert-Chisca-C... (s13)
- Leesburg-Gorgas-Allen (s66)
- Leesburg-Holston-Cloudla... (s45)
- Lobelville-Etowah-Ennis-E... (s5)
- Minvale-Dewey-Bodine (s49)
- Minvale-Fullerton-Bodine (s23)
- Minvale-Fullerton-Bodine (s40)
- Minvale-Fullerton-Bodine (s47)
- Minvale-Fullerton-Bodine (s58)
- Minvale-Fullerton-Decatur... (s36)
- Minvale-Lobelville-Fullerton (s55)
- Montevallo-Firestone-Con... (s50)
- Nauvoo-Gorgas (s28)
- Nella-Minvale-Fullerton-D... (s70)
- Nella-Nectar-Chisca (s12)
- Palmerdale (s61)
- Pruitton-Fullerton-Decatur... (s22)
- Remlap-Ketona-Decatur-... (s15)
- Roanoke-Ketona-Hollywo... (s24)
- Rock outcrop-Hartsells-Gorgas-... (s56)
- Rock outcrop-Pikeville-Hector (s224)
- Rock outcrop-Remlap-Barfield (s57)
- Shack-Minvale-Fullerton-... (s1609)
- Shack-Minvale-Lyerly-Col... (s39)
- Smithdale-Pikeville-Palme... (s107)
- Smithdale-Savannah-Luv... (s32)
- Smithdale-Savannah-Ochl... (s31)
- Smithdale-Savannah-Ora (s30)
- Stemley-Ennis-Chewacla (s43)
- Talbott-Rock outcrop-Bouldin (s228)
- Townley-Fullerton (s1612)
- Townley-Montevallo-Harts... (s38)
- Townley-Nauvoo-Hartsells... (s27)
- Townley-Nauvoo-Monteva... (s60)
- Townley-Nauvoo-Monteva... (s59)
- Townley-Nella-Hartsells-G... (s26)
- Townley-Rock outcrop-Hartsells-Gorgas (s42)
- Townley-Rock outcrop-Nella-Hartsells-All... (s53)
- Tupelo-Emory-Decatur-Ch... (s19)
- Tupelo-Holston-Etowah-D... (s20)
- Tupelo-Leadvale-Decatur-... (s25)
- Urban land-Tupelo-Decatur (s64)
- Water (s8369)
- Whitwell-Spadra (s29)
- Wickham-McQueen-Choc... (s41)
- Wynnville-Hartsells-Albert... (s54)
- Wynnville-Linker-Gorgas (s34)
- Wynnville-Townley-Sipsey (s68)
- Wynnville-Tupelo-Hollywo... (s16)

TITLE

FIGURE 2-9
Soils of the Cumberland Plateau of Alabama

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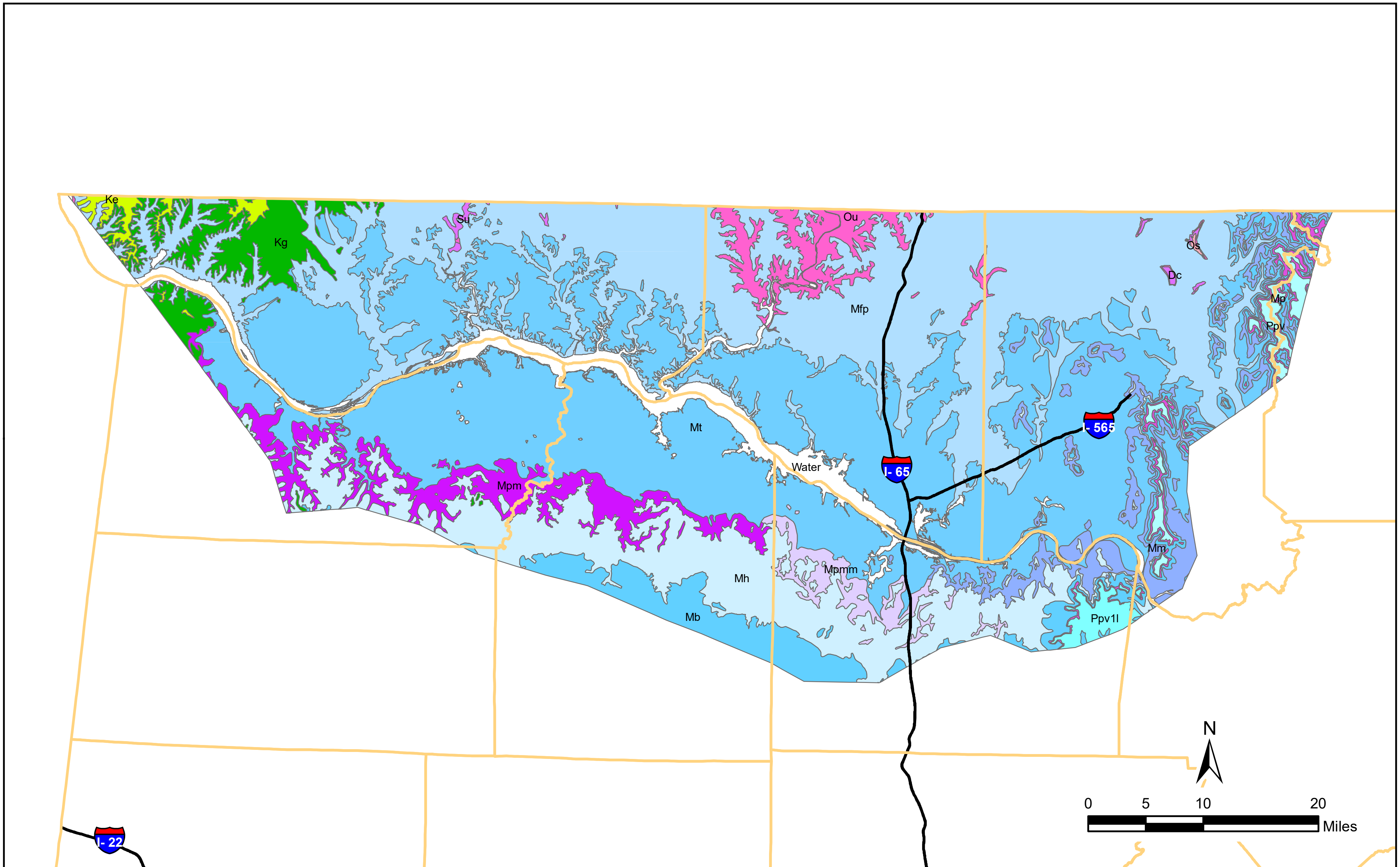

