Ohio Department of Transportation

GEOTECHNICAL DESIGN MANUAL

January 2026

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SECTION 100 GENERAL INFORMATION

101 GEOTECHNICAL EXPLORATION

When necessary, perform a geotechnical exploration in accordance with the Specifications for Geotechnical Explorations (SGE). The purpose of the geotechnical exploration is to provide geotechnical-related surface and subsurface information necessary for the planning, design, and construction of the project.

102 EXPLORATION TASKS

The extent of exploration is limited to the acquisition of geotechnical data for geotechnical conditions and properties that affect the planning, design, and construction of ODOT transportation projects, or existing ODOT transportation infrastructure and associated appurtenances. The geotechnical exploration is comprised of four primary tasks, presented as follows:

- Reconnaissance and Planning
- Boring, Sampling, and Field Testing
- Laboratory Testing
- Geotechnical Exploration Report

This manual serves as guidance for geotechnical design on ODOT projects for preparation of the Geotechnical Exploration Report, in addition to other ODOT design requirements such as presented in the ODOT Bridge Design Manual (BDM) and AASHTO Load and Resistance Factor Design Bridge Design Specifications (AASHTO LRFD).

103 GEOTECHNICAL ENGINEERING DESIGN CHECKLISTS

Geotechnical design features that arise in the development of roadway projects vary both in type and complexity. Cuts, embankments, wetlands, mine issues, and rock slopes are just some geotechnical issues encountered on transportation projects. Consistent and comprehensive reconnaissance, analysis, and plan preparation are necessary to ensure that all possible geotechnical issues that may occur on a project will be adequately identified and accounted for on the final plans.

A set of topical review checklists, a reference list, and a technical publications list have been developed to aid the project development personnel in their production of geotechnically sound project plans. All projects that contain geotechnical design will benefit from the use of this document. The checklists can be found on the Office of Geotechnical Engineering (OGE) website here.

104 GENERAL PLAN NOTES AND SPECIAL PROVISIONS

General Plan Notes and Special Provisions, made available on the OGE website here, have been developed and will continue to be developed by OGE for use by the designer. The designer needs to ensure that the notes and provisions are complete, revised accordingly, and apply to the specific project. Always check for available notes before creating custom notes for a project. If necessary, custom notes must be written to conform to the actual conditions that exist on each individual project.

105 CONSULTANT PREQUALIFICATION

ODOT maintains prequalification requirements for geotechnical engineering laboratories, field exploration services, drilling inspection services, and the performance of geotechnical engineering services. Prequalification requirements are set out in the ODOT Consultant Prequalification Requirements and Procedures, which can be viewed at Consultant Services' website. The ODOT Specifications for Consulting Services stipulates that consultants must be prequalified to perform these services, and further states that these services cannot be subcontracted to a non-prequalified firm. On local government projects that utilize federal funds, ODOT requires consultants hired by local governments be prequalified.

106 REVIEW OF THE PLANS

The consultant will ensure any plans submitted to ODOT for review according to the ODOT staged review process are reviewed by the registered engineer(s) responsible for preparation of the Geotechnical Exploration Report prior to submittal to ODOT. The registered engineer responsible for preparing the Geotechnical Exploration Report is required to submit letters certifying their review of the Stage 2 and final plan sets, and these letters are to be included in the Stage 2 and Final Plan Set submissions to ODOT. Certification letter templates can be found in the SGE, Appendix H.

107 REFERENCE DOCUMENTS

Within this manual reference is made to the latest version of many other manuals and documents that are integral to the comprehensive geotechnical design process here at ODOT. These referenced documents are listed as follows:

- ODOT Specifications for Geotechnical Explorations (SGE)
- ODOT Rock Slope Design Guide
- ODOT Bridge Design Manual (BDM)
- ODOT Location and Design Manual (L&D) Volume 1, Roadway Design
- ODOT Location and Design Manual (L&D) Volume 2, Drainage Design
- ODOT Location and Design Manual (L&D) Volume 3, Highway Plans
- ODOT Location and Design Manual (L&D) Volume 4, Survey, Mapping, and Subsurface Utility Location Services Specifications
- ODOT Traffic Engineering Manual
- ODOT Pavement Design Manual

- ODOT CADD Engineering Standards Manual (OHDOT)
- ODOT Construction and Materials Specifications (C&MS)
- AASHTO LRFD Bridge Design Specifications (AASHTO LRFD)
- AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS)
- FHWA-SA-95-037 Geotechnical Engineering Circular (GEC) 1 - Dynamic Compaction
- FHWA-SA-96-038 (GEC 2)- Earth Retaining Systems
- FHWA-NHI-11-032 (GEC 3) - LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations Reference Manual
- FHWA-IF-99-015 (GEC 4) - Ground Anchors and Anchored Systems
- FHWA-IF-02-034 (GEC 5) - Evaluation of Soil and Rock Properties
- FHWA-NHI-16-072 (GEC 5) - Geotechnical Site Characterization
- FHWA-IF-02-054 (GEC 6) - Shallow Foundations
- FHWA-NHI-14-007 (GEC 7) - Soil Nail Walls
- FHWA-HIF-07-039 (GEC 8) - Design and Construction of Continuous Flight Auger Piles
- FHWA-NHI-18-031 (GEC 9) - Design and Analysis of Laterally Loaded Deep Foundations
- FHWA-NHI-18-024 (GEC 10) - Drilled Shafts: Construction Procedures and LRFD Design Methods
- FHWA-NHI-10-024, 025 (GEC 11) - Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Vols. I and II
- FHWA-NHI-16-009, 010 (GEC 12) - Design and Construction of Driven Pile Foundations, Vols. I and II
- FHWA-NHI-16-027, 028 (GEC 13) - Ground Modification Methods Reference Manual, Vols. I and II
- FHWA-HIF-17-016 (GEC 14) - Assuring Quality in Geotechnical Reporting Documents
- FHWA-HIF-22-024 (GEC15) - Acceptance Procedures for Structural Foundations of Transportation Structures

- FHWA-NHI-06-088, 089 - Soils and Foundations Reference Manual – Vols. I and II
- FHWA-HIF-12-003 (HEC 18) “Evaluating Scour at Bridges”

SECTION 200 GEOTECHNICAL SUBMISSION GUIDELINES

The ODOT Project Development Process (PDP) is a phased approach to developing a transportation project from concept to completion. In the PDP, transportation projects are classified into one of five paths (paths 1 through 5) based on the anticipated level of project development complexity, with one being the least complex and five being the most complex. All projects must advance through five sequential phases: (1) Planning, (2) Preliminary Engineering, (3) Environmental Engineering, (4) Final Engineering/ROW, and (5) Construction. The project path identifies the recommended level of analysis, amount of stakeholder involvement, and activities performed during each phase. See the Project Development Process Manual in the Division of Planning for a complete description of the PDP.

Section 1400 of the Location and Design Manual, Volume 3 (L&D3) is focused on the design involvement in the PDP, with an emphasis on design review submittals. In it, an overview of plan development is provided, identifying the information to be included in each design review submission. However, the description of submission requirements is of a general nature, and the listing of the information to be submitted for a specific task is meant to be all inclusive, covering all the disciplines within ODOT. Specific detailed information on the submission requirements for a given discipline is not provided.

In this section, a detailed explanation of the Geotechnical Engineering submission requirements for the nine PDP tasks is presented. The geotechnical information and documents listed will be included as part of the complete project submission package required for each specific PDP task. Avoiding the submission of unnecessary information, but more importantly, avoiding insufficient or incomplete submissions, are the primary objectives of these guidelines. It is imperative that the submitting agency be aware of the expectations of the reviewing party, so that complete, correct, and consistent submittals are provided.

Details regarding the content of various geotechnical reports are presented in the SGE. Section 700 covers three geotechnical report types: Roadway Exploration, Structure Foundation Exploration, and Geohazard Exploration. Only one final report should be submitted for each project. The specific contents of these three report types are described so as to provide the necessary geotechnical information consistently from project to project. These reports and the Geotechnical Profile are part of the geotechnical deliverables for specific PDP task submissions and are referenced in these guidelines at the appropriate tasks. Refer to the SGE Section 700 for information regarding the electronic submission of geotechnical information and the role of the District Geotechnical Engineer (DGE).

Although guidelines are presented for the required geotechnical submissions in specific PDP tasks, it is understood that not all projects will have all geotechnical tasks. As well, in some projects, additional submissions may be desired throughout the plan development process. Meetings and additional correspondence between the appropriate parties may be beneficial. Subsurface explorations may be conducted in phases or for individual design concerns. Geotechnical data and recommendations for parts of a project or exploration should be submitted as necessary in the form of a Geotechnical Design Memorandum to the appropriate agency whenever this information will benefit the design process. Submissions for specific design features or compliance reviews may also be necessary as the project becomes more defined.

These guidelines are intended to provide guidance as to the typical content of the various geotechnical documentation and submissions that are part of the PDP. It is understood that the size and complexity of these documents are directly related to the specific requirements of the project. The type of report, amount of design calculations, and complexity of plan detailing are just some examples of the things that will differ from project to project.

201 TASK 1.2.C.B IDENTIFY GEOTECHNICAL ISSUES

The purpose of the Project Initiation Package (PIP) is to provide a summary or overview of potential issues and concerns that could create major scope, schedule, or cost issues during project development. Knowing about and avoiding or mitigating problematic issues early in the process will save time and money. The PIP is produced early in the Planning Phase by the ODOT District Staff and is required for projects following Paths 2-5.

The deliverables for this task will vary in detail based on the project size and complexity. Both an office and field review of the geotechnical aspects of the proposed project should be performed by the DGE. The geotechnical portion of the PIP form should be completed. In general, a description of the site geology, subsurface conditions, field observations, and any potential geotechnical issues are to be included in the project PIP. The required information should also be indicated on the proper study area basemap.

As the geotechnical work for the PIP is performed solely by the DGE within the District, there are no submission requirements. The product will be the geotechnical information being supplied on the PIP form and the project basemaps.

202 TASK 2 PRELIMINARY ENGINEERING PHASE

The development of preliminary alternatives in the Preliminary Engineering Phase involves Feasibility Studies (FS), Environmental Field Studies, and the creation of the Alternative Evaluation Report (AER). The Geotechnical Engineer will provide input for the FS and the AER. The FS is designed to analyze alternatives in order to identify a preferred alternative, or multiple alternatives, through the PDP. An FS is not required for Path 1 projects; will typically lead to the preferred alternative in Paths 2 and 3; and produces a limited number of alternatives for further study in Paths 4 and 5. The AER is designed for concurrent processing of preliminary engineering and environmental work, and the recommendation of the preferred alternative. An AER will not be compiled for Path 1 and 2 projects and is unlikely for Path 3 projects. The AER is primarily used on Path 4 and 5 projects.

The geotechnical deliverables for this task will typically be the geotechnical input into the FS and AER but may also include a Geotechnical Design Memorandum – Preliminary Geotechnical Exploration and/or a Geotechnical Design Memorandum – Subgrade.

For the FS, each proposed alternative should be analyzed to ascertain the impact of the geotechnical conditions present. The work consists of a thorough review of all existing geotechnical information, including that provided in the PIP. The location and extent of the geotechnical concerns for all alternatives are developed. A summary of the existing geotechnical conditions and possible construction and long-term geotechnical issues with each of the proposed alternatives would become part of the FS.

In the AER, typically 2 to 3 alternatives are compared, with the result being a recommendation for the preferred alternative. There may be several aspects of each alternative being studied that require geotechnical evaluation, such as:

- Effects on horizontal and vertical alignments due to soft foundation soils, mines, rock slopes, unstable soil slopes, etc.;
- The need for retaining walls, compared to steeper slopes or alignment changes;
- Foundation influences on the types of bridges or culverts at the alternative locations;
- Remediation or special construction techniques to alleviate challenging geotechnical conditions.

All these items influence the cost of a proposed alternative. In evaluating these areas, all existing geotechnical information should be utilized, and when necessary, reasonable design assumptions should be made. By using sound engineering judgment, reasonable evaluations and cost estimates can be developed for the alternatives without performing new borings. These evaluations and associated costs for each of the proposed alternatives would become part of the AER.

Although it should be a rare occurrence, there are times when a Preliminary Geotechnical Exploration is necessary to adequately perform the alternative evaluations for the AER. This exploration should only be performed when major design facets of an alternative require exploration to reasonably estimate project impacts because the existing geotechnical information does not allow for reasonable assumptions to be made. This exploration work must be approved by the DGE prior to beginning the work. Therefore, a cost proposal and boring plan should be submitted to the District as early as possible. The Geotechnical Design Memorandum – Preliminary Geotechnical Exploration issued as a result of the Preliminary Geotechnical Exploration should cover all the geotechnical alternatives considered, any design and remedial measures necessary to complete the project, and a cost summary for the geotechnical aspects of the proposed alternatives. A preferred alternative from the geotechnical perspective should be provided. The borings should be plotted on a plan and profile view. The Geotechnical Design Memorandum – Preliminary Geotechnical Exploration should be included in the AER.

Sometimes, pavement design is requested at this early phase. However, rarely is a Roadway Exploration necessary or prudent as alignments, profiles, and beginning and end points change, and assuming average design parameters is usually sufficient. If performed, this exploration work must be approved by the DGE prior to beginning the work. Therefore, a cost proposal and boring plan should be submitted to District as early as possible. The Geotechnical Design Memorandum – Subgrade issued as a result of the subgrade borings should include a subgrade analysis, specific subgrade stabilization recommendations, subgrade support values with the pavement design, and a cost summary for all the proposed alternatives. The borings should be plotted on a plan and profile view. The Geotechnical Design Memorandum – Subgrade should be included in the AER.

The general submission requirements for this task are presented in L&D3 Section 1400.

202.1 Task 2.1.A Prepare and Complete Feasibility Study

For a Feasibility Study, the geotechnical deliverables may include the following:

- Documentation summarizing existing conditions and possible construction and long-term geologic and geotechnical issues for each alternative.
- Mapping showing the areas of geotechnical and geological concerns.

202.2 Task 2.5 AER Submittal and Other Studies

The analysis of preliminary alternatives is near completion as the AER is finished and the preferred alternative is chosen for full design and plan development. For an AER, the geotechnical deliverables may include the following:

- Documentation of the geotechnical evaluations and cost estimates for the proposed alternatives, and a recommendation of a geotechnically based preferred alternative.
- Preliminary and/or subgrade borings plotted in plan and profile view if borings were performed.
- Geotechnical Design Memorandum – Subgrade, if only subgrade borings were performed.
- Geotechnical Design Memorandum – Preliminary Geotechnical Exploration, if preliminary borings were performed.

In addition to the AER, the analysis of alternatives may include Structure Type Studies and Retaining Wall Justifications. A Retaining Wall justification (RWJ) compares the impacts and costs (both right of way and construction) of the preferred alternative with and without retaining walls. RWJ would not be performed for Path 1 projects, but typically would be for Path 4 and 5 projects. Rarely would Path 2 and 3 projects have a need for RWJ. Geotechnical content may be included in the STS and RWJ reports but may also include delivery of a Structure Foundation Exploration report (for a standalone structure project) or a Geotechnical Design Memorandum - Structure for developing recommendations in the STS.

202.2.1 Task 2.5.D Structures

A Structure Type Study (STS) examines the project site in detail and evaluates conditions to determine the best structure alternative. Path 1 projects would not have an STS, but Path 4 and 5 projects, and often Path 2 and 3 projects, will have structure work that requires an STS. The work involved in an STS, including the geotechnical requirements, is prescribed in the Bridge Design Manual (BDM). Estimated resistances, a recommended foundation type, scour analysis inputs, and associated costs, comprise the geotechnical work. All existing geotechnical information should be utilized, and reasonable design assumptions should be made to provide the needed recommendations without requiring new borings, if possible. The STS may be a separate report but is often included as part of the AER.

There are times when a Structure Foundation Exploration (SFE) will be necessary to adequately develop the required foundation recommendations for the STS. This exploration should only be performed when the existing information at a critical structure location does not allow for reasonable foundation estimates to be developed. This exploration work must be approved by the DGE prior to beginning the work. Therefore, a cost proposal and boring plan should be submitted to District as early as possible. The report issued as a result of the SFE should cover all the geotechnical requirements of the STS provided in the BDM for the structure location being evaluated. The borings should be plotted on a plan and profile at the structure location. Either a summary of the SFE report, or preferably, the entire report, should be included in the STS report.

The geotechnical deliverables for this task are as follows:

- Geotechnical aspects of the STS, as required by the BDM, and included as part of the STS report.

- Existing geotechnical information presented on a plan and profile for any structure location being analyzed.

- Project borings (if any) plotted in plan and profile view for the structure location being analyzed for an STS.

- Draft version of the SFE report or Geotechnical Design Memorandum – Structure.

202.2.2 Task 2.5.E Retaining Wall Justification (RWJ)

An RWJ compares the impacts and costs of the project with and without retaining walls. Specific information on RWJ studies is found in the BDM and the Location & Design Manual Volume 3 (L&D3). Many areas of the project work affect the RWJ. Horizontal and vertical geometry, right of way acquisition, environmental and geotechnical concerns, types of usable wall systems, and associated costs of all these items, all need to be evaluated in an RWJ study. Making use of existing geotechnical information should be sufficient to perform the geotechnical evaluation in an RWJ study, so no further explorations should be necessary. The RWJ may be a separate report but is often included as part of the AER.

The geotechnical deliverables for this task are as follows:

- Geotechnical aspects of the RWJ study, as prescribed in the BDM and L&D3 and included as part of the RWJ report.

- Existing geotechnical information presented on a plan and profile for any wall location being analyzed.

202.3 Task 2.7.D.A Stage 1 Design, Geotechnical Services and Report

Stage 1 begins after the identification of a preferred alternative. Stage 1 Design refines and builds upon the preliminary engineering design completed for the AER. It provides a level of detail necessary to begin Preliminary Right-of-Way Plans, allows for an accurate estimation of required right-of-way acquisition, and allows for a refined estimate of construction costs. Path 1 projects would not have a Stage process, but Path 4 and 5 projects, and often Path 2 and 3 projects, will go through the Stage design process.

By the end of this task, design plans are developed to the Stage 1 level. The geotechnical exploration, testing, and a draft version of the exploration report should be complete. Cross-sections are nearly complete. Retaining wall and culvert plans are being developed. The geotechnical deliverables for Task 2.7.D.A include:

- Draft Geotechnical Profile,
- Draft Exploration Report,
- Completed applicable sections of the Geotechnical Design Checklists.

Details for what is presented in these documents can be found in the SGE Section 700. In general, on the Draft Geotechnical Profile, subsurface exploration locations will be shown on the plan view, with standard graphic logs shown on the profile view. If offset borings were obtained, the affected cross-sections with the graphic logs should be provided; presentation of offset borings along with nearby centerline borings on cross-sections is highly encouraged.

All geotechnical aspects of the project, including all structures, are to be addressed in the Draft Roadway Exploration Report. Geotechnical and geologic issues that are expected to be encountered should be explained and possible solutions presented with preliminary, but specific, recommendations. Multiple options can be developed, and analyses should be included. All completed test results should be presented. If additional borings are anticipated, the need and location for these borings should be explained.

All geotechnical aspects of a standalone bridge (structure) project are to be addressed in the Draft SFE Report if not already submitted as part of the STS. Geotechnical and geological issues that may be encountered should be explained and possible solutions presented. Preliminary foundation recommendations should be included for all structural alternatives which may include multiple options. All completed test results should be presented.

Pavement design is approved, and subgrade stabilization requirements are established in this task. A Subgrade Analysis Spreadsheet is required, and a design CBR is presented in the draft Roadway Exploration Report.

The general submission requirements for this task are presented in L&D3 Section 1400. For a complete geotechnical review to be performed at this point, the following items from the general submission package are needed:

- The geotechnical deliverables for this task
- Title Sheet, Typical Sections (showing subgrade stabilization if remedy is global), Plan/Profiles, and Cross-sections.

203 TASK 3.3.K.A STAGE 2, FINALIZE GEOTECHNICAL EXPLORATION AND REPORT

At the end of Stage 2, all design issues of any significance should be resolved. In general, Stage 2 plans should be developed to the point where plan preparation, design, and detailing are substantially complete. A Final Exploration Report and Final Geotechnical Profile are complete. These documents should meet all the requirements in the SGE. The geotechnical deliverables for Task 3.3.K.A include:

- Final versions of the Geotechnical Profile and Exploration Report;
- All final geotechnical analyses;
- Disposition of the geotechnical comments from the geotechnical review performed in Stage 1;
- A Certification of Review of the Stage 2 plans by the Geotechnical Engineer of Record;
- Completed applicable sections of the Geotechnical Design Checklists.

As a revised version of the Draft Exploration Report, the Final Exploration Report will reflect changes issued in the Draft review comments. The Final Exploration Report must contain specific and detailed recommendations for the chosen solution to each of the geotechnical issues anticipated on the project. Station limits, depths, specifications, notes, instrumentation, and calculations should be included. Foundation types, size and depths of deep foundations, bearing resistance, wall designs, specifications, notes, instrumentation and geotechnical calculations should be included. All required analyses should be completed and presented. A disposition of review comments pertaining to the Draft Geotechnical Profile and Draft Exploration Report should be developed and presented separately from the Final Exploration Report. More detail regarding the content of the Final Geotechnical Profile and Exploration Report submissions are presented in the SGE.

Based on the geotechnical recommendations presented in the Final Exploration Report, Stage 2 plans should be fully geotechnically developed. Plan and profile and cross-section sheets will be nearly complete. Culverts, retaining walls, and pertinent geotechnical features are to be presented in the plan views. Special benching and retaining walls are to be shown on the cross-sections. Rock cut layouts and catchment ditches are to be provided in the cross-sections (refer to Rock Cut Slope & Catchment Design Section). Show any subgrade undercuts (from subgrade analysis), embankment foundation stabilization, and any special embankment treatment on the cross-sections. Provide detail sheets and design calculations for culverts with possible foundation or settlement issues.

Retaining wall plans should be submitted in this task. The information to be included in the Stage 2 submittal depends on the wall type (cast-in-place, proprietary, special). Consult the BDM for design and submission requirements.

The general submission requirements for this task are presented in L&D3 Section 1400. For a complete geotechnical review to be performed at this step, the following items from the general submission package are needed:

- The geotechnical deliverables for this task
- Title Sheet, Schematic, Typical Sections, Plan/Profiles, Cross-sections, General Notes, and Culvert Details (if applicable)
- Retaining Wall plans and design calculations, if applicable
- General Notes (including all geotechnical and earthwork related plan notes), Bridge notes, and geotechnical specifications.

204 TASK 4.2 STAGE 3, DETAILED DESIGN PLANS

The Stage 3 Detailed Design should complete the design of the project. These plans must contain all details and quantities required to bid and construct the proposed work. There are no specific geotechnical deliverables for this task, although assistance in developing a disposition of Stage 2 comments may be needed. Do not modify the Final Exploration Report and submit the report again.

The general submission requirements for this step are presented in L&D3 Section 1400. If a geotechnical review is required at this step, the following items from the general submission package are needed:

- Full set of Stage 3 plans
- Disposition of Stage 2 comments

SECTION 300 GEOTECHNICAL EXPLORATION REPORT

301 GENERAL

Prepare a single Exploration Report and Geotechnical Profile to present the Geotechnical Exploration performed for each project in accordance with the requirements presented in SGE Section 700. The Exploration Report and Geotechnical Profile should be titled and presented to reflect the predominant scope of the project. Report title options are presented and described as follows:

- **Roadway Exploration Report**; a roadway project to include all structure explorations, if any.
- **Structure Exploration Report**; a new structure or structure replacement of any kind, including bridge, culvert, retaining wall, noise wall, high mast tower lighting, etc. If roadway work beyond bridge approach work is planned, title the report a Roadway Exploration Report.
- **Geohazard Exploration Report**; a geohazard remediation project, including landslide, rock slope, mine, and soft ground/peat.

302 REPORT

Provide a single Exploration Report with specific recommendations, addressing all geotechnical design aspects of the project, in accordance with the requirements presented in SGE Section 705. Do not repeat the requirements of the Construction and Material Specifications (C&MS). The requirements set forth in the C&MS are considered minimum requirements and will typically not be waived or reduced. Present any recommended additional requirements to those already presented in the C&MS along with justification.

Include recommended plan notes and details to prescribe or identify minimum acceptable performance criteria of geotechnical design features and recommendations. Before creating custom plan notes, check for existing specifications, special provisions, and notes in the Bridge Design Manual, L&D3, this manual, and on the OGE website. Fill in all necessary values for all plan notes (e.g., strength limit state bearing pressure, factored bearing resistance, etc.) so the notes are complete for insertion into the project plans.

Include calculations that are consistent with and support the recommendations presented. Present the calculations in a well-organized, easy to follow, legible format with all nomenclature clearly defined. Annotate calculations so that the source of equations used is clearly indicated. Ensure that the calculations include the initials of the designer and reviewer with a date of completion for both. Do not submit a report without calculations. If for some unique reason no calculations were performed, state this in the report. When presenting multiple sets of calculations, separate each set with a title page for ease of identification and review.

302.1 Draft and Final Report

Until ODOT has reviewed the report and all comments and questions have been resolved to the satisfaction of ODOT, the report is considered a draft report and should be titled as such. After

ODOT has reviewed the report and accepted all disposition of comments and changes, and the report content matches the final design of the project, submit the report as a final report, titled as such. If revisions to the Final Report are necessary, title the revised report “Revised Final Report” with sequential numbering corresponding to the number of revised reports submitted.

302.2 Geotechnical Design Memorandum

There will only be one final geotechnical report issued for each project which will include all geotechnical exploration and design content. If a project includes multiple design features, such as roadway, bridges, culverts, noise walls, retaining walls, etc., and it is necessary to present recommendations individually for one of these features during the project design process, do so in the form of a Design Memorandum. The Design Memorandum should be clear and concise, presented in a letter format with supporting documentation attached, such as boring logs, calculations, etc. Issue Design Memorandums as required, numbered sequentially, and titled appropriately for the content of the memorandum.

303 GEOTECHNICAL PROFILE

To be prequalified for Geotechnical Engineering Services, a consultant must be able to produce a Geotechnical Profile in accordance with the requirements of SGE Section 702. Guidance for the preparation of the Geotechnical Profile is presented in this Section.

303.1 Transition to OpenRoads Designer (ORD)

The Department has transitioned to the Bentley Systems CONNECT Edition generation of products as its standard drafting and design software packages. Refer to L&D3 Section 1204.1.2 for additional information.

All aspects of a project are to be created in the same CADD platform. Do not create the Geotechnical Profile in a different CADD platform if the roadway construction plan has been created in ORD.

Do not “mix and match” various CADD standard elements from one platform to the other. For example, do not use the standard sheet border cells from the now retired CADD Engineering Standards Manual (ODOTcadd) for MicroStation in ORD.

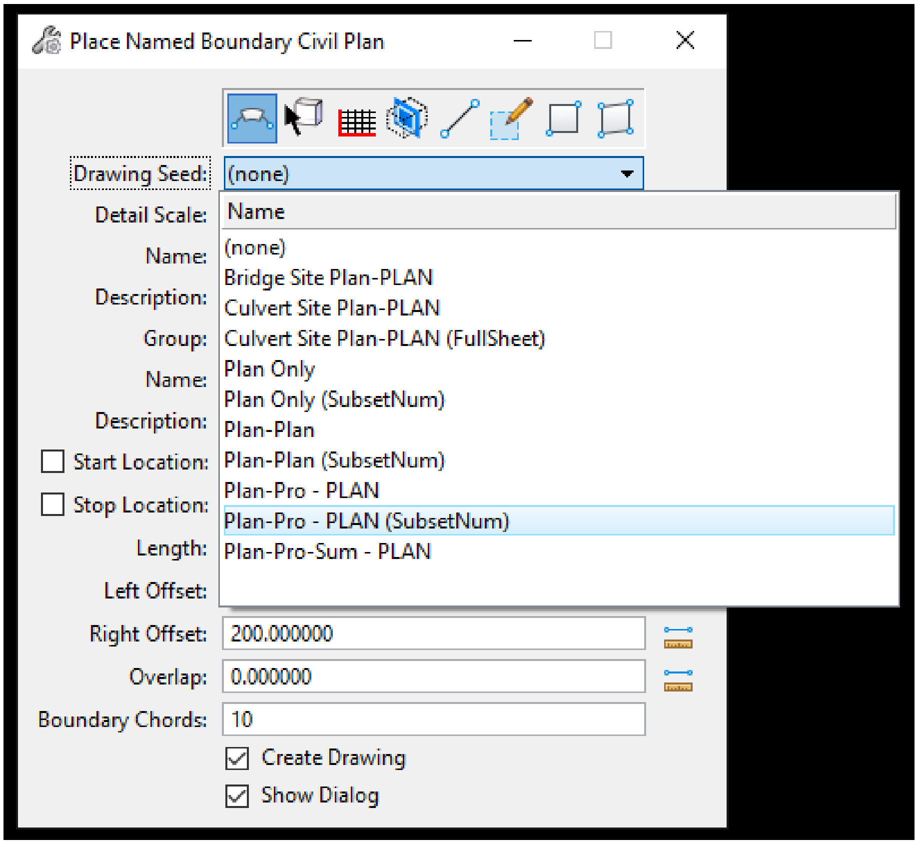
When working in ORD, the process for creating the Geotechnical Profile cover sheet, lab data sheets and boring logs sheets remains mostly unchanged. For guidance on creating Plan & Profile sheets and Cross Section sheets in ORD, refer to the ODOT OpenRoads for Design Training Guide and see the Sheet Clipping videos on the ODOT Bentley Wiki page.

When working in ORD consider the following:

- Do not place boring location targets in basemap files as plain graphical “cells”; use “geometry points” instead. Boring locations by station & offset can then be extracted by using one of the predefined “geometry reports”. See the video Import ASCII Points on the ODOT Bentley Communities Wiki Page.

- When creating Plan & Profile sheets or Cross Section sheets, select the drawing seed file set-up for sheet borders with subset sheet page numbering as shown in Figure 300-1.

Figure 300-1: Drawing Seed File Set-up for Sheet Borders



- Place boring graphics (stick logs) in the “drawing” models behind the “sheet” models. The Department does not yet have any tools for automated placement of boring graphics, and they will need to be placed “by hand”.

- Review the Bridge Site Plan section of Section 800 Plan and Profile Sheets of the Department’s OpenRoads Designer for Design Training Guide for guidance on the display of contour lines.

303.2 ORD Resources

There are many resources available for the ORD CADD platform, some of which are presented as follows:

- L&D3 – Highway Plans.

- ODOT ORD Training Guides.

- Associated training files, a set-up guide and videos that work through the training exercises can be found on the ODOT Bentley Communities Wiki Page.

https://communities.bentley.com/communities/user_communities/ohio_dot__consultants/w/ohio dot---wikis

- A schedule of Bentley's Special Interest Group (SIG) live webinar presentations for any of Bentley's products, videos of previous SIG presentations, and a schedule of live, interactive, on-line, Bentley instructor led training classes.

https://www.bentley.com/en/global-events/events-overview

- Bentley LearnServer site, which has learn at your own pace training videos on all of Bentley's products.

https://learn.bentley.com/app/Public

- Videos available on the ODOT Wiki page

https://communities.bentley.com/communities/user_communities/ohio_dot__consultants/w/ohiodot---wikis

    - Supplemental Videos: cover various topics and supplement the above training.
    - Custom Applications Videos: how to videos on ODOT's custom applications.
    - Webinars Videos: Recordings of ODOT's internal CADD/ProjectWise webinars.
    - Recordings of ODOT CADD User Group Meetings.

303.3 Common Geotechnical Profile Issues

The minimum requirements for performing a subsurface exploration and creating a Geotechnical Profile are presented in the SGE; however, these requirements cannot address every scenario. The Geotechnical Engineer is responsible for creating a clear, concise, and comprehensive Geotechnical Profile, depicting all the geotechnical data generated for a project, while meeting all the minimum requirements of the SGE. In most cases, the Geotechnical Profile is the only construction contract document representing the project geotechnical conditions. The Geotechnical Profile must be presented in a useful way for all parties involved to understand what subsurface conditions are expected to be encountered in construction. A list of common Geotechnical Profile issues that deviate from the intended clarity and usefulness of the Geotechnical Profile is presented as follows:

- Using an incorrect scale is one of the most common Geotechnical Profile issues, involving both the designer and the geotechnical engineer. As stated in SGE Section 702.2, it is not acceptable to use a scale other than 1" = 50' for projects or individual alignments more than 1250 feet in length. Using the same scale as the construction plans (a smaller scale such as 1"=20') leads to a larger Geotechnical Profile sheet package than is necessary. For borings drilled at the maximum 400-foot spacing, using such a small scale can result in a series of Geotechnical Profile sheets with only one graphical log shown on each sheet. This practice is not concise and hinders visual reference between borings for understanding the overall subsurface profile for the site.

- Boring data is not presented on cross sections nearly enough. Presenting boring data on cross sections, as well as along the profile, depicts geotechnical conditions left and right, as they relate to the centerline/baseline of the work. Presenting borings drilled at or near the same station on a cross section reduces congestion in the profile view while visually maintaining the vertical and horizontal relationship between borings. Refer to SGE Section 702.6.2 for guidance on the use of cross sections in a Geotechnical Profile.

- If historical borings are used in the Geotechnical Profile, use the best available log image (PDF or TIFF format). It is not necessary to reproduce the historical geotechnical information in the project boring log software.

- For a Geotechnical Profile – Bridge, if all the borings drilled are structure borings, do not include a Summary of Soil Test Data. When borings are drilled for dual purposes (structure and subgrade/roadway), the top portion of those borings relevant to the roadway (10 feet) or subgrade design (6 feet) can and should be displayed in the Summary of Soil Test Data along with the rest of the roadway/subgrade borings.

- For a Geotechnical Profile – Bridge, when sampling and testing for scour analysis, verify that boring depths and elevations reflect the latest survey data in the Cover Sheet report; they commonly do not. Only include samples relevant to the scour analysis in this report.

303.4 Common Boring Log Issues

In order to address a few common issues with boring logs, the following directions are presented:

- If using gINT to create boring logs, use the log forms in the ODOT gINT library available on the OGE website. If sulfate testing has been performed, use the gINT boring log report form that includes a column for sulfate content results.

- If using OpenGround to create boring logs, work must be completed in the ODOT cloud environment. If sulfate testing has been performed, the OpenGround logs are dynamic and will include a column for sulfate content results if they are present in the Geotechnical Chemistry Testing table.

- If using software other than gINT or OpenGround to create boring logs, a DIGGS (Data Interchange for Geotechnical and Geoenvironmental Specialists) file for the project must be submitted. The file must be in DIGGS schema 2.6 or greater. Details about the DIGGS file format and current schema can be found at https://www.geoinstitute.org/special-projects/diggs.

- All boring logs must include coordinates. ODOT's Geotechnical Data Management for both historical and current projects is based on the use of latitude and longitude for the position of borings and the various soundings. Reporting geotechnical data in northing & easting is not acceptable, regardless of any project specific scale factor.

- When a sample is subdivided into multiple specimens, make sure that all related data entry (including material descriptions and stratigraphic elevations) reflects this. Below is an example of what is expected for presentation of multiple specimen (source: SGE Sample Plans).

564.8	19	21 19 15	46	89	SS-13A	3.50	20	4	9	31	36	35	21	14	15	A-6a (8)
	20				SS-13B	-	48	8	23	14	7	NP	NP	NP	19	A-1-b (0)

303.5 Commentary on Unique Geotechnical Profile Scenarios

For projects where graphical logs displayed along the centerline or baseline profile would not best present the geotechnical data as it will be used in construction, such as for an R-Cut intersection project, OGE has utilized a strategy of only presenting the borings on cross sections and not on a profile view. All other Geotechnical Profile sheet components, including the cover sheet, Summary of Soil Test Data, plan views, etc. are prepared and included as normal.

For projects where no centerline or baseline profile is available, OGE has used two strategies depending on project features and designer preference. One strategy is to present a modified Geotechnical Profile that includes a cover sheet/Summary of Soil Test Data, plan view (boring plan), and boring logs (similar to Geotechnical Profile-[Structure] or Geotechnical Profile-[Geohazard]). A second strategy is to include all the relevant geotechnical data in a Geotechnical Data Report and reference it as a Special Provision in the plans. The report should include the Geotechnical Profile cover sheet information as described in SGE Section 702.3, all test results, and boring logs.

SECTION 400 ENGINEERING PROPERTIES OF SOIL AND ROCK

401 GENERAL

It is the Geotechnical Engineer's responsibility to plan and execute an appropriate subsurface exploration that meets or exceeds the minimum requirements of the SGE. The engineer must consider site risk, reliability, and variability when determining how and to what extent to characterize a site and determine design properties. All soil and rock strata that will be influenced by the project need to be adequately characterized. It is important that *representative* samples are being evaluated and considered.

Please refer to FHWA NHI-16-072, Geotechnical Engineering Circular No. 5 (GEC5), "Geotechnical Site Characterization" for a comprehensive and practical reference, to be used for planning and execution of geotechnical explorations, interpreting the acquired data, and developing reliable geotechnical design parameters. The terms "property" and "parameter" are used and defined in GEC5. In this document, the terms are used interchangeably, and the term "property" is typically used.

Values for design parameters can be established using some combination of direct and indirect methods. Direct methods typically consist of field and laboratory tests to determine specific values. Direct methods should be used to some extent on all projects, but should certainly be used to determine extreme properties, such as for very soft clay or very strong bedrock. Direct methods should also be used to determine highly sensitive properties, highly variable properties, and to substantiate atypical design and construction recommendations, such as ground improvement. Where site conditions are relatively consistent with straightforward stratigraphy and consistent properties, indirect correlations are considered reasonable for determining design properties. Use the design properties recommended in this Section where appropriate and applicable.

402 SUBSURFACE STRATIGRAPHY

Interpretation of stratigraphy is an important component of site characterization and geotechnical design. All the subsurface data collected as part of the subsurface exploration is to be presented in a Geotechnical Profile in accordance with SGE Section 700. Use all this data to create subsurface stratigraphy and associated properties for design analysis. It is impossible to accurately model all stratigraphic features at a site for design; some level of simplification and approximation is necessary. The challenge for site characterization is to accurately identify and model all important stratigraphic units within the constraints of practicality and the design and analysis methods that will be used.

Inspect the boring log(s) closest to the location of analysis or considered to be most representative of the soil conditions at the analysis location. Inspect and document any cut and fill exposures, such as bedrock outcrops, and use this data when creating stratigraphy. Also consider the geologic history and depositional environment of the subsurface materials. Divide the boring log into soil layers (i.e., stratigraphy) based on ODOT soil classification and consistency (stiffness of cohesive soils or density of granular [non-cohesive] soils). Soil classifications A-1-a through A-3a are considered granular soils, and A-5 through A-7-6 are considered cohesive soils. Soil classifications

A-4a and A-4b can be considered as either granular or cohesive depending on plasticity; a plasticity Index (PI) of greater than 6 is generally considered as a cohesive soil. Use engineering judgment on PI values of 6 or less, considering both how the behavior of the soil will affect the analysis, and the properties of adjacent, possibly related soils.

Soils of similar properties (either cohesive or granular of similar consistency or density) may be grouped together. For example, a layer of very stiff A-6a immediately overlying or underlying a layer of very stiff A-6b might be grouped together as a single cohesive soil layer. Depending on the type of analysis, thin layers of less than 3 feet thickness may often be ignored, if the Engineer judges that their exclusion will have little effect on the analysis.

Do not extend the bottom of analysis models beyond the available subsurface information. If the geotechnical conditions are homogeneous at depth between nearby borings, then the data from deeper borings may be used to extend the data for shorter borings with engineering judgment. However, if the available subsurface information is not deep enough for an analysis to be successfully completed, such as for deep foundations, then either design within the limits of the known geotechnical profile or collect additional, deeper data.

403 UNCONTROLLED FILL

Uncontrolled fill is a material description in the ODOT Soil Classification Chart (see SGE Figure 600-1) classified by visual inspection and presented with a description of the uncontrolled fill material. This description is intended to be used for fill material encountered in a subsurface exploration that appears or is known to have been placed in an uncontrolled manner. Uncontrolled refers to the type of material and/or the manner and condition in which it was placed. Uncontrolled fill material types include any landfill material, garbage, construction waste material (lumber, brick, concrete, steel, etc.), settling pond sludge, manufacturing waste material (such as foundry sand and slag), etc. Uncontrolled manner and condition of placement includes landfills, reclaimed strip mines, backfilled waste pits, irregular fills on private property, stockpiles, and temporary fills; all typically placed with no compaction, no moisture conditioning, variable and thick lifts, poor drainage, etc. The material properties of the uncontrolled fill are to be determined by the engineer based primarily on visual evaluation. Some laboratory testing may be warranted to identify adverse properties such as high moisture or organic contents, but otherwise, uncontrolled fills are not mechanically classified. The uncontrolled fill must be described in the report and on the Geotechnical profile, with estimated limits shown.

The uncontrolled fill description is not intended to be used for known ODOT roadway embankments built for the existing alignment.

404 ESTIMATING SHEAR STRENGTH USING SPT N_{60} BLOW COUNTS

Soil shear strength properties are likely the most used and important of all geotechnical design properties. The measurement and interpretation of soil shear strength properties are complex and depend on several factors. To the fullest reasonable extent, representative soil strength values should be determined from in-situ and laboratory tests considered appropriate for the design intent. Standard Penetration Testing (SPT) is still performed in virtually every subsurface exploration, and it is considered reasonable to use SPT N_{60} blow counts to estimate soil shear strengths in the absence of direct measurements for non-complex projects and geology.

404.1 Cohesive Soils

Estimating undrained shear strength (S_u) of cohesive soils using a hand penetrometer or hand torvane should be used in conjunction with, and in most cases, take precedence over estimating from SPT N_{60} blow counts. Nonetheless, for cohesive soils with SPT N_{60} blow counts of 52 blows per foot (bpf) or less, multiply N_{60} by 125 to arrive at an approximation of S_u for the soil in pounds per square foot (psf), resulting in a value ranging from 125 psf to 6500 psf. This linear relationship corresponds well with the charts produced by Terzhaghi & Peck (1967). For cohesive soils with SPT N_{60} blow counts greater than 52 bpf, this relationship becomes unconservative. In this case, use Stroud (1974, 1989):

$S_u = f_1 N_{60} p_a / 100$,

where f_1 is related to the soil plasticity index (PI) as given in Table 400-1, and p_a = 2116.5 psf (atmospheric pressure).

Table 400-1: Soil f_1 Value

Plasticity Index (PI)	f_1
0	5.7
8	5.6
15	5.5
26	5.3
31	5.2
36	5.1
40	5.0
43	4.9
45	4.8
47	4.7
50	4.5
51	4.4
55	4.1
56	4.0
57	3.9
58	3.8
59	3.7
60+	3.6

For values of PI or f_1 intermediate to those shown in Table 400-1, linearly interpolate between values. Alternately, if the soil PI is not definitively known, use Table 400-2 to approximate f_1:

Table 400-2: Alternate f_1 in the Absence of PI

Soil Class	f_1
A-1-a to A-3a	5.7
A-4a	5.6
A-4b	5.6
A-5	5.6
A-6a	5.5
A-6b	5.4
A-7-5	5.3
A-7-6	5.0

In the case of very soft cohesive soils with $N_{60} < 1$, the undrained shear strength may be approximated from 0 psf to around 100 psf, based on a visual observation of the soil using a hand penetrometer or Torvane, or determined by laboratory shear strength testing of relatively undisturbed Shelby tube (or similar) samples. Cone penetration testing (CPT) is also an excellent tool for exploration and determining strength properties of such soft soils and is highly encouraged. For peat soils, always assume $S_u = 0$ psf for pile bearing resistance; however, approximate a value of $S_u > 0$ psf for wave equation drivability analysis.

For very hard cohesive soils, do not use $S_u > 16{,}000$ psf. Analysis of approximately 150 CPT records collected around the state indicates that undrained shear strengths equal to or greater than 16,000 psf were encountered approximately 10% of the time; therefore, we consider this to be a reasonably conservative upper-bound value.

For drained analyses, it is a common practice to ignore cohesion of soils for long-term analyses, and to consider only the internal friction angle for even cohesive soils. However, for c-ϕ analyses, particularly in the case of overall (global) stability, we encourage the consideration of drained cohesion, so that overly conservative results are not produced. For soils with $S_u \leq 2000$ psf, the drained cohesion can be approximated as $c' = S_u/10$. For $S_u > 2000$ psf, use the following equation:

$$c' = 0.4{S_u}^{0.8} - \frac{S_u}{100} + 45$$

Where:

c' = drained cohesion (psf)
S_u = undrained shear strength (psf)

404.2 Granular Soils

For granular soils, use SPT $N1_{60}$ and the relative percentage of fine or coarse materials in the soil/soil classification, in accordance with the AASHTO LRFD Article 10.4.6.2.4 to estimate the drained friction angle of the soil. Use the middle of the range of friction angles presented on each line in AASHTO LRFD Table 10.4.6.2.4-1, and apply the adjustment according to Table 400-3:

Table 400-3: ϕ' Adjustment

Soil Class	Adjustment
A-1-a	+2.5°
A-1-b	+1.5°
A-2-4	+0.5°
A-2-5	-0.5°
A-2-6	-0.5°
A-2-7	-0.5°
A-3	-1.5°
A-3a	-0.5°
A-4a	-2.5°
A-4b	-2.5°

The values of SPT $N1_{60}$ on each line of AASHTO LRFD Table 10.4.6.2.4-1 may be linearly interpolated for intermediate values. For example, SPT $N1_{60}$ = 4 corresponds to a middle-range value of 29.5°, and SPT $N1_{60}$ = 10 corresponds to a middle-range value of 32.5°; therefore, SPT $N1_{60}$ = 8 corresponds to a middle-range value of approximately 31.5° by linear interpolation. If the soil was a Sandy Silt (A-4a) Soil class, the drained friction angle can be approximated as 31.5° - 2.5° = 29°.

For very dense granular soils, do not use a drained friction angle of $\phi' > 45°$. Considering the correlations published by Meyerhof (1956) and Bowles (1977), and the limits of the tabulated data in AASHTO LRFD Table 10.4.6.2.4-1 and publication FHWA-NHI-16-072 (GEC 5), we consider this to be a reasonably conservative limit for very dense granular soil.

404.3 Bedrock

For weak, augered bedrock, use the SPT N-value to estimate the unconfined compressive strength (UCS = Qu) per publication FHWA-ICT-17-018 “Modified Standard Penetration Test–based Drilled Shaft Design Method for Weak Rocks” (Stark et.al., 2017), Equation 2.2:

UCS (ksf) = $0.092 \times (N_{rate})_{90}$ (bpf).

For example, for a bedrock sampled by SPT using a hammer with ER = 81.5, resulting in a blow count value of 50/5”, the following would be the case:

$N_{81.5}$ = 50/5” × 12” = 120 bpf;

$N_{90} = 81.5/90 \times N_{81.5} = 0.9056 \times 120$ bpf = 108.7 bpf;

Qu (ksf) = $0.092 \times N_{90} = 0.092 \times 108.7$ bpf = 10.00 ksf = 69.4 psi.

There are additional possible modifiers to the equation for borehole diameter, sampler liner, and rod length; see FHWA-ICT-17-018, Table Q.1 and Skempton (1986) for additional details.

405 SOIL UNIT WEIGHT ESTIMATED FROM N_{60}

Estimate the total unit weight (γ_{tot}) of soils in pounds per cubic foot (pcf) based on the soil type (cohesive or granular) and the SPT N_{60} blow count value, according to Table 400-4.

Table 400-4: Soil Unit Weight Estimated from N_{60}

Cohesive N_{60}	Granular N_{60}	γ_{tot} (pcf)
0	-	100
1	-	105
2	-	108
3	0	110
4	-	112
5-6	1-2	115
7-9	3-5	118
10-13	6-8	120
14-19	9-14	122
20-27	15-24	125
28-35	25-34	128
36-39	35-44	130
40-43	45-54	132
44-51	55-64	135
52+	65+	140

For the effective unit weight (γ') of soils below the groundwater table, subtract 62.4 pcf from the above values. For the moist unit weight (soil above the groundwater table), subtract approximately 10 pcf from the values in Table 400-4.