Alabama County Boundary	Kg Gordo Formation	Ppv Pottsville Formation	Su Silurian System undifferentiated
Mb Bangor Limestone	Mh Hartselle Sandstone	Ppv1l Pottsville Formation (lower part), Appalachian Plateaus	Mt Tuscumbia Limestone
Dc Chattanooga Shale	Mm Monteagle Limestone	Mpm Pride Mountain Formation	Water
Ke Eutaw Formation	Ou Ordovician System undifferentiated	Mpmm Pride Mountain Formation and Monteagle Limestone undifferentiated	
Mfp Fort Payne Chert	Mp Pennington Formation	Os Sequatchie Formation	

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TITLE

FIGURE 2-10
Geology of the Highland Rim of Alabama

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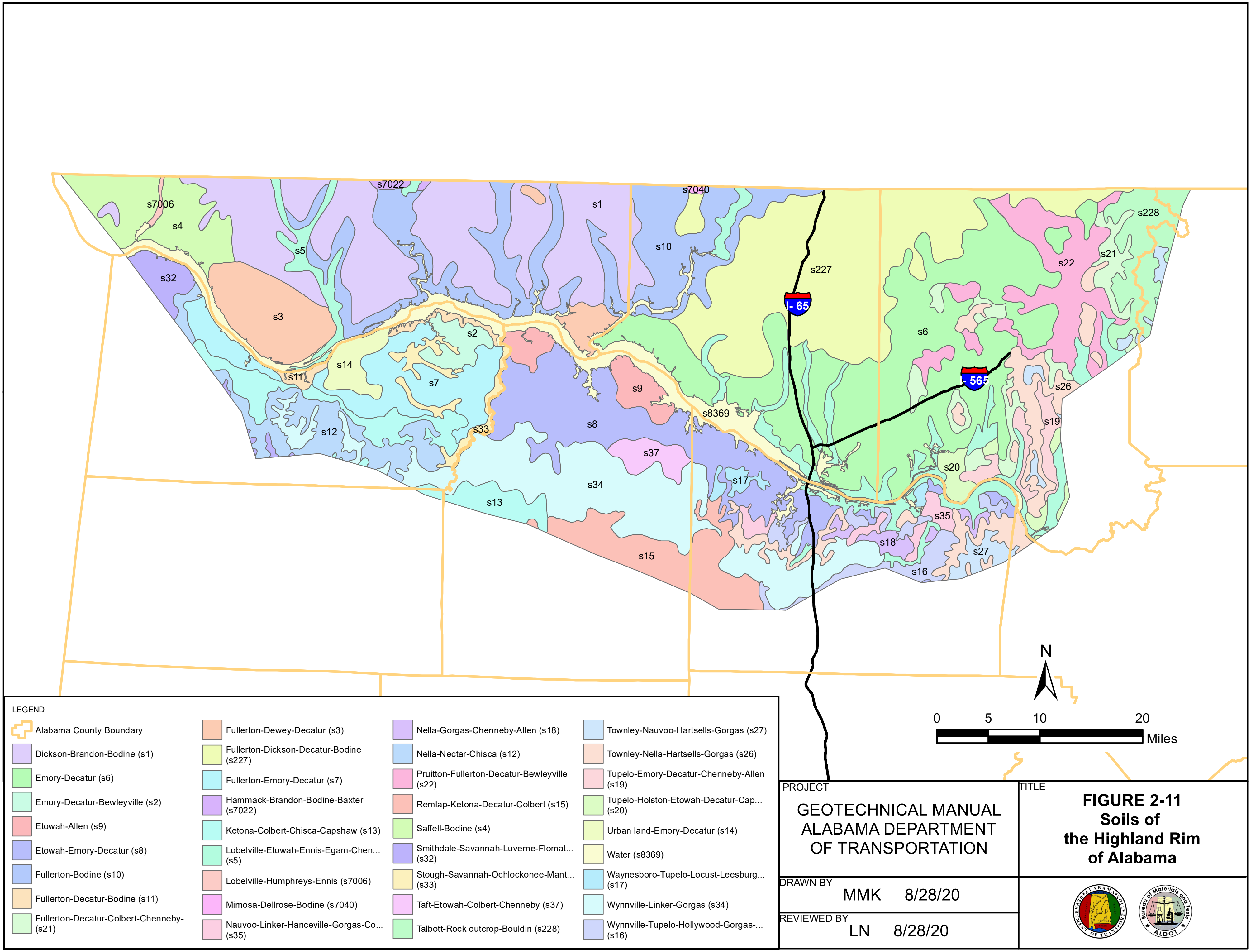