406 BEDROCK PROPERTIES

In most cases it is best to determine bedrock properties from field and laboratory testing. Properties can also be estimated from visual classification, correlations developed through research, and index testing.

From research (Masada and Han, 2013) estimated rock properties of typical rock types found in Ohio are presented in Table 400-5.

Table 400-5: Rock Properties of Typical Rock Types Found in Ohio (Masada and Han, 2013)

Rock Type	Unit Weight (pcf)	Unconfined Compressive Strength (psi)	Slake Durability Index (%)
Claystone	130-165	15-1400	0-60
Shale	155-165 (unweathered) 150-160 (weathered)	2100-4600 (unweathered) 100-400 (weathered)	20-90
Siltstone	160-170	3600-8100	65-90
Sandstone	155-165	1800-7800	85-100

Friable Sandstone	125-140	<3600	<85
Limestone	155-170	3500-16400	95-100
Dolomite	165-175	4100-10300	95-100
Coal	80-85	1300-7000	NA
Underclay	125-135	200-400	0-20

406.1 Estimated Bedrock Strength from Visual Description

For bedrock tested by uniaxial compressive strength testing (or other laboratory rock strength testing) that directly determines the UCS, use the laboratory determined value. For rock sampled by bedrock coring, and visually described by a geologist (but not laboratory strength tested), estimate Qu according to Table 400-6.

Table 400-6: Estimated Bedrock Strength Characteristics from Visual Description

Geologic Strength Descriptor	Q_u (psi)	E_i (psi)	Historic Hardness Descriptor (used prior to 2007)
Very Weak	200	18000	Very Soft
Very Weak to Weak	360	32000	Very Soft to Soft
Weak	750	68000	Soft
Weak to Slightly Strong	1125	100000	Soft to Medium
Slightly Strong	1500	140000	Medium / Medium-Firm
Slightly Strong to Moderately Strong	2250	200000	Medium to Moderately Hard
Moderately Strong	3600	320000	Moderately Hard
Moderately Strong to Strong	5000	450000	Moderately Hard to Hard
Strong	7500	680000	Hard
Strong to Very Strong	10000	900000	Hard to Very Hard
Very Strong	15000	1400000	Very Hard
Very Strong to Extremely Strong	20000	1800000	
Extremely Strong	30000	2700000	

406.2 Point Load Test

To obtain the unconfined compressive strength (UCS) from point load testing (PLT) a conversion factor is commonly used; UCS = conversion factor * I_s, where I_s = uncorrected point load strength index. ODOT research (Shakoor and Admassu, 2010) has found that for Ohio bedrock use a conversion factor of 24 for competent rocks (sandstones and limestones) and 12 for incompetent rocks (shales and claystones). Weaker non-durable rocks obtain less accurate compressive strength values based on point load testing.

SECTION 500 EMBANKMENTS

501 GENERAL

Roadways are often constructed on embankment fills (fill) to meet the vertical alignment requirements of the roadway. New roadway alignments are less common these days, while roadway reconstruction, widening and realignment are more common. For a project requiring new embankment construction, the embankment design focus will be on:

- the highest embankment fill sections, measured vertically from the same point;
- sidehill fills on existing ground that is steeper than 4(H):1(V), measured transverse to the proposed alignment;
- approach embankment fills, placed to accommodate the vertical profile as the roadway approaches a bridge;
- fills over weak and/or compressible foundation soils, including fills over existing drainages and waterways.

On roadway reconstruction, widening, and/or realignment projects, be aware of and consider any phased construction of embankments and determine how the embankment will influence adjacent new and existing infrastructure. Where new embankment will be placed overtop existing ditches, include quantities and details in the plans for an excavation of the ditch bottom to a nominal depth of 2.0 feet for the width of the ditch, unless the exploration justifies a deeper excavation, or the ditch bottom is composed of bedrock.

Determine the settlement and stability of a new embankment. For smaller projects with fill heights less than 10 feet, side-slope angles 2:1 or flatter, and consistently favorable foundation conditions, the designer should be able to predict that the planned embankment will be stable with acceptable amounts of settlement, without performing detailed calculations. Favorable foundation conditions include granular soils, non-organic cohesive soils with a Liquidity Index (LI) equal to or less than 0.7, and bedrock. For taller embankments with 2:1 side slopes, approach embankments, and embankments built on soft and/or compressible soils, perform detailed calculations to determine stability and settlement of representative sections and include in the geotechnical report.

Perform all exploration for embankment design in accordance with the requirements of the SGE. The depth of a boring drilled for embankment design is a function of the planned embankment height, so be sure to determine planned and maximum project embankment heights before drilling. Perform offset borings at critical embankment sections to develop a foundation profile (cross-section) for analysis. Perform additional exploration to delineate soft and/or compressible foundation soils, as necessary. Perform undisturbed sampling and laboratory strength and consolidation testing to adequately characterize foundation soils, especially where design recommendations include remediation of inadequate stability or excessive settlement.

502 EMBANKMENT STABILITY

Evaluate new embankments for stability. Stability problems most often occur when the embankment is built on: soft soils such as low strength clay or silt; peat; sloping ground; or residual/colluvial soils overlying shallow sloping bedrock. Once the geotechnical profile, soil strengths, and depth of groundwater table have been determined by field explorations and laboratory testing, analyze the stability of the embankment using a limit equilibrium method to estimate a factor of safety (FS) against failure. The embankment is considered to be stable if the estimated FS meets or exceeds the values presented in Table 500-1 for the corresponding conditions:

Table 500-1: Minimum Embankment Factor of Safety

FS (≥)	Embankment Analysis Condition
1.3	Short term (undrained) condition
1.3	Long term (drained) condition
1.1	Rapid drawdown, flood condition
1.5	Embankment containing or supporting a structural element

With regards to LRFD design, the global geotechnical stability of soil slopes is termed "overall stability," and is a Strength I Limit State condition. However, factored loads are incompatible with limit equilibrium analysis, therefore all load factors are 1.00, and resistance factors are the inverse of the governing FS. The codification of LRFD resistance factors by probabilistic calibrations for the design of slopes is being researched, but commercial slope stability analysis programs compatible with AASHTO LRFD procedures are not readily available. Therefore, in accordance with AASHTO LRFD specifications and FHWA publications, overall stability analysis is still performed by Allowable Stress Design (ASD) methods, utilizing FS, and this guidance expresses stability with regards to FS. However, in order to agree with LRFD terminology, stability of slopes should be expressed in terms of Capacity to Demand Ratio (CDR), where CDR is equal to the ratio of the factored resistance to the factored load. In ASD terms, when the calculated FS equals the minimum required FS, the CDR = 1.0.

A new embankment constructed in accordance with the Construction and Material Specifications (C&MS) is assumed to be internally stable. The standard embankment side slope angle is 2:1 or flatter; do not design or designate an unreinforced soil embankment with a side slope steeper than 2:1. If the side slope angle must be steeper than 2:1, such as to avoid right-of-way encroachment, design it using rock fill to a slope angle no steeper than 1.5:1, or design a reinforced soil slope no steeper than 1:1.

The Contractor is responsible for providing fill material that meets the C&MS Item 203 material requirements for construction of embankments unless otherwise specified in the plans. On rare occasions, the Department will specify fill material to be used to construct embankments. If a project consists of cuts and fills, ideally a balanced site, assume the embankment fill soils to be the same as the cut soils on the project for analysis purposes. If a project requires borrow to construct the fills, assume the embankment fill soils will be the same as the most prevalent soil type encountered on the project. The designer will need to assume the new embankment fill properties to analyze stability and settlement. Unless the embankment fill will be identified in the plans, determine the most prevalent soil classification in the project area and assume the corresponding properties for new embankment fill from Table 500-2.

Table 500-2: Assumed Embankment Fill Properties

Borrow Source Soil Class	**c (psf)**	**φ (deg)**	**c′ (psf)**	**φ′ (deg)**	**γ (pcf)**
Granular	0	32	0	32	125
A-4a/A-4b	2000	0	200	30	125
A-6a	2500	0	250	28	125
A-6b	2500	0	250	28	125
A-7-6	2000	0	200	26	125
Unknown	2500	0	250	26	125

Internal embankment stability problems generally result from poor-quality embankment materials and/or improper placement of the embankment fills. The infinite slope failure mode is an example of an internal stability problem. Based on the subsurface exploration, determine if poor-quality embankment soils are prevalent in the project area, such as highly plastic clays (in northwest Ohio), or "red bed" claystones (in south and east Ohio). Design embankments to ensure stability, considering weaker embankment soils as appropriate. Design adjustments will typically consist of flatter slopes and/or incorporating berms.

502.1 Improving Embankment Stability

If the embankment stability analysis of the original embankment configuration and location results in an FS less than the corresponding minimum value presented in Table 500-1, modify the design to ensure stability. There are usually several feasible solutions to a stability issue, however, design the most economical and feasible solution considering the following:

- Line and grade requirements
- Right-of-way constraints
- Construction time schedules and phasing plans
- Available materials
- Quantity and cost of materials

Consider the least expensive and least complex solutions first before progressing to more expensive and more complex solutions. A list of stability solutions to consider, generally in order of preference, is presented in Table 500-3:

Table 500-3: Options for Improving Embankment Stability

Bench sloping foundation soils	If the existing ground is sloping, creating a potentially unstable sidehill fill situation, bench the existing slope in accordance with the guidance presented in Section 800
Slow rate or staged construction	Conditions at the completion of full height embankment construction are the most critical. Reducing the rate of construction or building the embankment in partial heights (stages) with waiting periods will allow weak foundation soils to consolidate and gain strength. Consider wick drains to accelerate consolidation and strength gain of foundation soils.
Flatten side slopes	Incrementally consider side slopes flatter than 2:1 in ¼:1 increments if sufficient right-of-way and material supplies allow
Counterweight berm	Consider a counterweight berm (counterberm) built integral to the embankment to serve as a resisting moment to increase stability
Lower the roadway profile	Reduce the fill height and thus, the load, by lowering the fill profile.
Move the roadway alignment	If possible, move the roadway alignment away from soft foundation soils or sidehill fills.
Excavate soft foundation soils	Either partially or completely excavate soft foundation soils; consider a shear key. Must account for the potential of lateral squeeze.
Lightweight embankment	Reduce the driving force.
Ground Modification	Refer to FHWA GEC13, Ground Modification Methods, Volumes 1 and 2, and The Geoinstitute's Geotech Tools (https://www.geoinstitute.org/geotechtools/login) for determining appropriate ground improvement methods to ensure stability.
Bridge	In the most extreme cases, it may be necessary to span over extremely deep weak soils.

502.1.1 Slow Rate or Staged Construction

Embankment loading of a weak foundation soil may cause an excessive increase in pore water pressure resulting in instability. By reducing the rate of embankment construction, the increase in pore water pressure can be controlled. Weak foundation soils tend to gain strength during the loading process as consolidation occurs and pore water pressures dissipate. By building the embankment in partial height increments, or staging the embankment construction, the excess pore pressures will dissipate during waiting periods, and the foundation soils will consolidate and gain strength to resist failure under the full load of the embankment.

Use the SHANSEP method (Stress History And Normalized Soil Engineering Properties) developed by Ladd et al, to predict the consolidated strength of the foundation soils under the partial and full loads of the embankment. In the SHANSEP equation (shown below), use S = 0.22 and m = 0.8.

$S_u/S'_v = S(OCR)^m$

where S_u = undrained shear strength, S'_v = effective vertical stress, and OCR = over consolidation ratio.

Use the following design approach when considering slow rate or staged construction:

1) Through stability analysis, identify if using typical construction processes to construct the embankment to the plan dimensions will cause instability. Identify if instability is caused primarily by weak foundation soils (unstable without an elevated pore pressure) or temporary elevated pore water pressures (represented by an elevated piezometric head).

2) If an elevated pore pressure is the primary cause of instability, consider a slow rate of construction. Identify the critical pore pressure at which instability will occur and develop a plan note describing how to monitor the pore pressure and ensure the critical pore pressure is not exceeded. Include an action plan should the critical pore pressure be exceeded.

3) If weak soils are the primary cause of instability, consider staged construction. Identify the heights and limits of the stages, the length of the waiting periods, and construction monitoring required to ensure stable construction and service.

Proper instrumentation is desirable to monitor the state of stress in the soil during the loading period to ensure that loading does not proceed so rapidly that a shear failure occurs. Typical instrumentation consists of slope inclinometers to monitor stability, piezometers to measure excess pore water pressure, and settlement platforms to measure the amount and rate of settlement. Develop a plan of instrumentation, including monitoring depths, locations, and types of instruments. Identify threshold values and an action plan if threshold values are exceeded.

For soils that consolidate relatively fast, such as sand, sandy silts, and some silts, this method is practical by itself. Perform an adequate amount of consolidation testing to determine the time rate of settlement as well as the magnitude of total settlement. If the time required for embankment construction and waiting period(s) is excessive for the project, consider adding a surcharge load, if the foundation soils can support it, or designing and specifying prefabricated vertical drains (wick drains), in accordance with Report No. FHWA/RD-86/168, titled "Prefabricated Vertical Drains" in order to reduce the time required.

502.1.2 Excavate Soft Foundation Soils

Consider partially or completely excavating the soft foundation soils causing the instability. Ideally, if the soft soils are relatively shallow, they will be excavated in their entirety and replaced with Item 203 Embankment. However, if the soft soils are deep, consider partial excavation, either in a layer over the entire foundation or a key along the edge of embankment, and replacement with rock. Verify through stability analysis that the planned excavation limits and replacement material will result in a stable embankment. Identify the limits of excavation, replacement material, quantities, and construction sequence, if any, in the plans.

Soft foundation soils left in place and adjacent soft soils may cause a concern for lateral squeeze. The lateral squeeze phenomenon is due to an unbalanced load at the surface of the soft soil. The lateral squeeze behavior may be of two types, (a) short-term undrained deformation that results from a local bearing capacity type of deformation, or (b) long-term drained, creep-type deformation. Creep refers to the slow deformation of soils under sustained loads over extended periods of time and can occur at stresses well below the shear strength of the soil. This is of particular concern at approach embankments, causing lateral stress on piles. Refer to FHWA NHI-06-088 Soils and Foundations, Volume 1, Section 7.6 for evaluation of the potential for lateral squeeze.

502.1.3 Ground Modification

Ground modification (improvement) is defined as the alteration of site foundation conditions or project earth structures to provide better performance under design and/or operational loading conditions. Ground modification objectives can be achieved using a large variety of geotechnical construction methods or technologies that alter and improve poor ground conditions where traditional over-excavation and replacement is not feasible for environmental, technical or economic reasons. Ground modification has one or more of the following primary functions:

- increase shear strength and bearing resistance,
- increase density,
- decrease permeability,
- control deformations (settlement, heave, distortions),
- improve drainage,
- accelerate consolidation,
- decrease imposed loads,
- provide lateral stability,
- transfer embankment loads to more competent subsurface layers.

When considering ground modification refer to Publication No. FHWA-NHI-16-027 (FHWA GEC 13), "Ground Modification Methods Reference Manual" and the web-based *GeoTechTools* (https://www.geoinstitute.org/geotechtools/register) to determine the most appropriate ground modification method or methods based on applicability for the prevailing subsurface conditions, construction considerations, and costs. When one ground modification method is preferred or is determined to be the most appropriate, identify it in the plans and develop method-based specifications. When multiple methods are considered appropriate and acceptable, identify them in the plans and develop performance-based specifications. In every case, clearly identify:

- the limits of the ground modification;
- all required instrumentation with locations, reading schedules, threshold values, and action plans;

- performance requirements, such as limiting the total settlement prior to paving to a certain value, or increasing the undrained shear strength of the foundation soils to a certain value as determined by a certain testing method;

- how the contractor is to measure, test, and prove performance requirements have been met.

502.1.4 Chemically Stabilized Embankment
Chemically stabilized embankment construction performed in accordance with C&MS Item 205 is most commonly used to repair shallow landslide failures in clay soils. Mixing the soil with either cement or lime will increase the strength, reduce plasticity, and improve resistance to weathering effects. If embankment construction during late fall or early spring is required, the heat of hydration will resist freezing as long as the stabilized soil is covered quickly, and chemical drying will occur when conditions are not favorable for natural drying of wet soils.

502.1.5 Lightweight Embankment
Lightweight embankment fill can both improve embankment stability by reducing driving forces and reduce consolidation (settlement) of compressible foundation soils. Lightweight fills are most appropriate where the construction schedule will not allow for waiting periods using conventional fill or where underlying utilities or structures can tolerate little or no total settlement or additional load. Lightweight fills are substantially more expensive than conventional fill, therefore, a cost analysis will be necessary to justify their use. When designing lightweight embankments, identify the type, spatial limits, material property requirements, especially maximum unit weight, and construction procedures and sequence if necessary. The most common lightweight fills used on ODOT projects are presented here.

a) Expanded Polystyrene (EPS) Geofoam

Geofoam is approximately 1% of the weight of traditional fill and therefore, very effective in reducing driving forces and settlement loads. Typical geofoam embankment fills consist of a sand leveling course, geofoam blocks, membrane cover for separation and solvency (gasoline) protection, pavement system for proper load transfer to the geofoam, and side slope cover embankment. Geofoam is an excellent insulator, and therefore, may cause differential icing of the overlying pavement if placed above the frost depth. Do not leave geofoam exposed to UV light in service. Do not use geofoam where the groundwater table may rise and cause buoyancy issues. Design guidelines for geofoam embankments are provided in the NCHRP document titled, "Geofoam Applications in the Design and Construction of Highway Embankments" (Stark et al., 2004), developed as part of NCHRP Project 24-11.

b) Lightweight Aggregates

The most common lightweight aggregate used on ODOT projects is expanded shale. Expanded shale consists of an inert mineral aggregate that has a similar or greater shear strength to conventional fill but weighs roughly half as much. The primary disadvantage with expanded shale is that it is not readily available in Ohio.

c) Low Density Cellular Concrete Fill

Low density cellular concrete fill (LDCCF) is a mixture of portland cement and water slurry, combined with preformed foam to create air voids and produce a very lightweight porous concrete that can be poured in place. Typical unit weights range from 20 to 50 pcf. Since it is poured into place, it is well suited for void filling and confined spaces. Mass placements, such as for conventional embankments, require forming and placing in lifts. Temporary hydrostatic pressures must be considered in design.

502.2 Reinforced Soil Slope

Reinforced soil slopes (RSS) are commonly used to build embankments to a slope angle steeper than 2:1, but no steeper than 1:1, in order to avoid right-of-way encroachment and possibly to avoid construction of a retaining wall. Design RSS systems in accordance with publications FHWA-NHI-10-024 and FHWA-NHI-10-025, Geotechnical Engineering Circular 11 (GEC 11); and SS863. If the slope angle is steeper than 1.5:1, include a wrapped or shingled erosion control mat. This mat must be stabilized against ultra-violet light and should be inert to naturally occurring soil-born chemicals and bacteria. The erosion control mat serves to:

- protect the bare soil face against erosion until the vegetation is established;
- assist in reducing runoff velocity for increased water absorption by the soil, thus promoting long-term survival of the vegetative cover; and
- reinforce the surficial root system of the vegetative cover.

Include all RSS dimensions, details, and quantities in the plans, including:

- transverse, longitudinal, and vertical limits;
- geogrid (reinforcement) type(s), as listed in SS863, lengths and vertical locations;
- dimensions and locations of any required benching;
- surface treatments, if necessary, to address potential erosion.

503 ESTIMATING EMBANKMENT STRESS DISTRIBUTION

The critical first step for calculating settlement of the embankment foundation soils is to properly determine the stress distribution within the foundation soils due to the pressures imposed by the embankment fill. Stresses induced in the soil from an embankment load are distributed with depth in proportion to the embankment width, and the additional stresses in the soil decrease with depth. Use elasticity-based methods such as Boussinesq Theory to calculate the stress distribution in the foundation soils due to the embankment load. Simplifying charts and tables for typical embankment loading scenarios are readily available, such as in FHWA NHI-06-088, Soils and Foundations – Volume I, Section 7.3.1, and should be used when performing hand calculations. Calculations are typically programmed into a spreadsheet solution or performed using software such as FOSSA 2.0 (Adama Engineering) or Settle3 (Rocscience).

Review the design cross-sections to determine the sections of highest fill which will typically result in the greatest predicted settlement. Also review the Geotechnical Profile to find fill sections with the most compressible foundation soils, such as low-lying wet areas (wetlands), which should also be analyzed even though they may not be the highest fills.

504 SETTLEMENT

Determine the expected primary consolidation settlement of the embankment foundation soils, which may include both immediate and consolidation deformations depending on the type of foundation soils. Only consider settlement over a depth in which the increase in vertical effective stress in the foundation soil is 10% or greater; below this, settlement may be considered negligible. For short embankments (less than 10 feet high) with over consolidated foundation soils, it is acceptable to estimate settlement using empirical correlations with moisture contents and Atterberg values. For taller embankments and compressible foundation soils, use laboratory consolidation tests representing compressible foundation soil layers to calculate total settlement, especially if remedial measures are recommended. Ignore (do not calculate) bedrock settlement. See Section 1303.2.1 for other conditions in which settlement may be considered negligible.

Where a structure, utility, or other roadway infrastructure or adjacent property is not influenced by settlement of the embankment, a predicted total settlement of 3 inches or less is considered reasonable and should not require any corrective action. If predicted settlement is greater than 3 inches the designer will need to predict the effects settlement will have on the roadway. If the effects are considered detrimental to the performance of the roadway, the designer will need to recommend corrective actions to mitigate the settlement.

If the embankment is on top of or immediately adjacent to a structure or utility, determine the deformation effects the embankment will have on the relevant facility. Determine if the predicted deformation will adversely affect the facility and, if so, develop a mitigation plan. Determine the effects settlement may have on a culvert and design appropriate camber to accommodate it.

Wherever a waiting period or settlement mitigation measures are planned, especially for approach embankments, specify settlement platforms or settlement cells and designate their locations. Plan notes and plan details for settlement cells and platforms can be found on the OGE website. Settlement platforms should be placed in an area of maximum fill height but out of the way of construction traffic to avoid being damaged, such as near the crest of the embankment.

504.1 Time Rate of Settlement

Estimate the time rate of settlement using the coefficient of consolidation (C_v) obtained from consolidation testing. Since the time for 100 percent consolidation to occur is theoretically infinite, the time for 90 percent consolidation is usually considered the total time for primary settlement and the target settlement to achieve. If the time to achieve 90 percent consolidation is excessive, consider wick drains and/or surcharge loading to accelerate total settlement. Determine the wick drain layout, spacing, and tip elevation(s). Designate settlement platforms or cells with a reading schedule for measurement of settlement. Include the ODOT Special Provision – Wick Drains, available on the OGE website, in the plans.

504.2 Secondary Settlement

Secondary compression settlement occurs after the completion of primary consolidation settlement, and typically affects highly plastic (PI > 20) or organic (percent organics greater than 30%) clay-like soils. If such soils exist, predict the anticipated secondary settlement. In most cases, secondary settlement will not require mitigation, however, consider base (geosynthetic) reinforcement of the embankment to avoid differential secondary settlement and associated effects on the pavement.

504.3 Differential Settlement

Excessive differential settlement between the approach embankment and an adjacent structure is the most common service issue of the approach embankment. In an attempt to reduce settlements of the approach embankment, specify the placement of approach embankment materials in 6-in lifts. Use the appropriate plan note(s) found in the BDM Section 605 in the plans to control the approach embankment limits, sequence, and rate of construction, and ensure an acceptable amount of differential settlement.

504.4 Internal Embankment Consolidation

Internal consolidation (settlement) within embankments can be controlled by using fill materials that can resist the anticipated loads imposed on them. A well-constructed soil embankment will not excessively deform internally if quality control is exercised regarding material and compaction. Assume embankment consolidation will have a negligible contribution to the overall settlement of the embankment for all embankments less than 50 feet in height and not an approach embankment. For approach embankments greater than 50 feet high, consider embankment consolidation in the overall estimated approach embankment settlement.

SECTION 600 SUBGRADE

601 GENERAL

The guidance in this section originated as Geotechnical Bulletin 1 (GB1), Plan Subgrades. Use it for all projects that include new construction or pavement reconstruction involving pavement replacement, pavement widening, or Rubblize and Roll. The subgrade is the ground surface immediately underlying the proposed pavement with a depth of influence of approximately three feet. The Designer, based on the subsurface exploration, is responsible for identifying the method, location, and dimensions (including depth) of subgrade stabilization in the plans. Appropriate stabilization of the subgrade, if necessary, will ensure a constructible pavement buildup, enhance pavement performance over its life, and help reduce costly extra work change orders. This guidance is simplified so the Designer can easily apply the information from the subsurface exploration to provide reasonable limits and quantities for subgrade stabilization in the plans. However, the Designer must use engineering judgment when applying this guidance to a specific project. Limits and quantities must be verified and adjusted, as necessary, in the field based on proof rolling and visual observation.

602 LABORATORY TESTING

Perform a subsurface exploration according to the current version of the SGE. For Laboratory Testing:

- Perform visual soil classification and moisture content on each sample. Visually inspect each soil sample for the presence of gypsum (CaSO4·2H2O). Gypsum crystals are soft (easily scratched by a knife; they will not scratch a copper penny), translucent (milky) to transparent, and do not have perfect cleavage (do not split into thin sheets). Photos of gypsum crystals are shown in Supplement 1120.

- Perform mechanical soil classification (Plastic Limit (PL), Liquid Limit (LL), and gradation testing) on at least two samples from each boring within 6 feet of the proposed subgrade, preferably the top two samples of the proposed subgrade if the 6 feet is homogenous.

- Determine the sulfate content of at least one sample from each boring within the top 3 feet of the proposed subgrade, per Supplement 1122, Determining Sulfate Content in Soils. Determine the sulfate content of every sample that exhibits gypsum crystals.

- Never VISUALLY classify a soil as A-2-5, A-4b, A-5, A-7-5, A-8a, or A-8b within the top 3 feet of the proposed subgrade.

603 STANDARD PENETRATION TEST (SPT)

The standard penetration test (SPT) measures the number of blows per foot (N) required to drive the sampler through the soil and is an indicator of soil consistency and stiffness. N is corrected to equivalent rod energy of 60 percent (N_{60}). Refer to the SGE for more details. When evaluating the need for stabilization, the project may be evaluated as a whole or divided into segments, depending on the consistency of the soil conditions. Divide the project into segments if there are areas that have significantly lower or higher N_{60} values. To determine the stabilization option and depth, use the lowest N_{60} value (N_{60L}) recorded in the top 6 feet of the proposed subgrade from each boring. Calculate an average N_{60L}, to the nearest whole number, for a group of borings that represent a segment being considered for stabilization. Consider the following when calculating an average N_{60L}:

- When N_{60L} is greater than 30 blows per foot, use 30.
- When N_{60L} is a blow count in bedrock, exclude it.

Where subgrade requiring stabilization is positively identified (i.e., unstable subgrade), designate subgrade stabilization in the plans for those areas.

604 MOISTURE CONTENT (MC)

Comparing the existing moisture content of the soil to the optimum moisture content is an indicator of the need for subgrade stabilization. Estimated optimum moisture content for each soil classification is listed in Table 600-1. Some estimated optimum moisture contents are based on the PL of the sample. Where the optimum moisture content is calculated, minimum optimum moisture content has been established.

Table 600-1: Optimum Moisture Content

Soil Classification	Moisture Content	
	Optimum	Minimum Optimum
A-1	6	
A-3	8	
A-2	10	
A-4a	PL - 5	10
A-4b	PL - 5	10
A-6a	PL - 5	14
A-6b	PL - 5	16
A-7-6	PL - 3	18
Non-Plastic Silt	11	

Moisture contents that exceed the estimated optimum moisture content by more than 3 percent likely indicate the presence of unstable subgrade and may require some form of subgrade stabilization. Functioning drainage can reduce the subgrade soil moisture content. Therefore:

- For new construction projects, installation of construction underdrains as soon as possible is very important.
- For rehabilitation projects, the District should inspect and reestablish drainage as necessary as soon as possible and maintain this drainage until the project is sold. This will improve

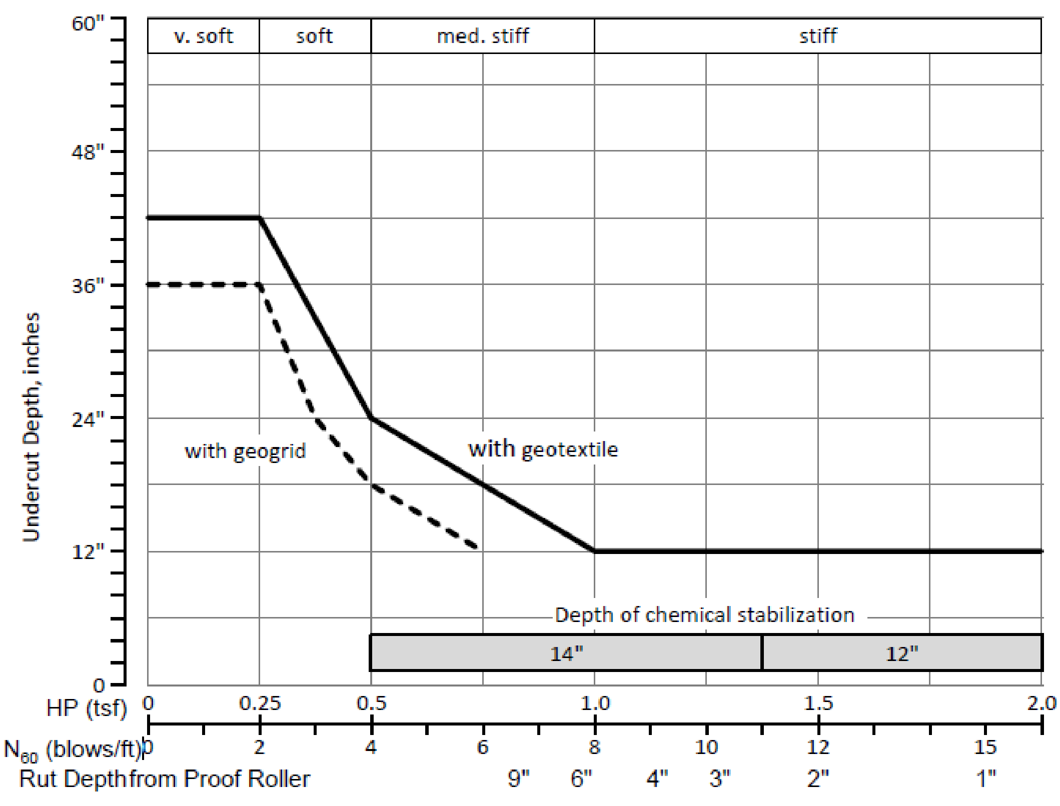
the subgrade soil conditions. The Contractor should maintain the drainage during the project. As a minimum, reestablishing drainage should include cleaning all the underdrain outlets.

- Consider installing new underdrain systems in advance of or at the start of rehabilitation projects.

605 DESIGNING SUBGRADE STABILIZATION

Currently the Department uses two options for establishing a stable subgrade, excavate and replace (also known as undercutting) or chemical stabilization. Figure 600-1 can be used to select the stabilization option and estimate the depth. For project subgrade design, use the Subgrade Analysis Spreadsheet. More information on each option is provided throughout the remainder of this Section and in the appropriate specification. The figure assumes uniform soil conditions at the bottom of the stabilization.

Figure 600-1: Subgrade Stabilization



- For N_{60L} greater than or equal to 12 and less than 15, no stabilization is necessary unless MC is greater than optimum plus 3 percent.
- This figure does not apply to A-1-a, A-1-b, A-3, or A-3a soils and soils with an N_{60L} of 15 or more. These soils should be reworked to stabilize the subgrade.

For all Interstates and other divided highways with four or more lanes more than 1-mile in project length, chemically stabilize the subgrade for the entire project (global stabilization). Where it is determined that soil is present where a majority of sulfate content values are found to be greater than 3,000 parts per million (ppm), or individual soil samples with sulfate contents greater than 5,000 ppm are present, contact the District Geotechnical Engineer to discuss options including stabilization as needed using excavate and replace methods.

For all other roadways, if it is determined that 30 percent or more of the subgrade area must be stabilized, consider stabilizing the entire project (global stabilization). This consideration should include a cost analysis of the options. Use bid tabs to generate this cost analysis.

Generally, chemical stabilization is more economical when stabilizing large areas (approximately greater than 1 mile of roadway) but should not be eliminated from consideration when remediating smaller areas. An exception to this may be segmented areas requiring multiple mobilizations of specialized equipment. It is noted that chemical stabilization may not work in very weak soils (N_{60L} less than 4 blows per foot), soils high in sulfates (greater than 5,000 ppm), and organic soils (A-8a or A-8b).

When choosing the method of stabilization, consider shallow underground utilities, whether they are active or not. Also consider the maintenance of traffic requirements next to a deep excavation and public access requirements to drives and businesses.

Where it is determined that the sulfate content of the soil is greater than 5,000 ppm do not perform chemical stabilization without prior consultation with the District Geotechnical Engineer to discuss options and risk. Attempting to delineate a high sulfate content area via additional tests is discouraged.

606 DYNAMIC CONE PENETRATION

Dynamic cone penetration (DCP) testing of the subgrade is an acceptable method of exploration in both design and construction. The depth of penetration is typically limited to 40 inches and, ideally, is performed using an automated system. In design, DCP data is used with classification testing and conventional borings to determine a design CBR value and predict subgrade stabilization needs.

Take representative subgrade samples for classification testing at each DCP location. Determine the design CBR value from all the exploration data (DCP and SPT explorations) using the correlation between group index value and CBR presented in Table 600-2.

Table 600-2: Correlation Between Group Index Value and Design CBR

Group Index	Design CBR	Group Index	Design CBR
>17.0	3	2.47 to 3.80	9
14.21 to 17.0	4	1.81 to 2.46	10
11.51 to 14.20	5	1.11 to 1.80	11
8.41 to 11.50	6	0.41-1.10	12
5.61 to 8.40	7	0.0-0.4	13
3.81 to 5.60	8		

The design CBR is calculated by entering the test data into the Subgrade Analysis Spreadsheet. Subgrade stabilization is also determined for SPT borings in this spreadsheet.

ODOT uses the U.S. Army Corps of Engineers' recommended equations for calculating in-situ CBR based on DCP penetration rates. For most soils, use

$CBR = 292/(DCP \times 25.4)^{1.12}$ for DCP in inches/blow.

For soils that predominantly have a $LL \geq 50$, use

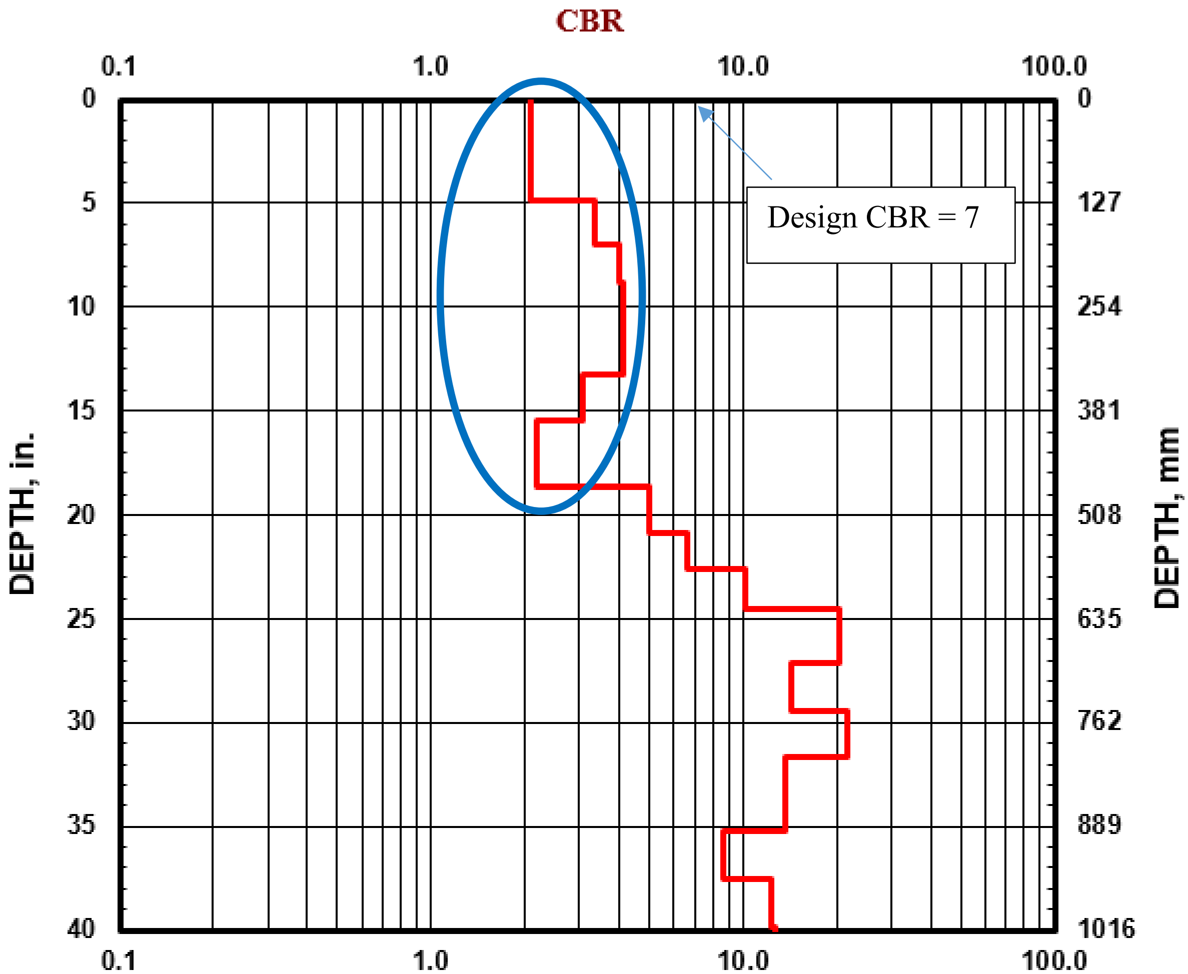
$CBR = 1/(0.072923 \times DCP)$ for DCP in inches/blow.

For cohesive soils with $LL<50$ and $CBR<10$ for the full depth of the sounding, use

$CBR = 1/(0.432283 \times DCP)^2$ for DCP in inches/blow.

Compare the in-situ CBR values at the DCP locations to the design CBR value of the project, and if the in-situ values are less, recommend excavate and replace stabilization methods and limits. See the example in Figure 600-2:

Figure 600-2: In-situ CBR from DCP Data Compared to Design CBR



607 PROOF ROLLING (ITEM 204) AND TEST ROLLING (ITEM 206)

According to C&MS Item 204, the top 12 inches of the subgrade is compacted, and the subgrade is proof rolled to 18 inches beyond the edge of the surface of the pavement, paved shoulders, or paved medians, including under new curbs and gutters.

Specify proof rolling and test rolling as follows:

For global stabilization, using either type of stabilization, the entire subgrade should be stabilized. Do not specify any proof or test rolling prior to the stabilization, since all the subgrade is being stabilized. After global stabilization, specify proof rolling for the stabilized area (entire project) to verify stability is achieved.

For spot stabilization using excavate and replace stabilization, specify proof rolling for the entire project to identify the unstable subgrade requiring stabilization. These locations and quantities may be different than what is shown in the plans. Also specify proof rolling for the planned stabilized areas to verify stability is achieved.

For spot stabilization using chemical stabilization, specify test rolling for the entire project to identify the unstable subgrade requiring stabilization. These locations and quantities may be different than what is shown in the plans. Specify proof rolling for the planned stabilized areas to verify stability is achieved. Spot chemical stabilization is discouraged by the Department. If considering this method, please contact the District Geotechnical Engineer.

An estimated quantity for Item 204 Proof Rolling or Item 206 Test Rolling should be determined as follows:

- Reconstruction: one hour per 2000 square yards of subgrade area
- New Construction: one hour per 3000 square yards of subgrade area

The proof rolling or test rolling deflections and soil conditions that are observed during construction will determine if there is a need to adjust the plan subgrade stabilization. **Adjustment of subgrade stabilization to fit field conditions is essential and is the responsibility of the Project Engineer.** Project Engineers should refer to the Manual of Procedures.

608 EXCAVATE AND REPLACE (ITEM 204)

Estimate the depth and limits of the excavation using Figure 600-1 and the Subgrade Analysis Spreadsheet. Actual depths and limits will be determined by the Project Engineer in the field based on the proof rolling.

An excavation replaced with granular material and underlain with Item 204 Geotextile Fabric can be used in any situation.

Consider replacement using geogrid when N_{60L} is less than 6; to avoid impact on shallow utilities below the subgrade; or to avoid difficult maintenance of traffic situations when using other stabilization methods. Do the following when using geogrid:

- If the replacement is less than or equal to 18 inches, place the geogrid at the bottom of the excavation.
- If the replacement is greater than 18 inches, place the geogrid in the middle of the granular material and a fabric on the bottom of the excavation.
- Use only Item 204 Granular Material Type B or C for the replacement material. Determine if the granular material meets the natural filter criteria for the subgrade as follows:

D_{15} (Type B or C) / D_{85} (subgrade) is less than or equal to 5; and

D_{50} (Type B or C) / D_{50} (subgrade) is less than or equal to 25;

where D_{xx} is the diameter of the soil particle measured in millimeters (mm) for which xx percent of the material is smaller.

As an example, for Item 204 Granular Material Type B the D_{15} is 0.4 mm and the D_{50} is 7 mm, so:

$0.4/D_{85}$ (subgrade) ≤ 5, so the D_{85} (subgrade) must be greater than or equal to 0.08 mm; and

$7/D_{50}$ (subgrade) ≤ 25, so the D_{50} (subgrade) must be greater than or equal to 0.28 mm

Use average gradation values for the subgrade. If both criteria are met, no fabric is necessary. Otherwise, include a fabric at the bottom of the excavation.

If not using Geogrid, replacement material will be Item 204 Granular Material Type ___ considering the following:

- Types B, C and D are all well-graded materials. Type B has a top size of 2 inches. Type C has a top size of 3 inches. Type D has a top size of 8 inches. The larger top size material will bridge unstable subgrade better than the smaller material.

- Use Type D or E when water levels are high and cannot be drained. Always choke the Type D or E with Type B or geotextile.

- Underdrains cannot be placed through Types D, E, or F or the geotextile or geogrid, without great difficulty. Use Type B or C, with no geotextile or geogrid, in the areas of the underdrains.

For severe conditions, including where a 12-inch excavation is not feasible, consider a cellular confinement system (geocell) or a manufacturer designed geosynthetic that exceeds the requirements of 712.09 or 712.15. If using Item 204 Special - Geocell, Subgrade, specify the depth of the system and the infill material, consisting of Item 411 Stabilized Crushed Aggregate. When considering use of a cellular confinement system or manufacturer designed geosynthetic, contact the District Geotechnical Engineer.

Excavate unstable subgrades to 18 inches beyond the edge of the surface of the pavement, paved shoulders, or paved medians, including under new curbs and gutters. Always drain the excavation to an underdrain, catch basin, or pipe. From L&D3, include plan note G121 in the plans.

609 CHEMICALLY STABILIZED SUBGRADE (ITEM 206)

The Designer, based on engineering judgment, should specify the chemical used to chemically stabilize the subgrade as follows:

- Cement may be used to stabilize unstable subgrades which have a Plasticity Index (PI) of 20 or less.

- Lime may be used to stabilize unstable subgrades which have a PI of 16 or greater, consisting of A-6b, A-7-5, or A-7-6 soils.

Chemical stabilization is not recommended for soils with an N_{60L} less than 4 because it is usually difficult for the stabilization equipment to operate on such soft soils.

Do not perform chemical stabilization if it is determined that soil is present with a sulfate content greater than 5,000 ppm.

When global chemical stabilization includes areas where shallow bedrock is anticipated to be encountered during the stabilization process, excavate bedrock to a depth of 18 inches below the top of proposed subgrade. Replace the excavation with Item 204 Embankment and then proceed with chemical stabilization. The Item 204 Embankment will need to be tested for sulfate content using the area/volume guidance in Supplement 1120.03.

Chemically stabilize subgrades to 18 inches beyond the edge of the surface of the pavement, paved shoulders, or paved medians, including under new curbs and gutters. To estimate the quantity of chemical, use the estimated rate and quantity formula from Table 600-3 for the specified chemical. When performing chemical stabilization design, use the dry density of the soil on the project as determined in the laboratory.

Table 600-3: Chemical Stabilization Rate and Quantity

Chemical	Estimated Rate (1)	Quantity Formula (2)
Cement	5 percent	C = 0.75 x T x 115 x 0.05
Lime	5 percent	C = 0.75 x T x 115 x 0.05

(1) By dry density of soil (using 115 pounds/cubic foot)
(2) Where: C = amount of chemical (pounds/square yard)
T = thickness of stabilization (inches)

Along with the pay items for chemical stabilization and the chemical, provide the following additional pay items for chemical stabilization:

- Item 206 Curing Coat, estimated at the same number of square yards as the stabilized area.
- Item 204 Proof Rolling, estimated in accordance with Section 607, using the same number of square yards as the stabilized area.
- Item 206 Mixture Design for Chemically Stabilized Soil is a lump sum item and requires the chemical stabilization to be designed according to Supplement 1120. This item should only be specified on projects with more than 40,000 square yards of chemically stabilized area.

Do not provide Item 204 Subgrade Compaction for areas that are being chemically stabilized.

When chemical stabilization is to be used on a project with multiple maintenance of traffic phases, coordinate the roadway work with the maintenance of traffic schemes such that an 8-foot minimum width for chemical stabilization exists. Typical chemical stabilization equipment cannot stabilize areas less than 8 feet in width. Small areas of less than 8 feet in width can be excavated out, mixed with the stabilization chemical, and compacted in place. However, excavation and mixing are not practical for large areas.

610 UNSUITABLE SUBGRADE

610.1 A-4b

When A-4b soil is encountered in natural ground or an existing embankment within 3 feet of the subgrade, regardless of its consistency or moisture content, remove or chemically stabilize because of its susceptibility to frost heaving. When constructing an embankment, the use of A-4b soils within 3 feet of the subgrade is prohibited per 203.03.

When excavating and replacing, A-4b should be excavated to 36 inches below proposed subgrade and replaced with Item 204 Embankment. If the subgrade is going to be chemically stabilized to a depth of 14 inches, A-4b soil does not have to be removed.

610.2 A-2-5, A-5, and A-7-5

According to C&MS Item 703.16.A and B, A-2-5, A-5, and A-7-5 shall not be used in a subgrade. A-2-5 soil is unsuitable because of its low weight, high optimum moisture, high LL and low PI and its propensity to sloughing in service. A-5 soil is unsuitable because it is highly elastic as indicated by the high LL. A-7-5 soil has a lower PI in relation to the LL than other clays. It is unsuitable because it may be highly elastic, and subject to considerable volume change.

When excavating and replacing, any A-2-5, A-5, and A-7-5 soils should be completely removed or excavated to 36 inches below proposed subgrade, whichever is less. Replace the excavation with Item 204 Embankment if the entire depth is removed. Otherwise, replace with granular material.

Chemical stabilization may be used to stabilize A-2-5, A-5, or A-7-5 soils. Appropriate laboratory testing should be performed to confirm.

610.3 A-8a and A-8b

A-8a and A-8b classifications are soils otherwise classified as A-4, A-5, A-6, or A-7, which have a liquid limit value after oven drying less than 75 percent of its liquid limit before drying, indicating an effect of the organic content. Excavate and replace is the only acceptable replacement procedure for A-8a and A-8b soils; do not chemically stabilize. If encountered in the subgrade, A-8a and A-8b soils should be completely removed, or excavated to 36 inches below proposed subgrade, whichever is less. Replace the excavation with Item 204 Embankment if the entire depth of this soil type is removed; otherwise, replace with granular material.

610.4 Liquid Limit (LL) > 65

According to C&MS Section 703.16.A, when a soil sample has a LL greater than 65, it shall not be used in an embankment or subgrade. When LL is greater than 65, it indicates a soil of high clay content and low load-carrying capacity.