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TITLE

FIGURE 2-11
Soils of
the Highland Rim
of Alabama

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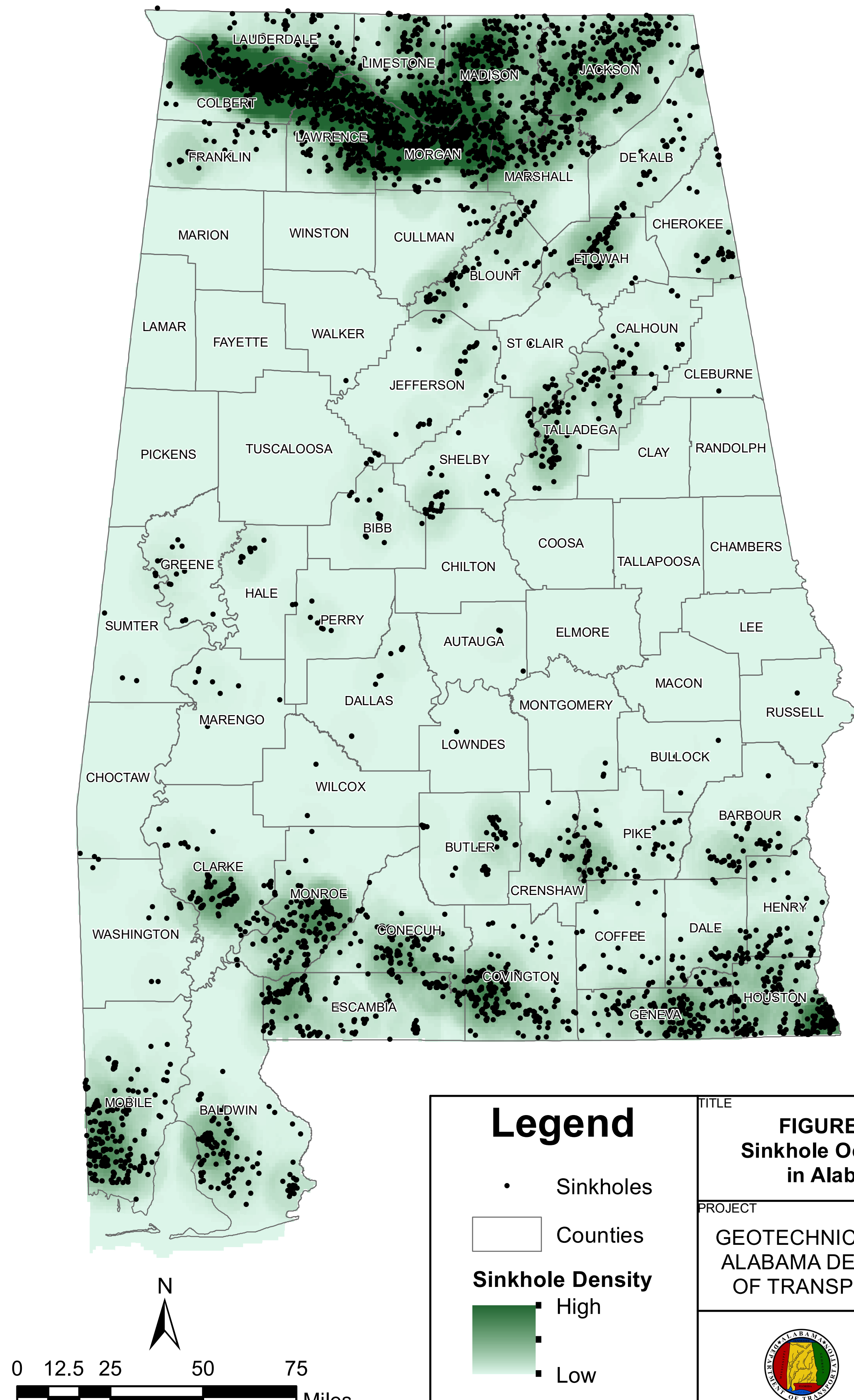