When excavating and replacing, any material with a LL greater than 65 should be completely removed or excavated to 36 inches below proposed subgrade, whichever is less. Replace the excavation with Item 204 Embankment if the entire depth of this soil type is removed; otherwise, replace with granular material.

Chemical stabilization may be used to stabilize soils with a LL greater than 65 rather than remove them. Appropriate laboratory testing should be performed to confirm.

610.5 Rock, Shale, or Coal

When rock, shale, or coal is encountered within 6 inches of the final subgrade elevation, it is to be removed according to 204.05 and replaced with suitable material conforming to 204.02 Granular Material Type B, with the gradation of 703.17. Remove the rock, shale, or coal to 12 inches beyond the edge of the surface of the pavement, paved shoulders, or paved medians, including under new curbs and gutters.

Replacing an existing pavement with a thicker pavement build-up can necessitate excavation of rock, which is difficult and expensive especially in confined work areas. Designers should try to adjust the pavement design and roadway profile to avoid excavating rock whenever possible.

611 RUBBLIZE AND ROLL (ITEM 320)

For Rubblize and Roll, consider the following:

- The Rubblize and Roll rehabilitation technique is not an option when the average N_{60L} value for the subgrade below the existing pavement is less than 12.

- Rubblize and Roll is not a "piecemeal" rehabilitation technique. It is only considered for the entire project, excluding any sections where vertical grade adjustments are proposed.

- During construction, the Rubblize and Roll is attempted before an area is selected for excavation and replacement. **The actual excavation areas will be selected based on the inability to Rubblize and Roll.**

Estimate excavation quantities based on soil samples within 3 feet of the existing subgrade in accordance with the guidance in this Section. The depth of excavation begins at the top of the existing subgrade. Where Rubblize and Roll is not planned (e.g., where grade is being lowered under a bridge), identify subgrade stabilization in accordance with guidance provided in this Section.

612 REPORT REQUIREMENTS

Prepare either a Geotechnical Design Memorandum or a Roadway Exploration Report according to the requirements of the SGE. The analyses and recommendations need to include, as a minimum, the following:

1) The method(s), locations, and dimensions (including depths) of planned subgrade stabilization. Identify problem subgrade as either unsuitable subgrade or unstable subgrade.

2) An electronic copy of the Subgrade Analysis Spreadsheet.

3) Average N_{60L} calculated to the nearest whole number.

    a) Report the average N_{60L} for the entire project.

b) If the project is broken up in segments, provide an average N_{60L} for each segment considered.

c) If the project involves rehabilitating the existing pavement and widening, also provide an average N_{60L} for the existing pavement and an average N_{60L} for the area where the pavement will be widened.

4) Average PI, calculated to the nearest whole number.

5) Results of all sulfate content tests, performed in accordance with Supplement 1122. Note the reporting requirements in 1122.04 regarding sulfate concentrations greater than 8,000 ppm. Provide electronic copy of sulfate content test results, either as part of gINT project file submittal or as standalone spreadsheet formatted per Supplement 1122.

6) Average design CBR for the entire project. This should be calculated as an average, to the nearest whole number. Do not calculate as a percentile.

613 PLAN REQUIREMENTS

L&D3 Section 1304.15 requires all subgrade treatments (subgrade stabilization) to be shown on the typical sections and cross sections in the plans. The sides of the excavation are typically shown as vertical. Be sure to identify unsuitable and unstable subgrade accordingly. The width of the stabilization should extend to 18 inches beyond the edge of the surface of the pavement, paved shoulders, or paved medians, including under new curbs and gutters. Include all necessary quantities and Plan Notes.

SECTION 700 LANDSLIDE EXPLORATION

701 GENERAL

This section provides guidance for site reconnaissance, survey limits, exploratory drilling, and in-situ and laboratory testing of soil and bedrock for landslide remediation projects. Recommendations for installation of instrumentation, in the form of inclinometer casing and monitoring wells, are also provided. Subsurface exploration is a necessity for the analysis and design of most landslide stabilization solutions. The analyses involved are quite rigorous, and the more data that is acquired; the more precisely the inputs can be estimated, and the more realistic the outputs will be.

702 SITE RECONNAISSANCE

702.1 Office Publication Search

Prior to making a site visit, endeavor to obtain as much knowledge about the geologic setting of the site as possible. A thorough search of various geologic, soil, and water resource publications can yield a large amount of information, which may provide greater insight to the probable causes and form of the landslide failure, and may immediately demonstrate which kinds of remediation options are possible and which kinds are not possible. See SGE Section 302.2, Office Reconnaissance, and the Geotechnical Engineering Design Checklists, Section II, Reconnaissance and Planning Checklist, for details and a list of publications recommended by OGE.

702.2 Historic Geotechnical Data Search

In many cases, a landslide may have been explored in the past, one or more times. In some cases, a historic geotechnical exploration may have resulted in a remediation which subsequently failed. Such data will be helpful for planning a geotechnical exploration and for assessing feasible remediation alternatives. Historic boring logs add to the amount of available subsurface data and mean that fewer new borings may potentially be required. An inventory of historic borings collected by ODOT is available from the Transportation Information Mapping System (TIMS). Also available at this website is an inventory of geohazards (landslides, rock slopes, abandoned underground mines) impacting the transportation infrastructure throughout the state, and an inventory of retaining walls. Past recommended, designed, or constructed remediation schemes may yield data which will be helpful in constructing a subsurface profile or in determining the reason for certain surface features. Full geohazard risk rating information and a history of past remediation work can be requested from the OGE or the DGE.

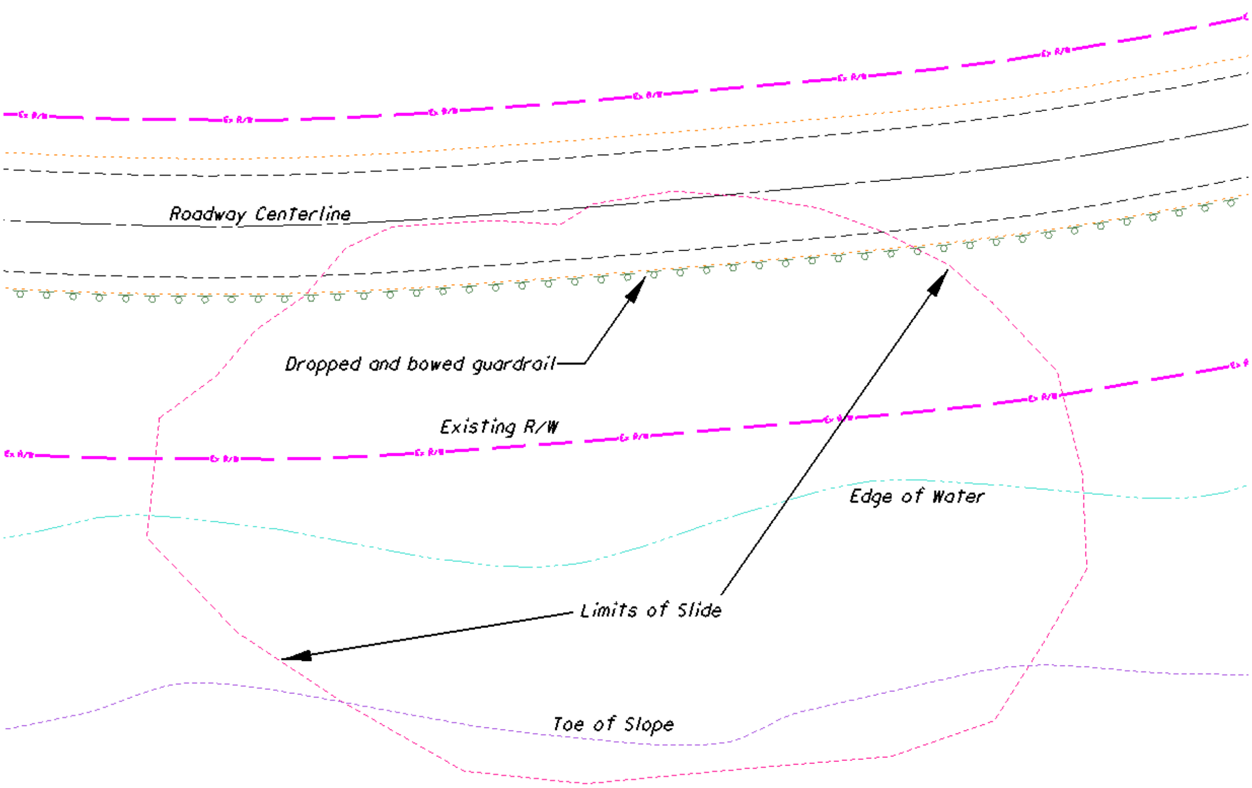
Past project plans, either for the original construction of the roadway, a subsequent reconstruction, or for the construction of a remediation scheme, can also yield useful data. Cross-sections will show both the historic “existing” ground profile, and the historic proposed ground profile, which can be compared to the existing ground line. This can show where soil or bedrock cuts or embankment fills were performed and can help with determining which soils are fill as opposed to natural, virgin soils. Features from historic remediation schemes, such as rock buttresses, shear keys, benched excavations, and rock channel protection (RCP) can yield helpful data for the construction of a subsurface profile.

702.3 Site Visit/Drilling Reconnaissance

Visit the site at least once, in order to gain a full understanding of the existing lay of the ground, evaluate the extent and severity of the landslide, and note surface features relevant to the geotechnical analyses. Include a drilling reconnaissance in the site visit, so that a geotechnical engineer may decide where drilling and installation of instrumentation should be performed, where there are obstacles to drilling access, and where drilling will be impractical. Also use this site visit to estimate the limits of the required site survey, and to make notes of features which the survey should locate.

Make a sketch of the site and note all the surface features which give evidence of the size, type, severity, and causes of the landslide. Figure 700-1 shows an example of a simple sketch of a typical landslide site, located along a river. We acknowledge that the toe of slope and limits of the slide are typically not known under water; however, they are shown in this example for completeness. This example site will be used in Section 900 Design of Drilled Shafts for Landslide Stabilization.

Figure 700-1: Example Landslide Site Sketch



The following is a list of typical landslide features which are often present and should be noted:

- Landslide "Anatomical" Features
    - Head scarp

- Toe bulge
- Tension cracking
- Hummocky ground
- Bowed, leaning, or overturned trees
- Leaning or overturned utility poles

- Erosion features
  - Surface seepage or wet, soft ground
  - Unusually verdant or wetland vegetation
- Roadway features
  - Longitudinal or transverse roadway cracking
  - Dropped or uneven sections of pavement
  - Deformed guardrail
  - Ditches, streams, and rivers, particularly if pinched by ground movement
  - Areas of pavement patching or "drag patching"
  - Apparent limits of roadway cut or fill
  - Rock cut slopes
  - Existing and likely future impact of the slide on the roadway and traffic
- Evidence of past remediation
  - Toe buttress
  - Driven piles, pipes, or posts
  - Retaining wall
  - Rock buttress or RCP
  - Drainage features

702.4 Site Survey

Establish the limits of the site survey during or immediately after the site visit. The sketch of the site is often useful to identify the area and features which the survey should locate, and it may be helpful to provide the surveyor with a copy of this sketch.

The following are minimum guidelines on the limits of the area to capture in the site survey:

- 100 feet beyond the limits of the failure area, in all directions
- 100 feet beyond the toe of the slope
- Both right-of-way limits

Extend the survey beyond these minimums as necessary in order to capture additional relevant features, or to set the site in a more general context. Also, if the slide is on a river bank or at the edge of some other body of water, it is preferable to obtain soundings of the bottom of the water feature, out beyond the "toe of the slope," where the bottom levels off.

Once the site survey is completed, develop a plan and profile, showing land usage and all pertinent topographical and geotechnical features, including right of way lines. Develop cross-sections at 25-foot intervals along the roadway centerline (or at right angles to the movement of the slide, if the slide is not nearly perpendicular to the roadway). These cross-sections will be used to develop a subsurface profile of the soil and bedrock, and to develop models for stability analysis.

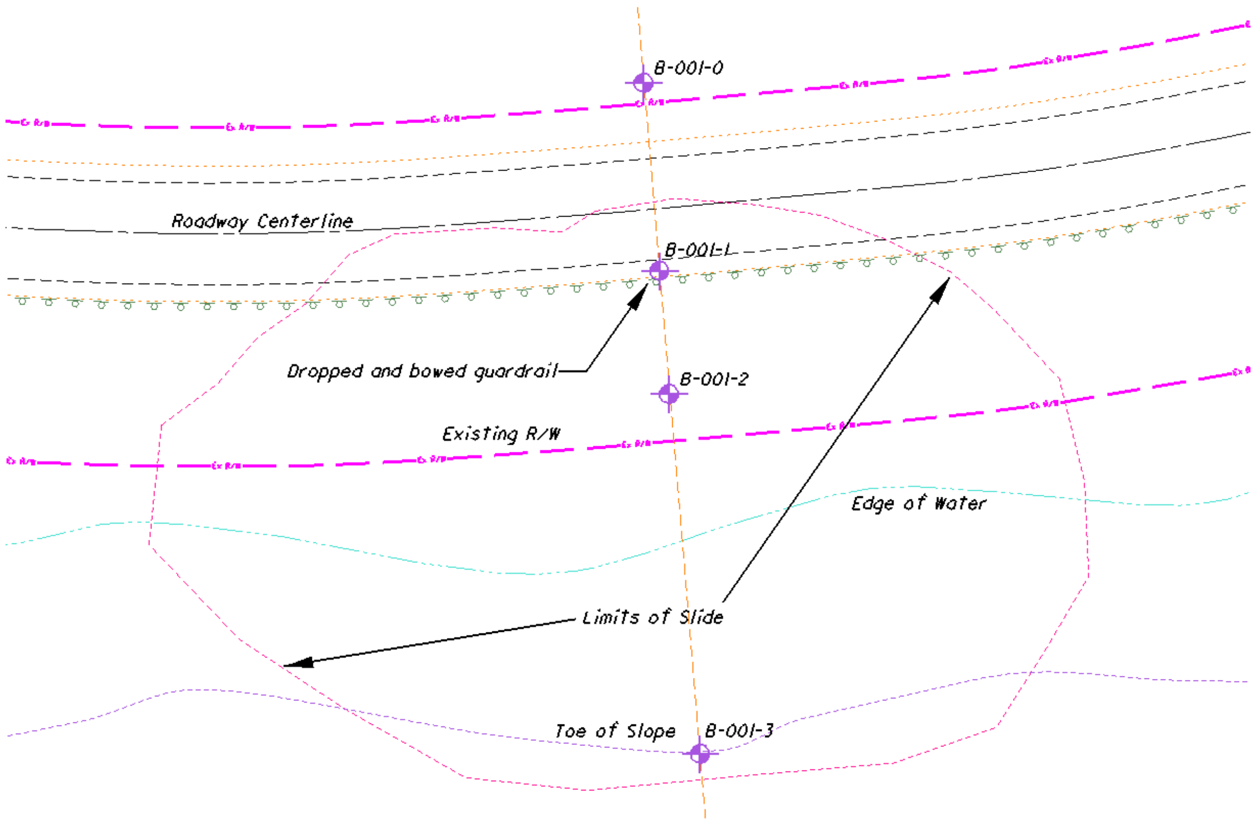
703 SUBSURFACE EXPLORATION PROGRAM

Develop a subsurface exploration program which yields sufficient subsurface data to develop realistic subsurface profiles and models, but which is conservative and within reasonable limits for the available time and money. The historic geotechnical data search may find historic site borings which will yield useful data for planning additional borings or reduce the number of necessary new borings. See SGE Section 303.5.5 for general guidance on planning an exploration program for a landslide geohazard site.

703.1 Primary Borings

The most basic site exploration consists of drilling a single primary line of borings, in a cross-section through the landslide. Perform enough borings such that the structure and slope of the soil strata and the bedrock surface, if necessary, may be determined. Figure 700-2 shows a plan view of the example landslide site again, with a proposed cross-section of borings plotted through the slide. The following paragraphs will discuss the placement of the borings in this example.

Figure 700-2: Example Primary Exploratory Boring Cross-section

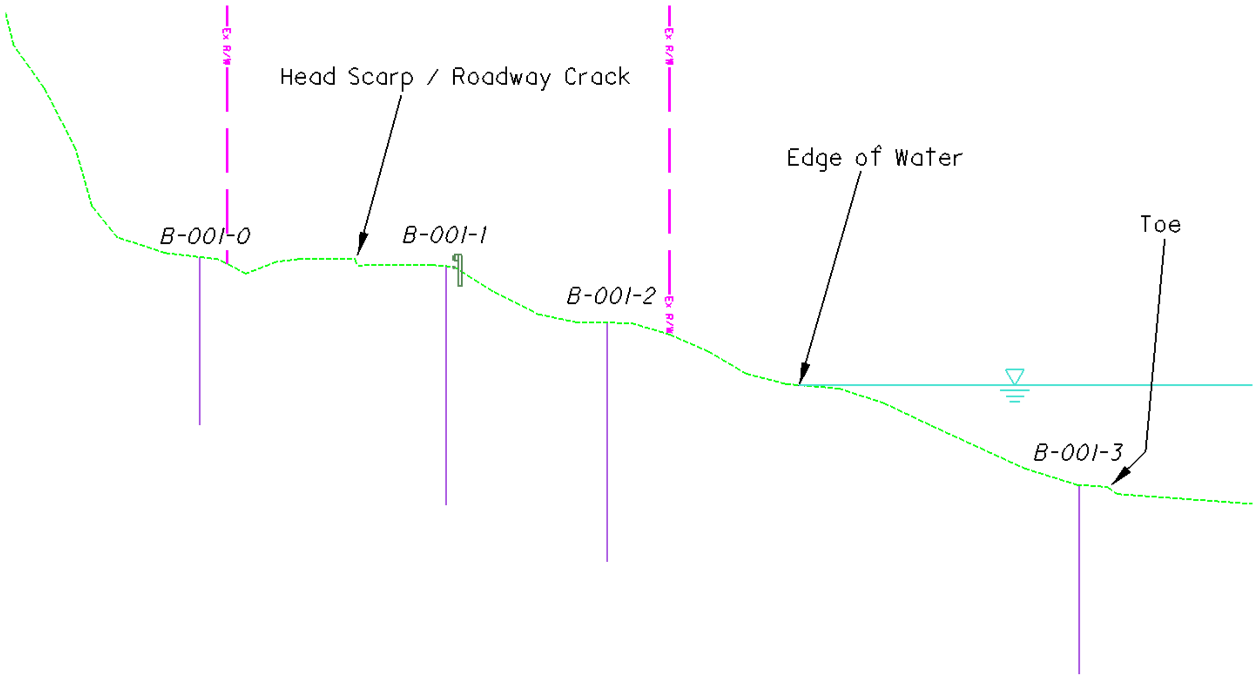


Boring B-001-1 has been placed at the approximate crest of the slope, on the outside shoulder of the road, near the head of the failure. This is an important location for a boring and is sometimes the only location a boring can be obtained, due to access difficulties. Always attempt to put a boring in this location, and to get it within the area of the slide, if at all possible. The further downhill this boring can be obtained, the better, although this could require removing guardrail.

Boring B-001-0 has been placed across the road and uphill from boring B-001-1. Often, the only locations at which borings will be possible are at the approximate locations of borings B-001-0 and B-001-1. The combination of borings B-001-0 and B-001-1 will allow a determination of the slope of the bedrock under the roadway and upper slide area, where it is relevant. If no borings are possible further downhill, the boring data from these two borings and the existing ground surface may need to be used to project and estimate a subsurface profile to the toe of the slide area.

Boring B-001-2 has been placed on the mid-slope. If there is a bench or nearly level area in the middle of the slope which is accessible to a drill rig, attempt a boring in this location. This will extend the knowledge of the subsurface information further down the hill and allow a better projection or estimation of the subsurface profile. If it is possible to cut a path to the mid-slope with a bulldozer, with minimal disturbance to the slope, this may also be attempted. However, use caution with this method. It is likely to introduce further instability into the upper slide area, and quicken or worsen the failure, by cutting too deep of a bench into the side of the existing slope.

Figure 700-3: Example Exploratory Boring Profile in Cross-section



Boring B-001-3 has been placed at the toe of the slope. In this example, the boring is out in the river, drilled into the river bottom. If possible, a boring at the toe of the slope or toe of the slide area is desirable to complete the subsurface profile. However, this is often not possible. Even when the slide is not on the bank of a river, the toe of the slope may be inaccessible to drilling equipment. If the slide extends into a body of water, a boring from a floating platform, or barge, may be attempted at the toe. If a barge is not available, it may be possible to drill a boring at the edge of the water. Regardless, the more subsurface data which can be obtained along the cross-section with the primary borings, the better. The proposed borings for this example are shown in profile view in Figure 700-3.

703.2 Secondary Borings

It is desirable to obtain additional borings up and down-station (transverse to) to the primary boring cross-section. These are helpful to define the limits of the slide area, to further refine the subsurface data, and to define the slope of the bedrock surface transverse to the direction of the slide. A better understanding of the top of bedrock across the site is especially helpful when planning the construction of drilled shafts.

If the landslide is very wide transverse to the direction of movement or is composed of a number of smaller slides along a length of roadway, it is also prudent to drill more borings along additional "primary" boring cross-sections. Each of these cross-sections may be individually analyzed to locate the most critical cross-section or to further refine the remediation design.

703.3 Soil and Bedrock Sampling

When drilling and sampling for subsurface exploration of a landslide, continuous soil sampling should be performed, per SGE Section 303.5, Boring Type C, Geohazard Borings. Perform undisturbed (Shelby tube) sampling whenever soft or very soft cohesive soils are encountered, and at the depth of the shear failure surface, if this is known. If auger refusal or SPT refusal in bedrock is encountered, core and sample the bedrock as required in the SGE.

703.4 In-situ Testing / Instrumentation

Most soil samples will be obtained by split spoon with the Standard Penetration Test (SPT) method, which will give a rough estimate of the soil consistency or density. Also perform a pocket penetrometer reading on cohesive soil samples, in order to obtain a second data point to determine the consistency and unconfined compressive strength. Exploration with Cone Penetrometer Testing (CPT) may also be performed in tandem with the drilling and soil sampling, in order to obtain a better estimate of the soil strength, and often to read the pore water pressure as well. In bedrock, a pressuremeter or dilatometer may be used to obtain an in-situ evaluation of the bedrock strength and stress-strain behavior.

Inclinometer casings are very often installed in landslide exploratory borings in order to determine the depth of the shear failure surface, and to obtain a better estimate of the rate and severity of the shear failure. The most advantageous location at which to install inclinometer casing is at the approximate center of the sliding mass. If the location of the head scarp and toe of the slide are known, determining the depth of failure at the center of the slide will give a good approximation of the shape of the entire failure surface. More than one inclinometer installation along the primary boring cross-section can further refine the data. In the example shown in Figures 700-2 and 700-3, inclinometer installations have been made at boring locations B-001-1 and B-001-2. Inclinometer installations in secondary boring locations or borings outside of the visible slide area are less critical, but may serve to define the size, shape, severity, and limits of the shear failure.

Groundwater monitoring wells or piezometers may also be installed in the landslide exploratory borings to determine the approximate level of the water table. Elevated groundwater is often a major contributing factor in landslide failures and determining the shape and depth of the "static" groundwater surface will aid in modeling the existing conditions. Do not cluster inclinometer installations and groundwater monitoring wells in the same borings. If a side-by-side installation is desired, drill an offset boring for the second instrumentation installation.

Geophysics have also been found useful with a good level of success to determine the top of bedrock, and with some success to delineate the shear failure surface itself. In this event, typically run one or more lines up and down the slope through the failure area and, if applicable, along the proposed alignment of a retaining structure. Survey the slope cross-section to be profiled and determine the elevation of each probe or geophone. Geophysics may not be successful in every situation, particularly if the top of bedrock is relatively deep, there are dense soils above bedrock that make a contrast difficult to determine, or if there is a large amount of background noise that may mask the signal being collected by the particular methodology in use. Geophysical methods should always be supplemented and confirmed by borings performed within the geophysical survey line.

703.5 Laboratory Testing of Soil and Bedrock Samples

The laboratory testing program for a landslide exploration should generally be more rigorous than the programs for either a roadway exploration or a structure foundation exploration. The number of soil samples is typically greater per boring, and additionally, we desire a greater refinement in the soil classification and shear strength determination. Classification testing of the soil aids in determining soil stratification for the subsurface profile. Moisture content testing aids in determining the ground water level. Undisturbed soil samples can be tested for unit weight and shear strength, which aid in determining engineering properties of the soils for the stability analysis modeling.

Subject all soil and bedrock samples to visual classification per SGE Section 602, and group them by similar classifications into preliminary strata. Subject at least one sample per stratum, per boring, to mechanical soil classification per SGE Section 603. Subject all undisturbed soil samples to unit weight testing, and subject samples near the failure surface to shear strength analysis by either unconfined compressive strength, unconsolidated-undrained (UU) triaxial compression, consolidated-undrained (CU) triaxial compression, or direct shear testing per SGE Section 604. Describe all bedrock samples per SGE Section 605. Intact bedrock cores may be subjected to unconfined compressive strength testing per SGE Section 606.

703.6 Boring Logs

Generate boring logs for every boring, per SGE Section 703.3, with visual descriptions of all soil and bedrock strata, and showing all available data from the exploratory borings, in-situ testing, and laboratory testing of the soil and bedrock samples. Also generate a Geotechnical Profile - Landslide, per SGE Section 704, with graphic boring logs, and at least one cross-section view per primary boring cross-section.

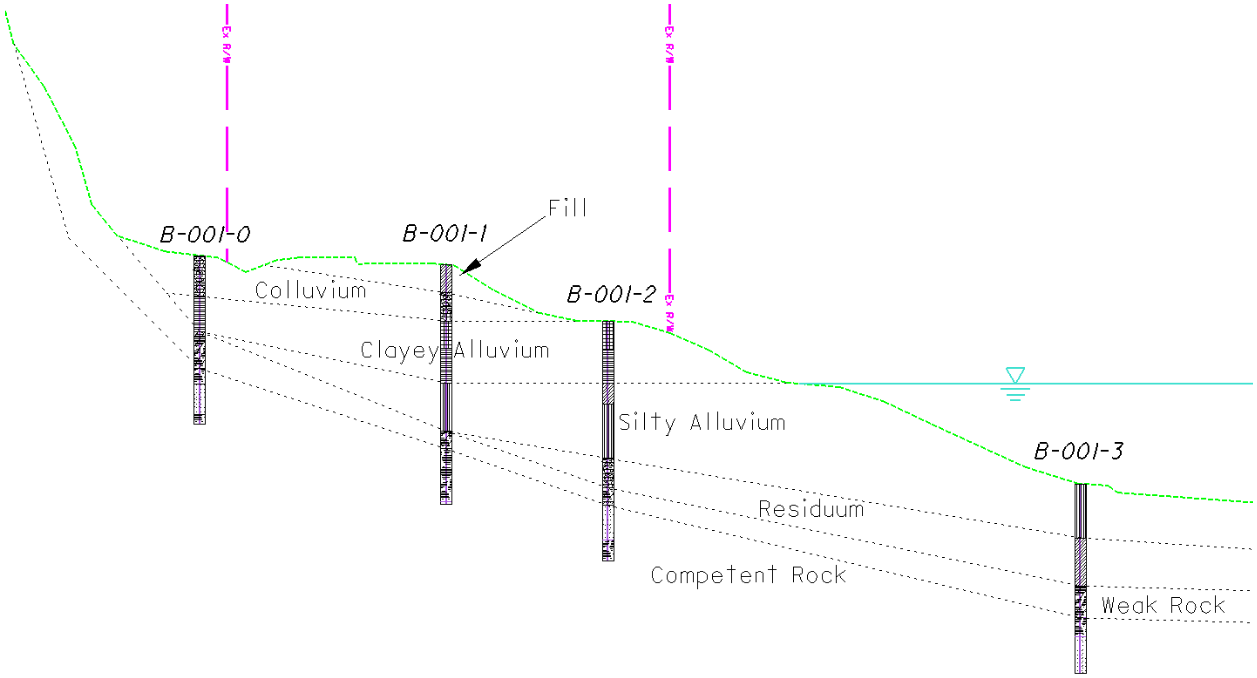
704 STABILITY ANALYSIS MODELING

Generate a subsurface profile of the soil and bedrock for each primary boring cross-section at which modeling and analyses are to be performed. Combine soil and bedrock information from the subsurface exploration and laboratory testing, as shown in the boring logs, with the cross-sections from the site survey and any historic data to build as realistic a representation of the subsurface conditions as possible.

704.1 Identify Soil and Bedrock Layers

Group soil samples across the subsurface profile into a finite number of discrete strata, so that modeling of the subsurface conditions can be performed for stability analyses. Lay the graphical boring logs on top of the site survey cross-section along the axis of the primary borings, or the cross-section views from the Geotechnical Profile - Landslide may be used. Analyze all the soil samples, comparing factors such as visual and mechanical classification, color, plasticity, gradation, SPT blow count, water content, and undisturbed laboratory test results, in order to group these samples into logical units which will define soil strata. Pay attention to the depositional environment of the site, apparent areas of cut and fill, exposed bedrock faces, and past construction plans, to add to the boring data and further aid in identifying the various strata. Figure 700-4 shows the subsurface profile for the example problem introduced in Section 702.

Figure 700-4: Example Soil and Bedrock Profile in Cross-section



If encountered, identify the top of bedrock in each boring across the subsurface profile. Make a distinction between weak bedrock, "competent" bedrock, and strong bedrock. Generally, the top of weak bedrock will correspond with the depth at which SPT blow count refusal (greater than 50 blows per 6") is reached, but where exploratory borings can still be advanced by soil auger. This rock will typically have a relative strength of very weak to weak, with an unconfined compressive strength in the range of 200 psi to 1500 psi. Weak bedrock is often highly weathered or broken, with a low rock quality designation (RQD). Weak bedrock, by this definition, is sometimes also called "Intermediate Geomaterial."

The top of competent bedrock will roughly correspond with the depth at which auger refusal is reached, and at which further bedrock sampling must be done by diamond-tipped core bit. This rock will typically have a relative strength of slightly strong to moderately strong, with an unconfined compressive strength in the range of 1500 psi to 7500 psi. Competent bedrock is often slightly to moderately weathered.

Strong bedrock may be slow and difficult to core, which is important to note for constructability reasons. This rock will typically have a relative strength of strong to extremely strong, with an unconfined compressive strength greater than 7500 psi. This rock is usually unweathered to slightly weathered.

704.2 Estimate Soil Engineering Properties

Estimate the engineering properties of the soil strata in order to model the subsurface profile for stability analyses. Interpret these values directly from the results of undisturbed soil testing or provide estimates through engineering judgment and experience using the results of soil classification testing and SPT blow counts. See Section 400 for guidance on estimating soil strength and unit weight using correlation to SPT blow counts and depth of sample.

704.3 Locate Groundwater Surface

Determine the groundwater surface in the subsurface profile for representation in the stability model. In some instances, complex hydrogeologic conditions may exist, such that there is not one single groundwater table with dry or moist soils above and saturated soils below. However, in most cases, a single groundwater surface may be approximated. In the subsurface, the groundwater surface may be located fairly accurately at single points through long-term observations with groundwater monitoring wells. Short-term observations (made during drilling) are often inaccurate, due to low permeability limiting the rate of water level recharge in the open boring hole, caving of soils from the walls of the open boring hole displacing free water, and the use of drilling fluids. However, short term observations may give a clue about the range of depths at which the groundwater surface lies, and sometimes, fairly accurate observations of the depth at which water was “first encountered” will be made. Water contents of the soil samples may also provide data to estimate the depth to the groundwater surface.

Utilize knowledge of hydrogeology and subsurface flow to connect the groundwater surface between known points. The groundwater surface should intersect with free water at the ground surface, and should slope downwards with a realistic potentiometric surface, generally following the lay of the land. If bedrock is shallow, the ground water surface often coincides with the top of bedrock. Figure 700-5 shows the groundwater surface in the subsurface profile for the example site. It can be seen that the groundwater surface in this example intersects with the free water surface of the river at the bottom of the slope and with the roadway ditch at the top of the slope. Monitoring wells were installed at boring locations B-001-1 and B-001-2 to provide additional known points for the groundwater level. The groundwater surface has been interpolated between all of the known points and estimated to the left of boring B-001-0.

If the landslide was triggered or aggravated by a rapid drawdown event, or if a rapid drawdown event is likely to occur at the site in the future, also perform the analyses for a groundwater surface in the rapid drawdown condition. For example, such an event occurred on the Ohio River, above the Belleville Lock and Dam, in January and February 2005, when runaway barges were caught in the flood gates of the dam. To construct a groundwater surface for a rapid drawdown event, the normal pool elevation, the flood elevation, and the rapid drawdown elevation (if different than the normal pool) of the river should be known. River gaging stations along most major waterways provide a useful historical record of the river level during flood events. If flood data cannot be found, a conservative estimate is to assume the flood water reached the crest of the failing slope. The exact subsurface level of the groundwater cannot be known, unless groundwater monitoring wells were already in place before the flood event, and readings were taken during the flood, but nevertheless, it is possible to approximate a reasonable potentiometric surface, similar to the normal groundwater condition. In this case, however, the groundwater surface will remain nearer to the existing ground surface and will probably meet the ground surface somewhere in the mid slope. The groundwater will typically continue at ground surface level down to the free water level below. Figure 700-6 shows an example of a rapid drawdown condition groundwater surface in the subsurface profile.

Figure 700-5: Example Groundwater Surface Profile in Cross-section

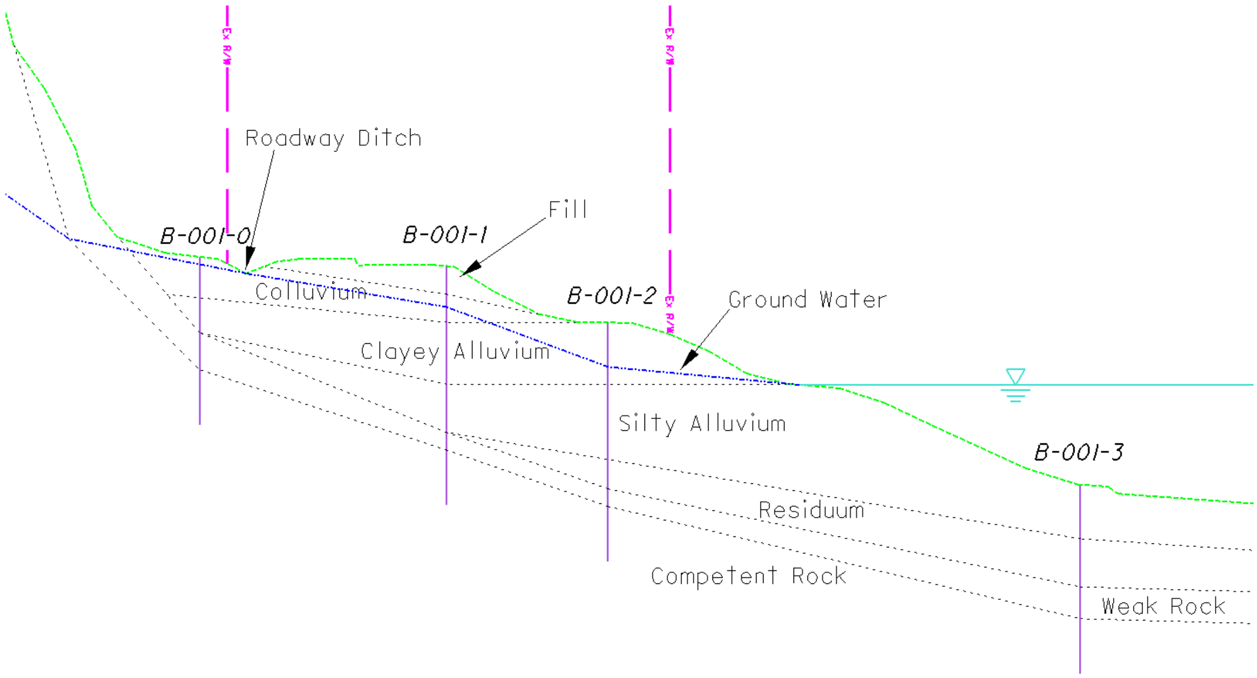
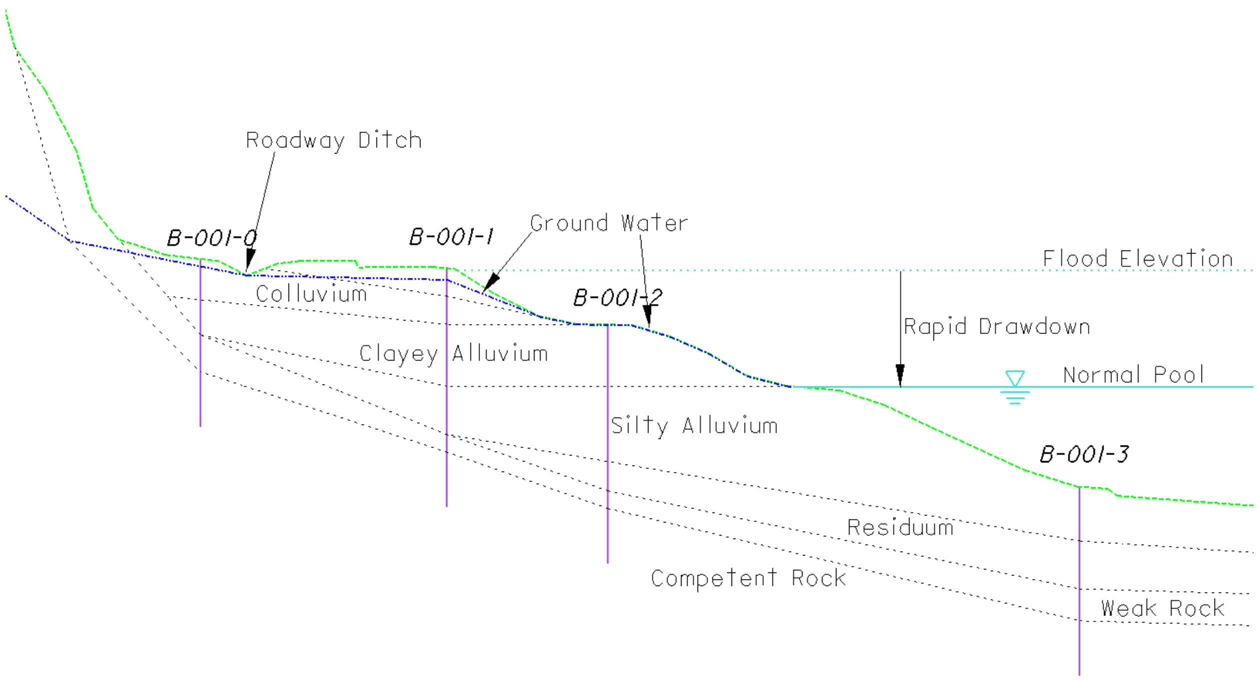


Figure 700-6: Rapid Drawdown Groundwater Surface Profile in Cross-section



704.4 Estimate Shear Failure Surface

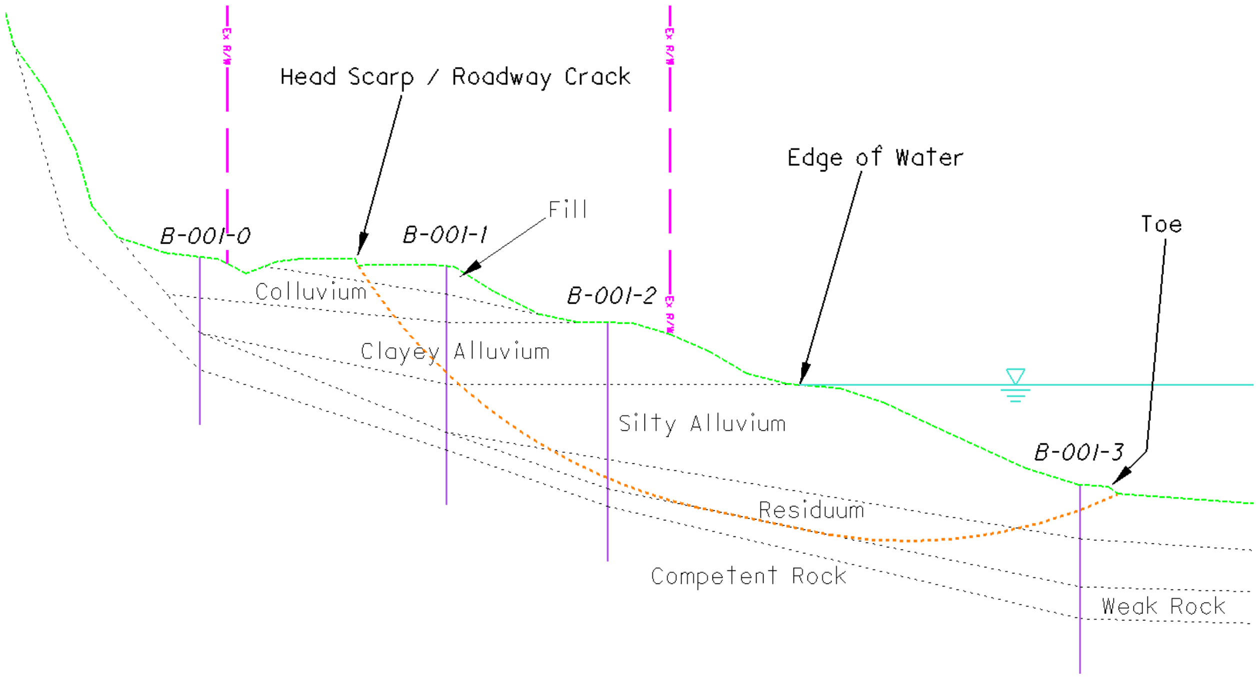
Before conducting a stability analysis, estimate the shear failure surface using engineering judgment and experience. Review the subsurface profile, inclinometer data, and landslide features noted in the site reconnaissance and site survey and construct a realistic representation of the shear failure surface which fits with the available evidence. This failure surface is merely an approximation, based on the engineer's interpretation of the mode of failure, but it should provide a useful starting point, and will give a metric with which to compare the results of computerized stability analyses, to determine whether the outputs seem reasonable.

In constructing the estimated shear failure surface, make sure it intercepts the ground surface at the head scarp and toe bulge (if evident), and make sure it conforms to the inclinometer data showing the depth of shear failure. If the bedrock is steeply sloping beneath the hillside and relatively shallow compared to the size of the failure, consider the likelihood that the failure surface intersects or travels along the top of rock. Highly weathered residual material at the top of the bedrock surface will often make up a thin, weak layer that provides a path of least resistance where a shear surface can develop.

If the toe of the landslide is in a river or other body of water, and is not visible from the ground surface, the engineer must use the other available data to project the failure surface out to its toe. Hopefully, soundings of the bottom of the waterway have been obtained. Depending on the detail and resolution of the soundings, the toe bulge may be apparent on the cross-sections. Otherwise, if the toe of the slide is not readily apparent, consider projecting the failure surface out to the natural toe of the slope.

An estimated shear failure surface for the example site is presented in Figure 700-7. The inclinometer at boring B-001-1 showed a shear failure at approximately 20 feet deep, near the bottom of the "clayey alluvium stratum," and the inclinometer at boring B-001-2 showed a shear failure at approximately 28 feet deep, near the top of rock. The estimated shear failure surface has been connected between these two points and the head scarp visible at the ground surface, and has then been projected along the top of rock. Although a small bulge at the toe is evident, this is at the bottom of the river, and was not visible during the site visit. The bottom of the river was surveyed through soundings, but the resolution of this survey is not high enough to be sure of a toe bulge feature. Therefore, the estimated failure surface has been projected to the toe of the slope, where the river bottom levels off.

Figure 700-7: Estimated Shear Failure Surface Profile in Cross-section



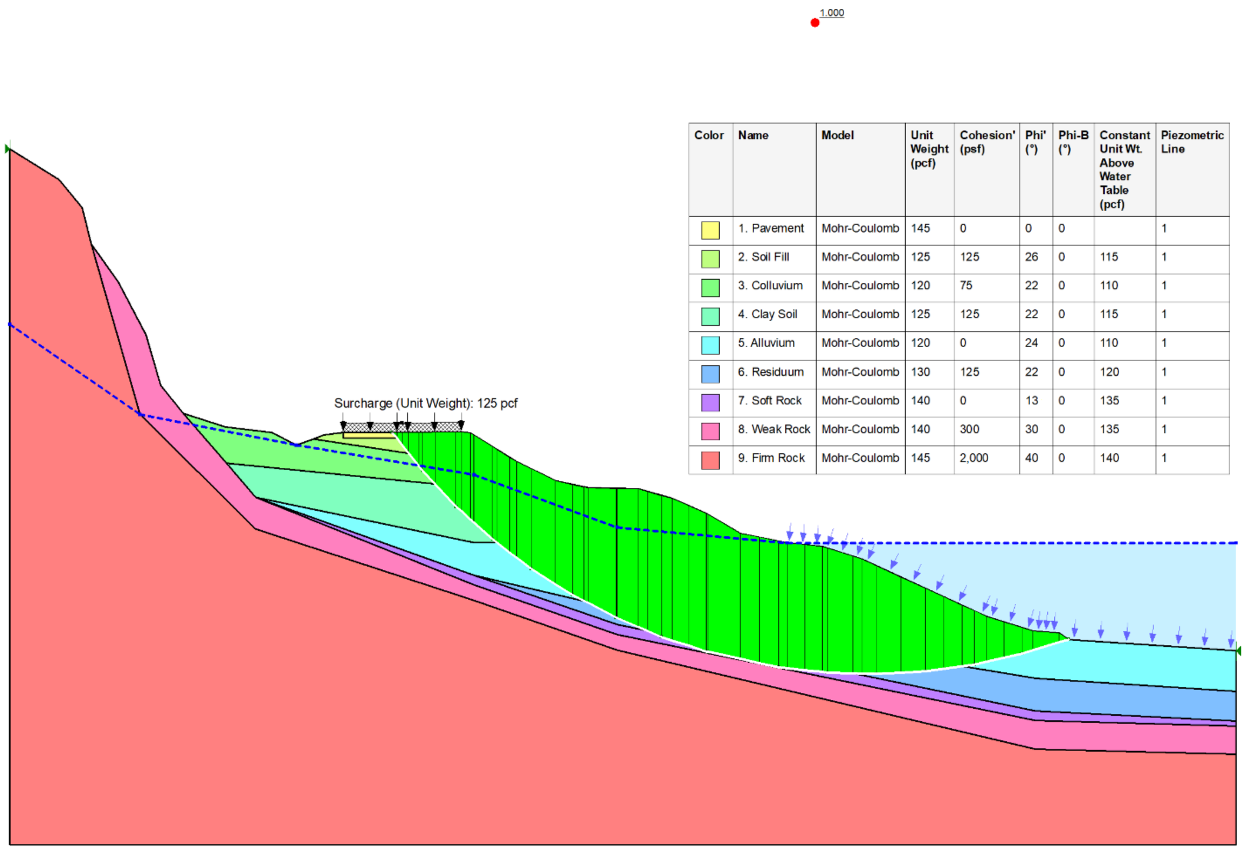
704.5 Stability Analysis

Perform a computerized limit equilibrium stability analysis of the existing condition, using a model based on the subsurface profile developed along the primary boring cross-section. Initially use estimated shear strength and unit weight values for each of the subsurface profile strata similar to the recommended values in Section 400. Refine these values during the analysis, through "back-calculation" of the engineering properties.

If the shear failure surface is estimated to move along the top of rock, add a new "soft rock" layer along the surface of the top of rock, through which a shear failure surface will develop. Through experience with many slides of this type, we have found that there is often a thin, weak boundary zone along the top of rock, with very low shear strength. This layer is often not identified in borings. Even when identified, this layer is typically not revealed as "soft" by vertical exploration and sampling methods such as SPT, but it exhibits residual strength properties due to historic landslide movement. OGE recommends representing this zone as a two-foot ± thick layer of very soft cohesive soil, with little to no cohesion, and an angle of internal friction usually between 12° to 18°.

The calculated shear failure surface that is output by the analysis should have a Factor of Safety of 1.0, and should roughly coincide with the estimated shear failure surface or with the known points of shear failure, as given by the head scarp, inclinometer data, and toe bulge. If the initial run of the analysis does not meet these criteria, adjust the engineering properties of the soil until the computerized analysis produces an output which conforms to these conditions. Assuming the slide event was triggered by excess pore water pressures within the slope, it may also be prudent to raise the piezometric surface above that for the static condition or that revealed by monitoring wells post-failure. Typically, a combination of both (adjusting soil properties and piezometric surface) is performed. Figure 700-8 shows the output of the computerized stability analysis for the example site in the existing condition utilizing GeoStudio 2018 R2 Slope/W. Note that a thin "Soft Rock" layer has been added to the top of the bedrock surface.

Figure 700-8: Computerized Stability Analysis of Existing Failed Condition



Do not include an artificially curved "layer" of residual failed soil which mimics the estimated failure surface. Such a layer will force development of the failure surface by the computerized analysis along the new layer. However, it will inaccurately predict the Factor of Safety for other portions of the slope, and for a reconstructed or retained slope. This method of artificially forcing the failure surface also inaccurately predicts resistance and loading in the remediation analysis. We are interested in the reasons for the initial development of the shear failure, and in preventing similar such failures. Therefore, the goal is to accurately model the entire slope and all strata in a "pre-failure" condition, so that we can see development of the failure surface along the expected path, and hopefully predict other likely failure surfaces, especially post-remediation. We acknowledge that the soil shear strength along the failure surface is lowered to residual strength values once the failure occurs and substantial movement has taken place. However, we would rather modify the strengths of all the soil strata and the level of the water table until an approximation of the initial failure conditions is achieved so that we can protect against any future such failures.

SECTION 800 SPECIAL BENCHING AND SIDEHILL EMBANKMENT FILLS

801 GENERAL

The guidance in this Section originated as Geotechnical Bulletin 2; use it when designing and specifying special benching of existing slopes in highway embankment construction. Guidance for standard specification benched embankment construction is provided in the ODOT Construction and Material Specifications, Item 203.05. Regarding standard specification benched embankment construction, Item 203.05 states the following:

If the existing slope is steeper than 8:1, bench into the existing slope as follows:

- Scalp the existing slope according to Item 201.
- Cut horizontal benches in the existing slope to a sufficient width to blend the new embankment with the existing embankment and to accommodate the placement, and compaction operations and equipment.
- Bench the slope as the embankment is placed and compact into layers.
- Begin each bench at the intersection of the existing slope and the vertical cut of the previous bench. Recompact the cut materials along with the new embankment.

Special benching should be used whenever the designer anticipates that there will be a stability problem and/or weak soils in an existing slope. Special benching is typically utilized to improve stability in a sidehill fill placed on an existing slope or to remediate an unstable existing slope. As opposed to standard specification benching, special benching is always shown on the cross-sections in the project plans. Special benching is performed in addition to, and in place of, standard specification benching, and has pay quantities for both excavation and embankment calculated for the benched areas and added to the plan General Quantities. Whenever special benching is used, Plan Note G109 from the ODOT Location and Design Manual, Volume 3 needs to be included in the General Notes. Plan Note G109 states, "Although cross-sections indicate specific dimensions for proposed benching of the embankment foundations in certain areas, no waiver of the specifications is intended. Bench all other sloped embankment areas as set forth in 203.05. No additional payment will be made for benching required under the provisions of 203.05."

Where sidehill fills are planned on the face of an existing slope which is steeper than 4H:1V, the OGE typically recommends special benching (as detailed in the ODOT Construction Inspection Manual of Procedures, Section 203.05, "Benching") to assure the embankment is "knitted" together. Compaction of fill soils placed upon an existing embankment, especially thin "sliver" fills of three feet or less thickness, is especially difficult. Conventional compaction equipment cannot be used on slopes as steep as a typical highway embankment, which means that fill placed directly on such a slope will generally be either poorly compacted or uncompacted, which leaves it highly susceptible to erosion or sliding failure. Even if compaction can be assured in such a fill, the sloping interface between the existing surface and the new fill provides a potentially weak continuous plane along which a shear failure may develop. Special benching creates a "stairstepping" surface, which improves stability by inhibiting the development of a contiguous shear plane along the interface.

The most common method of remediation for an unstable existing slope consists of digging out the failed soil mass in a benched excavation and reconstructing the slope with compacted engineered fill. This removes failed and sloughing material from the slope and inhibits the development of a shear plane along the interface with the existing ground and the new fill. If the reconstructed fill is well compacted and built with a stable slope, any potential shear surface will be forced deeper, below the special benched fill, thus improving the resistance against shear failure. Benched excavation and replacement are often used in combination with other methods, such as flattening of the slope, groundwater drainage, counter-berms, or retaining structures to further improve stability of the slope.

802 SHEAR STRENGTH OF PROPOSED EMBANKMENT FILL

On projects that propose new embankments, since the embankments have yet to be constructed, it is impossible to test for the shear strength of the embankment material. The designer, in order to determine both short-term and long-term stability, must estimate the shear strength parameters of the embankment material prior to construction.

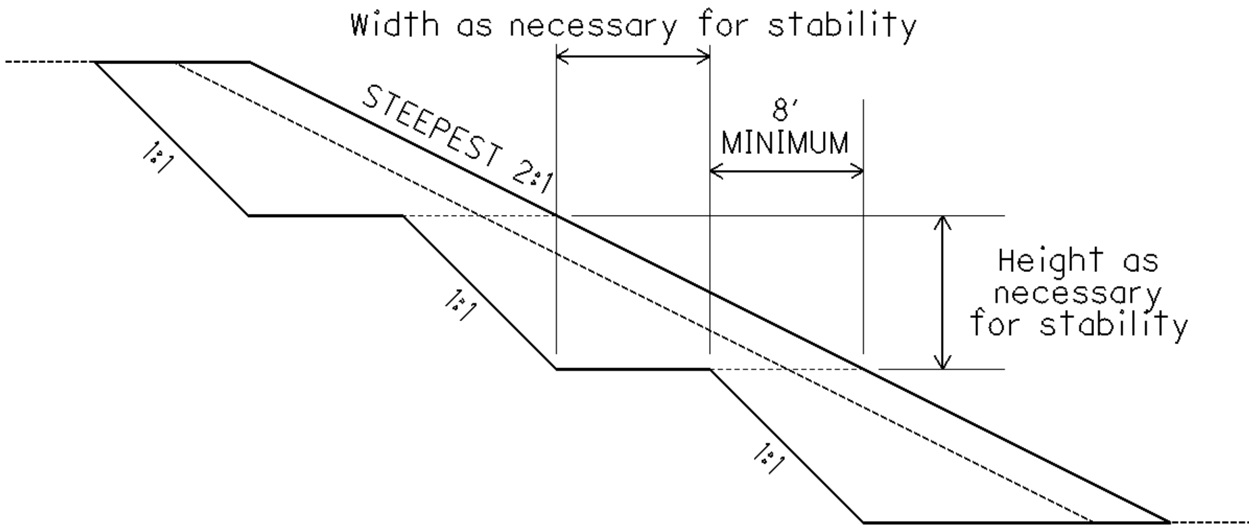
The primary purpose of this section is to provide a sound and consistent methodology for determination of shear strength parameters for proposed embankment fills that are not yet constructed. This section is therefore not applicable to in-situ soils that have not been moisture conditioned and compacted per ODOT C&MS Item 203 Embankment. Estimated shear strength parameters should not be used to analyze critical applications, such as stability analyses of existing embankments or evaluation of existing embankments supporting a structure. This section does not negate the need to perform strength testing as part of the geotechnical exploration when appropriate.

In most cases, the material used to construct a new embankment is obtained by excavating existing soil on or near the project site. In the geotechnical exploration performed for the project, the existing soils within the proposed right-of-way should have been sampled and undergone laboratory testing. Estimate the shear strength and unit weight of the new embankment fill using Table 500-2, based on the classification of the expected fill material.

803 GENERAL CASE: SPECIAL BENCHED EMBANKMENT CONSTRUCTION

As stated above, OGE typically recommends special benching where sidehill fills are planned on an existing slope which is steeper than 4H:1V. Figure 800-1 shows the details of a typical special benching scheme for a sidehill embankment fill. A similar detail drawing would be presented on each affected cross-section in the plans. Due to the variability of slope shape, height, steepness, and the position of benches across cross-sections, special benching details should never be shown on a typical cross-section. If the existing slope is stable and the new fill will be wide enough to accommodate construction equipment (≥ 8 feet) and will be built on a flat stable ground surface at the toe, specification benching will typically be adequate.

Figure 800-1: Example Section of Special Benching for Embankment Construction



The back slope of each bench is cut at a typical 1H:1V slope. This slope may need to be flattened based on the short-term stability of the soils in the existing slope. If the existing slope is made up primarily of granular soil, a slope approaching 30° (approximately 1.75 H:1V) may be more appropriate. However, if the existing slope is an embankment constructed of cohesive fill materials, which is typical in Ohio, a 1H:1V slope should stand for the short time period between excavation and placement of new fill material.

The final designed slope, after reconstruction with unreinforced compacted engineered fill, shall never be steeper than 2H:1V. A flatter slope may be desirable or necessary, depending on a stability analysis. If an embankment slope steeper than 2H:1V is required, it must be constructed as a Reinforced Soil Slope (RSS). For guidance on the design and construction of RSS, see document FHWA-NHI-10-025 (GEC 11) - Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Vol. II.

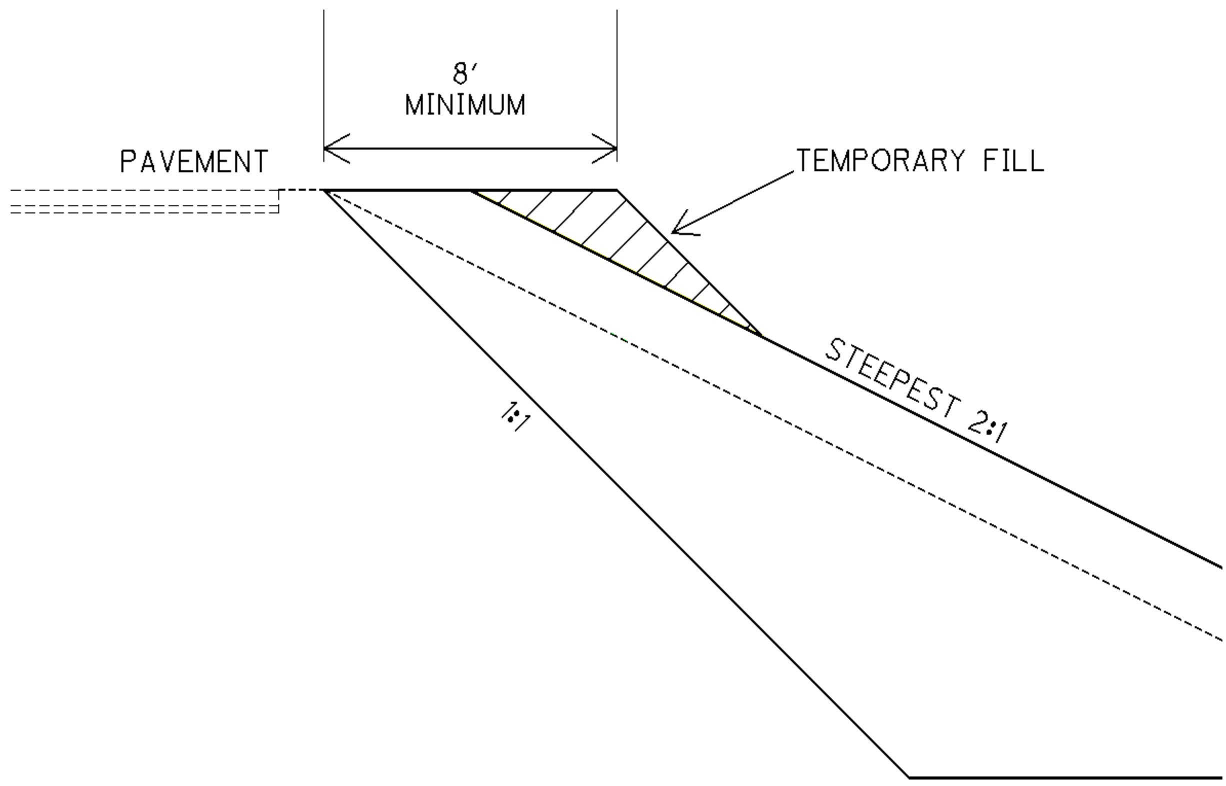
Each bench will be narrower at its top than at its base, as the back slope (typically 1H:1V) will be steeper than the final reconstructed slope (steepest 2H:1V). The top of each bench – the horizontal distance between the top of the back slope and the final reconstructed slope face – shall be at least 8 feet wide, as shown in Figure 800-1, above. This is to allow compaction and grading equipment to work on a level surface at any elevation from the bottom to the top of each bench. This also generally applies to the top bench, where it "daylights" through the top of the embankment.

However, when designing special benching for highway embankments, it will often be the case that the top bench will daylight through an existing roadway. Due to maintenance of traffic concerns, it is often not practical to cut through pavement on a heavily traveled road. In this situation, consideration needs to be given to adjustment of the benching scheme, to avoid impacting the existing roadway, guardrail, or shoulder, and to allow maintenance of traffic on the roadway during excavation and reconstruction of the embankment. A common method for accomplishing this is through the placement of temporary over-steepened fill (see Figure 800-2).

In this method, the back slope of the benched excavation is cut with a width of less than 8 feet between the top of the bench cut and the edge of the proposed fill. Temporary over-steepened fill is placed to make up the additional width. In Figure 800-2, this temporary fill is shown with a hatch pattern. The temporary fill is then “shaved” off the slope, to bring the cross-section back to the final proposed grade. The excess removed fill can be pushed to a different section of the embankment and reused.

The height of the back slope of each bench and the width of the base of each bench should be modified based on the geometry of the situation and as necessary based on a stability analysis. In general, the width and height of the benches should be arranged to minimize the required cut and fill quantities. Nevertheless, the minimum 8-foot horizontal clearance between the slope face and the back slope of each bench must be maintained, and no bench shall be taller than 20 feet in height, without a stability analysis and design by a Registered Professional Engineer per OSHA requirements.

Figure 800-2: Temporary Over-steepened Fill



The proposed new slope created by a sidehill fill may have a different resistance to shear failure than the existing slope. This must be checked with a slope stability analysis, to assure that the minimum required FS for slope stability is met (see Table 500-1). If the FS drops below the minimum required, the benches may need to be adjusted deeper and/or wider to improve stability.

Each individual bench should not vary greatly in elevation or horizontal location across the profile of the embankment (from station to station). In general, it is best to keep the benches either level or to follow the designed grade of the roadway (at an equal depth from the top of the embankment) along the length of the embankment. This ensures easier construction due to consistent grading. If drainage is to be installed on the back slopes of the benches (see Section 806, Special Benching for Landslide Stabilization), the bases of the benches need to have a graded slope of a minimum one percent grade, along their length, to allow water to drain along the bases of the benches.

If the embankment varies much in height from one end to the other, it may be necessary to add or subtract benches at the bottom or vary the height (or depth) of the bottom bench along its length. If this bench is to incorporate drainage, keep in mind the minimum one percent slope of the base, and/or provide suitable transverse drainage outlets at all low points. Additionally, the minimum horizontal clearance and/or cut for each bench will need to be increased by the width of the designed slope drain.

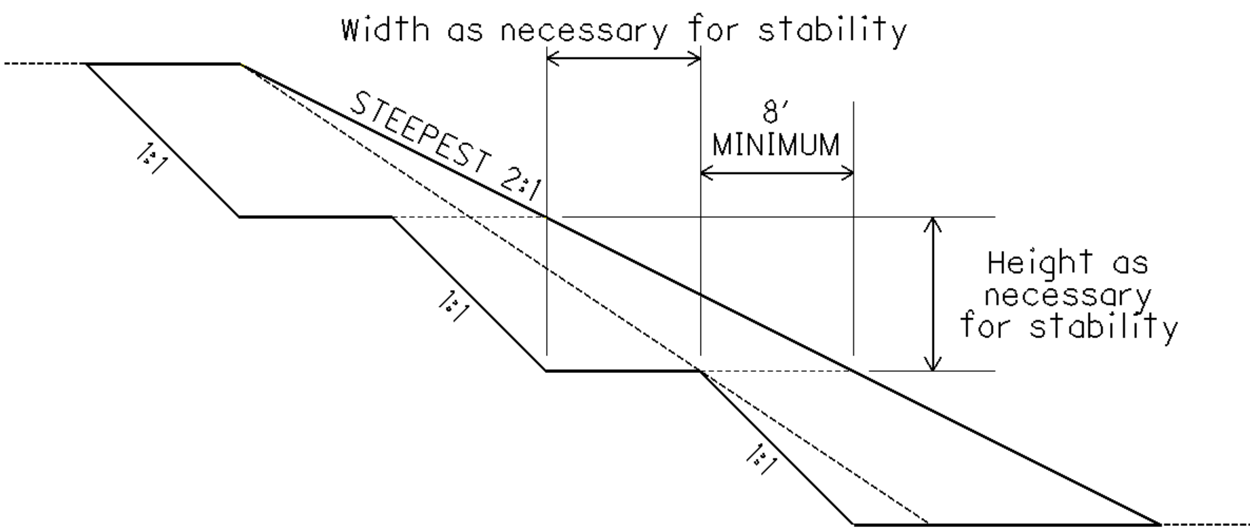
Wherever benches begin or end, the ends of the bench should be cut with a typical 1H:1V slope, in a similar manner to the back slope (as described above). This slope may need to be flatter, as defined by the short-term stability of the cut soils.

The benched excavation will be replaced with embankment fill, per C&MS Item 203.

804 SIDEHILL SLIVER FILL ON A STEEP SLOPE

Sidehill sliver fills are sometimes placed on a steep existing slope, widening the base of the embankment, and making the slope flatter; an example is presented in Figure 800-3.

Figure 800-3: Example Section of Special Benching for a Sidehill Sliver Fill on a Steep Slope



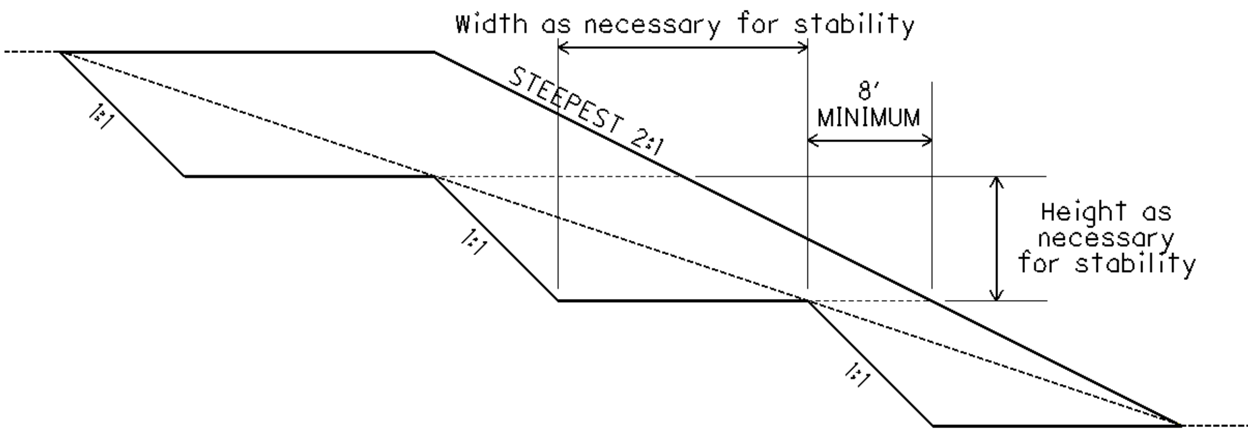
This case only differs from the general case in that the existing slope is being flattened, usually to improve stability. In this case, run a slope stability analysis to assure that the minimum FS for slope stability is met (see Table 500-1). If the FS drops below the minimum required, the benches may need to be adjusted deeper and/or wider to improve stability.

Due to the flattening of the slope, and widening of the base, the upper benches may need to be cut further into the existing slope than the lower benches in order to maintain the minimum 8-foot width at the top of each bench. See Section 803, General Case, for additional details on dimensions and construction of special benching.

805 SIDEHILL SLIVER FILL ON A SLIGHT SLOPE

Sidehill sliver fills are sometimes placed on a flatter existing slope, steepening the embankment, and widening the crest; an example is presented in Figure 800-4.

Figure 800-4: Example Section of Special Benching for a Sidehill Sliver Fill on a Slight Slope



The steepened slope created in this case may have a lower FS against shear failure; run a slope stability analysis to assure that the minimum FS for slope stability is met (see Table 500-1). If the FS drops below the minimum required, the benches may need to be adjusted deeper and/or wider to improve stability.

In this case, the benches may be cut an equal distance into the existing slope, but will tend to be deeply cut horizontally, due to the large difference between the existing slope and the back slope of the bench.

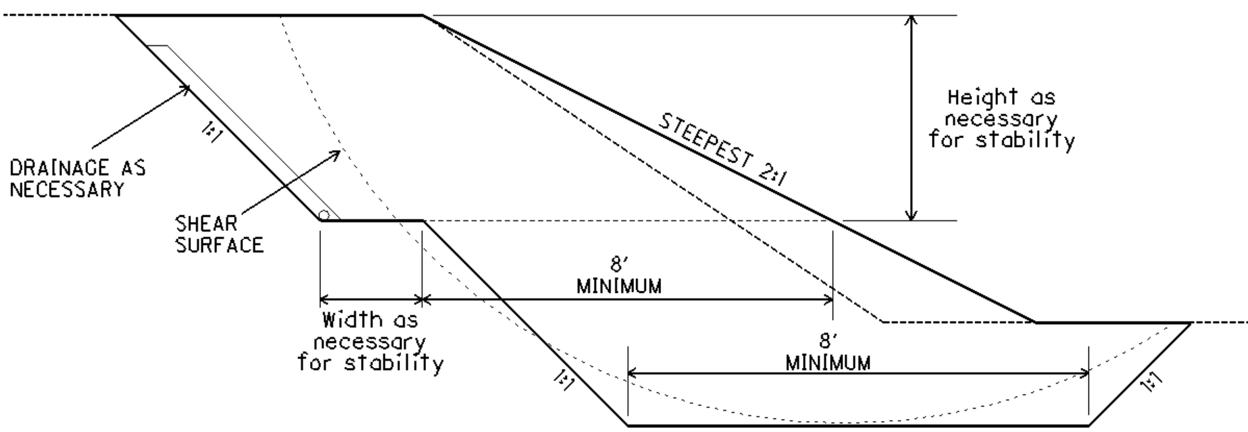
Due to the geometry of this case, the bottom bench should have the narrowest top. Pay special attention to maintain the minimum 8-foot width at the top of this bench. See Section 803, General Case, for additional details on dimensions and construction of special benching.

806 SPECIAL BENCHING FOR LANDSLIDE STABILIZATION

The most common method of remediation for an unstable existing slope consists of digging out the failed soil mass in a benched excavation, intercepting the failure surface, and reconstructing the slope with compacted engineered fill. If the reconstructed fill is well compacted and built with a stable slope angle, any potential shear surface will be forced deeper, below the benched fill, thus improving the resistance to shear failure. The development of a shear plane along the interface with the existing ground and the new fill is inhibited, in a similar manner to the "knitting" of a benched sidehill fill into an existing slope. Figure 800-5 shows the case of benching used to stabilize a landslide.

When designing benching for landslide stabilization, the benches should be designed to intercept the assumed shear failure surface. If excavation and replacement occur entirely above the failure surface, it will have little to no effect on stability, and the "repaired" slope will merely become a part of the moving soil mass. The benches need to be cut through the existing failure surface, and deepened vertically and widened horizontally into the slope, until adequate stability is achieved.

Figure 800-5: Example Section of Special Benching for Landslide Stabilization



The stability of the design must be checked with a slope stability analysis, to assure that minimum FS for slope stability is met (see Table 500-1). If the FS does not meet the minimum required, the depth and/or width of the benches will need to be adjusted again to improve stability. If the minimum required FS cannot be met by benching, additional measures, as discussed below, will be necessary to further improve stability of the slope.

If the FS is not adequate with benching and reconstruction alone, and the shear surface is expected to develop through the new fill material, consider substituting crushed rock as a fill material and/or including a rock shear key below the lowest bench. Crushed rock fill typically has a higher internal friction angle than typical Item 203 compacted embankment fill material, and therefore has more resistance to shear failure. A rock shear key can force the failure surface lower and improve stability.

Landslides are very often initiated or aggravated by elevated pore water pressures in the slope soils, either introduced by subsurface groundwater flow or through surface water infiltration. Lowering the groundwater level often increases the stability of the slope appreciably. Therefore, groundwater drainage is often incorporated into any design fix of a landslide with benching.

When groundwater drainage is to be incorporated, the back slope of each appropriate bench should have a slope drain installed along the entire length of the excavation. This slope drain is typically made up of an 18-inch thick layer of Item 203 Granular Embankment, as Per Plan (No. 8 Stone), with Item 690E12010 SPECIAL Geotextile Fabric, 712.09 Type A, to prevent infiltration of fines into the drain. The use of alternative equivalently performing materials may also be considered for installation as a slope drain. If the top bench is drained, the slope drain should stop 3 feet below the ground surface, so that properly compacted fill may be placed above, for pavement support and protection of the drain.

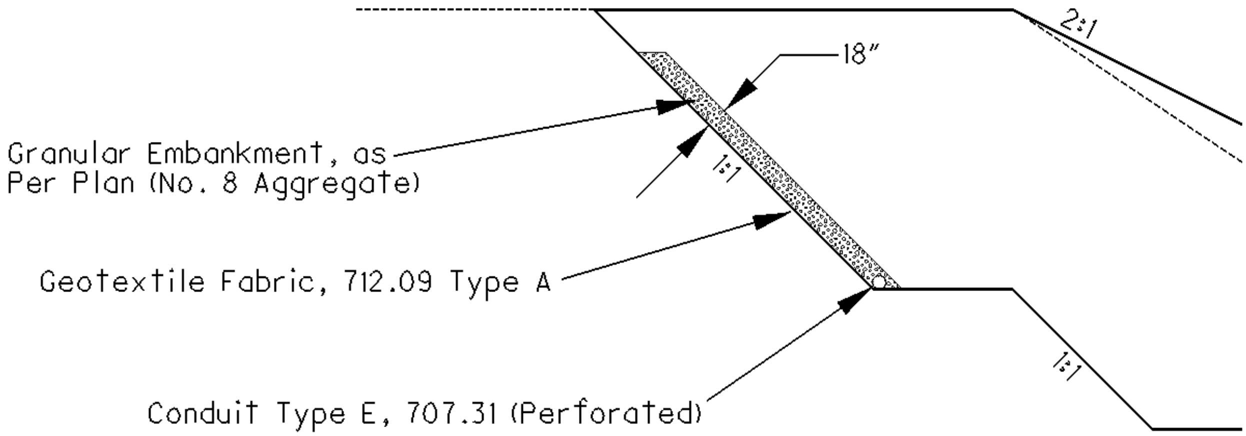
The bases of the draining benches need to have a graded slope of a minimum one percent grade, along their length, to allow water to drain along this path. A line of Item 611 Conduit Type E, 707.31 (Perforated) should be installed along the bottom of each slope drain, to allow an easy path for water to flow along the base of the bench. The Type E conduit should be perforated as per conduit for Item 605 Unclassified Pipe Underdrains. For bench cuts/fills of 10 feet or less in height, a 4-inch diameter drain pipe should be used. For bench cuts/fills of more than 10 feet in height, a 6-inch diameter drain pipe should be used. The larger pipe is preferred with a greater height of fill to prevent the weight of the fill from crushing the pipe.

Transverse outlet drains of Item 611 Conduit Type F, 707.33, should outlet from the aggregate drain at the low end of the benches. These drains should be installed at a minimum one percent slope, and outlet through the face of the slope. Outlet drains should utilize Item 611 Precast Reinforced Concrete Outlets. Tied concrete block mat or other erosion protection should be utilized below these outlets, extending to the toe of the slope. Alternately, drain pipes may be extended down the slope, to outlet at the toe. The following Plan Note, with a similar typical cross-section detail drawing as Figure 800-6, should be included in the General Notes whenever special benching slope drains are specified:

SPECIAL BENCHING SLOPE DRAINS

PLACE SPECIAL BENCHING SLOPE DRAINS AT THE LOCATIONS SHOWN ON THE PLAN & PROFILE AND CROSS-SECTION SHEETS. THESE DRAINS SHALL CONSIST OF ITEM 203 GRANULAR EMBANKMENT, AS PER PLAN (NO. 8 AGGREGATE), ITEM 690 GEOTEXTILE FABRIC, 712.09 TYPE A, AND ITEM 611 CONDUIT TYPE E, 707.31 (PERFORATED). THE TYPE E CONDUIT SHALL BE PERFORATED AS PER CONDUIT FOR ITEM 605 UNCLASSIFIED PIPE UNDERDRAINS. THE GRANULAR EMBANKMENT SHALL BE PLACED IN LIFTS AS THE SPECIAL BENCHING BACKFILL IS CONSTRUCTED. TRANSVERSE OUTLET DRAINS SHALL BE PROVIDED AT THE LOCATIONS SHOWN ON THE PLAN & PROFILE AND CROSS-SECTION SHEETS. THESE OUTLET DRAINS SHALL CONSIST OF ITEM 611 CONDUIT TYPE F, 707.33 WITH ITEM 611 PRECAST REINFORCED CONCRETE OUTLETS.

Figure 800-6: Example Section Showing Slope Drain Details



If special benching is not able to increase stability to the minimum required FS, it may be necessary to flatten the slope of the new benched fill, or to install a counter-berm. Flattening the slope is done identically to the case of a sidehill fill on a steep slope, as described in Section 804 above. A counter-berm consists of additional fill placed at the lower end of the slope, adding to the resisting force against the sliding mass of soil. The counter-berm is constructed identically to the replacement of a single bench with engineered fill, and merely increases the width of the base of the fill. When designing a counter-berm, make sure to keep a slope which ensures positive drainage. The top of the counter-berm should never be flat, nor should it have an inclination opposite to the slope on which it is built, otherwise, surface water runoff might be trapped at mid-slope, on top of the counter-berm, where it could negatively impact stability.

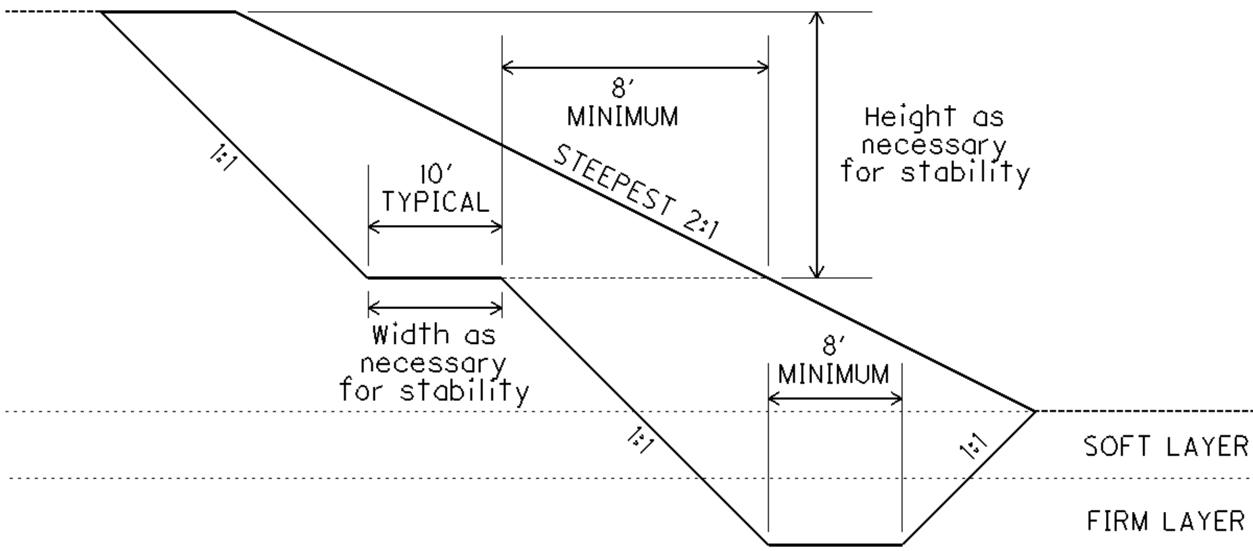
If none of the above measures achieve adequate stability, it may be necessary to use a retaining structure, such as drilled shafts or a wall. See Section 900 for design of landslide stabilization structures.

See Section 803, General Case, for additional details on dimensions and construction of special benching.

807 SPECIAL BENCHING FOR EMBANKMENT STABILITY OVER SOFT FOUNDATION SOIL

When an embankment is constructed over a layer of soft, weak soil, a shear failure may develop through the soft layer, failing the embankment. This may be remediated by excavating through the soft layer, and keying into a lower, firmer soil layer with a special benched embankment fill; see Figure 800-7.

Figure 800-7: Example Section of Special Benching for Embankment Stability over Soft Foundation Soil



Special benching is used to correct this problem in a similar manner to the case for Landslide Stabilization, in Section 806. If a shear surface has developed through the embankment, the special benching should intercept this shear surface, just as it would for a typical landslide.

In the case of a soft foundation soil, however, the lowest bench should penetrate the soft layer, so that the excavation and replacement form a key into firmer soil. The base of this key must be at least 8 feet wide, to allow compaction and grading equipment to operate in the constrained space. The depth of this key and the depths of the benches into the embankment will be defined by stability. The design must be checked with a slope stability analysis, to assure that minimum FS for slope stability is met (see Table 500-1). If the FS does not meet the minimum required, the key, and/or the benches will need to be deepened, vertically and horizontally into the slope, until stability is achieved.

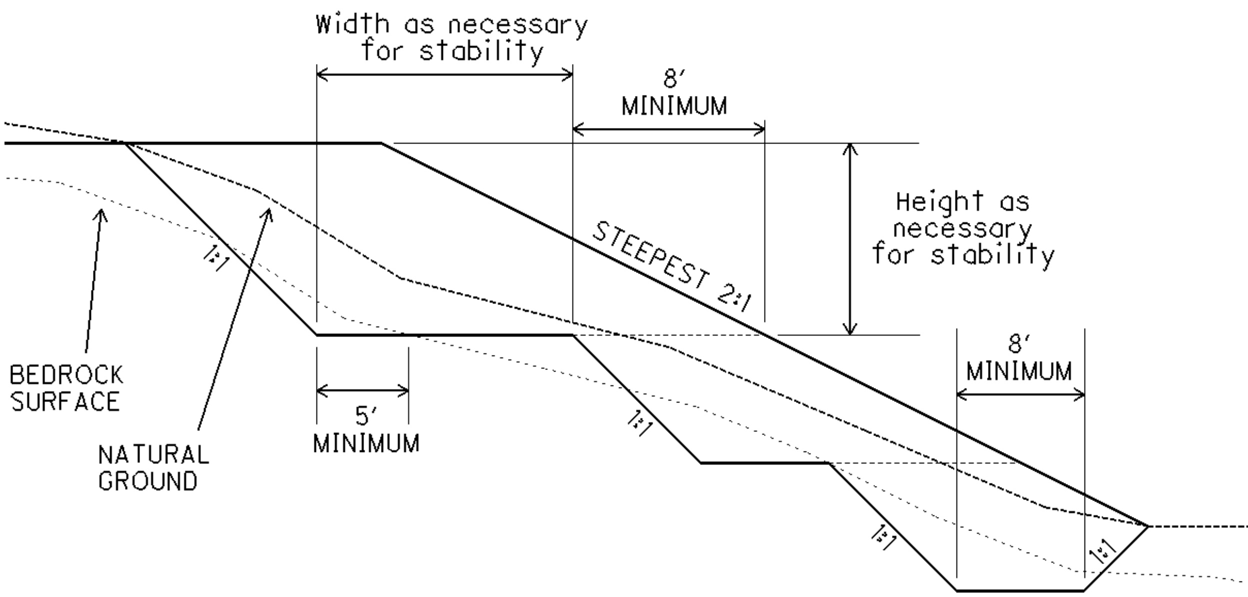
Although the back slope of each bench is typically cut at a 1H:1V slope, this slope may need to be flattened where it extends through the soft layer, or the length and duration of an open excavation may be restricted. The cut slope angle is based on the short-term stability of the existing soils, and the soft layer may not stand at a 1H:1V slope. If the length and duration of the open excavation is restricted, typically to a maximum length of 50 feet and a maximum duration of one day, a plan note will be needed to specify this.

If the minimum required FS cannot be met by special benching, additional measures, such as flattening of the slope, groundwater drainage, counter-berms, or retaining structures to further improve stability of the slope will be necessary. See Section 806, Landslide Stabilization, for details on these methods.

808 SPECIAL BENCHING FOR EMBANKMENT STABILITY KEYING INTO BEDROCK

In Appalachian regions, natural slopes often have rock close to the surface. The residual soils on these slopes, weathered from the underlying parent rock, are typically low in strength. These slopes are often marginally stable or unstable in their natural state. The additional weight of a sidehill embankment fill will only worsen the stability of the existing slope, and shear failures are often induced in this situation, with the soils sliding on top of the bedrock surface. To avoid the potential of such a shear failure, it is necessary to key into the shallow bedrock surface with special benching to improve the stability of the slope by intercepting the failure surface at the soil-bedrock interface. Figure 800-8 shows an example of this case.

Figure 800-8: Example Section of Special Benching for Embankment Stability by Keying into Bedrock



When keying benches into bedrock, the base of each bench should extend a minimum of 5 feet into the bedrock to provide a firm key. The designed benches may need to be adjusted in the field, in order to meet the actual bedrock surface. It may be necessary to make a deeper key at the base of the slope, to improve stability. The base of this key must be at least 8 feet wide to allow compaction and grading equipment to operate in the constrained space.

The depth of the bedrock key for these benches will be defined by stability. The design must be checked with a slope stability analysis, to assure that minimum FS for slope stability is met (see Table 500-1).

809 PLAN REQUIREMENTS

When specifying special benching on a project, include the following:

1) Excavation limits of special benching on the individual cross-sections. Establish pay quantities for both excavation and embankment calculated for the benched areas.

2) Plan Note G109 from the ODOT Location and Design Manual, Volume 3.

3) Limits of any temporary over-steepened fill. Establish a pay quantity for the temporary fill.

4) All necessary quantities and details of groundwater drainage, if any. When slope drains for special benching are specified, included the plan note found in Section 806.

SECTION 900 DESIGN OF DRILLED SHAFTS FOR LANDSLIDE STABILIZATION

901 GENERAL

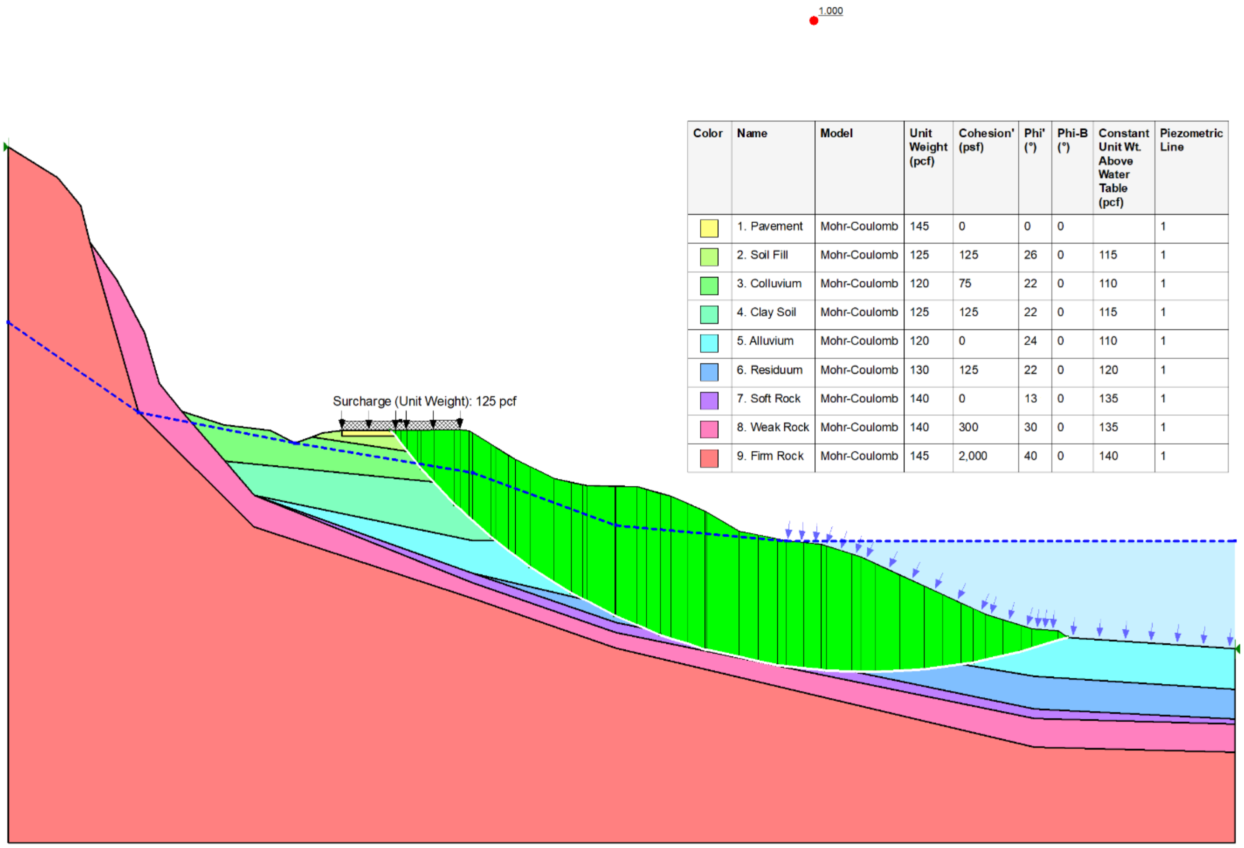
The most common method of remediation for an unstable slope consists of digging out the failed soil mass in a benched excavation and reconstructing the slope with compacted engineered fill, as described in Section 800. However, in some situations with adverse slope geometry, limited right-of-way, or the failure of a riverbank where the toe of the failure extends out into the bottom of the river, a structural solution is likely the most logical remediation. Drilled shafts can often be used to stabilize the existing unstable slope by embedding the shafts into a lower, stable stratum, preferably bedrock, and utilizing the mechanism of soil arching between the shafts to increase the nominal resistance against sliding to the point of stability. Where soil arching does not yield adequate resistance, a more robust structural solution such as a retaining wall may be necessary.

The example site presented in Section 700 is used here as an example site to develop guidance for the design of a drilled shaft solution for landslide stabilization. The limit equilibrium model presented in Figure 700-8 is presented again here in Figure 900-1 for easy reference. A user's guide for the University of Akron Slope Analysis Program, UA Slope 2.3, which is used in the Liang method analysis to determine the single shaft load for one row of evenly spaced shafts using soil arching for landslide stabilization, is also presented. Finally, guidance for utilizing the program LPILE, developed by Ensoft, Inc., for design of drilled shafts reinforced with structural steel sections is presented.

902 LIANG METHOD ANALYSIS

Dr. Robert Liang of the University of Akron published two papers, one in December, 2002, titled "Drilled Shaft Foundations for Noise Barrier Walls and Slope Stabilization," and one in November, 2010, titled "Field Instrumentation, Monitoring of Drilled Shafts for Landslide Stabilization and Development of Pertinent Design Method," as part of the results of a research project conducted for the Ohio Department of Transportation. The goal of this research was to develop a methodology for design of drilled shafts to stabilize unstable slopes. This research built upon the results of two earlier research projects by Dr. Robert Liang and Sanping Zeng, "Numerical Study of Soil Arching Mechanism in Drilled Shafts for Slope Stabilization," and "Stability Analysis of Drilled Shafts Reinforced Slope." These research projects used two-dimensional finite element modeling as well as centrifuge testing of physical models to study the effects of soil arching between pairs of equally spaced, laterally loaded drilled shafts in a single row. As a result of these research studies, Dr. Liang developed a mathematical model to predict the percentage of lateral load from the soil which is transferred to the shafts as opposed to the percentage which is passed between the shafts. The percentage of lateral load which passes between the shafts from the uphill soil mass to the downhill soil mass, the "Load Transfer Factor" (η), is a function of the soil cohesion (c), the soil angle of internal friction (φ), the drilled shaft diameter (D), the center-to-center spacing between the drilled shafts (S), the location of the drilled shafts on the slope (ξ) and the angle of steepness of the slope from horizontal (β). The two factors relating to shaft geometry, S and D, are typically expressed as a spacing-to-diameter ratio (S/D). We refer to this analysis methodology as the Liang method.

Figure 900-1: Computerized Stability Analysis of Existing Failed Condition for Example Site



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (pcf)	Piezometric Line
	1. Pavement	Mohr-Coulomb	145	0	0	0		1
	2. Soil Fill	Mohr-Coulomb	125	125	26	0	115	1
	3. Colluvium	Mohr-Coulomb	120	75	22	0	110	1
	4. Clay Soil	Mohr-Coulomb	125	125	22	0	115	1
	5. Alluvium	Mohr-Coulomb	120	0	24	0	110	1
	6. Residuum	Mohr-Coulomb	130	125	22	0	120	1
	7. Soft Rock	Mohr-Coulomb	140	0	13	0	135	1
	8. Weak Rock	Mohr-Coulomb	140	300	30	0	135	1
	9. Firm Rock	Mohr-Coulomb	145	2,000	40	0	140	1

A computer program called UA Slope 2.3 was developed as part of the "Field Instrumentation, Monitoring of Drilled Shafts for Landslide Stabilization and Development of Pertinent Design Method," and the subsequent "Enhancement of UASLOPE for Improving Implementation Efficiency" research studies. This program uses inter-slice forces from the method of slices for slope stability calculation, together with the results of the mathematical model for η, to calculate the force imposed on one shaft in a single row of equally spaced drilled shafts by a moving soil mass above an assumed soil shear failure surface. The failure surface is either estimated or determined by a stability analysis; see Section 704.5. The UA Slope 2.3 program requires as its inputs the geometry of the ground surface and soil strata, the geometry of the shear failure surface, the geometry of the phreatic surface, the physical properties (cohesion, friction angle, and unit weight) of each soil stratum, and the drilled shaft geometry (offset location, diameter, and spacing). A user's manual for UA Slope 2.3 is attached as an appendix. The user's manual fully describes how to input data and run the program, however, some of the major points will be reiterated and expanded upon in this section.

The geometric origin for all data input in the UA Slope 2.3 program is at the top left, and the slope must always be represented from uphill on the left to downhill on the right. In other words, all geometric data is entered in X,Y coordinates, where 0,0 is at the top left, X increases from left to right, and Y increases from top to bottom.

The geometric data for the UA Slope 2.3 program must be input in vertical slices. Each soil stratum (including the ground surface) is defined by points along its top, where it intersects each vertical slice. The maximum number of vertical slices is 50, and these include the boundaries at the right and left limits of the represented cross-section. Therefore, depending upon the complexity of the geometry, the vertical slices must be chosen with care in order to represent each change in geometry along the top of each stratum. Strata must be input from top to bottom, and must be represented across the entire cross-section, whether they extend all the way across or terminate somewhere in the middle. Wherever a layer terminates, the coordinates defining its top surface will be the same as those for the layer below – in other words its thickness will be zero. The maximum number of layers is 30. One extra layer – which does not count in the maximum number of soil layers – is always input, defining the bottom of the lowest layer. This bottom layer must be flat, with the same Y-coordinate all the way across the cross-section.