Legend

- Sinkholes
- Counties

Sinkhole Density

High

Low

TITLE

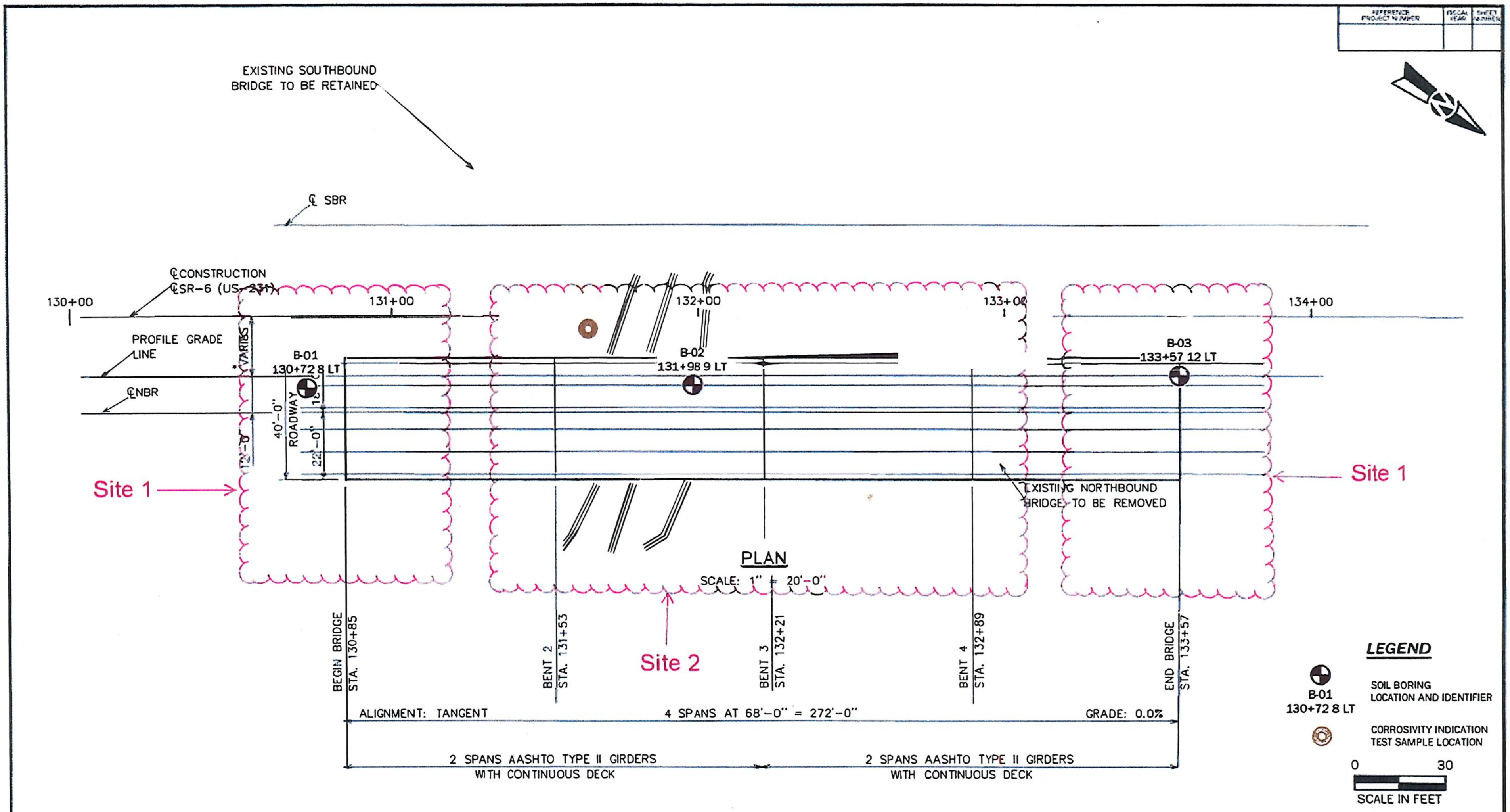
FIGURE 2-12
Sinkhole Occurence in Alabama

PROJECT

GEOTECHNICAL MANUAL
ALABAMA DEPARTMENT OF TRANSPORTATION

Appendix C

Example Figures for Delineation of Geotechnical Sites and Subsurface Stratigraphy



NOTE: BORING OFFSETS ARE FROM BRIDGE CENTERLINE, NOT THE PROJECT CENTERLINE.

DRAWING SOURCE:
AS PROVIDED BY ALABAMA DEPARTMENT OF TRANSPORTATION, UNDATED.