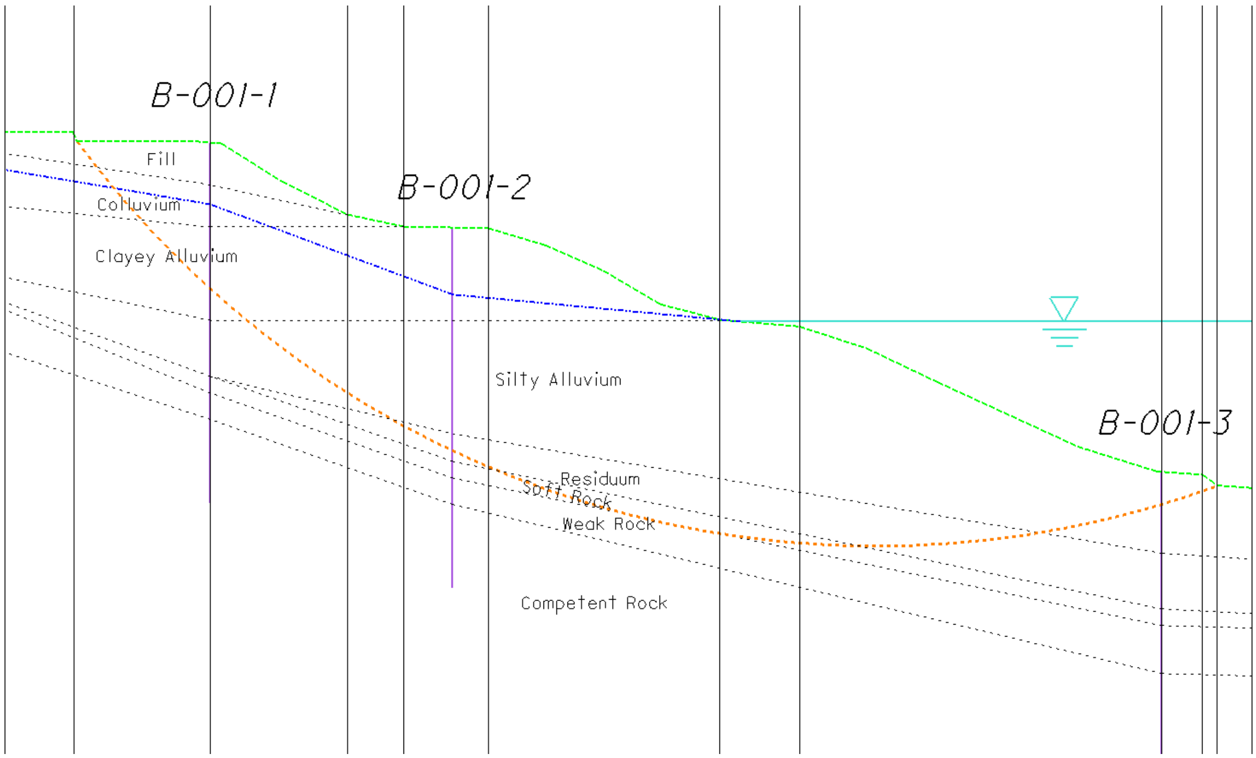
The coordinates defining the ends of the shear failure surface must exactly meet with the ground surface, or the program will output an erroneous Factor of Safety. We recommend that vertical slices should be positioned at the end points of the shear failure surface, so that the ground surface coordinates and the shear surface end coordinates may coincide at these points. The points defining the shear failure surface do not otherwise have to be on the vertical slices, and points in-between vertical slices will usually need to be entered in order to properly define the shape of a curving failure surface. However, we recommend that wherever the failure surface intersects a vertical slice, this should be represented by an input point. The UA Slope 2.3 program allows a maximum of 50 points to define the shear failure surface.

The points defining the phreatic surface do not have to be on the vertical slices, and it will often be advantageous to enter points in-between vertical slices to properly define the shape of the phreatic surface. However, the phreatic surface needs to extend across the entire cross-section, from the first to the last vertical slices, which are the boundaries at the right and left limits of the represented cross-section. The UA Slope 2.3 program allows a maximum of 50 points to define the phreatic surface.

It is recommended to build a geometric model of the analysis cross-section in a CADD drawing, with each vertical slice drawn in, and all the intersections between the slices and strata determined before attempting to input the data into the UA Slope 2.3 program. Building a geometric model will also allow determination of all the points to define the shear failure surface and phreatic surface. It is also recommended to enter all the coordinates into a spreadsheet or similar matrix, to consult while entering the data into the UA Slope 2.3 program. This will limit the number of data entry errors.

The geometric model constructed for the example site is presented in Figure 900-2. This model has twelve vertical slices, including the left and right boundary limits. Note that the "Fill," "Colluvium," "Clayey Alluvium," and "Residuum" layers do not extend all the way across the model. Where the "Fill," "Colluvium," and "Clayey Alluvium," layers terminate on the right, their defining coordinates become the same as the ground surface for the rest of the width of the model. Where the "Residuum" layer terminates on the left, its coordinates become the same as the "Soft Rock" layer below. Note also that the geometric model constructed for UA Slope 2.3 is not as wide as the model constructed for stability analysis (as shown in Figure 900-1); it is not necessary to make the UA Slope 2.3 model any wider than the limits of the shear failure surface.

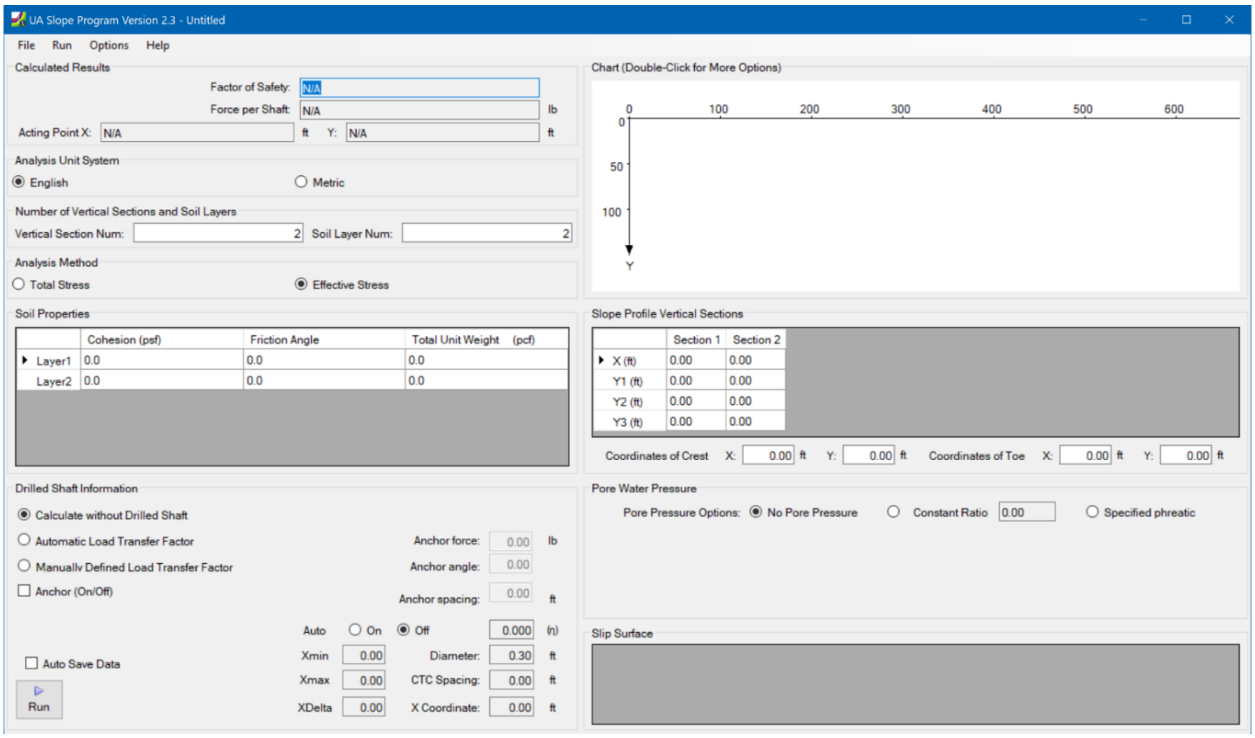
Figure 900-2: Geometric Model with Vertical Slices



The UA Slope 2.3 program requires the input of three engineering soil properties per stratum; cohesion, friction angle, and total unit weight. The cohesion and friction angle for each stratum should be approximately the same as those utilized in the computerized stability analysis. However, the unit weight must be approximated to a single value for each stratum. We recommend utilizing the moist unit weight for those strata entirely above the phreatic surface, and we recommend utilizing the saturated unit weight for those strata entirely below the phreatic surface. If the phreatic surface passes through a stratum, we recommend selecting a unit weight somewhere in-between the moist and saturated unit weights, based on the relative percentage of the stratum which is above or below the phreatic surface.

After the UA Slope 2.3 program is started for the first time, or if File, New is selected from the drop-down menus, the Main Menu screen will open, with "UA Slope Program Version 2.3 – Untitled" displayed on the title bar. Figure 900-3 shows the UA Slope 2.3 program Main Menu. The Main Menu screen is the only screen in the UA Slope 2.3 program; all data is input on this screen, the program is executed from this screen, and program outputs appear on this screen. The program also has four drop-down menus, for file functions, program execution, program options, and "Help" information about the program. The Options drop-down menu will display two small dialogue boxes for changing file paths (most of which should not be changed or the program will cease to function) and for Chart Options, which allows the user to change the colors used in the "Chart" graphic that appears in the upper right corner of the Main Menu screen. It should be noted that, by default, the program only has colors defined for 7 soil layers: if your model has more layers than this, the additional layers will change colors randomly every time the mouse scrolls over the Chart, which can become visually bothersome. Therefore, we recommend assigning colors to at least as many layers as you will have in your model.

Figure 900-3: UA Slope 2.3 Program Main Menu



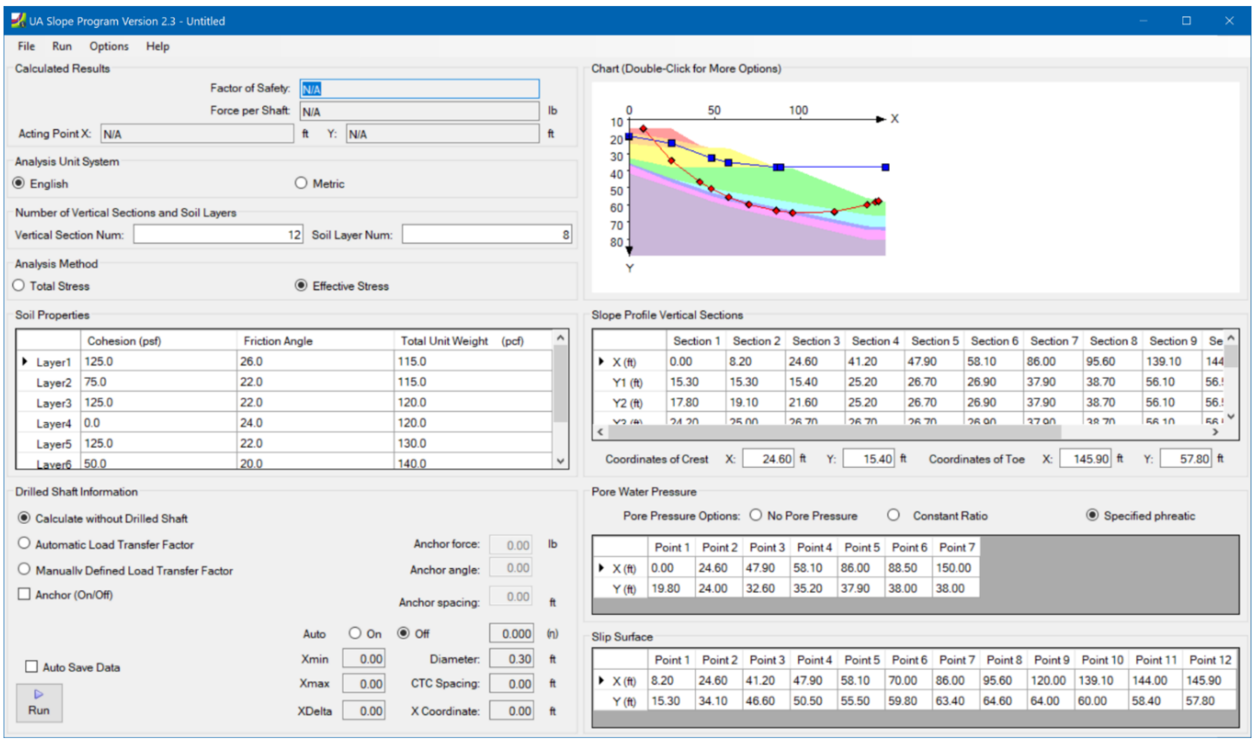
Before beginning to input data, the user must input the units of measurement (English or Metric), number of vertical sections, and number of soil layers. The user must also select the analysis method as either total stress method or effective stress method. We recommend using the effective stress method, as the load transfer factor equation coded in the program was based on effective stress concepts and is not calibrated or verified for total stress analysis.

Pore Pressure Options, on the lower right of the Main Menu, allows the option to select "No Pore" pressure, "Constant Ratio" pore pressure, or "Specified Phreatic" surface. No pore pressure does not take into account pore water pressure effects, constant ratio pore pressure applies the same pore pressure effect to every slice in the method of slices analysis, and specified phreatic surface allows the user to define a ground water table to set the effect of pore water pressure on the analysis. We recommend utilizing the specified phreatic surface, as this is the most realistic alternative.

The geometry for the Slope Profile Vertical Sections, Specified Phreatic surface, and Slip Surface are input into spreadsheet-like grids. The X-coordinates are entered on the top row, and Y-coordinates are entered on each lower row, corresponding to each X-coordinate for each surface. The number of Slope Profile Vertical Sections and Soil Layers are specified as mentioned above on the upper right portion of the Main Menu screen, and their selection will automatically format the Slope Profile Vertical Sections grid for the appropriate number of entries. To select the number of points for the Specified Phreatic surface and Slip Surface, right-click with the mouse pointer in the box containing the grid, and select either Add Point, Set Points, or Delete Point. For the Set Points option, a second small dialogue box opens: the user should type the number of points in the dialogue box and then hit Enter on the keyboard. Soil Properties are entered similarly, in a grid on the left of the Main Menu, in which each row represents a single soil layer or stratum.

Although the data-entry grids in the UA Slope 2.3 program may look similar to spreadsheets, the data cannot be cut, copied, or pasted like a spreadsheet. The Main Menu screen with the geometry and soil properties entered for the example problem is presented in Figure 900-4.

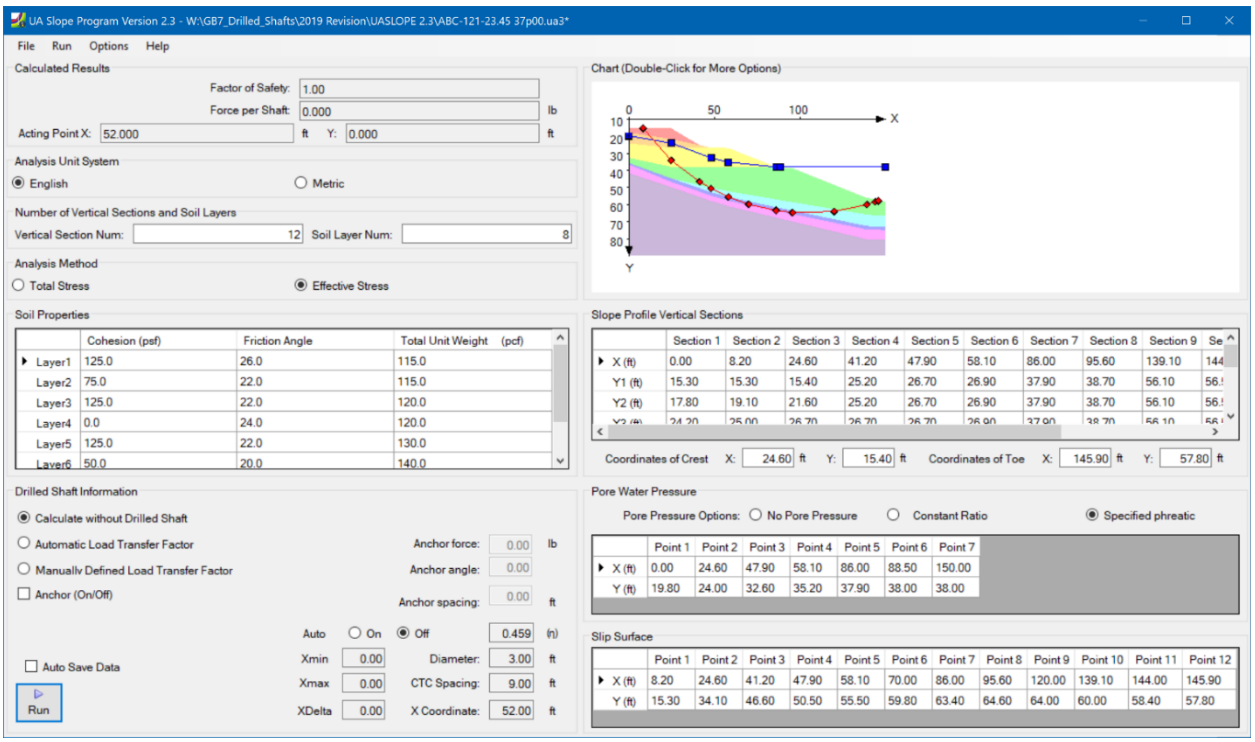
Figure 900-4: UA Slope 2.3 Program Main Menu with Geometry



Finally, the user must define the Coordinates of [the] Crest of the slope and the Coordinates of [the] Toe of the slope. These coordinates have two purposes: first, to calculate the angle of steepness of the slope from horizontal (β); and second, to calculate the location of the drilled shafts on the slope (ξ). The program does not allow slope angles steeper than sixty degrees, since the load transfer factor equation coded in the program is not calibrated for steeper slope angles. Furthermore, if the crest and toe coordinates are not defined, ξ cannot be determined, and η will always evaluate as "0," invalidating the results of the analysis. In the example problem, the Coordinates of Crest are at the outside edge of the roadway embankment, and the Coordinates of Toe are at the toe of slope, where the failure surface exits. The drilled shafts must be placed between the Coordinates of Crest and the Coordinates of Toe.

Run the analysis first without the drilled shaft effect (Calculate without Drilled Shaft), so that the Factor of Safety for the existing condition may be appraised and correlated with the Factor of Safety from the computerized stability analysis (this should be 1.0). If the initial run of the UA Slope 2.3 program does not have a Factor of Safety of 1.0 (or very close), adjust the soil properties or phreatic surface slightly until the Factor of Safety equals 1.0. The Main Menu screen with the Calculated Results of the analysis without the drilled shaft is shown in Figure 900-5; the Factor of Safety of 1.00 is displayed at the upper left corner of the Main Menu.

Figure 900-5: UA Slope 2.3 Program Results without Drilled Shafts



To perform an analysis with the drilled shaft effect, the user must input the Drilled Shaft Information (geometry). Drilled Shaft Information consists of Diameter (D), center-to-center (CTC) Spacing (S), and X Coordinate (location). The X Coordinate is the offset of the centerline of the single row of drilled shafts from the Y-axis (the uphill boundary slice). For this reason, it is helpful to make the left (uphill) boundary at an even-numbered offset from the roadway centerline or baseline. If the failure scarp does not extend beyond the roadway centerline, the roadway centerline can be used as the boundary slice; this allows the X Coordinate of the drilled shafts to be the same as the roadway offset of the drilled shafts. If the failure scarp does extend beyond the roadway centerline, some other reference point must be used, and the difference between the boundary slice location and the roadway centerline must be subtracted from the X Coordinate of the drilled shafts to get the roadway offset of the drilled shafts.

For example, in the example problem, the landslide head scarp is at 1.8 feet left of the roadway centerline. Therefore, the left (uphill) boundary of the model in the UA Slope 2.3 program has been set at 10.0 feet left of the roadway centerline. If the drilled shafts were placed at X=50.0 feet in the model, they would be at an offset of 50.0 – 10.0 = 40.0 feet right of the roadway centerline.

In Figure 900-6, the Main Menu screen with the Calculated Results of the analysis with the drilled shaft effect "Automatic Load Transfer Factor" is presented. In the Drilled Shaft Information on the lower left of the Main Menu screen, the drilled shaft Diameter is 3.00 ft, the CTC Spacing is 9.00 ft, and the X Coordinate is 52.00 ft. Note that in the Chart, in the upper right corner of the Main Menu screen, a representation of the drilled shaft (to scale with the cross-section) is displayed. In the Calculated Results, in the upper left corner of the Main Menu screen, the Factor of Safety of 1.30, the Force per Shaft of 169311.232 lb, and the Acting Point of the resultant, X: 52.000 ft and Y: 44.019 ft are displayed. The X coordinate of the Acting Point is always the same as the X Coordinate of the drilled shafts, while the Y coordinate of the Acting Point is two-thirds of the distance from the ground surface to the defined Slip Surface (the program represents the horizontal earth pressure load as a triangularly distributed load with an intensity of 0 at the ground surface).

Drilled shafts should be analyzed for a variety of geometric configurations of diameter (D) and spacing-to-diameter ratio (S/D). For each combination of D and S/D, the drilled shafts should be analyzed at varying offset locations (X Coordinate). Tables and graphs should be made for each geometric configuration of the drilled shafts, showing the relationship between drilled shaft location (X Coordinate) and Factor of Safety, and between drilled shaft location (X Coordinate) and Force per Shaft. If the drilled shaft X Coordinate is not the same as offset from roadway centerline (if the Y-Axis in the UA Slope 2.3 program is not at roadway centerline), then the drilled shaft location should be adjusted to roadway offset in the graphs. Figure 900-7 shows an example of the Factor of Safety versus Offset graph for the example problem.

Figure 900-6:UA Slope 2.3 Program Results with Drilled Shafts

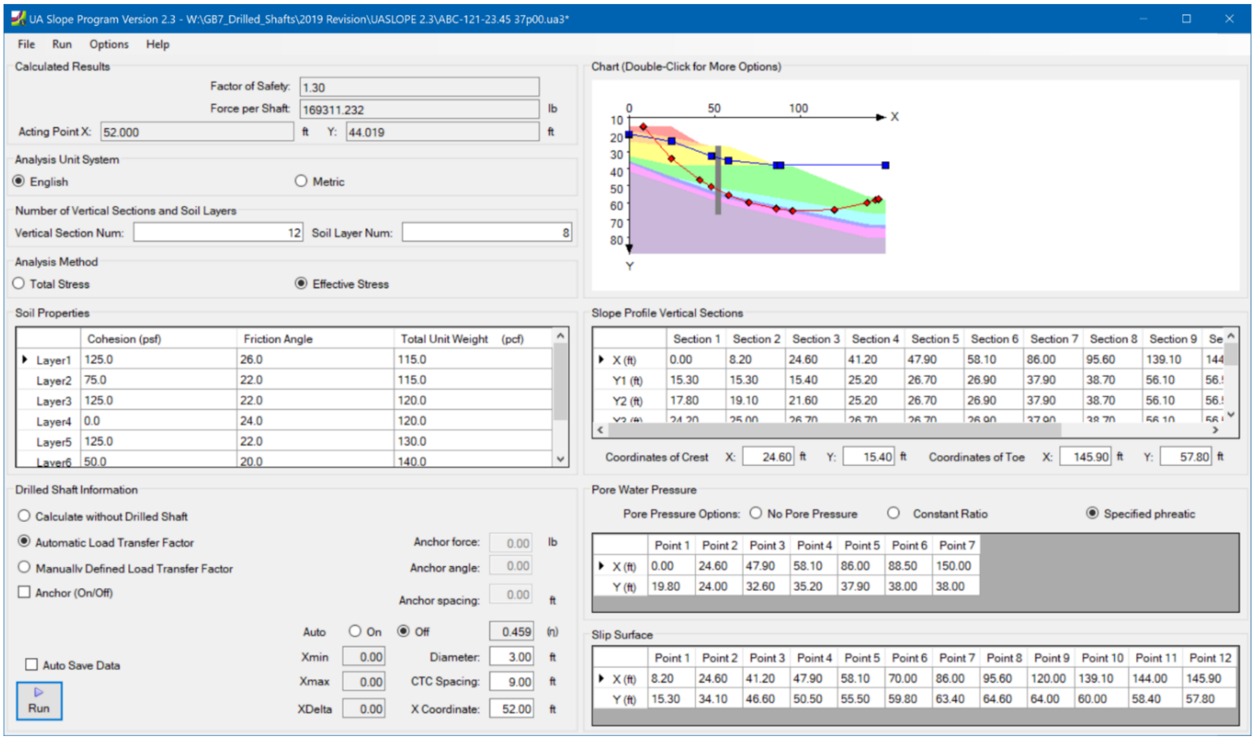
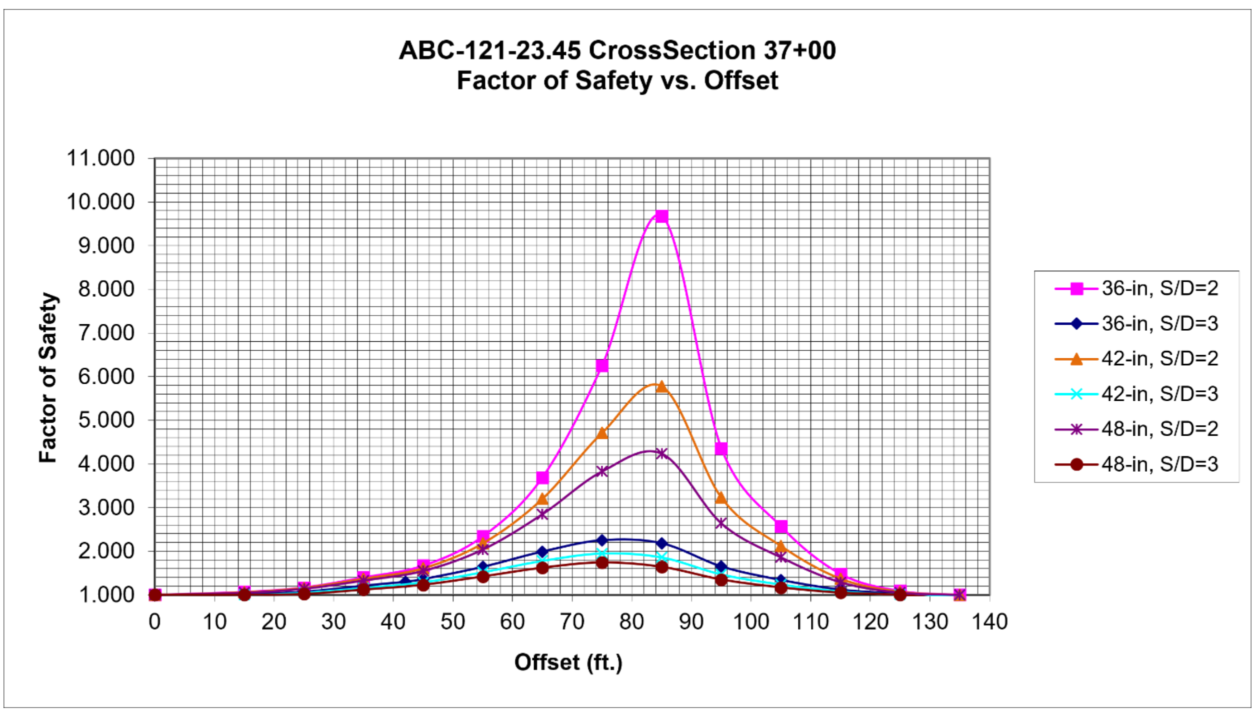


Figure 900-7: Factor of Safety vs. Shaft Offset from Roadway Centerline



Note that in the example graph, the X-axis represents offset from roadway centerline, which in this case, is 10 feet less than drilled shaft location (X Coordinate) used in the UA Slope 2.3 program. Also, the diameter of the shafts is provided in inches, and shaft geometry is expressed as a spacing-to-diameter ratio (S/D). The Factor of Safety generally increases from 1.0 to a maximum somewhere near the middle of the failure, and then decreases back to 1.0 at the toe of the failure. Note also that the Factor of Safety decreases as the spacing between shafts increases, and as shaft size increases – as a larger shaft size at the same S/D ratio has a larger space between shafts.

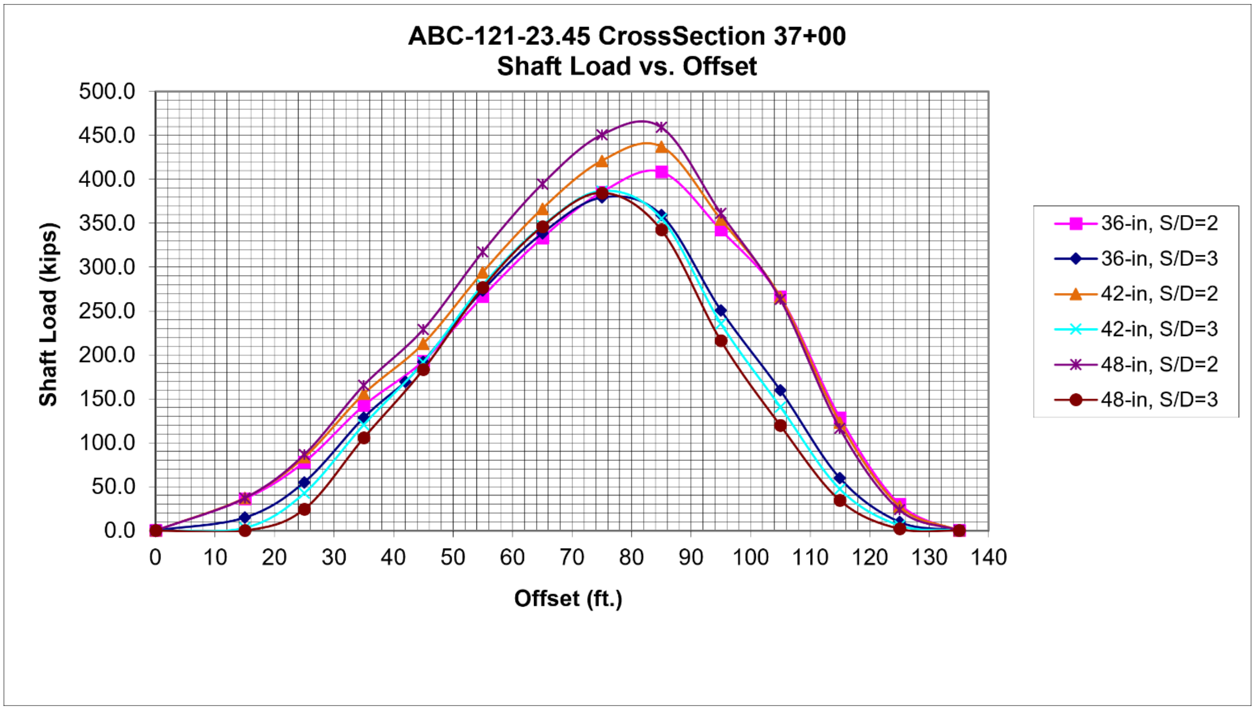
The UA Slope 2.3 program has a built-in feature to speed up the graphing process. The program includes a template input spreadsheet, titled "UASlope_Default.xlsx," into which the problem geometry can be input per Section 4.4 of the UA Slope 2.3 Program User's Manual. A separate template spreadsheet will have to be created for each combination of D and S/D; however, the spreadsheet can be set up to iterate through a range of X Coordinates at a defined interval. This greatly reduces the number of times the program will have to be run for a full analysis. A new copy of the spreadsheet is saved on the executing computer at the file path "C:\spile\Excel_Results," which will contain a tabular set of results along with a pair of graphs for Factor of Safety versus Offset and Shaft Load versus Offset. It should be noted that the graduate student who programmed this behavior did not understand the difference between X Coordinate and Offset from Roadway Centerline, therefore the first line in the table (with the minimum X Coordinate) will always be Offset 0 in the excel output file. The offset will have to be manually corrected by the engineer performing the analysis.

The associated Shaft Load versus Offset graph for the example problem is shown in Figure 900-8. It can be seen from this graph that the shaft load tends to follow a similar trend to the Factor of Safety, increasing from 0 to a maximum somewhere near the middle of the failure, and then decreasing back to 0 at the toe of the failure; this behavior is typical.

Based on the two sets of plots, a shaft size, geometry, and offset should be chosen which will maximize Factor of Safety, minimize shaft load, and conform to other constraints, such as right-of-way restrictions, and other issues which might affect constructability, such as slope steepness and waterways. The minimum Factor of Safety for slope stability required by ODOT is 1.3, therefore, an offset and geometry are typically chosen to meet this minimum Factor of Safety, and then the drilled shafts are assessed structurally, to determine if they are capable of resisting the predicted shaft load. In the example problem, drilled shafts of 36-inch diameter, with S/D=3 (9-foot center-to-center spacing) at an offset of 42 feet from roadway centerline (X Coordinate of 52 feet) had a Factor of Safety of 1.30, and were selected for design (see Figure 900-6 for the results of this analysis).

In some instances, it may be impossible to achieve the minimum Factor of Safety with a row of spaced drilled shafts, or right-of-way or constructability restrictions may mean that a sufficient offset to achieve the minimum Factor of Safety cannot be utilized. In this case, a drilled shaft wall may be necessary, the most common of which are a drilled shaft soldier pile and lagging wall, and a "plug-pile" lagging wall.

Figure 900-8: Shaft Load vs. Shaft Offset from Roadway Centerline



Soldier pile walls consist of a row of drilled shafts spaced at typically a 4-foot to 8-foot center-to-center spacing. HP-section or W-section steel beam reinforcements (soldier piles) are inserted vertically into the shafts, with the webs of the steel sections placed parallel to the direction of the landslide movement. Structural concrete is poured into the shafts up to the bottom of the proposed depth of lagging. Low-strength grout is often poured on top of the structural concrete to finish filling the holes. The soil and grout are then excavated down to the top of the structural concrete, and then precast concrete lagging panels are inserted in-between the steel beams, held in place by the flanges of the beams.

A plug-pile lagging wall consists of a row of structural drilled shafts spaced at a two-shaft-diameter center-to-center spacing (S/D=2). These structural shafts are reinforced with steel cages or steel beam sections, and then filled for their full length with structural concrete. Plug-piles of the same diameter are then drilled in-between the structural shafts, tangent to the structural shafts. The plug-piles are typically shallower (they usually do not penetrate bedrock), and serve the purpose of lagging, however, they are not connected to the structural shafts. The plug piles are filled with structural concrete but are not steel reinforced.

If the lateral earth loads are so high that a wall cannot be built to stand in a cantilever condition, any of the above wall types can be reinforced with grouted ground anchors (tiebacks) or with deadman anchors.

The UA Slope 2.3 program may also be used to calculate the loading on a drilled shaft retaining wall. For this case, set the drilled shaft Diameter (D) and the CTC Spacing (S) between shafts as presented above. However, in this case, the Manually Defined Load Transfer Factor setting under

Drilled Shaft Information should be used. This setting allows the Load Transfer Factor (η) to be explicitly defined; in the case of a wall, η should be set to zero (0), meaning that the drilled shafts take up the full load from the uphill soil slice, and no load is passed through to the downhill soil slice. The Factor of Safety calculated by the UA Slope 2.3 program should be ignored in the case of analysis for a wall. This Factor of Safety is for the *entire* defined shear failure surface, not for the drilled shaft retaining structure. If we use a wall, we often assume that the slope below the wall will continue to fail, and that the Factor of Safety for the slope will continue to be marginal or inadequate. Factor of Safety for the wall is assessed through a structural analysis and is separate from the Factor of Safety for stability of the slope.

In some cases, generally where the slope is fairly flat, and there is a large amount of the failure surface downhill of the drilled shaft retaining structure which may offer resistance to movement, the UA Slope 2.3 program may not be able to successfully calculate a factor of safety. This is because the downhill soil mass, in combination with the drilled shaft retaining structure, offers so much more resistance than the magnitude of the load on the back of the structure, that the factor of safety of the system exceeds a program limit of 100. In this event, the program will output a factor of safety of "100;" the load on the back of the drilled shaft retaining structure may still be conservatively utilized in further structural analysis, but it will be impossible to estimate the factor of safety of the entire system in the existing configuration.

903 LPILE ANALYSIS

Once the load on the shafts is determined, the reaction of the shaft to the load needs to be determined, including the drilled shaft head displacement, the shear and moment distributions. In this step it is also necessary to determine whether the drilled shaft is structurally capable of resisting the load. Any capable p-y analysis software, such as LPILE, or FBPIER may be used. Perform p-y analyses in accordance with Section 1501.7, except as modified in Section 903. ODOT OGE currently uses the program LPILE, developed by Ensoft, Inc., therefore, the examples in these sections will refer to this program.

903.1 Conversion of Force per Shaft to Distributed Lateral Loading

The UA Slope 2.3 program calculates an unfactored horizontal earth pressure (EH) resultant load per shaft, however, for proper structural analysis of drilled shaft reaction, we need to convert this to a more realistic distributed load. We note that the actual load distribution is complex, and impossible to determine without direct measurement or back-calculation through measurement of displacements, however, we feel that a triangular load distribution is a close enough approximation of the actual condition to develop a realistic calculation of distributed shear, moment, and displacement in the drilled shaft.

The depth to the shear surface, depth to bedrock, and single shaft resultant force need to be determined at the chosen offset for the placement of the drilled shafts, based on the desired Factor of Safety predicted by the UA Slope 2.3 program. Utilizing these known factors, we can calculate a triangular distribution of loading, from zero (0) at the ground surface to a maximum at the depth of the shear surface. This horizontal earth pressure (EH) load is represented solely as a horizontal distributed load, with no vertical component, as this is a more conservative assumption, providing the maximum lateral loading. Research has shown that the vertical load component is either insignificant or tends to provide a small amount of compression to the shaft, which marginally increases bending resistance. If using LPILE, the distributed load must be converted into units of pounds per inch (lb/in) of length along the drilled shaft.

If the horizontal distance between the drilled shafts and traffic loading is less than or equal to half the depth to the shear surface at the location of the drilled shafts (d_τ), also apply an (unfactored) vehicular live load surcharge (LS) to the drilled shafts equal to two feet of soil with a unit weight γ_s = 125 pcf, per AASHTO LRFD, Article 3.11.6.4.

Do not represent the load on the shaft as a single resultant point load, and "cut off" the top of the shaft at the point of application of this resultant load. This method does not realistically predict either the shape or magnitude of shear and moment distributions and cannot predict the displacement at the drilled shaft head.

903.2 p-y Modification Factors for Group Action

If the drilled shafts are at a center-to-center spacing closer than about 3½ diameters, a reduction in the soil resistance p, for the p-y curve behavior of the soil, must be considered. The loss in capacity is due to soil-structure-soil interaction, and an overlap in the region of the soil that provides passive resistance to the deflection of the drilled shafts when placed in a closely spaced group. This effect does not occur where drilled shafts are embedded in a relatively much stiffer material, such as bedrock or concrete, where the stress field effects are very limited, and the material does not deform substantially under the design loadings. Therefore, in LPILE, apply the p-multiplier (Pm) from the ground surface (or artificially lowered ground surface) to the top of bedrock or to the bottom of the drilled shaft, whichever is shallower.

Reese, Isenhower, and Wang, "Analysis and Design of Shallow and Deep Foundations" (2006) publish an equation for the pile group p-reduction factor for a single row of piles placed side by side,

$P_m = 0.64(S/D)^{0.34}$, for $1 \leq S/D < 3.75$, where $0.5 \leq P_m \leq 1.0$

This is an empirical relationship based on testing by several researchers in a number of different soil types. Utilize this equation for determination of the p-multiplier.

903.3 Soil Layering and p-y Models

LPILE will calculate a passive resistance, "Mobilized Soil Reaction," for the soil mass in reaction to the lateral load imposed on the shaft. The actual passive resistance of the downhill soil mass in a landslide will be reduced due to translation of the soil mass away from the drilled shafts; however, it will not usually become zero, as "full depth crack" theories assert. In order to model the loss of passive resistance of the downhill soil mass, we recommend three separate methods, depending on the type of drilled shaft retaining structure installation.

903.3.1 Method 1

For the case of a row of spaced drilled shafts, in which the downhill soil mass must remain in place, and where soil arching and downhill load transfer are dependent mechanisms for retention of the uphill soil mass, use a load transfer reduction, based on the Factor of Safety (FS) of the existing slide plane, calculated by the UA Slope 2.3 program with the inclusion of the drilled shafts. Please note, that for this design case, the UA Slope FS must meet the minimum required

Factor of Safety for an unreinforced slope (FS ≥ 1.30), and the slope must have a steepness of 2H:1V or flatter, as outlined in Section 800. In this case, the p-multiplier, as determined in Section 903.2, is reduced above the shear plane. The reduced p-multiplier is equal to:

[(1-1/FS) x P_m], or [P_m – (P_m/FS)]

For example, if S/D = 3.0 and FS = 1.30, then P_m = 0.930, and the p-multiplier above the shear plane = [0.930 – (0.930/1.30)] = 0.215.

903.3.2 Method 2

For the case of a retaining wall in which the downhill soil mass will be left as-is, and if stability analysis downhill of the proposed wall shows that the downhill soil mass does not meet the minimum required Factor of Safety for an unreinforced slope (FS ≥ 1.30), consider the downhill soil mass to be unstable. In this case, we may assume that the downhill soil mass will continue to fail away from the wall, while the wall retains the uphill soil mass. In order to model the anticipated loss of passive resistance of the downhill soil mass, artificially lower the ground surface in the LPILE analysis, completely discounting the passive resistance of the soil between the existing ground surface and the artificially lower ground surface. To do this, first determine the angle of steepness of the slope from horizontal, downhill of the drilled shafts (β_{dh}); then determine the depth to the shear surface at the location of the drilled shafts (d_τ). For slopes of steepness from β_{dh}=0° to 45°, lower the ground surface by an amount equal to d_τTAN(β_{dh}). For slopes of steepness β_{dh}=45° or more, discount the entire soil mass from the actual ground surface to the depth of the shear failure surface. Model all soil layers below the artificially lower ground surface normally, per the LPILE user's manual.

903.3.3 Method 3

For the case of a retaining wall in which the downhill soil mass will be regraded to a stable slope (lower at the base of the wall than behind the wall), set the ground surface in LPILE equal to the proposed regraded ground surface and model all soil layers below the proposed ground surface as in the proposed condition.

903.4 Drilled Shaft Length

Embed the drilled shaft in a solid stratum below the shear failure such that deflection at the drilled shaft head will be constrained to appropriate serviceability limits (see Section 903.8, below, for details of the required serviceability limits). Ideally, embed the shaft into bedrock to provide resistance to deflection. Regardless, extend the drilled shaft a minimum of 10 feet below the shear surface. Also select the total drilled shaft length such that the drilled shaft is geotechnically stable (see Section 903.9, below).

903.5 Steel Reinforcement

In the past, it has been common to reinforce concrete drilled shafts with steel reinforcing bar (re-bar) cages. However, due to the expense in time and money for labor to construct re-bar cages, and the relative fragility of these cages before they are embedded in the concrete shaft, it has recently been the practice to more commonly utilize steel HP-section or W-section beams as reinforcement. These steel sections are generally cheaper per pound of steel than re-bar cages, often require less weight of steel per length of drilled shaft, require no cost in labor for fabrication, and can be easily and quickly installed with little danger of distorting or otherwise damaging them during or before installation. However, we leave it up to the designer to choose whether to use a concrete shaft reinforced with steel re-bars or with a steel beam section. Alternately, the designer may design for both options, so that the contractor may choose which method to employ.

903.6 Section Type, Dimensions, and Cross-section Properties

Choose the appropriate Section Type in accordance with Section 1501.7.5 depending on whether a steel re-bar reinforced concrete shaft or a steel section embedded in a concrete drilled shaft is being used.

The ground surface should be represented as level, not inclined. Inclination of the ground surface is usually used in LPILE to represent pile batter, and in any event, represents a reduction in the soil resistance near the ground surface, which is not relevant, as we are already discounting soil resistance near the ground surface.

Unless there is a constructability concern which dictates a smaller rock socket diameter, set the diameter of the drilled shaft the same over its entire length. Use ASTM A709 minimum 50 ksi yield strength steel for the structural steel used in a steel reinforcing beam section. Use 60 ksi yield strength epoxy-coated steel for re-bar reinforcement, per C&MS Item 524. Typically, use Class QC 5 concrete, per C&MS Item 524. If using a drilled shaft reinforced with a steel beam section, see the Plan Note “DRILLED SHAFTS FOR SLOPE STABILIZATION” found on the OGE website. If the drilled shafts are of 7 feet or greater in nominal diameter, use QC 4 Mass Concrete instead.

903.7 Pile-Head Loadings and Options

Choose Pile-Head Loadings and Options in accordance with Section 1501.7.6.

Run two p-y analyses for each loading case; running one analysis with unfactored loading for the Service Limit State, to determine head deflection of the drilled shaft; and one analysis with factored loading for the Strength Limit State, to check the structural and geotechnical resistance of the drilled shaft.

903.8 LPILE Output

Review the LPILE output in both the Strength and Service Limit States in accordance with Section 1501.7.7. Regardless of the results of the “Top Deflection versus Length” plots, embed the drilled shaft a minimum of 10 feet below the shear surface.

In the Strength Limit State, pay particular attention to the possibility of artificial plastic hinging, and modify the analysis as necessary in accordance with Section 1501.7.8.

LPILE also generates a plot of Lateral Deflection versus Depth and calculates a (maximum) “Pile-head deflection.” For the unfactored Service Limit State analysis, limit the maximum Pile-head deflection to 1% or less of the drilled shaft length above bedrock (if not embedded in bedrock, this is 1% of the total drilled shaft length); however, if the drilled shafts are to be installed within 10 feet of the edge of pavement, and the pavement is not to be replaced as part of the same project, limit the Pile-head deflection to 2” or less. Use whichever serviceability limit requires the least deflection. If the drilled shaft deflects more than the required serviceability limit, we consider this to represent failure, and a stiffer reinforcement or larger diameter drilled shaft will have to be selected and re-analyzed.

903.9 Geotechnical Resistance

Perform a check of geotechnical resistance against overturning of the drilled shaft either with p-y analysis methods using LPILE or with moment equilibrium analysis methods.

Do not utilize the Geotechnical Strength Limit State check per FHWA-NHI-18-024 (GEC 10) – "Drilled Shafts: Construction Procedures and LRFD Design Methods", Section 12.3.3.3.1. This check is not intended for retaining structures and will produce overly conservative results.

903.9.1 LPILE Deflection Analysis

Check geotechnical overturning resistance using p-y analysis methods in accordance with Section 1501.7.9. This is by far the simpler method to check geotechnical resistance.

903.9.2 Moment Equilibrium Analysis

Check geotechnical overturning resistance using moment equilibrium analysis methods in accordance with Section 1501.4(B).

904 STEEL BEAM SECTION DESIGN

After determining the Service Limit State lateral deflection of the shaft and the Strength Limit State moment and shear distributions for the single shaft load by analysis with an appropriate p-y analysis software, such as LPILE, or FBPIER, check that the shaft reinforcement is capable of resisting the calculated factored maximum moment and maximum shear force. This section is provided to give guidance for the design of steel beam W-sections or HP-sections as drilled shaft reinforcement, as this is not typical practice at this time. If designing a conventional re-bar reinforced concrete shaft, utilize the LRFD design procedures for laterally loaded drilled shafts, per FHWA GEC 10, Chapter 12 and Chapter 16.

ODOT is now utilizing LRFD methods, per AASHTO LRFD, for the design of steel beam sections resisting shear and moment due to lateral earth loadings. Per FHWA Policy Memorandum Related to Structures, dated June 28, 2000, LRFD specifications are required for all new culverts, retaining walls, and other standard structures on which States initiate preliminary engineering after October 1, 2010. It is no longer acceptable to use Load Factor Design (LFD) methods or Allowable Stress Design (ASD) methods.

Use steel with a minimum yield stress (Fy) of 50 ksi (ASTM A709 grade 50) for beam sections used for landslide stabilization drilled shaft reinforcement. Per ODOT Bridge Design Manual (BDM) Section 302.4.1.1.C, ASTM A709 grade 36 is not recommended and is being discontinued by the steel mills.

904.1 Minimum Concrete Cover for Reinforcing Steel

Whether designing a drilled shaft reinforced with a steel reinforcing bar (re-bar) cage or with steel HP-section or W-section beams, ensure that the reinforcement can fit within the drilled shaft with the minimum required concrete cover per BDM 301.5.7 and C&MS 509.04.B. The minimum concrete cover between soil and steel reinforcement for a drilled shaft of 4 feet or less in diameter is 3 inches. The minimum concrete cover between soil and steel reinforcement for a drilled shaft greater than 4 feet in diameter is 6 inches.