ALABAMA DEPARTMENT OF TRANSPORTATION

APPROVED BY

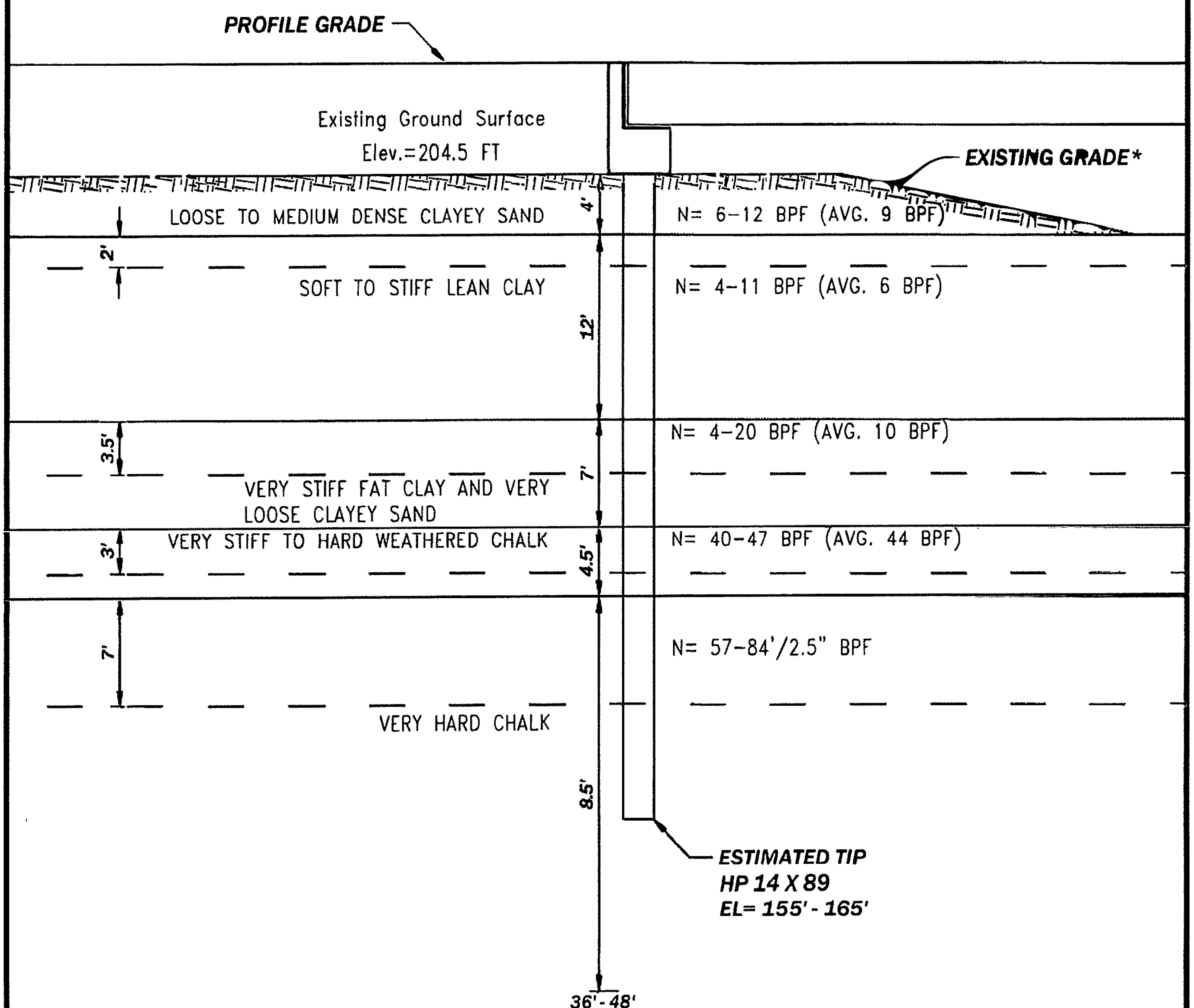
DATE

BORING LOCATION SCHEMATIC
SHEET PLN - 01 OF 01

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

SITE 1

ABUTMENTS 1 & 5



* Existing Grade generally matches Proposed Grade

NOT TO SCALE

ALABAMA DEPARTMENT OF TRANSPORTATION

Station 130+85

APPROVED BY:

DATE:

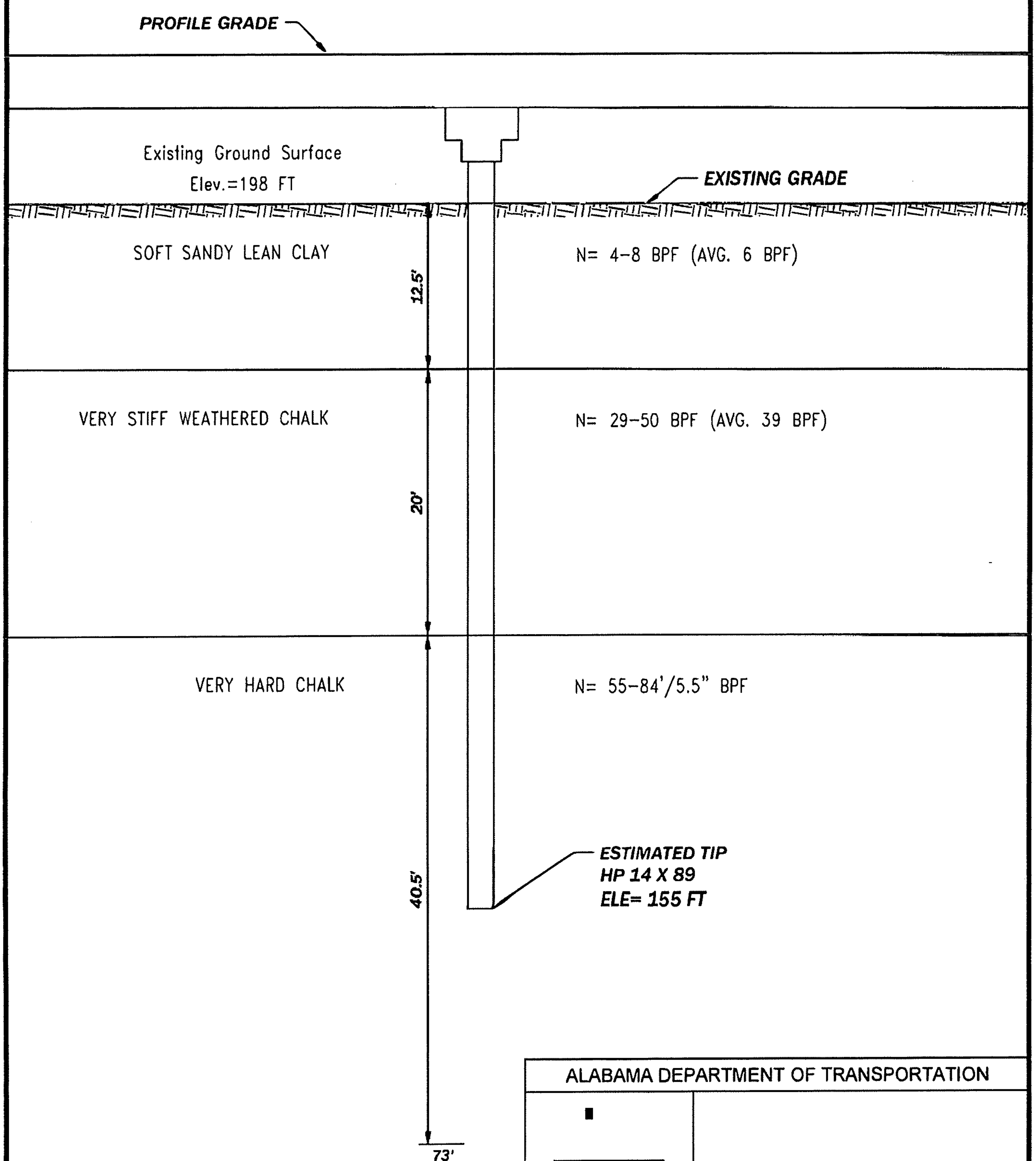
TYPICAL SOIL PROFILE

SHEET PRF - 01 OF 02

REFERENCE PROJECT NUMBER	FISCAL YEAR	SHEET NUMBER

SITE 2

BENTS 2, 3 &4



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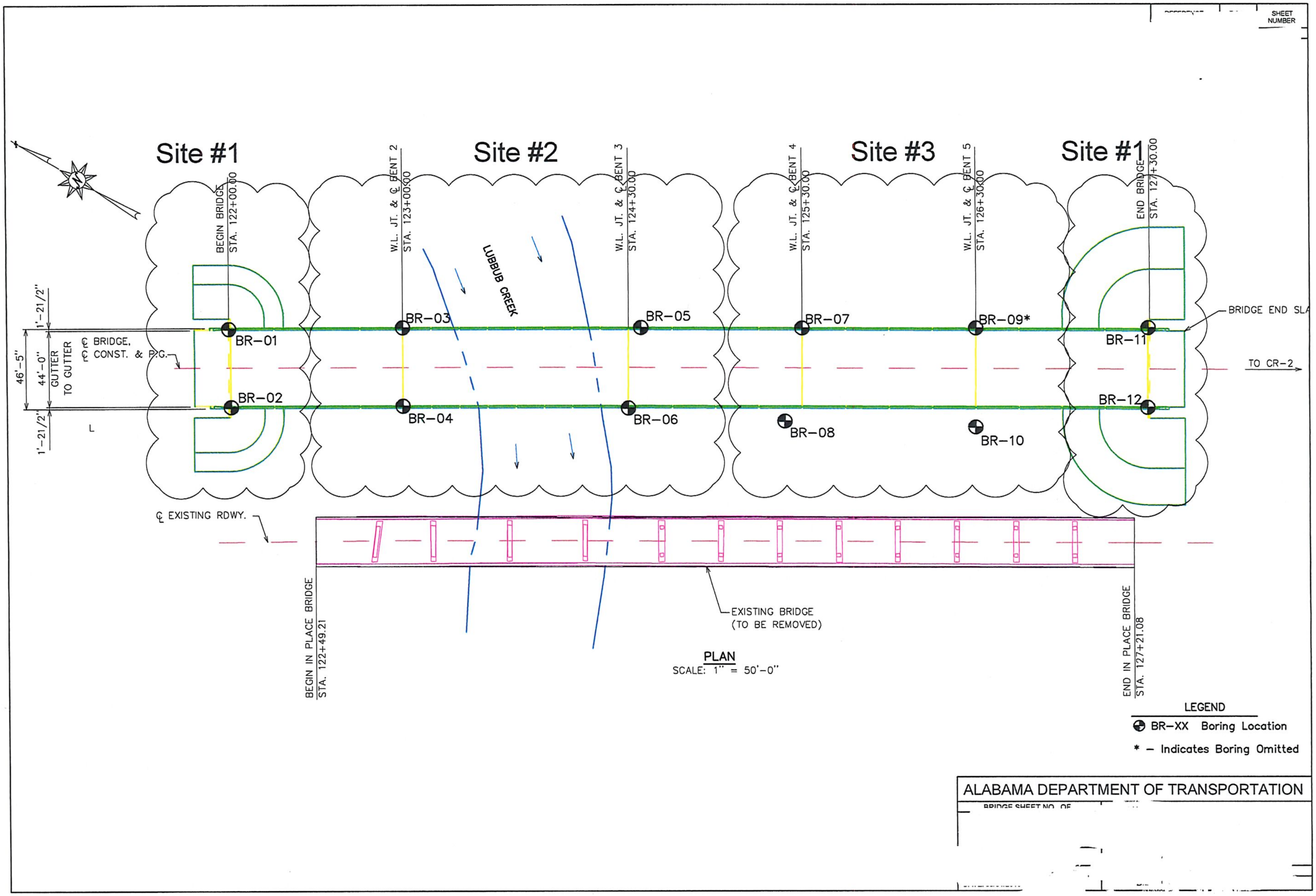
DATE:

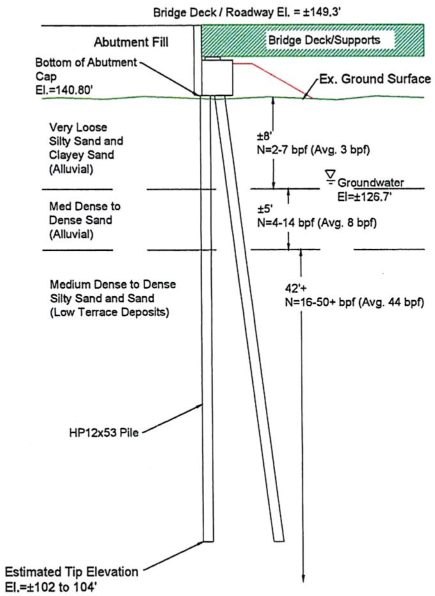
TYPICAL SOIL PROFILE

SHEET PRF - 02 OF 02

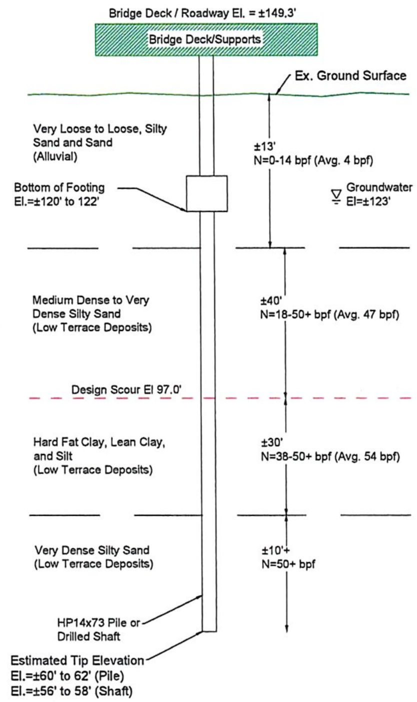
NOT TO SCALE



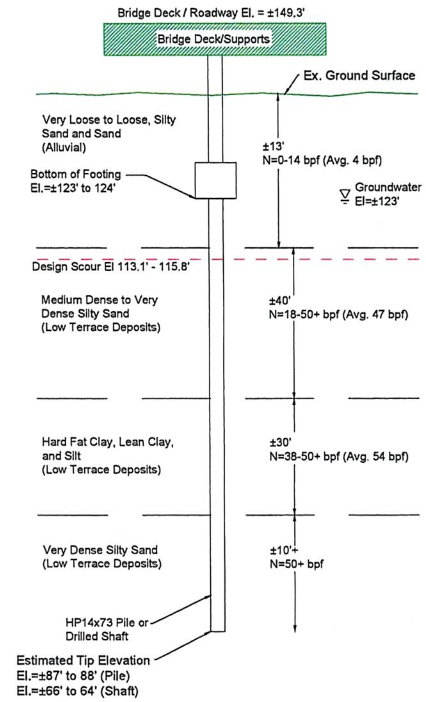
SHEET NUMBER
Site #1
Site #2
Site #3
Site #1
BEGIN BRIDGE
STA. 122+00.00
W.L. JT. & ℄ BENT 2
STA. 123+00.00
W.L. JT. & ℄ BENT 3
STA. 124+30.00
W.L. JT. & ℄ BENT 4
STA. 125+30.00
W.L. JT. & ℄ BENT 5
STA. 126+30.00
END BRIDGE
STA. 127+30.00
LUBBUB CREEK
BR-01
BR-02
BR-03
BR-04
BR-05
BR-06
BR-07
BR-08
BR-09*
BR-10
BR-11
BR-12
BRIDGE END SLA
TO CR-2
℄ BRIDGE,
℄ CONST. & P.G.
46'-5"
44'-0"
GUTTER TO GUTTER
1'-2 1/2"
1'-2 1/2"
℄ EXISTING RDWY.
EXISTING BRIDGE
(TO BE REMOVED)
BEGIN IN PLACE BRIDGE
STA. 122+49.21
END IN PLACE BRIDGE
STA. 127+21.08
PLAN
SCALE: 1" = 50'-0"
LEGEND
BR-XX Boring Location
* – Indicates Boring Omitted
ALABAMA DEPARTMENT OF TRANSPORTATION



Site #1



Site #2



Site #3

ALABAMA DEPARTMENT OF TRANSPORTATION

BRIDGE SHEET NO. OF

APPROVED

DATE

Appendix D

Testing Procedures for Topsoil Analysis

ALABAMA
DEPARTMENT OF TRANSPORTATION

Testing Procedures for Topsoil Analysis

5/12/09

Determination of Deleterious Materials

1. Oven-dry the sample at 105±5°C to a constant mass.
2. Take a sample of the material and record the weight. (1500g suggested)
3. Screen the 1500g sample over a No. 10 (2.00mm) sieve and identify any deleterious materials as rock, gravel, slag, cinder, sod, etc...
4. Record the weight of the deleterious materials and calculate the percentage of deleterious materials by mass.

$$\left(\frac{A}{B}\right) * \mathbf{100} = C$$

A = Wt of deleterious material(Wt retained on #10)
B = Sample wt (1500g suggested)
C = %Deleterious Materials

Determination of Organic Material

1. Take a sample of the fraction passing the No. 10 sieve (50g is suggested) and determine the percentage by mass of organic matter fractions by ashing at 440°C.
    a. Find weight of bowl being used.
    b. Add 50g of topsoil.
    c. Place in muffle furnace at 440°C overnight.*(4hour minimum)*
    d. Carefully remove the sample from the oven and place it directly in the dessicator and allow it to cool.
    e. Weigh ashed sample.
    f. Find percentage by mass of organic material by subtracting the Bowl Weight+ sample after ashing from the Bowl Weight + 50g sample, and dividing the difference by 50g.

Determination of Silt and Clay Content