904.2 Load and Resistance Factor Design (LRFD)

For Strength Limit State design, use a load factor of γ_{LS}=1.75 for the vehicular live load surcharge (LS) and a load factor of γ_{EH}=1.50 for the horizontal earth pressure (EH), per AASHTO LRFD, Article 3.4.1. Use factored loading and resistance for structural capacity (flexure and shear) design of the steel beam section reinforcement. Use a resistance factor φ_f=1.00 for flexural resistance and a resistance factor φ_v=1.00 for shear resistance per AASHTO LRFD Article 6.5.4.2. Check the flexure resistance of the steel beam section according to AASHTO LRFD Article 6.10.8. Check the shear resistance of the steel beam section according to AASHTO LRFD Article 6.10.9. If the steel section is embedded in a concrete drilled shaft, assume that it has continuous lateral bracing and transverse stiffening. If the steel section extends above the drilled shaft and is unbraced (as in a soldier pile wall) analyze the steel section for flexural buckling with an unbraced length equal to the exposed length, per AASHTO LRFD Article 6.9.4.1.2.

904.3 Iterative Design Process

Use an assumed steel section for LPILE (or other comparable software) analysis to determine the drilled shaft head deflection (Service Limit State) and distributed and maximum moment and shear (Strength Limit State) for the beam. Check that the selected steel section is capable of resisting the calculated maximum moment and maximum shear per AASHTO LRFD procedures, and check that the drilled shaft head deflection is less than the required serviceability limit (see Section 903.8 above). If these requirements are not met, select a more capable steel section. If the minimum capable steel section will not fit within the selected nominal drilled shaft diameter with the minimum required concrete cover, consider a larger diameter drilled shaft. If the deflection, flexure, and shear requirements are greatly exceeded, consider selecting a lighter steel section (and possibly smaller diameter drilled shaft) to save cost.

Every time a new steel section or a new nominal shaft diameter are selected, recalculate the drilled shaft reaction with LPILE, and check the deflection and the flexure and shear resistance of the steel section per AASHTO LRFD specifications.

905 SLOPE PROTECTION AND REGRADING

Once a structural solution has been designed to remediate the slope failure or retain the soil mass, either with a row of spaced drilled shafts or with a wall, ensure that critical parts of the remainder of the slope geometry will remain in place, so that the structural solution will not fail. Failure of the structural solution does not necessarily mean failure of the structural elements. Failure also includes any movement of the soil mass, independent of the structural fix, which compromises the integrity of the facility the drilled shafts are designed to protect.

If using a row of spaced drilled shafts, per the Liang method, and if the slope below the shafts continues to move, or if a new instability develops below the shafts, this system is jeopardized. This solution depends on soil arching and a percentage of the load being transferred between the shafts, through to the downhill soil mass. This mechanism increases the Factor of Safety (the nominal resistance versus load) of the entire landslide by effectively decreasing the method of slices interslice force. However, it depends on the soil mass downhill of the shafts to support the remaining interslice force. If the soil mass downhill of the shafts is removed, the passive support of this soil is eliminated, and the soil mass uphill of the shafts will fail between the shafts, causing the entire system to fail. In this case, the drilled shafts could remain in place without suffering a structural failure.

However, if the structural solution is designed to depend on the passive resistance of the downhill soil mass below the shear failure surface to provide support for the structural elements, and that soil mass fails away – either through a new shear failure or by erosion – the structural elements of the system may fail. Furthermore, if a new shear failure develops uphill of the structural retention system, we regard this as failure of the system, even though no failure of the structure occurs.

Therefore, it is important to protect the integrity of the remainder of the slope once a structural solution has been selected and designed. Perform slope stability analyses of the slope both uphill and downhill of the structure. If the soil mass uphill of the structure is found to have inadequate stability, the uphill slope may need to be regraded, may need reconstruction with special benched excavation and embankment backfill (see Section 800), may require the addition of subsurface drainage, or the structure may need to be modified (either moved to a new lateral location or increased in height) to properly retain the uphill soil mass. If using unreinforced embankment fill material or natural soil per C&MS Item 203, the uphill slope may not be regraded to steeper than 2H:1V. If using a Reinforced Soil Slope (RSS), the uphill slope may be steeper than 2H:1V, but no steeper than 1H:1V. However, analysis of RSS stability is beyond the scope of this document.

If the slope downhill of the structure is found to have inadequate stability, other measures may have to be taken, depending on the type of structural solution. If using a row of spaced drilled shafts, the downhill slope must be stabilized by one of the methods recommended above, or the row of shafts must be moved further downhill, in order to increase the stability of the lower slope. If the shafts are moved, the shaft loading will change, and the entire system will need to be redesigned. Regrading or reconstructing the lower slope may not be practical, due to limiting geometry at the toe of the slope. If neither movement of the shafts nor regrading of the lower slope can successfully increase the stability, a wall will be necessary.

If a wall is used, failure of the downhill soil mass may not contribute to failure of the system overall, as long as the material into which the drilled shafts are embedded (usually bedrock) does not fail, and as long as the drilled shafts are capable of standing cantilever or are reinforced by tiebacks. If there is a chance that the material into which the drilled shafts are embedded may fail along with the downhill portion of the landslide, design the drilled shafts for this condition, and increase the embedment.

Regardless of structural stability of the drilled shaft reinforcing system, there may be other reasons: environmental, aesthetic, or otherwise, for which the downhill soil mass may need to be stabilized. In this case, the downhill slope may need to be flattened, and/or the soil may need to be cut to below the existing shear failure surface. If the slope failure is along the bank of a river or other stream, erosion control – typically C&MS Item 601 Rock Channel Protection (RCP) – may be necessary at the base of the slope in order to protect against erosion undercutting the toe of the slope and inducing an additional instability. Also, if along a body of water which is prone to flooding, design the system to resist loadings under the "normal" condition, the 100-year recurrence flood condition, and the rapid-drawdown condition. In general, design the structural system for the worst-case conditions which may be anticipated during the life of the structure.

SECTION 1000 ROCK CUT SLOPE & CATCHMENT DESIGN

1001 GENERAL

The guidance in this section originated as Geotechnical Bulletin 3; use it when designing rock cut slopes, rockfall catchment, and rockfall controls. This guidance is based on research presented in Shakoor and Admassu (2010) entitled "Rock Slope Design Criteria" (State Job Number 134325), previous FHWA co-sponsored research, and the experience of the OGE. This guidance should be viewed as the philosophy of the OGE regarding rock cut slope and catchment design. It is not possible to provide design guidance for all potential scenarios. If a scenario is encountered that falls outside those described here, perform the design in consultation with the OGE and DGE.

The Designer is responsible for preparing a design that is based on a site-specific geotechnical exploration and achieves the optimal balance of safety, construction costs, and future maintenance costs. Avoid "template" designs; instead, use appropriate information regarding the site geology, slope of the natural hillside, and the condition of cut slopes in similar geology within proximity to the project to determine the appropriate cut slope configuration(s). The designed configuration will be influenced by lithology, rock properties, and bedrock structure. Research and experience have shown that a consistent design methodology can be formulated by using properties such as intact rock strength, rock durability, fracture frequency, regional joint characteristics, and other common rock properties.

The design approach first satisfies the overall global stability of the rock cut. It is recognized that in nearly all cases typical geologic and geometric conditions exist throughout Ohio, namely, near horizontally bedded sedimentary rock strata with a range of lithologies that include limestone, dolomite, sandstone, siltstone, shale, claystone and coal. Those strata defined as shale in the ODOT C&MS Item 203.02.P are considered a rock type and are included in this guidance. Based on practice, OGE experience, and results of research (Woodard, 2004; Shakoor and Admassu, 2010), it is recognized that the primary causes of degradation and failure of rock slopes in Ohio are the differences in durability of rock units and intersecting discontinuities. The design approach presented here accounts for differences in durability of geologic units and anticipated geologic structure of rock slopes in Ohio.

The typical geologic structure in Ohio makes the necessity for rigorous rock mechanics structural analysis (kinematic analysis) rare for cut slope designs. If the designer believes a detailed kinematic analysis is necessary, contact the DGE or OGE, as the design may be beyond the considerations of these guidelines.

The Designer should be aware that a design following these guidelines may result in a slope design with varying slope angles and benches where the excavation quantities and/or costs are similar to creating a continuous 1.5H: 1V cut slope, for example. Although the use of a uniform slope through varied geologic formations may simplify construction, it likely won't be effective against long-term weathering. Therefore, replacing the designed rock slope with a constant slope is generally not recommended.

OGE recognizes that rockfall poses a serious geologic hazard, and the selection of appropriate slope configurations, as well as rockfall catchment controls, will minimize this hazard. Use the guidelines presented in this section, which are based on OGE experience and FHWA co-sponsored research, to design adequate rockfall catchment and controls.

Expanded discussion of the topics presented in this section can be found in the ODOT Rock Slope Design Guide on the OGE website. All references to slope angles by ratio, such as 2:1, are H (horizontal):V (vertical).

1002 DEFINITIONS

1) Lithologic Unit: A body of rock comprised of a similar mineral composition, grain size, and engineering characteristics.

2) Competent Unit: A lithologic unit described as a limestone, sandstone, or siltstone and based on the following guidelines:

    a) Any limestone or sandstone visually described as moderately strong or stronger based on SGE Section 605.5

    b) Any limestone or sandstone visually described as very weak, weak, or slightly strong based on SGE Section 605.5; have a unit weight of 140 pcf or greater; or a unit weight less than 140 pcf with a second cycle (Id_2) SDI value of 85 percent or greater as based on ASTM D 4644

    c) Any siltstone with a second cycle (Id_2) SDI greater than 85 percent as based on ASTM D 4644.

3) Incompetent Unit: A lithologic unit described as shale or claystone, or a competent lithologic unit described as slightly strong, weak, or very weak based on SGE Section 605.5, with a unit weight less than 140 pcf and an Id_2 value less than 85 percent.

4) Design Unit: A portion of a slope, or the entire slope, that can be cut at a consistent angle. A design unit may be comprised of single or multiple lithologic unit(s). A design unit can be selected based on lithology and the anticipated slope failure(s). The thickness of a design unit can range from a relatively small thickness (minimum 10 feet) to the height of the entire cut slope. Three (3) design units are considered in this guidance, defined as follows:

    a) Competent Design Unit: Consists of greater than 90 percent competent units. The failures anticipated to occur in this design unit are those controlled by unfavorable orientation of discontinuities (plane, wedge, or toppling failures).

    b) Incompetent Design Unit: Consists of greater than 90 percent incompetent units. The failures anticipated in this design unit include raveling, mudflows and rotational slides.

    c) Interlayered Design Unit: Consists of interlayered competent and incompetent units, each ranging in proportion from more than 10 percent to 90 percent. Undercutting-induced failures (rockfalls) and mudflows are the anticipated primary failures in this design unit. However, raveling and rotational slides are possible.

5) Daylight: Where the cut slope intersects with the existing ground surface at the top of the cut; no additional cut above and beyond this point.

1003 SUBSURFACE EXPLORATION

Exploration for a rock cut slope, which includes geologic explorations, data collection, and presentation of information, are vital to the design and construction of rock cut slopes. Each specific project involves unique situations, and the explorations should be planned accordingly.

When designing the rock cut slope configuration, evaluations of the current conditions of existing cuts or nearby exposures of the same material should be performed, if available, and used to confirm the appropriateness of a design slope angle. For example, if a slope is stable at 1:1, then a new slope cut at 1:1 should also be stable. Additionally, the County Manager, or an individual knowledgeable of the area, should be contacted to determine the maintenance and performance history of an existing cut slope and/or nearby cut slopes. Periodic maintenance can mask the true performance of a slope.

Evaluate existing rock cuts and slopes comprised of similar geology and located near a new cut area to assess the performance of the relevant rock units. New rock cuts in areas where there are few, if any, rock exposures, will require a detailed subsurface exploration in the absence of existing rock exposures.

Follow the guidelines in SGE Section 303 for planning the subsurface exploration of a rock cut. The subsurface exploration program (e.g., borings) should be tailored to the site-specific conditions determined after the site reconnaissance has been performed. Variations can occur, even in similar geology, both vertically and horizontally; sometimes, in just a few feet.

Existing rock cut slopes that are to be rehabilitated have rock exposures that are to be studied as part of the exploration efforts. Depending on the amount of information available from the existing slope and the scope of the remediation, subsurface explorations (e.g., borings) may be limited, or not required at all. As an example, a rehabilitation project that is being completed as part of widening of an existing cut that is performing well may not need additional borings. A review of archived subsurface data and characterization of the existing cut slope may be enough for design.

Follow procedures for bedrock sampling and testing, including bedrock description and strength testing, in SGE Sections 400 and 600, in addition to the following requirements:

1) Perform a representative number of unconfined uniaxial strength tests (ASTM D 7012) for limestone or sandstone units visually described as slightly strong, weak, or very weak (estimated strength less than 3,600 psi). At least one test should be run for each unit. Include the unit weight of the design unit in the presentation of the test results. If the samples are too small for unconfined strength tests, use the point load test. See Section 406.2 regarding details and a discussion of the point load test.

2) Perform at least one Slake Durability Test (ASTM D4644) per lithologic unit for the following units:

   a) Competent Design Units, described as slightly strong, weak, or very weak limestone or sandstone, with a unit weight less than 140 pcf.

   b) Siltstone.

   c) Incompetent Design Units.

   d) Interlayered Design Units, on the incompetent lithology only.

If the Designer believes additional test procedures are warranted for a specific project, such as those used to determine direct shear strength, Young's Modulus, Poisson's ratio, tensile strength, hydraulic conductivity, or others, contact the DGE.

Additional guidance for rock slope design exploration can be found in Section 200 of the ODOT Rock Slope Design Guide.

1004 DESIGN OF SLOPE ANGLE

The design of rock cut slopes is a progressive process, with the number of steps dependent on the size of the project. A major project (Path 4/5) will likely have a Preliminary Engineering Phase which may include a preliminary exploration program consisting of a limited amount of drilling and testing, resulting in preliminary design(s). In this phase, alternative alignments are usually considered. Once an alignment is established, a final exploration program of drilling, testing and rock mapping is executed in the Stage 1 Design Phase. Be sure to identify any missing data or gaps in the data from the preliminary program when planning the final program. The preliminary design is refined using the final program data. For smaller projects with known alignments the entire exploration is typically performed at the same time in the Stage 1 Design Phase.

The rock cut slope configuration is influenced by a number of factors; lithology, strength, weathering resistance, bedding characteristics, joint inclination, and discontinuities. Three design units; competent, incompetent, and interlayered, have been identified as representing the conditions typically encountered in Ohio. The following sections outline the guidelines for each of the Design Units.

1004.1 Competent Design Units

Once it is established that the rock slope will be comprised of competent design units, the cut slope inclination may be determined based on Rock Quality Designation (RQD) as presented in Table 1000-1.

Table 1000-1: Recommended Cut Slope Angles for Competent Design Units

RQD (%)	Recommendations
0-50	1:1; identify any concerns
51-75	Review global stability and design based on engineering judgment
76-100	0.5:1; consider 0.25:1 for thickly bedded units

1004.2 Incompetent Design Units

The design slope angle for an incompetent design unit is based on the average second-cycle SDI (Id_2); guidelines are presented in Table 1000-2.

Table 1000-2: Recommended Cut Slope Angles for Incompetent Design Units

SDI (Id_2) (%)	Recommendations
< 20	2: 1 or flatter -Special design; consult the DGE
20-60	2:1
60-85	1.5:1
85-95	1:1
95-100	1:1 or steeper; consult the DGE

For incompetent design units that have SDI less than 20 percent or result in a design slope steeper than 1:1, use engineering judgment and consult with the DGE.

1004.3 Interlayered Design Units

Interlayered design units exhibit significant stratigraphic variations which need to be considered in a cut slope design. Four stratigraphic configurations for interlayered units, designated as Type A through Type D, are identified and presented in Table 1000-3. Provide adequate catchment and drainage for all designs.

Table 1000-3: Stratigraphic Configurations for Interlayered Units

Type	Description
Type A	Very thick (>3 ft) competent unit underlain by incompetent unit
Type B	Medium to thick bedded (10 inches to 3 ft) ***sandstone and/or siltstone*** units interbedded or interlayered with incompetent units in variable proportions
Type C	Medium to thick bedded (10 inches to 3 ft) ***limestone*** units interbedded or interlayered with incompetent units in variable proportions
Type D	Thin bedded (2 to 10 inches) limestone units interbedded with incompetent units in variable proportions

1004.3.1 Design Approach for Type A Stratigraphy

Type A stratigraphy consists of a very thick competent unit underlain by an incompetent unit, and typically produces topples of flat or cubical rockfall debris caused by undercutting of the competent unit. The design should focus on avoiding toppling failures of the competent unit by minimizing excessive weathering of the incompetent unit. The design unit should follow the contour of the unit contact. The following slope design options are recommended for Type A stratigraphy:

1) Cut the competent unit at 0.5:1 to avoid toppling failures. If the incompetent unit is significantly thick (10 feet thick or greater), cut it at an angle as specified in Table 1000-2.

2) If the incompetent unit is 3 feet to 10 feet thick, cut it at 1:1. If the incompetent unit is less than 3 feet thick, cut it at the same angle as the overlying competent rock.

3) If the incompetent unit is 3 feet thick or greater, provide a geotechnical bench following guidelines in Section 1005 at the contact between units.

4) If the competent unit consists only of very thick to massive-bedded sandstone, the slope may be cut at 0.25:1. Use of this steeper cut slope angle may result in the need for localized or patterned stabilization using rock bolts. This design will be based on engineering judgment through consultation with the DGE.

1004.3.2 Design Approach for Type B Stratigraphy

Type B stratigraphy consists of medium to thick bedded sandstone and/or siltstone units interbedded or interlayered with incompetent units, which typically results in flat or cubical rockfall debris. The design approach for this type of stratigraphy should be to either cut the slope at a uniform flatter angle to reduce undercutting or cut it at a steeper (0.25:1) angle and provide an effective catchment ditch. Due to the thinness of the sandstone/siltstone units, treating each unit as a separate design unit with multiple benches is impractical.

If the design unit is comprised of more than 50% competent units, either:

1) Cut the slope at a uniform angle of 1.5:1, or

2) Cut the slope at 0.25:1 and evaluate if stabilization of the sandstone/siltstone units is needed. This option will likely require more maintenance.

If the design unit is comprised of less than 50% competent units, cut the slope at 1.5:1.

1004.3.3 Design Approach for Type C Stratigraphy

Type C stratigraphy consists of medium to thick bedded limestone units interbedded or interlayered with incompetent units, and typically produces cubical rockfall debris. The incompetent units are too thin to be independently designed.

If the design unit is comprised of more than 50 percent competent units, cut at 0.25:1 for design unit thicknesses less than or equal to 25 feet or 0.5:1 for design unit thicknesses greater than 25 feet. The preferred design approach is to cut the design unit at a steeper angle and provide adequate catchment. Stabilizing limestone units in the upper portions of the slope where there is a greater potential to generate rockfall may be justified and should be evaluated. Coal seams are common within this stratigraphy and should be protected from weathering. Placing benches on top of coal seams might not necessarily prevent undercutting.

Where the design unit is comprised of less than 50 percent competent units, the incompetent units are usually comprised of red bed claystone. The design approach should focus on reducing the degradation of the thick incompetent units and retaining the weathered material on the slope face. This can be accomplished by constructing a serrated slope, a series of small, 3 to 4-foot wide benches. The design should be based on the incompetent rock units and follow guidance in Table 1000-2. If the slope contains significant thicknesses of red beds, refer to the Special Care Formations sub-section.

1004.3.4 Design Approach for Type D Stratigraphy

Type D stratigraphy consists of thinly bedded limestone units interlayered with incompetent units. This type of stratigraphy is likely to generate flat-shaped rocks that can have long trajectories in the presence of steep slopes. Thinly bedded limestone units are commonly associated with marine limestones that can be identified as fossiliferous (typically found in Southwestern Ohio). Field observations indicate that where limestone comprises more than 50 percent of the slope, toppling and other types of undercutting-induced failures can occur. Type D design units should be cut at 1:1 or flatter based on engineering judgment.

1004.4 Special Care Formations

Through experience and observation, special care formations have been identified throughout Ohio. Special care formations are rock formations that are potentially prone to:

- Rapid weathering because of low durability

- Gradual reduction in shear strength caused by weathering

- Landsliding where over-steepened

The guidance in this document may need to be modified using local experience and engineering judgement when designing a rock cut in a special care formation. The special care formations identified in Ohio include:

- Conemaugh formation; specifically, the red beds identified as Round Knob Shale (below the Ames Limestone) and Clarksburg Red Shale (below the Connellville Sandstone)

- Monongahela formation: specifically, the red beds identified as Upper Uniontown Shale and Tyler Shale

- Washington formation; specifically, the red beds identified as Creston Red Shales

- Fairview/Kope formation; generally, highly weatherable shale in Cincinnati Area

- Miamitown formation; generally, weatherable shale in Cincinnati Area

- Sharon and Blackhand formations; specifically, friable sandstone

1005 BENCHES

The purposes of a bench include:

- Providing erosion protection of a less durable rock underlying a more durable rock where weathering and erosion may result in undercutting.

- Allowing for overall steeper angles of a slope where weaker lithologic units are present.

- Providing construction access and constructability of the slope.

- Providing slope configuration transitions due to design unit changes, overburden thickness changes, and existing ground relief.

Even though rock slope benches provide some degree of protection against rockfall, this is considered a secondary attribute of benches. Benches should be located to enhance overall slope stability, provide construction access, and transition configuration changes; not as rockfall catchment.

Benches constructed with the specific intent of catchment should be avoided. FHWA discourages mid-slope benches because they are rarely cleaned and could become launching features for rocks (FHWA, 1998). In general, mid-slope benches are not effective for rockfall control unless they are directly beneath a near vertical slope (0.25:1 or steeper). Design of slopes that include maintained (cleaned) benches will require access points and enough width (accounting for weathering) for equipment access.

Three types of benches are identified in this guidance and are described below.

1005.1 Overburden Bench

Overburden is the soil overlying the bedrock on and at the top of a slope. The thickness of overburden soil and depth to bedrock along a cut slope will vary. Since the overburden is usually cut at a flatter angle than the underlying bedrock, the soil-bedrock interface is a common slope angle transition point. If the overburden thickness is different than what is shown on the plans, an overburden bench, constructed at the soil-bedrock interface, provides space to make transverse adjustments to the cut line in construction, in order to maintain the plan cut slope angles and offsets. The overburden bench is typically 10 feet wide. Consider run-off drainage from above the cut; The overburden bench may need to be modified to handle this water, especially in large recharge areas.

Slopes in the soil overburden zone (where the soil is over 10 feet thick) should typically be cut at a 2:1 angle; confirm that such a slope will be stable. If a 2:1 soil slope does not daylight within a reasonable distance, a slightly steeper slope may be necessary to minimize right-of-way and excavation and avoid “chasing the slope” for a great distance.

In some cases, the overburden may be very thin and/or comprised mostly of severely weathered bedrock. If the overburden zone is less than 10 feet thick or the natural slope is 1:1 or steeper, rounding of the top of the cut to blend into the natural slope is permissible; refer to L&D1, Section 307.2.3 and use Sample Plan Note G101.

1005.2 Geotechnical (Lithologic) Bench

A geotechnical or lithologic bench is a bench placed at the top of a less durable design unit, such as shale or claystone, that underlies a more durable design unit, such as sandstone or limestone. The purpose of a geotechnical bench is to provide protection against undercutting of the more durable design unit as the less durable design unit weathers and erodes. Guidance for the design of geotechnical benches is provided below.

1) For an incompetent design unit 10 feet thick or less, the bench should be 10 feet wide.

2) For an incompetent design unit thicker than 10 feet, the bench should be made wider as necessary, depending on the anticipated weathering of the unit. The designer should consider and predict the anticipated rate of weathering and the angle of repose of the weathered slope. For example, if a 15-foot thick design unit is cut at a 1:1 slope angle and is anticipated to weather to a 2:1 slope angle, the designer should consider a 15-foot wide geotechnical bench to prevent future undercutting. The designer should evaluate any existing exposures of the same design unit in the area to assist in predicting the weathered slope angle and rate.

3) For interlayered design units, provide a minimum 10-foot wide bench at the contact between different design units; a wider bench may be required and should be evaluated.

4) Where a permeable formation overlies an impermeable formation (including areas of fractured flow), which may indicate a potential aquifer zone, consider configuring the bench for drainage.

5) The longitudinal grade of a bench should follow the base of the competent rock; adjustments in construction are expected. The bench should typically have a grade towards the road of 3 to 10% to promote drainage away from the slope. An exception may be considered where the bench is used to drain an aquifer or recharge runoff, in which case a negative grade (batter) may be used to capture water and transmit it longitudinally. Drainage likely will be necessary at and below coal seams. Be aware that bench grades are difficult to control when rock is blasted; slight grades may not get constructed correctly.

6) Install a bench drain along the contact between competent and incompetent rock units where groundwater is encountered or anticipated. Install a backslope drain or a diversion berm behind the slope crest to reduce runoff on slope face.

7) For coal, clay, or mineral seams of mineable thickness, or in the case of known or suspected underground mines that will be located within the cut slope, a 20-foot wide bench should be placed below suspected mine voids and above unmined seams.

8) Where there is a competent unit overlying an incompetent unit near the base of the slope, include a 10-foot wide bench below road grade to prevent undercutting of the competent unit during maintenance procedures.

9) Where using these guidelines results in different types of benches in the vicinity of each other (e.g., a construction bench and a geotechnical bench within a few feet vertically), use engineering judgment to combine benches for a practical constructible design.

10) Access roads to benches will most likely require additional right of way. Sufficient width for equipment access on maintained benches will also be necessary.

11) The face of the slope will not be uniform and will have relief transverse to the slope, especially where there are drainage channels. Because of this, bench widths will likely vary; being narrowest (but at least as wide as required) in the vicinity of these drainage channels.

12) A minimum burden thickness (width of the cut material) needs to be maintained to safely support construction traffic. Approximately 30 feet of rock burden (excluding overburden soil) measured transverse to the slope is necessary to safely allow for two-way construction traffic.

13) Geotechnical benches must be field adjusted during construction to align with elevation changes in the bedding contact.

1005.3 Construction Bench

A construction bench is a 5-foot bench used to accommodate construction practices. Where design unit thicknesses are greater than 30 feet, blasting typically occurs in maximum 30-foot lifts. In order to maintain the cut line, especially in tall slopes, construction benches provide transverse slope offsets to accommodate drill access for pre-split of underlying lifts, and to account for variances that occur in drilling, such as tool wander.

For slopes steeper than 1:1, or where pre-splitting is specified for a 1:1 slope, include a 5-foot wide horizontal construction bench every 30 vertical feet of a rock cut slope where no geotechnical benches are required. The actual widths of construction benches in construction will vary from the design.

1006 ROCKFALL CATCHMENT

OGE has established a rockfall catchment design criteria of 95% catchment at the edge of pavement or shoulder (where shoulder exists). Adequate rockfall catchment must be included in the design of rock cut slopes. An effective method of minimizing the hazard of rockfalls is to control the distance and direction the rocks travel. The recommended and most frequently used method to control rockfall in Ohio is the appropriate sizing of a catchment area or ditch. Other rockfall control and protection methods include barriers, wire mesh fences, and mesh slope drapes (Refer to ODOT Rock Slope Design Guide Section 504 for additional information). A common feature of all these protection methods is their energy-absorbing characteristics in which the rock is either stopped over some distance or is deflected away from the roadway.

Catchment ditches are considered the primary rockfall control method along new slopes. Barriers and other similar controls are considered to be solutions to rockfall problems along existing rock slopes. When designing a new rock cut slope, a barrier or a combination of a catchment ditch and a barrier should only be considered in special circumstances, such as:

- to satisfy roadside grading requirements in accordance with L&D1 Section 307.2.1;
- to address right-of-way concerns;
- to address changing slope condition of bedrock quality; or
- where the cut is very high.

Design charts and rockfall computer simulation programs are used to select and design effective rockfall protection measures. If design charts are used as the basis for catchment design, check the design by analyzing representative critical sections along the rock cut slope using a rockfall simulation program (Colorado Rockfall Simulation Program [CRSP] or equivalent software). The simulation program will either validate the design chart or indicate a need for a larger catchment area. The final catchment design should be the greater dimension determined from the two design tools.

Geotechnical (lithologic) benches are not to be designed as a rockfall mitigation measure, however, their ability to attenuate rockfall hazards should not be ignored. Include any geotechnical benches in the rockfall simulation models, considering both end of construction and long term (weathered) conditions. Do not include construction benches in the model.

Utilizing a combination of sources, including other state DOT standards, FHWA cosponsored research, and ODOT research, recommended catchment widths have been determined for varying cut slope heights and angles and varying catchment foreslope angles; see Table 1000-4. These recommendations are based on minimum rockfall containment criteria of 95% within the catchment area. Two typical ditch configurations are considered, a constant angled foreslope (Figure 1000-1) and a flat bottom with a foreslope (Figure 1000-2) and. The flat bottom configuration allows for easy equipment access for ditch clean-out. These configurations may need to be modified for hydraulic design.

For a slope with multiple slope angles, user the lowermost slope angle, where the slope intersects the ditch, to determine the catchment width. The catchment width for an individual rock cut slope section should not vary and should be based on the critical section, or widest resulting catchment area, of the slope design. The critical section is typically the maximum rock cut slope height, however, shorter slope sections where larger block sizes are anticipated may be the critical section. The cut slope height, H, is defined as the vertical distance from overburden bench or lowest 2:1 or flatter slope of more than 10 feet in height, to the base of the slope.

Figure 1000-1:Typical Configuration for Catchment Area with Constant Angled Foreslope

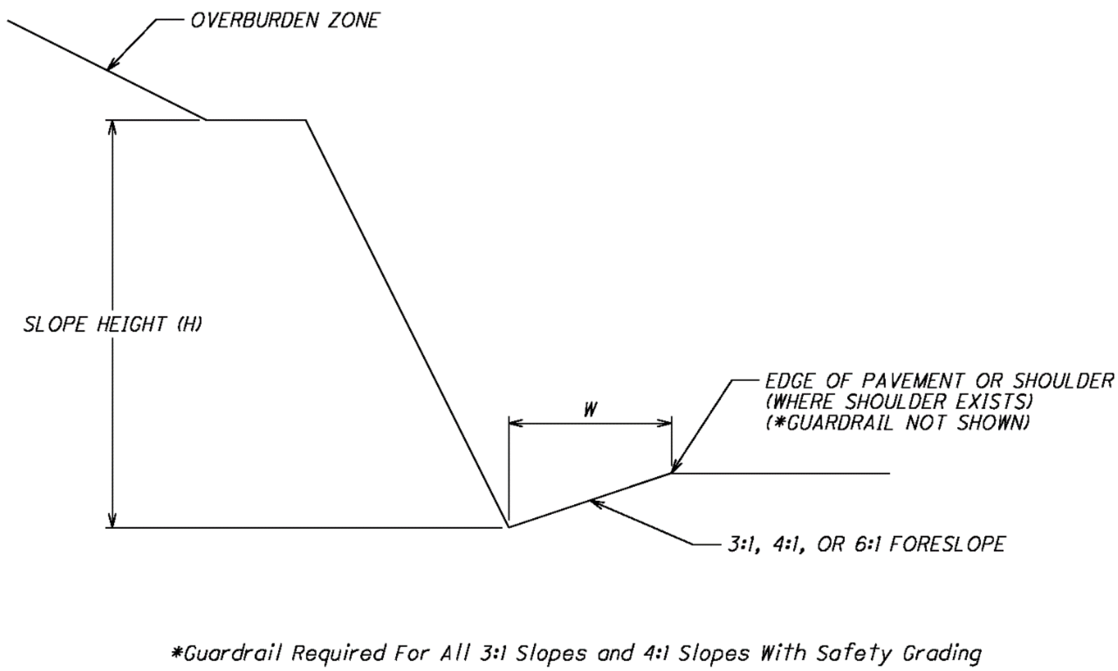
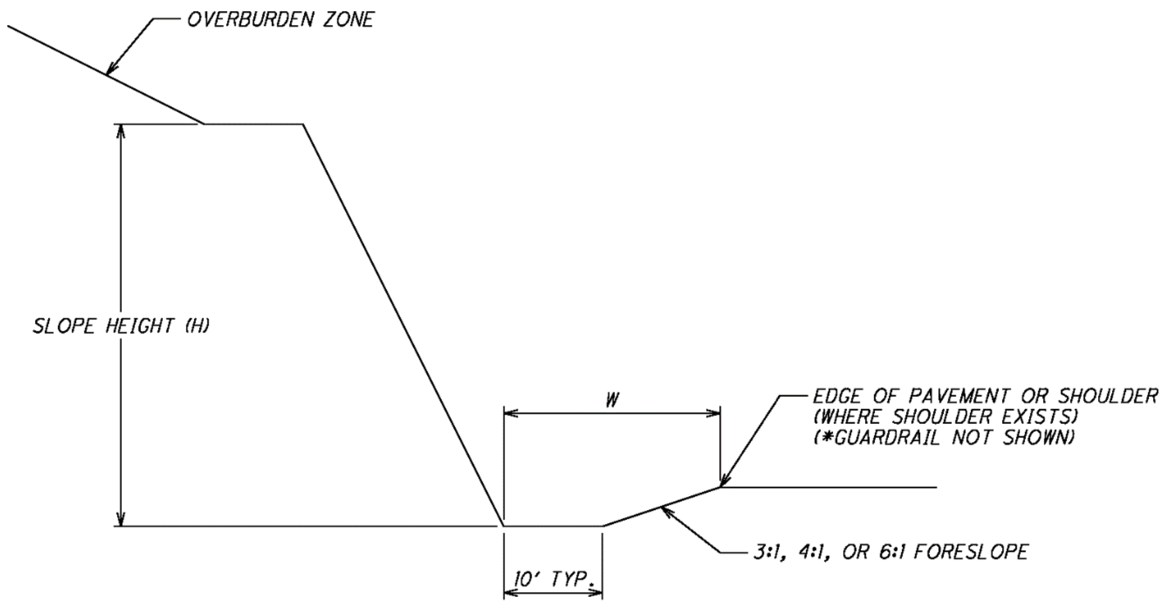


Figure 1000-2: Typical Configuration for Catchment Area with a Flat Maintenance Bench (Bottom) and an Angled Foreslope



*Guardrail Required For All 3:1 Slopes and 4:1 Slopes With Safety Grading

Table 1000-4: Recommended Catchment Widths for Varying Cut Slope Angles, Heights, and Foreslope Angles

	Cut Slope Height, H (ft)					
	0-40	**50**	**60**	**70**	**80**	**>90*** **
Cut Slope Angle	**Catchment Ditch Width, W (ft)**					
3H:1V Catchment Foreslope Angle*						
0.25:1	10	15	15	15	20	25 min.
0.5:1	10	15	20	20	20	25 min.
1.0:1	15	20	20	20/25**	25	30 min.
1.5:1	15	20	20	20/25**	25	30 max.
4H:1V Catchment Foreslope Angle*						
0.25:1	10/15**	15	20	20	25	30 min.
0.5:1	15	15	20	20	25	30 min.
1.0:1	15/20**	20	20/25**	25/30**	30	35 min.
1.5:1	15/20**	20	20/25**	25/30**	30	35 max.
6H:1V Catchment Foreslope Angle*						
0.25:1	15	20	25	30	35	40 min.

0.5:1	20	20	25	30	35	40 min.
1.0:1	25/30**	25/30**	30	35	40	40 min.
1.5:1	25/30**	25/30**	30	35	40	40 max.

* For new slopes only, refer to L&D1, Section 307.2.1 for guidance on catchment foreslope angles.

** Catchment widths for single foreslope angle only (Figure 1000-1)/flat maintenance bench and foreslope angle (Figure 1000-2)

*** For slopes with a height (H) greater than 90 feet and an angle of 1:1 or steeper, use these values as a minimum and increase or modify as necessary based on design.

The designer should use engineering judgment and rockfall simulation modeling to determine the appropriate catchment ditch width for a rock cut slope where the portion of the slope intersecting the ditch is flatter than 1.5:1. Guidance on rockfall simulation modeling using CRSP can be found in Section 503.1 of the ODOT Rock Slope Design Guide.

1007 OTHER GEOTECHNICAL CONSIDERATIONS

Refer to Section 600 of the ODOT Rock Slope Design Guide for information regarding:

- Mines and Mine Seals
- Groundwater
- Surface Water
- Transitions
- Karst

Refer to the ODOT Rock Slope Design Guide Section 700 for construction considerations.

1008 PLAN REQUIREMENTS AND DELIVERABLES

The exploration and design of rock cut slopes will require the preparation of an Exploration Report and Geotechnical Profile. The design features will be presented in the project plans. Refer to SGE Section 700 for report and Geotechnical Profile requirements. The Geotechnical Profile must include all rock test results and pictures of rock cores. The report must include all rockfall analyses, including input parameters and output results. The report must also include catchment area configurations and catchment features considered with cost estimates and final recommendations.

In the plans, all benches, overburden, geotechnical, and construction, must be shown and labeled on the cross-sections.

SECTION 1100 ABANDONED UNDERGROUND MINES

1101 GENERAL

The Abandoned Underground Mine Inventory and Risk Assessment (AUMIRA) process was developed in 1998 in response to a collapse of an abandoned underground mine (AUM) which resulted in the closure of Interstate Route 70 in Guernsey County. Since the implementation of the AUMIRA process, ODOT has completed a statewide inventory of all known AUMs and the relative risk they pose to the statewide transportation network. The purpose of this section is to provide general guidelines and advice for the remediation of AUM sites. The remediation effort consists of a site exploration, selection and design of remediation method(s), development of construction documents, and construction. During the exploration, if data or information contrary to the existing inventory record is found, then the AUMIRA process should be reinitiated for the site.

Although much exploration, likely using multiple methods, will be performed to determine the physical limits and conditions of the AUM, the actual conditions and limits will likely be different, as discovered in construction. AUMs are relatively old, man-made, underground structures that may not be accurately mapped, or not mapped at all; and have undergone varying amounts of deterioration since their construction. In many cases, the overburden soil and rock have also undergone deterioration due to subsidence.

The remediation of AUM sites can be as simple as District maintenance personnel excavating and backfilling a subsidence feature with controlled material, or as complex as deep void and overburden grouting with a need for extensive groundwater management and environmental permitting. While the focus of AUM remediation is on surface deformation, all site details and characteristics must be considered when selecting a remediation method, including type, size, limits and condition of mine workings, associated shafts and entries, and overburden characteristics. Remediation is performed to protect ODOT's assets, but impacts to, and influences from, areas beyond the right-of-way must also be considered.

1102 SUBSURFACE EXPLORATION

The subsurface exploration begins with an extensive office reconnaissance review of the inventory data previously gathered in the AUMIRA process as well as any newly discovered resources, such as a new AUM map, digital elevation models, or the revised georeferencing of an existing map. ODOT historical resources, such as construction plans and records, geotechnical records, and aerial imagery that may not have been available at the time of the site inventory, should also be reviewed. Refer to ODOT Manual for AUMIRA (AUMIRA Manual) Sections 303 and 304 and SGE Section 302.2 for additional guidance.

After completing the office reconnaissance, the site reconnaissance (visit) is performed to observe and verify previously recorded site features, as well as record any newly discovered features according to the AUMIRA process. If private property access is necessary, coordinate with the District in advance of the visit so associated features outside of the right-of-way can be inspected.

If the office and site reviews confirm that remediation should be performed, gather as much information as necessary to establish preliminary work limits. The level of effort and information detail will be proportional to the scope of anticipated remediation options and ODOT asset(s) impacts. Refer to AUMIRA Manual Section 400 and SGE Section 302.3 for additional guidance.

The subsurface exploration of an AUM site is unique in comparison to most ODOT projects. It is often an iterative process that utilizes both conventional and unconventional methods such as drilling and sampling and geophysical, respectively. The preliminary work limits will often be adjusted based on the findings of the exploration.

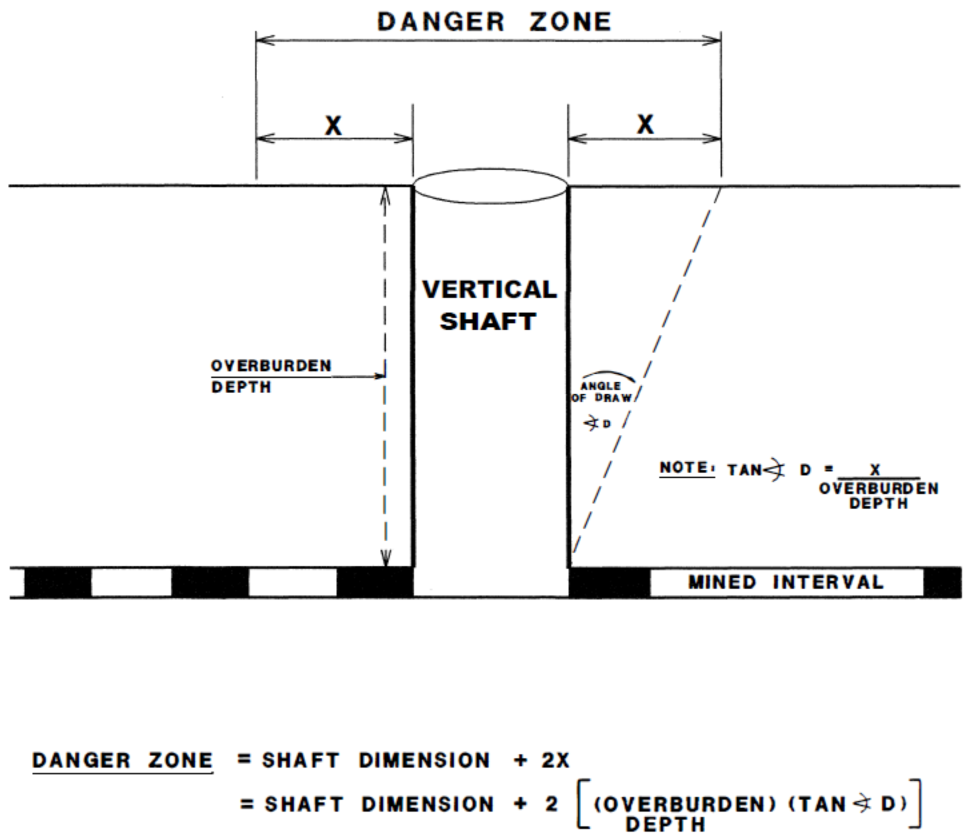
The exploration of AUM openings, including slope entries, drift entries and vertical shafts, may cause vertical and horizontal ground movements or collapse near the openings. Subsidence may occur due to settlement of unstable AUM opening backfill material or failure of existing mine supports. Subsidence may be induced by physical disturbance of the opening, vibrations from drilling activities or the weight of drilling equipment. Any exploration work to be performed in the vicinity of a mine opening should address the necessary safety considerations for potential disturbance.

A ”Danger Zone” should be designated around any suspected vertical shaft location prior to any work being performed. The Danger Zone should be defined by the estimated vertical shaft dimensions plus an additional distance away from the outside edge of the vertical shaft, equivalent to the depth of the overburden multiplied by the trigonometric tangent value for the angle of draw (see Figure 1100-1). Any drilling exploration of the vertical shaft to define its location and condition should be performed outside the Danger Zone using angled drilling. A similar Danger Zone concept would be established when exploring slope and drift entries.

1102.1 Geotechnical Drilling

Drilling geotechnical borings in accordance with the SGE is the most common method of AUM exploration. This method can definitively characterize soil, bedrock, groundwater, and mine related features such as voids, but it is limited to the location of the boring. Boring locations, spacing, sampling intervals and depths are dependent on the site characteristics. For a preliminary boring plan, borings should extend to between 5 and 20 feet below the lowest known mine interval, as determined in the Office Reconnaissance. Consider possible thickness and elevation variations within the mined seam. From the preliminary borings attempt to determine if the mined seam is intact; if a void is present; characteristics and elevation of the overburden soil and bedrock; groundwater conditions; and any anomalous conditions. Additional borings, if any, should extend 5 feet below the lowest known mine interval with limited overburden sampling and testing for further characterization. Refer to SGE Section 303.5.4 for further guidance.

Figure 1100-1: Vertical Shaft Danger Zone



1102.2 Geophysical Methods

Geophysical exploration is the second most common method of AUM exploration. Performing a geophysical exploration is typically quick, non-invasive/destructive, and covers a large linear area (vertical x horizontal). There are several different geophysical exploration methods, all of which have limitations of acceptable data collection. Ideally, multiple surface geophysical methods will be completed, and results compared to develop a model of the subsurface conditions across the project area. Based on results from the geophysical surveys, additional drilling may be warranted and is common to verify anomalous features such as voids.

Geophysical data can be complex, requiring significant effort to "clean up" or process. Such data processing should be performed by an experienced professional for each method. Typical geophysical methods utilized for underground void detection are presented in Table 1100-1.

Table 1100-1: Common Geophysical Methods for AUM Void Detection

Cross-hole seismic tomography	Multichannel Analysis of Surface Waves (MASW)
Electric Resistivity (ER)	Spectral Analysis of Surface Waves (SASW)
Full Waveform Inversion (FWI)	Seismic Refraction

Gravity	Shear-wave seismic reflection
Ground Penetrating Radar (GPR)	Refraction Microtremor (ReMi)

OGE is currently capable of performing ER, seismic refraction, and ReMi. The limitations of ER include nearby overhead powerlines, buried metallic structures, and highly conductive zones above the mined interval that may contribute to inaccurate results. The limitations of seismic refraction include the potential for noise from traffic or construction. The limitations of ReMi include the potential for erroneous results when a low-velocity layer underlies a high-velocity layer. When there is minimal difference in stiffness between adjacent layers, geophysical results are often ambiguous.

Refer to the following resources for a comprehensive description of the benefits and limitations of several geophysical methods:

- ASTM D6429: Standard Guide for Selecting Surface Geophysical Methods
- ASTM D420: Standard Guide to Site Characterization for Engineering Design, and Construction Purposes
- FHWA-IF-04-021: Application of Geophysical Methods to Highway Related Problems

1102.3 In-situ Exploration Methods

Several in-situ (down the borehole) methods of exploration can be performed in conjunction with drilling. Upon completion of sampling and drilling, casing will typically be installed within the overburden soil and highly fractured upper bedrock strata. After installation of the casing a waiting period will be required to allow for any disturbance of the fines within the void to settle. This is necessary in both water filled and air-filled voids.

Sending a camera down the borehole is the most common in-situ method of exploration. The camera should be sized to fit within the casing and rock core hole and be able to enter into any encountered void. Once in the void, the camera can be panned to view the condition of the void. The main limitation of the borehole camera is line of sight which is directly proportional to the light source and turbidity within the void. A compass can be used within the camera field of view to determine the alignment of the void(s) in relation to assets on the surface.

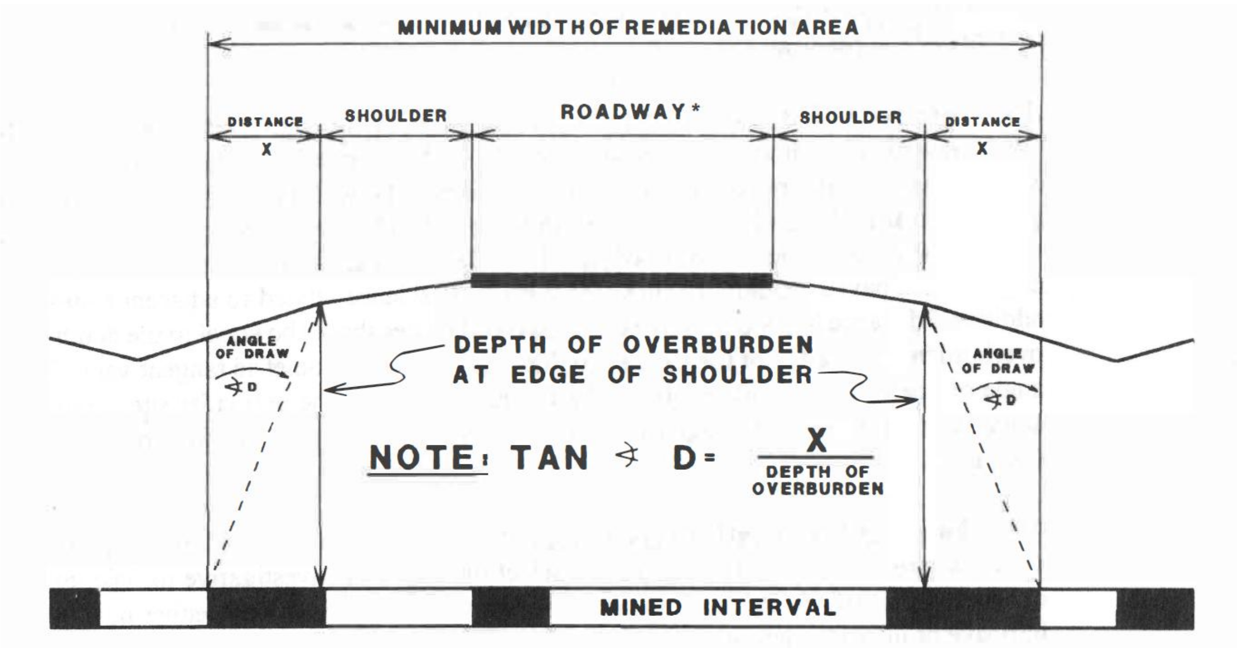
Mapping of the void can be completed utilizing either borehole LiDAR or sonar. If the void is air-filled, a borehole LiDAR unit can be inserted into the void through a minimum 6-inch casing. If the void is water filled, a sonar unit can be inserted into the void through a minimum 4-inch casing. For both sonar and LiDAR the void needs to be filled with a "clean" medium (water or air); otherwise, the return signal is dampened, reducing the imaging.

1103 DETERMINING WORK LIMITS

Due to the varying level of accuracy in determining the mine limits, linear work limits (up and down station of roadway centerline) should be extended beyond the area expected to need remediation. This guideline is recommended because the actual extent of the needed remediation will only be revealed through the execution of the work. An increased project work area cleared for construction activities will minimize or eliminate the need to expand project limits during construction due to unanticipated additional remediation areas.

The width of the remediation area perpendicular to the roadway centerline should be defined by the area from the outside edge of shoulder to outside edge of shoulder, plus an additional distance beyond the edge of each shoulder, to allow for surface subsidence related to adjacent abandoned underground mine workings. This additional distance beyond the edge of the shoulder (see Figure 1100-2) is equal to the depth of the overburden (soil and bedrock) times the trigonometric tangent value for the angle of draw. If a specific angle of draw is not known for the site geology, use 35 degrees.

Figure 1100-2: Determining Minimum Width of Remediation Area



* REPRESENTS BOTH LANES OF A TWO LANE ROAD OR ALL LANES OF A DIVIDED HIGHWAY AND THE MEDIAN.

On some sites it may be advantageous to remediate additional width to accommodate future widening of the roadway. When remediating all lanes of a divided highway, the entire median should be remediated.

1104 FACTORS AFFECTING REMEDIATION METHOD SELECTION

When AUM remediation is warranted, selecting the most appropriate remediation method or combination of methods is important. All sites are unique, and the design engineer must consider site-specific conditions when deciding which remediation method(s) is/are most appropriate for the anticipated construction conditions. Consider the following conditions when evaluating AUM remediation method(s).

1104.1 Hydrogeologic Setting

The AUM may be flooded or partially flooded and function as a confined aquifer. The water levels and fluctuations within the AUM must be determined as part of the exploration. The potential for a sudden release (hydraulic “blow-out”) of pooled water in AUM workings must be considered when evaluating remediation alternatives. The dewatering of an AUM may have a destabilizing effect, inducing or accelerating subsidence, both inside and outside the right-of-way.

Aquifers above the AUM must be characterized and possible effects on them considered. Nearby water wells must be identified as part of the exploration, and their potential impacts due to the remediation considered.

1104.2 Geometry, Size and Condition of the AUM

The geometry of the AUM workings is a function of the mining method(s) utilized in the layout and operation of the mine. The thickness of the extracted mineral(s) impacts the likelihood of surface deformation. The lithology and condition of the bedrock forming the mine roof and floor contribute to the likelihood of various failure mechanisms. Identify the method and current condition of roof support, if possible.

Conditions within an AUM may be highly variable. Varying degrees of roof rock failure, roof support pillar/post punching into the mine floor, floor heave, support pillar crushing, etc. are commonly encountered. Large amounts of roof fall in the mine workings may result in random compartmentalization of the mine. Some mines are found to be in about the same condition as they were at the time of abandonment.

1104.3 Type and Condition of Overburden

The type of bedrock overlying the mine (overburden bedrock or roof rock), its bedding and fracture patterns and its compressive strength greatly affect the remediation method selection. The design engineer should consider how to:

- incorporate the natural strength of the overburden bedrock into remediation methods which do not radically disturb it;
- physically increase the existing strength of the overburden bedrock, or;
- physically remove and replace the overburden.

1104.4 Site Constraints

There are several site constraints to consider when evaluating remediation methods. Above and below ground utilities will need to be identified and relocated if necessary. All nearby structures, both inside and outside the right-of-way, will need to be identified, and the pre-construction condition documented. The design will need to consider protective measures for all structures and utilities within the influence of the project. Right-of-way limits will impact the maintenance of traffic scheme and construction methods and staging areas.

1104.5 Roadway Classification and Pavement Build-up

The roadway classification and pavement build-up will influence the method of remediation and maintenance of traffic scheme. Multi-lane highways with full width shoulders create the possibility of lane closures while maintaining traffic and allow flexibility in choosing remediation method(s) and sequencing of construction operations, whereas two-lane roads are more restrictive. The pavement build-up will control how the pavement can bridge over loosened and settled subgrade; for example, an original slab pavement with asphalt overlay will bridge much better than rubblized pavement or full-depth asphalt.

1104.6 Type and Volume of Traffic

The type and volume of traffic using the roadway directly affect the choice of remediation method(s). This does not mean that different levels of safety are desired for different types of roadways. Rather, roadways carrying greater volumes of heavier vehicles, such as interstate routes, must be made structurally stronger, requiring a more extensive remediation effort to ensure satisfactory levels of service and safety. This may be achieved with a less extensive remediation effort on a roadway carrying a lower volume of lighter weight vehicles.

1104.7 Nearby Traffic

If an acceptable detour of traffic away from the remedial work area is available, almost any form of remediation that can be constructed within the site limits can be performed. Otherwise, traffic must be maintained during remediation which will affect the method, sequence, and footprint of the remediation. While maintaining traffic, the remedial construction operations must be conducted a safe distance away from the traffic so as not to create dangerous driving conditions. The minimum safe distance is equal to the depth of the overburden times the trigonometric tangent value for the angle of draw. As previously stated, if a specific angle of draw is not known for the site geology, use 35 degrees.

1105 REMEDIATION METHODS

There are several methods of remediation to be considered as presented below, each with advantages and disadvantages. It is somewhat common to use more than one method on a project in response to variable site conditions. All the aforementioned factors must be considered in determining the method(s) of remediation. A cost comparison analysis should be completed for all feasible methods.

1105.1 Excavate and Replace with Controlled Backfill

Excavating the overburden and AUM and properly backfilling to roadway grade is the most desirable remediation method because it knowingly eliminates the mine and any subsided overburden within the remedial work area. Two drawbacks associated with this method; complete reconstruction of the pavement and a complete closure of the road are necessary, requiring a traffic detour during construction. Excavation should only be considered when the maximum depth is less than 50 feet.

1105.2 Drilling and Grouting

Drilling access holes and pumping cement grout into the mine void to fill and stabilize it is the most common method used by the Department to remediate AUMs. This method begins by building a grout barrier around the area to be remediated. Barrier boreholes are drilled external to the roadway area to be stabilized, and low slump grout is pumped. The barrier boreholes are located to ensure the roadway is beyond the influence (angle of draw) of non-remediated AUM workings (See Figure 1100-2). The barrier will effectively isolate a portion of the mine to be grouted under and adjacent to the road (within the barrier).

After building the barrier, higher slump grout is placed through patterned production boreholes located internal to the completed barrier. The higher slump grout "floods" the mine void space within the barrier. Production grouting should proceed from the geological "down-dip" end to the "up-dip" end of the isolated portion of the mine so as to "squeeze" any groundwater out of this portion of the mine.

The advantages of drilling and grouting remediation include the ability to construct adjacent to traffic (safe distance beyond the angle of draw) in most cases, and salvage of the existing pavement structure. This method is most common for remediation of deeper and/or water filled mines, on the order of 40 to 100 feet. The disadvantages include a need for environmental permitting, groundwater management, and changes to plan quantities of grout.

1105.3 Pneumatic Stowing

Pneumatic stowing consists of filling the mine void with pneumatically placed aggregate. This method is most common for remediation of shallow subsidence features, exposed mine voids in a cut face, and opened drift or slope entries and entries that can be cleared of debris by excavation. This method is relatively inexpensive and quick to construct, however, it is limited to the remediation of relatively shallow dry mine voids.

1105.4 Land Bridge

A heavily reinforced concrete pavement can be built to bridge over the mine void, also known as a land bridge. Mine openings beneath the roadway can also be remediated by use of this method. The minimum length of potential subsidence which a land bridge would be expected to span for a site should first be determined when considering this remediation alternative. As a minimum, the span should extend the length of the known AUM longitudinally along the roadway, plus some additional distance beyond each end. This additional distance should be equal to the depth of the overburden times the trigonometric tangent value for the angle of draw.

The primary advantage of building a land bridge is avoiding underground construction and associated challenges such as groundwater control, environmental permitting, underground utilities, more extensive maintenance of traffic, etc. The primary disadvantage is the expense of reconstructing the existing pavement with a pavement structurally capable of spanning over voids and future subsidence. This method would not apply to median or grass shoulder remediation and would be most appropriate for a relatively short site length and deep mine void with overburden thicknesses more than 40 feet. Some options for land bridges are presented as follows:

- Continuous Reinforced Concrete Pavement (CRCP) – concrete pavement continuously reinforced in only the direction of travel, intended to span minor subsidence-related settlements.

- Double Reinforced Pavement – concrete pavement continuously reinforced in both directions, intended to span subsidence-related settlements. This pavement is, in effect, acting as a bridge deck founded on the subgrade.

- Pre-Cast Concrete Spans – pre-cast concrete sections such as box beams or bridge spans to form a structure to span small areas of potential subsidence-related settlements. Accurate placement of these box beams or bridge spans relative to the AUM workings is necessary for this system to be effective. Precast sections should be founded on bedrock.

1105.5 Dynamic Compaction

Dynamic compaction consists of repeatedly dropping a heavy weight in a pattern over the site via a crane to induce collapse and void filling of the AUM and consolidate the overburden. The weight, drop height, grid spacing, and number of passes are selected based on site conditions and the targeted depth of improvement. Groundwater and overburden characteristics and depths will determine feasibility of this method. The deeper the mine and the thicker and more competent the roof rock, the less likely this method is applicable. Other site constraints include underground utilities and nearby infrastructure. The site will need to be regraded requiring borrow. Dynamic compaction should only be considered for new construction.

1105.6 Implosion

The AUM may be remediated by completely collapsing or imploding the mine void using explosives. Obviously, this method is quite invasive and should only be considered for new construction. Explosives must be placed by an experienced blaster. Proper timing delays and explosive placement within the abandoned underground mine pillars and associated overburden are critical to the successful use of this remedial alternative. A pre-blast survey will need to be performed, along with a groundwater study. Dynamic compaction and/or controlled backfill may also be used in combination with implosion depending on site characteristics.

1105.7 Remediation of Mine Openings

As was previously discussed in Section 1102.3, mine openings present a heightened risk of subsidence, and the remediation method must be robust enough to ensure the risk for potential subsidence in the vicinity of the opening is mitigated. Drift entries beneath the roadway can typically be excavated and replaced with controlled backfill material. Drift entries found to be in stable condition, open or containing debris which can be efficiently and effectively removed by equipment, can also be remediated by excavating and backfilling. Backfill material can be placed by mechanical means or by pneumatic stowing. A mine drain should be considered if impounding mine drainage behind backfill material is a possibility. A mine vent may also be appropriate to vent accumulating mine gas.

All remediation work for vertical shafts and slope entries should be performed with all equipment and materials being located beyond the Vertical Shaft Danger Zone (See Figure 1100-1). Vertical shaft and slope entries should be opened and backfilled with controlled fill. If a vertical shaft or slope entry cannot be opened and cleared of unstable material, it should be remediated by a combination of other methods. Unstable material can be consolidated by dynamic compaction (for new construction only) or by drilling and grouting, and any open volume can be filled with controlled backfill. After remediation, confirmation drilling from directly atop the center of the vertical shaft should be performed.

1106 CONSTRUCTION

Since the actual limits, material quantities and stabilization efforts required to remediate an AUM site will only be fully realized in construction, the project engineer and inspectors must be aware of work progress and changed conditions encountered. When most AUM remediation projects are completed, the "as-built plans" are usually quite different than the design plans, and the completed work must be well documented. The project engineer needs to be familiar with all data obtained and presented in the exploration and design and should be fully aware of the intent of the remediation design.

1106.1 Record Keeping

Close inspection of the remediation work as it progresses is important. Inspectors should expect to encounter changed conditions in construction and will need to report them accurately and quickly as they are encountered. The construction schedule should identify any points during the progress of the work where informed decisions will need to be made before proceeding. Any changes to the work plan or project plans need to be documented and well communicated. The project engineer should advise the design engineer and District Geotechnical Engineer of any unusual or unexpected conditions as they are encountered which may alter the original scope and plan. If the project engineer or contractor proposes a significant deviation from contract requirements, the design engineer and District Geotechnical Engineer will need to be consulted to ensure the intent of the design is not compromised.

For drilling and grouting projects, complete and accurate boring logs and grout mix and placement records are critical. These records will be required for compliance with the reporting requirements of the OEPA Injection Well Permit.

Photograph and record the locations of subsidence events that occur during construction, as well as previously unknown mine openings and other mine related features or conditions encountered during construction. As a minimum, record the location(s) using a hand-held GPS device. Include this information in the permanent construction record.

Actual time and material usage as compared to that anticipated in the construction documents should be monitored. This information will provide an indication of potential overruns of contract items, and/or changed subsurface conditions different than anticipated in design.

"As-built" drawings should be produced for all mine remediation projects. These drawings should contain all modified or new information developed as the result of the project. This information should include the location of all planned and new borings, confirmation borings, AUM workings encountered, work performed under contingency items, and actual work limits.

All remediated AUM openings should be marked with a permanent monument located at the center of the slope and drifts openings, or directly over the center of a vertical shaft. These monuments should be located by ground survey and become a permanent record for the roadway site.

Complete and accurate construction records, as described above, are necessary for the future monitoring of the site. Post-construction inspection of completed work items, in the traditional sense, will typically not be possible due to the work being below final grade. Accurate construction records will be invaluable as a reference for post-construction monitoring and review of conditions adjacent to the project area.

1106.2 Site Monitoring and Confirmation Testing

The site will need to be monitored for possible changes during construction. Existing conditions may change, or new conditions may develop. Certain methods of remediation may induce mine-related settlements. A construction site monitoring program will need to be developed, considering potential impacts of induced settlement. The design engineer should refer to Table 1100-2 for common methods of site monitoring during construction. The site should be re-inspected within one year after completion of construction activities. Subsequent re-inspection frequencies will be performed in accordance with AUMIRA Manual Section 700.

Table 1100-2: Construction Site Monitoring Methods

Method of Monitoring	Applications	Limitations
Borehole GPR	Detect subsurface anomalies. A radar signal is transmitted from one hole to an adjacent hole.	Maximum borehole spacing for this method may be limited to 10± feet and may require a greater amount of data processing time.
Time Domain Reflectometer (TDR)	Detect lateral (shear) and vertical (subsidence) movements. TDR is relatively inexpensive, particularly when drilling is performed for other purposes. Data collection can be performed by one person. Data can be easily interpreted and is usable at the time of collection on the site.	Rental of a grout pump, with operator, is required for cable installation. The project will need to purchase a TDR meter for the data collection from installed cables.
Slope Inclinometer	Detect lateral subsurface movement (shear); may have application in detection of “side-draw” related to adjacent subsidence activity.	Relatively expensive as compared to TDR monitoring. Requires sophisticated data gathering equipment, time consuming data collection, and extensive data analysis to produce usable information.

Borehole Camera	View overburden bedrock conditions, condition of subsurface voids, grouted portions, etc. A video record of viewed conditions is created. This work can be performed by ODOT personnel and equipment.	Requires stable overburden conditions, otherwise, camera will be at risk when lowered in the borehole. Quality of video imagery below the groundwater table is usually of a lesser quality.
Piezometers	Detect and monitor piezometric surface within a particular aquifer. This form of monitoring is relatively inexpensive, particularly if a drilling program is to be performed. Data collection can be performed quickly by one person. Data is easily interpreted and usable at the time of collection on the site.	Requires stable overburden conditions to allow for installation of well casing. Measurement is at a particular aquifer elevation, complete with the appropriate sealing(s) of the annular space, to only allow water from a particular aquifer to enter and rise in the slotted casing. Location of piezometers must not interfere with the safety of the traveling public.
Observation Wells	Detect combined groundwater head for a given borehole location. This form of groundwater monitoring has application in areas where fractured overburden conditions allow for co-mingling of originally separate aquifers.	Requires stable overburden condition to allow for the installation of a well casing. Location of observation wells must not interfere with the safety of the traveling public.

All remediation projects should include some means of confirming the effectiveness and completeness of the remediation effort and that the full and necessary vertical and lateral (both transverse to and along the roadway) limits of the AUM workings have been remediated. This is generally accomplished by confirmation drilling, which consists of drilling borings to below the depths of the AUM to verify no voids or unacceptably loose or soft overburden exists. Any such conditions encountered will require further remediation. All confirmation drilling and boring sealing must be performed in accordance with the SGE. The Contractor will need to provide a boring log and digital photographs of the rock core. Geophysical testing may also be considered to verify the effectiveness of remediation efforts.

SECTION 1200 FOUNDATIONS FOR TOWERS, POLES, AND OVERHEAD SIGNS

1201 GENERAL

In general, light towers and other poles are founded on drilled shafts, usually bolted onto the top of the shaft or sometimes with the pole embedded into the shaft. The typical controlling geotechnical failure modes are overturning (in the Strength Limit State) or excessive lateral deflection (in the Service Limit State) due to wind loading or large eccentric loads from cantilever mast-arms or supports. Bearing is not generally a valid failure mode and is only checked for signal- and sign-support foundations, which can carry some substantial vertical loads. However, the typical failure mode for mast-arm, signal, and sign-support foundations is generally neither overturning nor bearing, but torsion of the drilled shaft foundation due to unbalanced wind load causing rotation of the mast-arm. Sliding (or lateral translation) is not checked for drilled shaft foundations. Overall (global) stability is not specifically checked for these foundations, but it could require checking if they are installed on or near a slope.

In some special circumstances (typically when there are shallow utilities that conflict with the installation of a drilled shaft), poles can be founded on a shallow (spread footing) foundation. In this case, the valid failure mode checks will be as for any structure founded on shallow foundations, in accordance with BDM Section 305.2 and the AASHTO LRFD Article 10.6. There is no minimum soil shear strength requirement. Service Limit State lateral deflection is not checked for poles founded on spread footings, but settlement is checked: particularly differential settlement and its influence on angular displacement of the pole.

Borings for Towers, Poles, and Overhead Sign Foundations are Type E5 Structure Borings, in accordance with SGE Section 303. If the minimum length foundation identified in this section or in the associated standard drawing will be entirely embedded within a stable embankment, then no boring or design is necessary; use the minimum length foundation. If the new structure will replace an existing in-kind structure that is stable at basically the same location, consider reusing the existing foundation, or duplicating the existing foundation for the new structure or using the current design standard, whichever results in a longer foundation.

When performing a Standard Foundation Design in accordance with the Office of Roadway Engineering (ORE) HL, TC, or ITS Standard Construction Drawings (SCD), average the shear strength properties of all soil layers within the proposed foundation depth, according to the soil properties encountered in the boring. For this averaging, generally ignore any soils within the frost depth (approximately the top three feet). Compare the average shear strength of the soils to the minimum shear strength values on the SCD to determine if a Standard Foundation Design can be used.

When performing a Special Foundation Design, always assume the top 3 feet starting from the proposed ground surface to be a soft clay with undrained shear strength of S_u = 250 psf (correlating approximately to an SPT N_{60} = 2 bpf), regardless of the soils revealed in the boring, unless shallow bedrock is at a depth of less than 3 feet below proposed grade. This is to account for soils weakened by freeze-thaw or other disturbance.

Always assume the groundwater at depth is at D_w = 3.0 feet below the proposed ground surface, unless more shallow groundwater is identified on the boring log.

If performing a p-y analysis, divide the soil boring data up into layers according to the soils encountered in the boring (generally one layer per sample). However, include the assumed freeze-thaw weakened soil in the top 3 feet with S_u = 250 psf. Derive p-y soil properties for each soil layer and enter the soil model into the p-y analysis program.

If performing a Broms' analysis, average the soil properties over the foundation length, L, including the assumed freeze-thaw weakened soil in the top 3 feet with S_u = 250 psf. Calculate the average effective friction angle ϕ', effective unit weight γ', and Rankine passive earth pressure coefficient k_p; and calculate the foundation diameter D at a depth of ⅔ L

1202 LIGHT TOWER OR LIGHT POLE FOUNDATIONS

For light tower or light pole foundations, see standard drawings HL-20.21 and HL-20.11 for basic dimensions of the drilled shaft foundation unit: nominal drilled shaft diameter, D; No.4 steel reinforcing bar spiral cage or hoops outside diameter, B; and bolt circle maximum diameter, BC. Diameter of the drilled shaft foundation unit is generally controlled by the required bolt circle diameter for attachment of the tower mounting plate or pole base.

1202.1 Light Tower Foundations

Length of the drilled shaft foundation unit for a light tower is dependent on an Extreme Limit State check against overturning and a Service Limit State check against excessive lateral deflection. According to HL-20.21, "If solid rock is encountered before reaching required depth, the remaining foundation depth may be decreased by 50 percent." For the purposes of HL-20.21, consider "solid rock" as bedrock with an unconfined compressive strength of 1500 psi or greater (corresponding to a strength of bedrock description of Slightly Strong or greater). However, this provision only applies if there is not already a design soil boring at the light tower location that shows bedrock within the length of the drilled shaft foundation unit; in this case, consider the encountered bedrock in the design of the foundation and restrict the foundation length from being shortened by plan note.

Calculate the wind load against the support tower and the luminaries in accordance with AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS) Articles 3.4 and 3.8.

In the Extreme Limit State, the only geotechnical check is for resistance against overturning. The load to be resisted is from wind against the support tower and against the luminaries, which is transferred to the top of the foundation as a lateral shear load and a moment. In accordance with LRFDLTS, the Extreme Limit State design wind speed is 115 miles per hour, and the wind (W) load factor is 1.00.

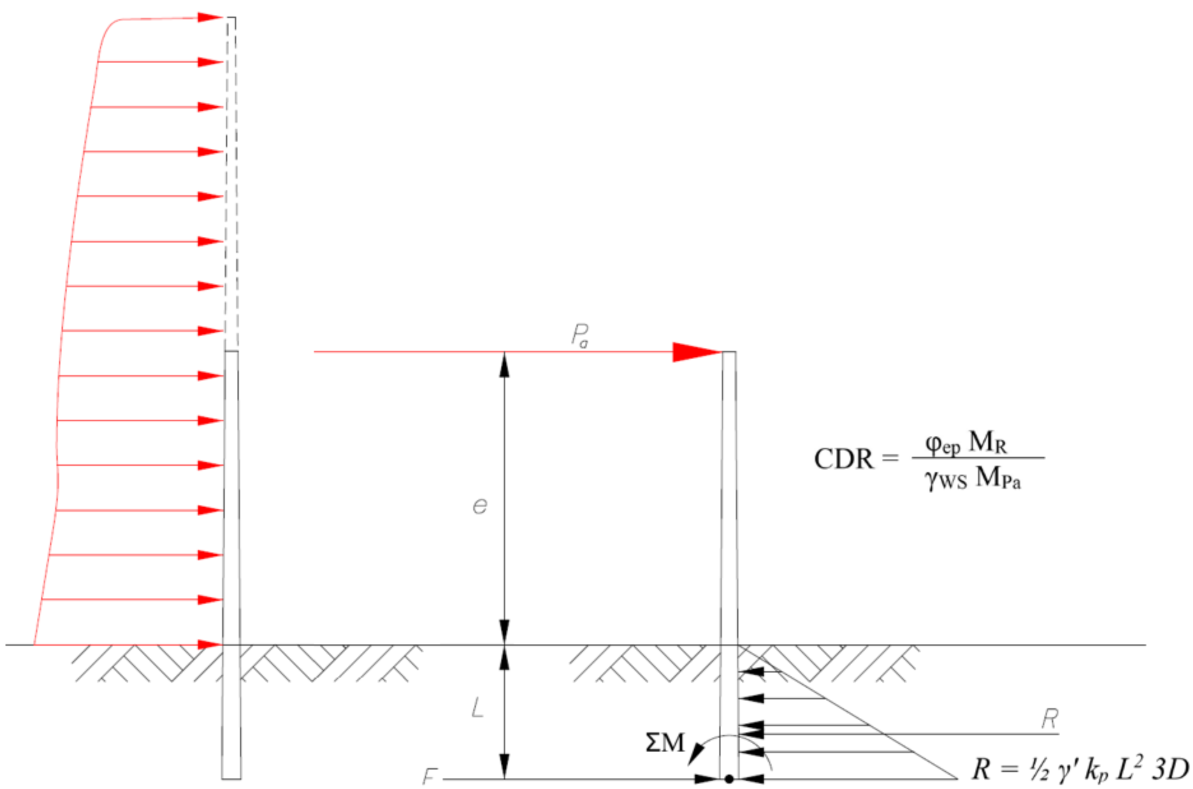
Check overturning resistance for light tower drilled shaft foundations with either:

1) A p-y analysis at the Extreme Limit State, checking that the vertical foundation element does not have very large deflection (typically around 100 inches) or infinite or incalculable deflection (failure of the program to converge at a solution). It is otherwise acceptable for the

Extreme Limit State analysis deflection to be quite large (on the order of 10 to 20 inches). For input into this analysis, calculate the resultant factored shear and moment loads at the top of the vertical foundation element. Input the full geotechnical profile over the drilled shaft foundation length with separate soil properties for each identified soil stratum. For the weight of the tower, use a minimum dead load of components (DC) load factor of 0.90.

2) A moment equilibrium analysis about the tip of the vertical foundation element using the Broms' Method with Extreme Limit State load factors and resistance factors, in accordance with the commentary in LRFD C11.8.4.1. To perform this analysis, see the following definitions and formulas (see Figure 1200-1):

Figure 1200-1: Broms' Method for a Tower or Pole Foundation



P_a is the resultant horizontal load from wind pressure, calculated at some height of effect, e, above the top of the vertical foundation element.

L is the length of the vertical foundation element. D is the diameter of the vertical foundation element, at a depth of ⅔ L (the same location as resultant force R).

γ' is the average of the effective unit weight of the foundation soil over length L. Keep in mind the depth to water, D_w, when calculating the average; for soils above the water table the effective unit weight is the same as the total unit weight, γ_{tot}. If γ_{tot} is assumed the same over length L, the following approximate equation may be used:

$$\gamma' = \gamma_{tot} (0.5 + 0.5 (D_w/L))$$

k_p is the Rankine passive earth pressure coefficient, calculated using the average effective friction angle ϕ' over length L:

$$k_p = (1+\text{SIN}(\phi'))/(1-\text{SIN}(\phi'))$$

Therefore, the resultant forces in the ground, according to Broms' Method are a concentrated force F at the tip of the vertical foundation element, and a resultant R of the distributed passive resistance along the length of the vertical foundation element, where:

$$R = \tfrac{1}{2} \gamma' k_p L^2 3D$$

We will define the Moment Arm of force, F as MA_F, therefore, when summing moments about the base of the vertical foundation element,

$MA_R = \tfrac{1}{3} L$ and $MA_{Pa} = e + L$

Calculating the moments for these moment arms, we have:

$$M_R = MA_R\, R = (\tfrac{1}{3} L) (\tfrac{1}{2} \gamma' k_p L^2 3D) = \tfrac{1}{2} \gamma' k_p L^3 D$$

$$M_{Pa} = MA_{Pa}\, P_a = (e + L) P_a$$

Therefore, for LRFD Extreme Limit State design, we have:

$$CDR = \frac{\varphi_{ep} \tfrac{1}{2} \gamma' k_p L^3 D}{\gamma_W (e + L) P_a}$$

where:

φ_{ep} = 0.75, the resistance factor for passive resistance of vertical elements, per AASHTO LRFD Table 11.5.7-1

γ_W = 1.00, the Extreme Limit State load factor for wind, per LRFDLTS Article 3.4

CDR = Capacity-Demand Ratio, and CDR $\geq$ 1.00.

The check of overturning resistance will give the minimum length and diameter of the vertical foundation element necessary to satisfy the requirements of stability.

Determine the maximum factored shear and moment in the drilled shaft for considerations of structural resistance in accordance with AASHTO LRFD Article 10.8.3.9.

In the Service Limit State, check against excessive lateral deflection. The Service Limit State deflection criterion is a maximum resultant projected deflection at the top of the tower of 10% of the height of the tower in accordance with LRFDLTS Article 10.4.2. The resultant projected deflection is the sum of the deflection at the top of the foundation plus the lateral rotation in radians at the top of the foundation times the height of the tower above the foundation. The design wind speed for the Service Limit State is 76 mph in accordance with LRFDLTS Figure 3.8-4b.

1202.2 Light Pole Foundations

Light pole foundations are designed in accordance with the ODOT Office of Roadway Engineering (ORE) Traffic Engineering Manual (TEM) Section 1140-6, and ORE standard drawing HL-20.11. According to HL-20.11, a Standard Foundation Design can be used if the average shear strength of the soils over the foundation length, L, meets or exceeds either S_u = 2000 psf (for cohesive soils) or ϕ_f = 30° (for granular soils). Otherwise, perform a Special Foundation Design.

If performing a Special Foundation Design, calculate the wind load against the support pole, truss arm, and luminaire in accordance with LRFDLTS Articles 3.4 and 3.8. Check for Extreme Limit State overturning and Service Limit State excessive lateral deflection as for a light tower foundation.

In addition, perform an Extreme Limit State check for resistance to torsion (rotation of the mast-arm in the direction of the wind) in accordance with Section 1204.1 as for Mast-arm Signal and Sign Support Foundations.

1203 CCTV POLE FOUNDATIONS

CCTV poles are typically precast fiber-reinforced spun concrete poles preordered to set lengths. In this case, the pole is embedded into the drilled foundation. The foundation could be a drilled shaft of greater diameter than the pole, with the pole embedded in the drilled shaft. However, it usually consists of only the base of the pole itself, embedded into the ground in a drilled hole, backfilled with concrete, low strength mortar, or polyurethane expanding foam (PEF) in accordance with Supplemental Specification 809, Intelligent Transportation System (ITS) Devices and Components and in accordance with Supplemental Specification 864, Polyurethane Expanding Foam Backfill. In this case, the ordered length of the pole will be equal to the height above ground (typically 70 feet) plus the foundation embedment length, L. See standard drawing ITS-12.10 for basic details of the concrete poles. The CCTV pole could also be a metal (aluminum or steel) pole, bolted onto a drilled shaft foundation in the same way as other lighting and traffic control poles; in this case the length of the pole will only be above the drilled shaft foundation.

For CCTV poles, both a Strength Limit State check against overturning and a Service Limit State check against excessive lateral deflection are performed. Because of the need for a stable platform for the camera, the Service Limit State check usually controls. If using a drilled shaft foundation unit, also determine the maximum factored shear and moment in the drilled shaft for considerations of structural resistance in accordance with AASHTO LRFD Article 10.8.3.9.

Perform the Strength Limit State check against overturning in the same way as for a Light Tower or Light Pole Foundation, as described in Section 1202. However, since the pole is typically embedded into the ground in a drilled hole backfilled with PEF, in this case use the diameter of the pole as the diameter of the vertical foundation element. The PEF is generally similar in strength to a stiff soil, and so is ignored in the analysis. The poles are tapered; for a Broms' Method analysis, use the diameter of the pole at a depth of ⅔ L as the diameter (D) of the vertical foundation element. Note that due to the taper, whenever L is changed in the analysis, D will change likewise. For a p-y analysis, represent the tapered profile of the pole, with a fixed diameter at the top of ground, and a diameter that varies with length.

For Service Limit State design, it is preferable to fix the tip of the foundation to minimize both the deflection of the foundation and the resultant projected deflection at the top of the pole. We consider three cases for the fixity of the tip of the foundation (the butt of the pole):

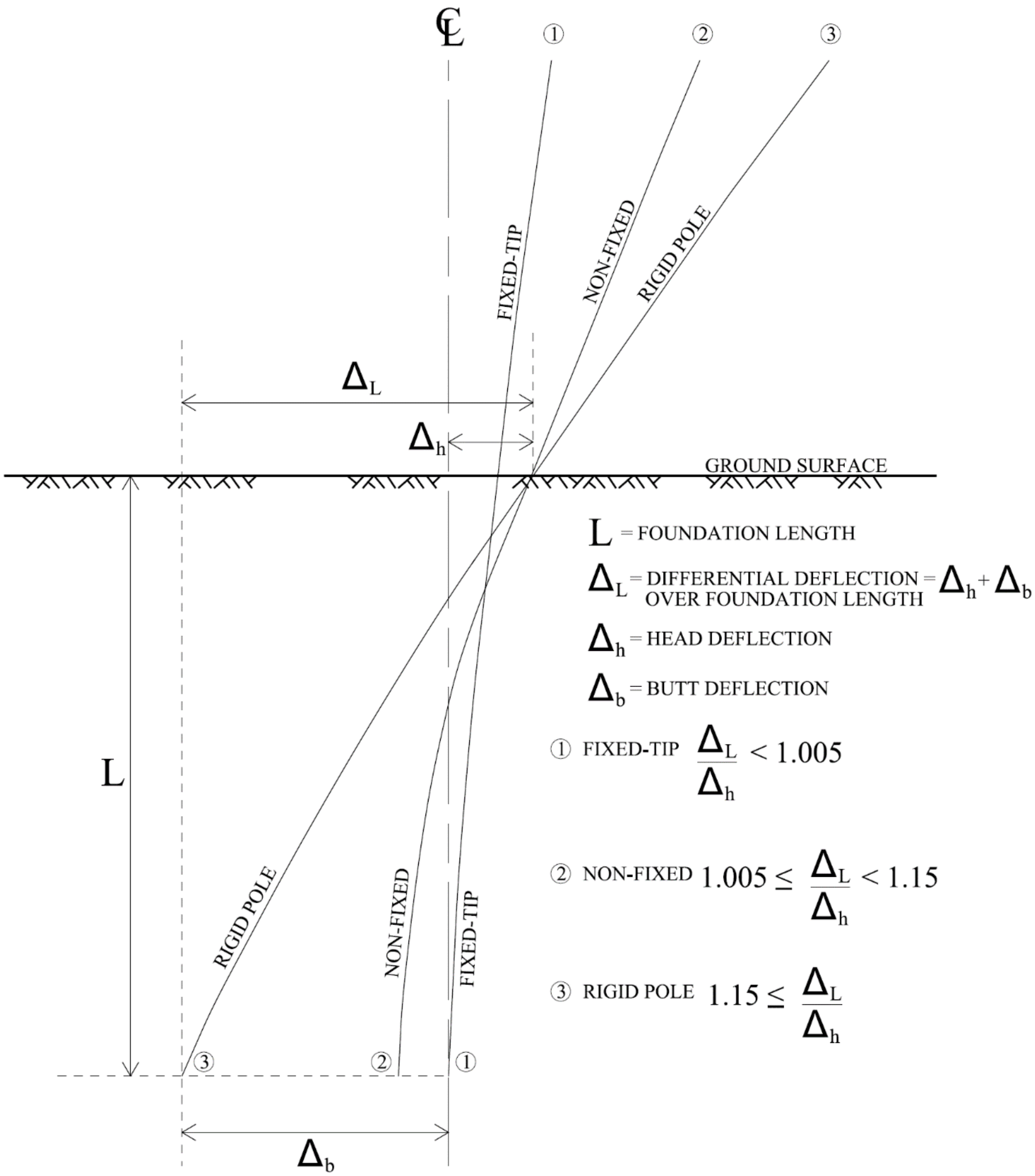
1) Fixed-Tip, where the tip of the foundation does not move at the design loading. All deflection in this case is structural bending of the pole.

2) Non-Fixed, where the tip of the foundation moves some small amount, but most of the deflection is still structural bending of the pole.

3) Rigid Pole, where the pole does not bend substantially, and most of the movement is at the tip of the foundation.

The fixity of a pole foundation, where the pole is embedded into the ground in a drilled hole, in accordance with SS809 (not a drilled shaft foundation), is assessed through a concept called deflection ratio. Deflection ratio is defined as the differential horizontal deflection over the length of the vertical foundation element, Δ_L, divided by the horizontal deflection at the head of the vertical foundation element, Δ_h, (see Figure 1200-2 for a full definition of the terms involved).

Figure 1200-2: Deflection Ratio for a CCTV Pole Foundation



If $\Delta_L/\Delta_h = 1$, then there is no movement at the tip of the foundation (a Fixed-Tip). Realistically, this is often not possible, so we define Fixed-Tip as $\Delta_L/\Delta_h < 1.005$. Non-Fixed is defined as $1.005 \leq \Delta_L/\Delta_h < 1.15$ and Rigid Pole is defined as $\Delta_L/\Delta_h \geq 1.15$.

It is preferred that a Fixed-Tip condition be achieved at the tip of the foundation. If a Fixed-Tip cannot be achieved within a foundation length of L ≤ 20 feet, then a Non-Fixed tip condition is acceptable. If a Non-Fixed tip cannot be achieved within a foundation length of L ≤ 20 feet, then increase the foundation length beyond 20 feet. The maximum foundation length is 25 feet, even with a Rigid Pole deflection. The minimum foundation length is 12 feet in soil.

Generally, if the soils are cohesive and the average SPT N_{60} blow count revealed in the design soil boring is 12 bpf or higher to a distance of 12 feet below the proposed ground surface, then a 12-foot foundation embedment can be used by inspection. This is colloquially called "12 for 12." This average N_{60} blow count does not include the assumed freeze-thaw weakened soil in the top 3 feet in accordance with Section 1201. There are several exceptions to this rule, for which a custom foundation length based on a p-y analysis must be designed:

- If a weak layer (N_{60} < 12 bpf) exists at the top of the boring to deeper than 3 feet;
- If a weak layer (N_{60} < 12 bpf) of more than 3 feet thickness exists at the bottom of the 12-foot depth;
- If there are granular soil layers of greater than 3 feet in thickness;
- If there is shallow bedrock within the 12-foot depth.

If using the Ensoft LPILE program for the p-y analysis, enter the following properties for an embedded pole (if using a different p-y analysis program, make equivalent entries):

- Section Type: Elastic Section (Non-yielding);
- Length of Section (ft): Pole Foundation Length L;
- Structural Shape: Pipe;
- Elastic Modulus: 6,244,000 psi (per the precast pole manufacturer);
- Wall Thickness: 3.55 in. constant (at top and at bottom);
- Elastic Section Diameter (At Top): 23.10 in.;
- Elastic Section Diameter (At Bottom): 23.10 in. + L × 0.18 in/ft, due to batter of the pole;
- Compute the moment of inertia and areas (at top and at bottom) and draw section.

If running a p-y analysis in the Service Limit State, for a standard 70-foot tall pole, use the following values and calculation steps:

1) Pile Head Loading: Shear, V = 253 lb; Moment, M = 99,208 in-lb; Weight, W = 11,061 lb;
2) Vary L typically from 20 feet to point of failure in one-foot increments;
3) Recalculate the Pile Properties for each increment of L;
4) Determine the design foundation length L to achieve the Service Limit State deflection criteria.

If running an analysis for a metal pole bolted to a drilled shaft foundation, or a concrete pole set into a drilled shaft foundation, use the properties of the drilled shaft in the p-y analysis model. If the pole has a concrete drilled shaft foundation, the Service Limit State deflection criterion is a maximum resultant projected lateral deflection at the top of the pole of 0.10 inch. This is the sum of the deflection at the top of the foundation plus the lateral rotation in radians at the top of the foundation times the length of the pole above the foundation.

If running a p-y analysis in the Strength Limit State, no load or resistance factors are used (all loads are nominal). For a standard 70-foot tall pole, use the following values and calculation steps:

1) Pile Head Loading: V = 1,264 lb; M = 496,038 in-lb; W = 11,061 lb;

2) Vary L typically from 20 feet to point of failure in one-foot increments;

3) Recalculate the Pile Properties for each increment of L;

4) Determine the design foundation length L to achieve the Strength Limit State deflection criteria (the same as for a Light Tower or Light Pole Foundation, as described in Section 1202).

For poles other than the standard 70-foot tall precast fiber-reinforced spun concrete pole, calculate the weight of the pole based on the dimensions and unit weight of the pole materials; calculate the wind load on the pole in accordance with LRFDLTS Articles 3.4 and 3.8. However, the design wind speed for the Service Limit State is 30 mph; the design wind speed for the Strength Limit State is 100 mph. In the Strength Limit State, use a wind (W) load factor of 1.15 for a Broms' method analysis.

1204 SIGNAL- AND SIGN-SUPPORT FOUNDATIONS

All signal- and sign-support foundations are designed in accordance with the ODOT Office of Roadway Engineering Traffic Engineering Manual (TEM), and Office of Roadway Engineering (ORE) standard drawings. According to the foundations standard drawings, TC-21.11, TC-21.21, ITS-30.12, ITS-35.12, and ITS-36.12, a Standard Foundation Design can be used if the average shear strength of the soils over the foundation length, L, meets or exceeds either S_u = 2000 psf (for cohesive soils) or ϕ_f = 30° (for granular soils). For mast-arm designs (TC-16.22, TC-81.22), if the soils do not meet these minimums, the standard design may be lengthened by the inverse of the ratio of the site soil strength to the required soil strength. For example, if we have S_u = 1700 psf cohesive site soils, it is acceptable to lengthen the standard design drilled shaft foundation by 2000/1700 = 1.18 times. Otherwise, perform a Special Foundation Design.

If performing a Special Foundation Design, calculate the wind load against the support structures and appurtenances in accordance with LRFDLTS Articles 3.4 and 3.8. The Extreme Limit State design wind speed is 115 miles per hour for signal- and sign-supports, it is 120 miles per hour for DMS Supports, and the wind (W) load factor of 1.00. These loads produce wind pressures of 33 psf for signal- and sign-supports, and 36 psf for DMS supports, respectively. For the calculation of vertical load acting at the top of the drilled shaft foundation unit, use a maximum dead load of components (DC) load factor of 1.25 for consideration of Strength Limit State bearing failure; use a minimum dead load of components (DC) load factor of 0.90 for considerations of Extreme Limit State overturning, uplift, and torsion; and use a dead load of components (DC) load factor of 1.00

for consideration of Service Limit State lateral deflection. The design wind speed for the Service Limit State is 76 mph in accordance with LRFDLTS Figure 3.8-4b. Sum the effects of moment, shear, and vertical load at each foundation unit, and determine the vertical, lateral, and moment loads transmitted to the top of each drilled shaft for each limit state and load combination.

1204.1 Mast-arm Signal and Sign Support Foundations

Perform Extreme Limit State design of mast-arm signal and sign pole foundations in accordance with TEM Sections 240-4.3(2), 440-3, and 440-4 and standard drawings TC-16.22, TC-81.22, and TC-21.21. If the soil shear strength meets or exceeds the requirements of TC-21.21, a Standard Foundation Design can be performed, using the following steps:

1) For each appurtenance attached to the mast-arm (generally signals and overhead signs), determine the cross-sectional area in square feet, but ignore the cross-sectional area of the mast-arm itself.

2) For each attached appurtenance, determine the horizontal distance from the center of the cross-sectional area to the axis of the signal pole in feet.

3) For each attached appurtenance, multiply the cross-sectional area times the distance to arrive at an “area moment” in feet^3, and sum all “area moments” to arrive at the “area moment design factor” value “K” in feet^3.

4) Consult the Table in TEM Section 440-3, Step 4, and reference the K value to the maximum allowable K value to determine the design number. According to TEM Section 440-3, Step 5, “As long as the K value is not exceeded, it is acceptable to exceed the “Maximum Design Area” shown on TC-81.22.”

5) Consult standard drawing TC-21.21 to determine the appropriate foundation dimensions and details for the appropriate design number under “TC-81.22 Type Supports.”

6) Ignore the Service Limit State check for a Standard Foundation Design.

If the K value exceeds the maximum for design number 14, or if the soil shear strength values are below the minimum requirements for a Standard Foundation Design according to TC-21.21, perform a Special Foundation Design.

For a Special Foundation Design for mast-arm signal and sign pole foundations, Extreme Limit State torsion (rotation of the mast-arm in the direction of the wind) typically controls the design. However, there are three Extreme and Strength Limit State checks for which the design must have adequate resistance:

- Extreme Limit State torsion from wind-load on the appurtenances and mast-arm,
- Extreme Limit State overturning moment, and
- Strength Limit State vertical bearing load.

Perform the following steps for Extreme and Strength Limit State Special Foundation Design:

1) For each appurtenance attached to the mast-arm, for the mast-arm itself, and for the pole, determine the cross-sectional area in square feet.

2) Multiply the cross-sectional area for each item by a wind pressure of 33 psf to arrive at a wind force.

3) Multiply each wind force by the wind (W) load factor 1.00 to arrive at factored wind force.

4) For each factored wind force, calculate two moments:

   a) for the distance from the axis of the signal pole in feet, M_h, and

   b) for the distance from the top of the foundation in feet, M_v.

5) Sum the M_h moments to arrive at a factored torsion from wind-load, ΣM_h.

6) Sum the M_v moments to arrive at a factored wind overturning moment, ΣM_v.

7) For each appurtenance attached to the mast-arm, for the mast-arm itself, and for the pole, determine the weight, W, in pounds-force. For the mast-arm and the pole, fabricated from ASTM A595 Grade A steel, assume a unit weight of 0.282 pci.

8) Multiply each weight by the dead load of components (DC) load factor 1.25 for Strength Limit State and the Extreme I Max Limit State, and by a DC load factor 0.90 for the Extreme I Min Limit State, to arrive at factored weight.

9) Multiply all of the factored weights by the distance from the axis of the signal pole to the center of gravity of the items in feet to arrive at a set of overturning moments, M_o. Sum the overturning moments and add the wind overturning moment M_v for the pole alone to arrive at a factored weight overturning moment ΣM_w.

10) Consider the factored wind overturning moment, ΣM_v, and factored weight overturning moment ΣM_w in both the X and Y axes in both the Extreme and Strength Limit States. Use a resultant of the factored overturning moments for the check against overturning moment, ΣM_o.

11) Sum all the factored weights in pounds-force to arrive at the factored vertical bearing load, ΣW.

12) Perform a single Strength Limit State drilled shaft axial resistance analysis in accordance with AASHTO LRFD Article 10.8.3.5. Calculate both the factored side resistance and the factored tip resistance separately.

13) For the check against Extreme Limit State factored torsion from wind-load, ΣM_h, compare only the factored side resistance for the drilled shaft foundation. Apply a resistance factor for side

resistance, $\varphi = 1.00$, for the Extreme Limit State in accordance with AASHTO LRFD Article 10.5.5.3.

14) For the check against factored overturning moment, ΣM_o, in the Extreme and Strength Limit States, perform either:

  a) A detailed p-y overturning resistance analysis, in the same way as for a Light Tower or Light Pole Foundation as described in Section 1202, with the factored overturning moment, ΣM_o. Also use the factored vertical bearing load, ΣW, with both a minimum dead load of components (DC) load factor of 0.90 and a maximum dead load of components (DC) load factor of 1.25.

  b) A Broms' Method overturning resistance analysis in the same way as for a Light Tower or Light Pole Foundation as described in Section 1202. For this analysis, assume e = Pole Height (according to the standard drawing) minus 1.5 feet, and $P_a = \Sigma M_o/e$.