1. Take a sample of the fraction passing the No. 10 sieve (200g is suggested) and place it in a container with water containing a deflocculating agent and allow it to soak for a minimum of 2 hours. When the soaking period is finished pour the sample over the No. 200 sieve and wash the material by means of a stream of water from a faucet. The material may be lightly manipulated by hand to facilitate the washing process. Continue the washing until the water coming through the sieve is running clear.
2. When washing is complete the material may be dried in the sieve or by flushing the material into another container. Excess water can be removed by decanting or suctioning to speed drying time. Take care not to lose any particles by removing only clear water.

3. Dry the sample to a constant mass at a temperature of 110±5°C and determine the mass retained on the No. 200 sieve.
4. Calculating the sand and silt/clay percentage by mass.
   a. Percentage of sand by mass

$$\left[\frac{C}{B}\right] * 100 = A$$

A = Percentage of sand by mass
B = Original dry mass of sample (200g)
C = Dry mass of sample retained on the No. 200 sieve

   b. Percentage of silt/clay by mass

$$100 - A - D = E$$

A = Percentage of sand by mass
D = Percentage of organic material by mass (found from earlier ashing)
E = Percentage of silt/clay by mass

Determination of Topsoil pH

1. Prepare the calcium chloride stock solution by dissolving 147g of $CaCl_2$ in 1 liter of distilled water. This stock solution will be used to make the calcium chloride working solution for testing pH.
2. Prepare the calcium chloride working solution by diluting 5ml of calcium chloride stock solution with distilled water to a volume of 500ml.
3. Calcium Chloride method:
   a. Weigh approximately 10g of air dried soil and place into a glass container.
   b. Add approximately 10ml of calcium chloride working solution mix thoroughly and let stand 1hr.
   c. Using the pH meter and electrode measure the pH of the sample to the first decimal place.
   d. Acceptable values range from 5 to 7.
4. Distilled Water method:
   a. Weigh approximately 10g of air dried soil and place into a glass container.
   b. Add approximately 10ml of distilled water mix thoroughly and let stand 1hr.
   c. Using the pH meter and electrode measure the pH of the sample to the first decimal place.
   d. Round the values to the nearest whole number.
   e. Acceptable values range from 5 to 7.