15) For the Strength Limit State check against factored vertical bearing load, ΣW, compare the sum of the factored side resistance and the factored tip resistance for the drilled shaft foundation. Apply the appropriate resistance factors for Nominal Axial Compressive Resistance of Single Elements, φ_{stat}, for a drilled shaft in accordance with BDM Table 305-1.

16) Determine the maximum factored shear and moment in the drilled shaft for considerations of structural resistance in accordance with AASHTO LRFD Article 10.8.3.9.

If performing a Special Foundation Design, in the Service Limit State, check against excessive lateral deflection. There are two Service Limit State deflection criteria:

1) A maximum resultant projected lateral deflection at the top of the signal pole of 2.5% of the height of the pole. This is the sum of the deflection at the top of the foundation plus the lateral rotation in radians at the top of the foundation times the height of the pole above the foundation.

2) A maximum vertical deflection of the mast-arm of $\ell/150$, where ℓ is the length from the centerline of the pole to the end of the mast arm.

1204.2 Overhead Sign Support Foundations

Perform Extreme Limit State design of semi-, center mount, and cantilever overhead sign support foundations in accordance with TEM Sections 240-4.3(1) and 240-4.4 and standard drawings TC-9.11, TC-9.31, TC-12.31, and TC-21.21. If the soil shear strength meets or exceeds the requirements of TC-21.21, a Standard Foundation Design can be performed for semi- or center mount overhead sign support foundations by comparing sign area with the design numbers listed in either standard drawing TC-9.11 or TC-9.31, or for cantilever overhead sign support foundations by comparing sign area and distance to the centerline of the sign (moment arm) with the design numbers listed in TEM Figure 298-12. Then consult standard drawing TC-21.21 to determine the appropriate foundation dimensions and details for the appropriate design number under "TC-9.11 Type Supports," "TC-9.31 Type Supports," or "TC-12.31 Type Supports." Ignore the Service Limit State check for a Standard Foundation Design.

If the sign area or the moment arm exceeds the limits for the design numbers in standard drawings TC-9.11 or TC-9.31 or Figure 298-12, or if the soil shear strength values are below the minimum requirements for a Standard Foundation Design according to TC-21.21, perform a Special Foundation Design.

Perform Special Foundation Designs for overhead sign support foundations in the same way as for mast-arm signal and sign pole foundations, as given in Section 1204.1. For the Service Limit State, consider the value of ℓ as the length from the centerline of the pole to the outside corner of the sign or the length of the sign support truss, whichever is greater.

1204.3 Span-Wire Strain-Pole and Sign Foundations

Perform Extreme Limit State design of span-wire strain-pole, signal and sign foundations in accordance with TEM Sections 440-5 and standard drawings TC-81.11 and TC-21.21. Load analysis can be performed with the Span Wire Signal Support Design Software (SWISS) developed by the ODOT Office of Roadway Engineering. If the soil shear strength meets or exceeds the requirements of TC-21.21, a Standard Foundation Design can be performed by comparing the moment output by SWISS with the "Minimum Factored Flexural Resistance At Base (Ft Kips)" for the design numbers listed in standard drawing TC-81.11. Then consult standard drawing TC-21.21 to determine the appropriate foundation dimensions and details for the appropriate design number under "TC-81.11 Type Supports." Ignore the Service Limit State check for a Standard Foundation Design.

If the moment at the base of the strain-pole exceeds the "Minimum Factored Flexural Resistance At Base (Ft Kips)" for design number 14, or if the soil shear strength values are below the minimum requirements for a Standard Foundation Design according to TC-21.21, perform a Special Foundation Design.

At the top of each drilled shaft foundation unit, calculate the overturning moment from the wind load and the lateral tension load, the shear load from the wind load, and the vertical load from the dead load. Calculate the wind load against the strain-poles, messenger wires, and appurtenances in accordance with LRFDLTS Articles 3.4 and 3.8. Unless they are tethered (not allowed to swing freely), use an area factor of 0.50 for the signals, and an area factor of 0.10 for the signs. To calculate the lateral tension load from the messenger wires against the strain-poles, use the self-weight of the wires and appurtenances distributed across the length of the span-wire. For the dead load acting vertically at the top of the drilled shaft foundation unit, calculate the sum of the weight of the signals, signs, electrical wires, messenger wire, and strain-poles.

For the Extreme Limit State calculation of lateral tension load from the messenger wires against the strain-poles, use a maximum dead load of components (DC) load factor of 1.25. For the Strength Limit State calculation of vertical load acting at the top of the drilled shaft foundation unit, use a maximum dead load of components (DC) load factor of 1.25. For the Extreme Limit State calculation of vertical load acting at the top of the drilled shaft foundation unit, use a maximum dead load of components (DC) load factor of 1.25 for the Extreme I Max Limit State and use a minimum dead load of components (DC) load factor of 0.90 for the Extreme I Min Limit State.

If performing a Special Foundation Design, in the Service Limit State, check against excessive lateral deflection. There are two Service Limit State deflection criteria:

1) A maximum lateral rotation in radians at the top of the foundation of 0.0055 radians.

2) A maximum resultant projected lateral deflection at the top of the strain-pole of 1.5% of the height of the pole. This is the sum of the deflection at the top of the foundation plus the lateral rotation in radians at the top of the foundation times the height of the pole above the foundation.

If performing a Special Foundation Design, in the Extreme Limit State, check resistance against overturning in the same way as for a Light Tower or Light Pole Foundation, as described in Section 1202. Determine the maximum factored shear and moment in the drilled shaft for considerations of structural resistance in accordance with AASHTO LRFD Article 10.8.3.9.

1204.4 Span-Wire Sign Support Foundations

Perform Extreme Limit State design of span wire sign support foundations in accordance with TEM Section 240-4.5(1) and standard drawings TC-17.11 and TC-21.21. If the soil shear strength meets or exceeds the requirements of TC-21.21, a Standard Foundation Design can be performed by comparing sign area and span length with the design numbers listed in TEM Figure 298-18. Then consult standard drawing TC-21.21 to determine the appropriate foundation dimensions and details for the appropriate design number under "TC-17.11 Type Supports." Ignore the Service Limit State check for a Standard Foundation Design.

If the sign area or the span length exceeds the limits for the design numbers in standard drawings Figure 298-18, or if the soil shear strength values are below the minimum requirements for a Standard Foundation Design according to TC-21.21, perform a Special Foundation Design.

Perform Special Foundation Designs for overhead sign support foundations in the same way as for span-wire strain-pole signal and sign foundations, as given in Section 1204.3. However, note that with two span wires, the signs are not free-swinging.

1204.5 Span Truss Sign and DMS Support Foundations

The foundations for span truss sign and DMS support foundations consist of two foundation units, one on each side of the roadway, each consisting of a tie beam and two drilled shafts. Because the drilled shafts have a fixed connection to the tie beam, they are rigid in the longitudinal direction of the drilled shaft group and are only capable of translation (not rotation), or of failure by bearing or pullout. In the direction transverse to the drilled shaft group, the shafts are free to laterally rotate at the head, but the loading is small in this direction and the shafts are partially constrained by the connection to the truss, therefore deflections in this direction are expected to be insignificant. Due to the head rigidity and constraint on deflections, Service Limit State checks are not applicable to the design of span truss foundation units and are relevant only to structural analysis of deflections in the supported span truss.

Perform Extreme Limit State design of span truss sign and DMS support foundations in accordance with TEM Sections 240-4.5(2) or 1303-5 and standard drawings TC-15.116 and TC-21.11 or ITS-35.13 and ITS-35.12 or ITS-36.12. If the soil shear strength meets or exceeds the requirements of TC-21.11, ITS-35.12, or ITS-36.12, a Standard Foundation Design can be performed for sign support foundations by comparing span length and sign area with the design numbers listed in TEM Figure 298-13 or for DMS support foundations by comparing span length with the maximum span for the design numbers listed in standard drawing ITS-35.13. Then consult standard drawing TC-21.21, ITS-35.12, or ITS-36.12 to determine the appropriate foundation dimensions and details for the appropriate design number.

If the sign area or the span length exceeds the limits for the design numbers in Figure 298-13 or standard drawing ITS-35.13, or if the soil shear strength values are below the minimum requirements for a Standard Foundation Design according to TC-21.11, ITS-35.12, or ITS-36.12, perform a Special Foundation Design.

In the Strength Limit State, check the foundations for bearing resistance. In the Extreme Limit State, check the foundations for resistance against uplift (for overturning), and lateral load stability. At the top of each foundation unit, calculate the overturning moment and the shear load from the wind load, and the vertical load from the dead load of the span truss structure and supported signs, catwalks, and other appurtenances.

Keep in mind that Extreme Limit State design wind pressures are 33 psf for span truss sign supports, and 36 psf for span truss DMS supports, respectively, according to Section 1204.1.

Perform a drilled shaft axial resistance analysis in accordance with BDM Section 305.4 and AASHTO LRFD Article 10.8.3.5. Consider group effects in accordance with AASHTO LRFD Article 10.8.3.6. Calculate both the factored side resistance and the factored tip resistance separately. For Strength Limit State bearing resistance, consider both the factored side resistance and the factored tip resistance. Apply the appropriate resistance factors for Nominal Axial Compressive Resistance of Single Elements, φ_{stat}, for a drilled shaft in accordance with BDM Table 305-1.

If the Extreme Limit State overturning moment couple at a foundation unit results in uplift on one of the two drilled shafts, perform a calculation of drilled shaft uplift resistance in accordance with BDM Section 305.4.1.3 and AASHTO LRFD Article 10.8.3.7. For uplift resistance, consider both the drilled shaft factored side resistance and the foundation self-weight. For side resistance, apply a resistance factor for uplift resistance, $\varphi_{up} = 0.80$, for a drilled shaft in accordance with AASHTO LRFD Article 10.5.5.3. For foundation self-weight, apply the minimum dead load of components (DC) load factor of 0.90.

Perform an analysis of the lateral load effects on the drilled shaft foundations in accordance with BDM Section 305.1.2 and AASHTO LRFD Article 10.8.3.8, using p-y analysis methods. Consider group effects and p-multipliers. For loading in the longitudinal direction of the drilled shaft group, utilize p-multipliers in accordance with AASHTO LRFD Article 10.7.2.4. See FHWA GEC 10, Section 11.5.1, Table 11-1 regarding p-multipliers with a more comprehensive table of values for drilled shafts. For loading in the direction transverse to the drilled shaft group, utilize a p-multiplier in accordance with BDM Section C305.1.2. Determine the maximum factored shear and moment in the drilled shaft for considerations of structural resistance in accordance with AASHTO LRFD Article 10.8.3.9. In the Extreme Limit State p-y analysis, check that the deflection is not very large (typically around 100 inches) or infinite or incalculable (failure of the program to converge at a solution). It is otherwise acceptable for the Extreme Limit State analysis deflection to be quite large (on the order of 5 to 10 inches).

1204.6 DMS Pedestal Support Foundations

Perform Extreme Limit State design of DMS pedestal support foundations in accordance with TEM Section 1303-5 and standard drawings ITS-30.13 and ITS-30.12 or ITS-36.12. If the soil shear strength meets or exceeds the requirements of ITS-30.12 or ITS-36.12, the Standard Foundation Design can be used (there is only one standard design). Consult standard drawings ITS-30.12 or ITS-36.12 to find the drilled shaft foundation dimensions and details. Ignore the Service Limit State check for the Standard Foundation Design.

If the soil shear strength values are below the minimum requirements for the Standard Foundation Design according to ITS-30.12 or ITS-36.12, perform a Special Foundation Design.

Perform Special Foundation Designs for DMS pedestal support foundations in the same way as for overhead sign support foundations, as given in Section 1204.2, with the exception that wind load for the Extreme Limit state is at the 1700-year Mean Recurrence Interval (MRI), 120 mph design wind speed, producing a wind pressure of 36 psf against the full cross-sectional area of the DMS and pedestal support. For the Service Limit State, consider the value of ℓ as the maximum length from the centerline of the pole to the outside end of the supported truss, signs, and catwalks, wherever this is greatest.

SECTION 1300 BRIDGE FOUNDATIONS

1301 GENERAL

In addition to the guidance and requirements set forth in this document, design all bridge foundations in accordance with the latest requirements of the BDM and AASHTO LRFD.

1302 SCOUR

The design engineer (typically the hydraulic engineer) needs to perform a scour evaluation for all waterway crossing structures with the exception of pipe and four-sided culverts in accordance with L&D2. They will compute scour depth for all bridge foundations except for spread footings founded on scour resistant bedrock. See BDM Section 305.2.1.2.b.(B) for the definition of scour resistant bedrock and non-scour resistant bedrock. When evaluating scour for a replacement structure, review all inspection reports for evidence of stream degradation, scour or previous scour countermeasures.

Scour evaluations require an interdisciplinary approach. Structural engineers perform or have oversight in routine bridge inspections that identify concerns due to scour, stream degradation/aggregation, meandering, and prior use of scour countermeasures. Hydraulic engineers provide the hydraulic loading applied to the bridge foundation created by the stream or channel flow. Shear forces are generated based on equations in publication FHWA-HIF-12-003 (HEC 18) "Evaluating Scour at Bridges." Geotechnical engineers evaluate the resistance to the hydraulic loading – the critical shear stress at which mobilization of soil particles occurs – based on streambed soil or rock properties. The resulting scour depth, based on the interaction between the flow, hydraulic depth, shear forces, and critical shear stress, is determined by the hydraulic engineer, in coordination with the geotechnical engineer.

For bridges and 3-sided structures over water, perform scour sampling and testing in accordance with SGE Section 303.7.1. If the predicted depth of scour exceeds the 6.0-foot soil profile represented by the continuous scour sampling and testing required, subsequent samples and strata must be considered in analyzing the potential for deeper scour.

1302.1 Critical Shear Stress

The design engineer will perform scour depth calculations by stratum in accordance with the requirements of L&D~~V~~2, Section 1008.10 using the FHWA Hydraulic Toolbox. If the channel shear stress (τ) predicted by a hydraulic model or by equations such as those in HEC 18 exceeds the scour critical shear stress (τ_c) for any stratum, then scour occurs.

1302.1.1 Granular and Cohesive Soils

For granular soils, critical shear stress is determined to be a function of the mean particle grain size using the equation in HEC 18 Figure 4.6, "Critical shear stress vs. particle grain size (Briaud et al. 2011)."

$$\tau_c\ (Pa) = D_{50}\ (mm)$$

where:

τ_c = Critical shear stress (Pa)
D_{50} = mean particle grain size (mm), ≥ 0.2 mm

For cohesive soils, The Hydraulic Toolbox has a cohesive soil computation method which requires τ_c as an input, however, FHWA and L&D2 only allow for the use of τ_c as determined by laboratory testing using the Erosion Function Apparatus or similar device. Unfortunately, this testing is not readily available nor regularly performed. In the absence of this data, L&D2 directs the designer to consider a cohesive soil with a $D_{50} < 0.2$ mm as a granular soil with a $D_{50} = 0.2$ mm. Therefore, for a cohesive soil, determine and report either the laboratory derived τ_c (Pa) or the average D_{50} (mm) for each stratum.

Through interdisciplinary research, ODOT and FHWA have a goal of improving scour analysis and providing more accurate scour depth estimates for bridge foundation design. FHWA is developing the NextScour Research Initiative. ODOT is executing a research project titled, "Scour Critical Shear Stress of Ohio Soils" with a primary goal of determining an acceptable equation for predicting the critical shear stress of cohesive soils in Ohio. OGE is participating in an FHWA Erosion Testing Services pooled fund study by sending cohesive soil samples to FHWA for laboratory erosion testing.

1302.1.2 Bedrock

Determine scour critical shear stress of a non-scour resistant bedrock by rearranging HEC 18 Equations 7.38 for 'Critical Stream Power' and 7.39 'Approach Flow Stream Power' to derive the critical shear stress for non-scour resistant bedrock as follows:

$$\tau_c = \rho \left(\frac{1000\, K^{0.75}}{7.853\, \rho} \right)^{2/3}$$

where:

τ_c = Critical shear stress (Pa)
ρ = Mass density of water (1000 kg/m^3)
K = Erodibility Index (dim.)

Determine the Erodibility Index, K, in accordance with BDM Section 305.2.1.2.b(B.6.a), using HEC 18 Equation 4.17. Use the guidance of BDM Section 305.2.1.2.b(B) and HEC 18 Section 4.7.2 on the selection of values for the several parameters; Intact Rock Mass Strength Parameter, M_s, Rock Joint Set Number, J_n, Joint Roughness Number, J_r, Joint Alteration Number, J_a, and Relative Joint Orientation Parameter, J_s.

Choose reasonable values for these parameters, based on a geologist's direct evaluation of the bedrock core and rock exposures, if present. Do not automatically assume the worst-case values of $J_n = 5$, $J_r = 1$, $J_a = 5$, and $J_s = 0.4$ as given in the BDM. For reference, these assumed values would mean: $J_n = 5$, there are multiple joint/fissure sets running through the rock that control its behavior; $J_r = 1$, the joints/fissures are either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation; $J_a = 5$, the joints/fissures contain shattered or micro-shattered (swelling) clay gouge, with or without crushed rock; and $J_s = 0.4$, that the rock bedding is both inclined steeply in a direction adverse to the stream flow and that the rock is made up of long, thin plates that are easy to rip out. The combination of these values is unrealistic, and OGE considers evaluation of bedrock scour with these rock properties suspect. Typical values for Ohio bedrock are as follows:

- Rock Joint Set Number ($J_n \leq 5$): Statewide, $J_n = 1.50$; One joint/fissure set plus random

- Joint Roughness Number ($J_r \geq 1$)

  - Unweathered sandstone or limestone, $J_r = 4.0$, Stepped joints/fissures

  - Moderately weathered sandstone or limestone, $J_r = 3.0$, Rough or irregular, undulating

  - Unweathered shale, highly weathered sandstone or limestone, $J_r = 2.5$, Rough or irregular, undulating

  - Highly weathered shale, $J_r = 2.0$, Smooth undulating

  - Decomposed rock (residuum), $J_r = 1.5$, Slickensided undulating

  - Decomposed rock in a shear failure zone, $J_r = 1.0$, Smooth planer

- Joint Alteration Number ($J_a \leq 5$)

  - Unweathered rock, $J_a = 0.75$, Tightly healed, hard, non-softening impermeable filling

  - Unweathered to slightly weathered rock, $J_a = 1.0$, Unaltered joint walls, surface staining only

  - Slightly weathered to moderately weathered rock, $J_a = 2.0$, Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling

  - Moderately weathered to highly weathered rock, $J_a = 3.0$, Non-softening, strongly over consolidated clay mineral filling, with or without crushed rock

  - Highly weathered to severely weathered rock, $J_a = 4.0$, Softening, moderately over-consolidated clay mineral filling, with or without crushed rock

  - Severely weathered to decomposed rock (residuum), $J_a = 5.0$, Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock

- Relative orientation parameter ($J_s \geq 0.4$), Statewide, $J_s = 1.0$, Joints oriented roughly flat with respect to stream flow

If RQD = 0, do not set Block Size Parameter $K_b = 0$ and subsequently Erodibility Index K = 0. In this case, set the minimum value of $K_b = 0.10$. In the scour calculations that depend on K, it is in the denominator, and K = 0 will result in a divide by zero error.

For example, for a very weak and weathered rock with Q_u = 43 psi = 0.30 MPa, RQD = 4, J_n = 1.50, J_r = 1.0, J_a = 5.0, and J_s = 0.9:
for Q_u < 10 MPa, $M_s = 0.78\ Q_u^{1.05} = 0.78\ (0.30)^{1.05} = 0.22$;
$K_d = J_r/J_a = 1.0/5.0 = 0.20$, and $K_b = RQD/J_n = 4/1.50 = 2.67$;
$K = (M_s)(K_b)(K_d)(J_s) = (0.22)(2.67)(0.20)(0.9) = 0.106$.
$\tau_c = \rho\ (1000\ K^{0.75} / 7.853\ \rho)^{2/3} = 1000\ (\cancel{1000} \times 0.106^{0.75} / 7.853 \times \cancel{1000})^{2/3} = 1000\ (0.185/7.853)^{2/3}$ = 82.4 Pa = 1.72 psf.

1302.2 Erosion Category
If the ultimate scour calculated by the design engineer proves excessive, and the flood hydrograph for the scour flood is available, then the design engineer can perform a time-rate of scour analysis. For this analysis, the geotechnical engineer will need to provide the erosion category (EC) for each stratum, which is dimensionless.

For cohesive soils, $EC = 4.5 - \frac{3}{1.07^{PI}}$

where:

PI = Plasticity Index (dimensionless)
$1.5 \leq EC \leq 4.5$

For granular soils, $EC = 1.2\big(1.83333 + Log(D_{50})\big)$

where:

D_{50} = mean particle grain size (mm), ≥ 0.2 mm
$1 \leq EC \leq 6$

For bedrock, $EC = 1.2(1.83333 + Log\left(\frac{S_v}{2.5}\right))$
where:
S_v = average vertical spacing between horizontal discontinuities (mm), ≥ 0.25 mm,
$1 \leq EC \leq 6$

Report soil and bedrock scour data for use in the scour evaluation in a tabular form in the Geotechnical Profile, as shown in the examples below:

Soil Scour Samples (EXAMPLE)				
Boring No.	Sample	Elevation	D_{50} (mm)	Erosion Category (EC)
B-001-0-19	SS-4	920.9' - 919.4'	0.0185	2.975
	SS-5	919.4' - 917.9'	0.1230	2.211
	SS-6	917.9' - 916.4'	0.1731	1.286
	SS-7	916.4' - 914.9'	0.0662	2.361

Bedrock Scour Samples (EXAMPLE)					
Boring No.	**Sample ID**	**Sample Elevation**	**K Index**	**τ_c Value (Pa)**	**Erosion Category**
B-003-0-22	NQ-1	654.0' - 643.8'	4524	17022	4.422
	NQ-2	643.8' - 633.8'	155	3152	3.229
	NQ-3	633.8' - 631.3'	19.74	1124	2.561
	NQ-4	631.3' – 620.0'	14.6	967	2.295
	NQ-5	620.0' - 608.8'	1077	8306	4.133
	NQ-6	608.8' - 606.5'	646	6434	3.898

1302.3 Foundation Design for Scour

The controlling scour elevation may occur with the design flood, check flood or an overtopping flood of lesser recurrence interval. Use the scour elevation resulting from the design flood with Strength and Service Limit State checks. Use the scour elevation resulting from the check flood with Extreme Event II Limit State checks. Neglect the side resistance (axial or lateral) provided by soil or bedrock above the controlling scour elevation. For a definition of the scour design flood and scour check flood, see AASHTO LRFD Article 2.6.4.4.2.

Ensure that friction deep foundation elements extend a minimum of 15 feet below either the Thalweg elevation or the controlling scour elevation, whichever is deeper. If the design flood scour reaches top of rock, do not use driven piles; use either spread footings bearing on or within the bedrock or drilled deep foundation elements socketed into the bedrock. The foundations must penetrate a specified distance below the controlling scour elevation for non-scour resistant bedrock. See BDM Section 305.2.1.2.b.(B) for the definition of scour resistant bedrock.

If spread footings are bearing on non-scour resistant bedrock, locate the bottom of footings at least one foot below the controlling scour elevation in the bedrock; if no bedrock scour is anticipated, locate the bottom of footings at least one foot below the top of rock. If spread footings are bearing on scour resistant bedrock, key the bottom of footings at least 3 inches into rock. Spread footings bearing on soil are not allowed for support of bridge structures when scour is a possibility; however, spread footings bearing on soil are allowed for support of retaining walls, three-sided culverts, and culvert headwalls. If bearing in soil, spread footings subject to scour must bear a minimum of 2 feet below the controlling scour elevation; the effective depth of footing, D_f, will become the depth of the footing below the controlling scour elevation. See Section 1400 for other requirements of spread footing foundations for three-sided culverts and culvert headwalls.

If drilled deep foundation elements are socketed into non-scour resistant bedrock, when the rock has Q_u < 2500 psi, extend the foundation elements to penetrate a minimum of 10 feet below the controlling scour elevation in the bedrock; when the rock has $Q_u \geq$ 2500 psi, extend the foundation elements to penetrate a minimum of 5 feet below the controlling scour elevation in the bedrock. If socketed into scour resistant bedrock, design the length of the bedrock socket based solely on the requirements of axial resistance and of lateral stability.

Because deep foundation elements will lose support above the controlling scour elevation, investigate the structural capacity of the foundation elements considering the length above the controlling scour elevation as an unbraced length. Analyze the deep foundation elements for buckling and lateral stability as unsupported columns above the point of fixity with scour losses included in accordance with Section 1301.6 and BDM Section 305.3.5.5. Also perform a p-y analysis on the deep foundation elements according to Section 1301.6 and BDM Section 305.1.2 to demonstrate:

- Lateral stability against overturning failure in the Strength Limit State with the scour elevation estimated at the design flood.

- Lateral stability against overturning failure in the Extreme Event II Limit State with the scour elevation estimated at the check flood.

- Lateral stability against excessive deflection in the Service Limit State with the scour elevation estimated at the design flood.

Keep in mind that if deep foundation elements are not tangent and if the scour progresses below the deep foundation cap or footing by a depth greater than the width of the footing, lateral loads from retained soil will also disappear as the retained soil is scoured away (see Section 1403 regarding the “second and third stages of scour.”) Also, if a bridge approach is scoured away, eliminate loads related to live load on the bridge (vehicles cannot reach the bridge if the approaches are gone). For stability of drilled deep foundation elements socketed into bedrock, the extension into rock creates a point of fixity. If necessary for lateral stability, increase the length of the rock socket.

Any time soil subject to scour is composed of layers of both cohesive and granular soils, perform a layer-by-layer scour analysis, in accordance with L&D2 Section 1008.10.4, treating the cohesive and granular soils as separate layers. Also, any time scour progresses through soil overburden and reaches top of rock, perform a layer-by-layer scour analysis, as a minimum treating the soil overburden and bedrock as separate layers. Often, divide the bedrock into multiple layers, considering the type, strength, intactness, and state of weathering of the bedrock in accordance with Section 1302.1.2.

1303 SPREAD FOOTINGS

See BDM Section 305.2 for specifications regarding the design of spread footing foundations. Design spread footings for the support of bridge piers in accordance with ASHTO LRFD Article 10.6, using resistance factors in accordance with ASHTO LRFD Article 10.5.5.2.2. Design spread footings for abutments and retaining walls, including footings for three-sided culverts and culvert headwalls, in accordance with LRFD Section 11.6, using resistance factors in accordance with ASHTO LRFD Article 11.5.7. The reason for the different sets of resistance factors is that the load combinations are different for bridge piers than for abutments and retaining walls. The resistance factors in LRFD Table 10.5.5.2.2-1 are calibrated to previous practice factors of safety for bridge pier footings. The reason that the resistance factors in this table are in general less than in Table 11.5.7-1 for retaining walls is due to the lower load factors specified in LRFD Table 3.4.1-2 for the most significant sources of loading carried from a bridge superstructure. The use of different resistance factors to offset the different predominant load factors results in a similar reliability for the shallow foundations between bridge piers and retaining walls.

Also use the resistance factors in ASHTO LRFD Article 10.5.5.2.2 if AASHTO LRFD is used to design spread footing foundations for:

- Building interior columns
- Building walls
- Luminaires, sign, or signal support poles
- Noise barriers
- Any other structure, where the most significant sources of loading are Dead Load (DC or DW), Live Load (LL or PL), or Wind (WS or WL).

Use the resistance factors in ASHTO LRFD Article 11.5.7 whenever AASHTO LRFD is used to design spread footing foundations for any other structures where the most significant sources of loading are Earth Pressure (EH, EV) or Surcharge (ES, LS).

1303.1 Design Considerations

Proportion and optimize spread footing dimensions for the controlling Limit State and failure mode, including settlement (both total and differential) in the Service Limit State, and bearing, overturning (limiting eccentricity), sliding, and overall stability in the Strength and Extreme Event Limit States. For other design considerations relating to spread footings for the support of bridge structures, see BDM Section 305.2.

1303.1.1 Bearing Depth

Refer to BDM Section 305.2.1.2 and Figure 305-2 for bearing depth requirements for bridge pier and abutment spread footing foundations. Otherwise, for spread footings bearing on soil supporting all other structures, the bearing depth shall be:

- At or below the frost line in accordance with BDM Section 305.2.1.2.a(E) and Figure 305-3.

- At or below the depth of probable disturbance due to adjacent future utility or roadway excavations. If utilities or pavement will be present within a lateral distance of 4B from the middle of the foundation (where B is the footing width), assume a minimum depth of soil disturbance of 3 feet.

- A minimum of 2 feet below the predicted scour elevation, for foundations subject to the effects of scour.

For spread footings bearing on bedrock, the footings shall penetrate a minimum of 3 inches into the rock. See Section 1302.3 and BDM Sections 305.2.1.2 and 606.6 for additional requirements related to scour and detailing for sloping bedrock.

1303.1.2 Effective Footing Dimensions

Calculate eccentricity and effective footing dimensions for each limit state and load combination. Calculate eccentricity as $e = \Sigma M/\Sigma V$ (as shown in AASHTO LRFD Figure 11.6.3.2-1), where:

e = eccentricity in the horizontal direction of interest
ΣM = sum of moments about the center of the footing in the direction of interest
ΣV = sum of vertical loads, including the weight of the footing

For footings bearing on soil, calculate effective footing dimensions in accordance with ASHTO LRFD Article 10.6.1.3. Use the most critical combination of effective footing dimensions and load combination for the calculation of external stability in each limit state.

Effective footing area, A', is defined as $A' = B' \times L'$, where:

B' = effective footing width
L' = effective footing length

1303.1.3 Bearing Stress Distributions

Use a bearing stress distribution for settlement in the Service Limit State and bearing in the Strength and Extreme Event Limit States, in accordance with ASHTO LRFD Article 10.6.1.4. Calculate the uniform and linearly varying bearing stress for all spread footing foundations in accordance with AASHTO LRFD Article 11.6.3.2. Take note of whether e is less than or equal to B/6 in the calculation of the linearly varying bearing stress: if e is less than or equal to B/6, use AASHTO LRFD Equations 11.6.3.2-2 and 11.6.3.2-3; if e is greater than B/6, use AASHTO LRFD Equations 11.6.3.2-4 and 11.6.3.2-5. Provide the value of linearly varying bearing stress to the structural designer for the internal reinforcement design of the spread footing foundation. For footings bearing on soil, check bearing resistance versus the uniform bearing stress over the effective footing area. For footings bearing on rock, check bearing resistance versus the maximum value of the linearly varying bearing stress. For undrained sliding resistance on cohesive soils, use a stress distribution (q_s) equal to half the linearly varying bearing stress in accordance with AASHTO LRFD Figure 10.6.3.4-1.

1303.2 Service Limit State Design

See BDM Sections 305.1.3 and 307.1.6 for total and differential vertical deformation limits for spread footing foundations for bridges and retaining walls. For spread footing foundations for all other structures, limit total and differential vertical deformations to the values necessary to prevent unacceptable deformations or overstress of the supported elements, in accordance with AASHTO LRFD Articles 2.5.2.6, the Service and Strength Limit State requirements of AASHTO LRFD Sections 5 through 9, or other requirements according to the structural designer. Otherwise, the Service Limit State total vertical deformation limit for spread footing foundations is 2 inches or three percent (3%) of the minimum footing dimension, whichever is less, and the differential vertical deformation limit is 1/500 over either the width or length of the footing.

1303.2.1 Settlement

Calculate Service Limit State bearing stress for spread footings in accordance with Section 1303.1.3. Perform settlement analyses for spread footings in accordance with BDM Section 305.2.1.1 and AASHTO LRFD Article 10.6.2.4. Alternately, settlement may be estimated by CPT direct methods; see Section 1303.2.3. Only consider settlement over a depth in which the increase in vertical effective stress in the foundation soil or bedrock is 10% or greater; below this, settlement may be considered negligible. For shallow foundations, start with the bearing stress over the effective bearing area, A′. For deep foundation groups, start with the bearing stress over the perimeter of the foundation group. For shallow foundations, the point of bearing stress application is the base of the foundation. For deep foundations, see AASHTO LRFD Article 10.7.2.3 for the location of the point of bearing stress application. Decrease the bearing stress with depth below the point of bearing stress application in an acceptable manner; typically, apply the bearing stress over an area that increases with depth at a 1(H):2(V) slope. Other acceptable methods include Westergaard or Boussinesq stress distribution, Osterberg Influence Factor, and other methods referenced in AASHTO LRFD Article 10.6.2.4.1.

Do not analyze any single layer thicker than 10 feet; subdivide thicker soil strata.

For foundations bearing on cohesionless soils or bedrock, determine elastic settlement due to the incremental loading during the construction sequence. Settlement of foundations bearing on cohesionless soils may be assumed to be negligible if the Service Limit State bearing stress (in psf) is less than fifty (50) times the friction angle of the foundation soil (in degrees). Settlement of foundations bearing on bedrock may be assumed to be negligible if the maximum Service Limit State bearing stress is less than fifty (50) times the rock mass modulus, E_m. Take note that the mass modulus, E_m, is not the same as the intact modulus (E_i or E_R). Mass modulus and intact modulus can be correlated through uniaxial compressive strength, q_u, and RQD or GSI. See AASHTO LRFD Table 10.4.6.5-1 for recommended correlations.

For foundations bearing on cohesive soils, report total settlement and report separately the different components of settlement: immediate elastic settlement, consolidation settlement, and secondary settlement. Settlement of foundations bearing on cohesive soils may be assumed to be negligible if the Service Limit State bearing stress over the effective footing area, A′, is less than fifty percent (50%) of the undrained shear strength of the foundation soil.

1303.2.2 Service Limit State Bearing and Sizing

In addition to meeting the requirements of Strength and Extreme Event Limit State design, size footings to meet the requirements for Service Limit State design in accordance with Section 1303.2. Preliminary sizing of footings, to be used as a starting point in the analyses, can be done using Presumptive Bearing Resistance in accordance with AASHTO LRFD Article 10.6.2.5. Do not use Presumptive Bearing Resistance for Strength and Extreme Event Limit State design.

1303.2.3 Direct Design by CPT

Final bearing resistance analysis of spread footings is typically performed in the Strength Limit State in accordance with Section 1303.3. However, a CPT direct design method, as put forth by "Generalized Direct CPT Method for Evaluating Footing Deformation Response and Capacity on Sands, Silts, and Clays," Mayne and Woeller (2014), and modified by Dettloff (2016), uses a displacement-based method, which is usually limited by the Service Limit State. However, bearing resistance in the Strength and Extreme Even Limit States is also based on a limiting displacement corresponding to soil behavior type. This section describes the implementation of this method, which we will call the "Mayne-Woeller method."

The Mayne-Woeller method takes the form of the following equation:

$$q = h_s\, q_{tnet}\, (s/B)^{0.5} < q_{ult}$$

where:

q = applied foundation stress (bearing stress) = σ_v
h_s = empirical fitting term that depends on soil behavior type, $h_s = f(I_c)$
I_c = soil behavior type index
q_{tnet} = net total cone resistance (same units as q), $q_{tnet} = (q_t - \sigma_{vo})$
q_t = corrected CPT tip resistance (same units as q), $q_t = q_c + u_2\,(1 - a)$
q_c = CPT cone (tip) resistance (same units as q)
u_2 = CPT instantaneous pore pressure, measured just behind the cone (same units as q)
a = CPT probe net area ratio
σ_{vo} = total in-situ vertical stress (same units as q)
s = vertical displacement of the foundation (units of length)
B = foundation width (same units as vertical displacement, s)
s/B = normalized displacement (or pseudo-strain)
q_{ult} = ultimate (nominal) bearing resistance = q_n

See "Guide to Cone Penetration Testing for Geotechnical Engineering," Robertson and Cabal (2015) for more details on the above variables and factors.

The Mayne-Woeller method equation can be rewritten in the form:

$$q_{ult} = q_n = h_s\, q_{tnet}\, (s_{max}/B)^{0.5}, \text{ where } s_{max}/B = (s/B)_{max}$$

where:

s_{max} = maximum vertical displacement at the ultimate bearing resistance

$(s/B)_{max}$ = ultimate normalized displacement (or pseudo-strain), $(s/B)_{max} = f(I_c)$

In the Mayne-Woeller method, the value of $(s/B)_{max}$, corresponding to q_{ult}, is dependent on soil type, and ranges from 3% for clays to 12% for sands. However, this produces a variable level of reliability with respect to the Service Limit State and potentially warrants a variable resistance factor for the Strength Limit State. To establish a consistent level of reliability and use a consistent resistance factor, Dettloff (2016) compared the predicted stress versus displacement behavior of the Mayne-Woeller method with the theoretical nominal bearing resistance predicted by the Terzaghi bearing capacity equation for a variety of foundation soil types and shear strengths. It was discovered that the Terzaghi allowable (service load) bearing resistance generally occurred around a predicted normalized displacement of approximately (s/B) = 1% (0.01) particularly for granular soils. Given the typical foundation sizes for transportation structures, this correlates to a deflection in the range of 1 inch. Therefore, to establish a consistent level of reliability, a fixed displacement criterion of 1 inch can be established for the Service Limit State. This criterion will usually control the design, as the Strength Limit State bearing resistance generally occurs at a displacement limit beyond this, except for smaller foundation sizes on clay soils. We will refer to this criterion as:

s_{lim} = limiting vertical displacement criterion at the Service Limit State = 1.00 inch
$(s/B)_{lim}$ = limiting normalized displacement (or pseudo-strain) at the Service Limit State

For structures that are less settlement critical, a larger Service Limit State displacement criterion (s_{lim}) may be established with the agreement of the structural engineer, as long as the displacement does not compromise the performance of the structure or exceed the requirements of BDM Sections 305.1.3 and 307.1.6. If a larger Service Limit State displacement criterion is established, the Strength Limit State will more often control the design.

The step-by-step design methodology for direct design by CPT is as follows:

1.) Establish spread footing foundation dimensions.

2.) Determine loads, eccentricities, effective footing widths, and vertical stresses, σ_v, for the spread footing foundation at the various Service and Strength Limit States (and Extreme Event Limit States, when applicable) in accordance with Section 1303.1.

3.) Average the CPT data over a depth of 1.5 B below the bottom of the foundation elevation to determine average I_c and q_{tnet} soil properties for the bearing foundation soil.

4.) Calculate h_s from the following equation as a function of the average I_c:

$$h_s = 2.8 - \frac{2.3}{1 + (I_c/2.4)^{15}}, \text{ where } 0.50 \leq h_s \leq 2.8$$

5.) Calculate the limiting Service Limit State vertical stress, q_{lim}, based on the Mayne-Woeller method equation as follows:

$$q_{lim} = h_s\, q_{tnet}\, (s_{lim}/B)^{0.5}, \text{ where typically, } s_{lim} = 1.00 \text{ inch}$$

6.) Calculate $(s/B)_{max}$ from the following equation as a function of the average I_c:

$$(s/B)_{max} = 0.03 + \frac{0.09}{1+[(I_c-0.67)/1.7]^{15}}, \text{ where } 0.03 \leq (s/B)_{max} \leq 0.12$$

7.) Calculate the Strength Limit State ultimate (nominal) bearing resistance, q_n, based on the Mayne-Woeller method equation as follows:

$q_n = q_{ult} = h_s\, q_{tnet}\, (s_{max}/B)^{0.5}$, where $s_{max}/B = (s/B)_{max}$

8.) Calculate the Strength Limit State factored bearing resistance $q_R = \varphi_b q_n$, where the bearing resistance factor is determined as follows:

$\varphi_b = 0.70$ for bridge pier footings

$\varphi_b = 0.75$ for retaining wall or abutment footings

9.) When applicable, calculate the Extreme Event Limit State factored bearing resistance $q_R = \varphi_b q_n$, where the bearing resistance factor is $\varphi_b = 1.00$ for the Extreme Event Limit State.

10.) Compare σ_v to q_{lim} at the Service Limit State and calculate $CDR_{svc} = q_{lim}/\sigma_v$.

11.) Compare σ_v to q_R at the Strength Limit State and calculate $CDR_b = q_R/\sigma_v$.

12.) Compare σ_v to q_R at the Extreme Event Limit State and calculate $CDR_b = q_R/\sigma_v$.

13.) If either CDR_{svc} or CDR_b is less than 1.00 at any of the Service, Strength, or Extreme Event Limit States, then resize the footing, and return to Step 1.) until both CDR_{svc} and CDR_b are greater than or equal to 1.00 at all applicable limit states.

Consider overturning (limiting eccentricity) in accordance with Section 1303.3.4.

Failure by sliding cannot be assessed through direct design methods at this time. For consideration of failure by sliding, average the CPT data over a depth of 1.0 foot below the bottom of the foundation elevation to determine average I_c, S_u, and ϕ' soil properties for the sliding foundation soil. Undrained shear strength, S_u, is conservatively considered to be equal to the CPT probe sleeve resistance, f_s. Drained friction angle, ϕ', must be calculated, based on the estimated equivalent N_{60} and $N1_{60}$ value over the soil properties averaging depth. N_{60} is correlated according to Robertson (2012) by the following equation:

$$\frac{(q_t/p_a)}{N_{60}} = 10^{(1.1268\,-\,0.2817 I_c)}$$

where:

p_a = atmospheric pressure (same units as q_t)

$N1_{60}$ is calculated in accordance with AASHTO LRFD Bridge Design Specifications Article 10.4.6.2.4, where:

$N1_{60} = C_N N_{60}$, and $C_N = 0.77 \log_{10}(40/\sigma'_v)$, and $C_N < 2.0$

Calculate the effective vertical stress, σ'_v, using the correlated total unit weight (and effective unit weight) of the overburden soil above the soil properties averaging depth, in consideration of the estimated groundwater depth from the CPT sounding. Total unit weight, γ, is correlated according to Robertson (2010) by the following equation:

$$\gamma/\gamma_w = 0.27 \log_{10}R_f + 0.36 \log_{10}(q_t/p_a) + 1.236$$

where:

R_f = Friction Ratio = (f_s/q_t) x 100%
γ_w = Unit Weight of Water
p_a = atmospheric pressure (same units as q_t)

Use $N1_{60}$ to estimate the drained friction angle, ϕ', in accordance with Section 404.2. In place of Table 400-3, use the following Table 1300-1.

Table 1300-1: ϕ' Adjustment

I_c	Adjustment
$I_c < 1.00$	+2.5°
$1.00 \le I_c < 1.25$	+1.5°
$1.25 \le I_c < 1.50$	+0.5°
$1.50 \le I_c < 1.75$	-0.5°
$1.75 \le I_c < 2.00$	-1.5°
$2.00 \le I_c < 3.00$	-2.5°
$I_c \ge 3.00$	-3.5°

Complete the calculation of sliding resistance in accordance with Section 1303.3.5. If Ic ≤ 2.2, then consider drained sliding resistance using ϕ' as for a granular soil. If Ic > 2.2, then consider undrained sliding resistance using S_u and σ_v as for a cohesive soil.

1303.3 Strength Limit State Design

Analyze bearing, overturning (limiting eccentricity), sliding, and overall stability for spread footings in the Strength Limit State. For sliding and limiting eccentricity, use the “Strength Ia” Limit State, where vertical loads (DC, DW, ES, EV, LL, etc.) use minimum load factors, and horizontal loads (BR, EH, LS, TU, WS, etc.) use maximum load factors; exclude transient vertical loads from the analysis in accordance with AASHTO LRFD Figure C11.5.6-3. For bearing, use the “Strength Ib” Limit State, where all loads use maximum load factors. While overall stability is a Strength Limit State consideration, do not use factored loads for the analysis.

Note that “horizontal” earth pressure loads (EH and LS) are not applied horizontally, but inclined at an interface friction angle δ. These inclined loads are factored into horizontal and vertical components for mathematical convenience to make the stability computations easier to calculate. Do not use separate maximum and minimum load factors for the respective components of the loads. Treat each inclined load as a single load, which either serves to stabilize or destabilize the foundation, then apply the same load factor (either maximum or minimum) to both the horizontal and vertical components of the load. If it is uncertain whether a single, inclined load will either serve to stabilize or destabilize the structure, perform the calculations both ways.

If the foundation soils are too soft to satisfy the requirements of bearing and sliding, and over-excavation and replacement of the foundation soil under the footing with Item 203 Granular Material is performed, analyze bearing at the bottom of over-excavation, treating the granular material over-excavation and replacement as an extension of the footing. Analyze sliding both at the top and bottom of the granular material; at the top, the foundation soil will have the properties of the granular material in accordance with BDM Table 307-1; at the bottom, the foundation soil will have the in-situ soil properties, but sliding resistance will gain the benefit of passive resistance against the granular material over-excavation and replacement.

For each Strength Limit State failure mode, report the capacity-demand ratio (CDR) as the factored resistance divided by the factored load. CDR < 1.00 is unacceptable.

1303.3.1 Bearing Resistance of Soil

Calculate bearing resistance of foundation soils over the effective footing area, A′, for spread footings on soil in accordance with AASHTO LRFD Article 10.6.3.1.2a. The bearing capacity factors, Nc, Nq, and Nγ may be determined from AASHTO LRFD Table 10.6.3.1.2a-1, or may be calculated from the following equations:

$$N_c = \frac{N_q - 1}{\tan(\phi')} \text{ if } \phi' > 0;\ N_c = \pi + 2 \text{ if } \phi' = 0$$

$$N_q = e^{\pi \tan(\phi')} \tan^2\left(45° + {\phi'}/{2}\right) \text{ if } \phi' > 0;\ N_q = 1 \text{ if } \phi' = 0$$

$$N_\gamma = 2\left(N_q + 1\right)\tan(\phi') \text{ if } \phi' > 0;\ N_\gamma = 0 \text{ if } \phi' = 0$$

Note that the depth of footing, D_f, and depth of water, D_w, are both measured from the ground surface. D_w is not measured from the bottom of the footing (i.e., it is not a depth below the footing). For footings with a differential height of earth on two sides (including for retaining walls), measure D_f and D_w from the ground surface on the lower side.

Use the maximum likely seasonal water level for design. In locations where it makes sense, conservatively assume $D_w = 0$, unless there is a justification for assuming otherwise (for example, for a footing in a free-draining sandy slope, where the piezometric surface is sloping up from a free water surface below, but not as steeply as the ground surface). Utilize knowledge of hydrogeology and subsurface flow to connect the groundwater surface between known points. The groundwater surface should intersect with free water at the ground surface, and should slope downwards with a realistic potentiometric surface, generally following the lay of the land. Pay attention to groundwater levels in the borings (these will indicate the deepest possible location for the potentiometric surface), water contents, interfaces of free water at streams, ponds, and ditch lines, and the general drainage schema (water flows downhill to open bodies, connect ditch lines, water tends to rise up gradually in embankments, etc.).

For the calculation of shape factors, s_c, s_q, and s_γ, use the effective footing dimensions, B′ and L′, in place of B and L.

For the calculation of the depth correction factor, d_q, use the following equation, taken from "A Revised and Extended Formula for Bearing Capacity" (Hansen, 1970). This correction factor is a modification to the surcharge component for the additional length of the shear rupture surface necessary to reach the surface. AASHTO LRFD sets limits for the use of this equation for drained friction angle $32° \leq \phi' \leq 42°$, for depth of footing ratio $1 \leq D_f/B' \leq 8$, and for cases where "*the soils above the footing bearing elevation are as competent as the soils beneath the footing level*;" however, there are no such limits in the original Hansen derivation. These limitations are a leftover of an earlier method used in AASHTO LRFD, and do not apply to the use of the depth correction factor; therefore, use the equation regardless of the values of ϕ', D_f, B′, or surcharge soil strength.

$$d_q = 1 + 2 \tan \phi' (1 - \sin \phi')^2 \arctan (D_f / B')$$

For other depth correction factors, d_c and d_γ, AASHTO LRFD does not recognize depth correction factors other than d_q; therefore use $d_c = d_\gamma = 1.00$ for all ODOT projects.

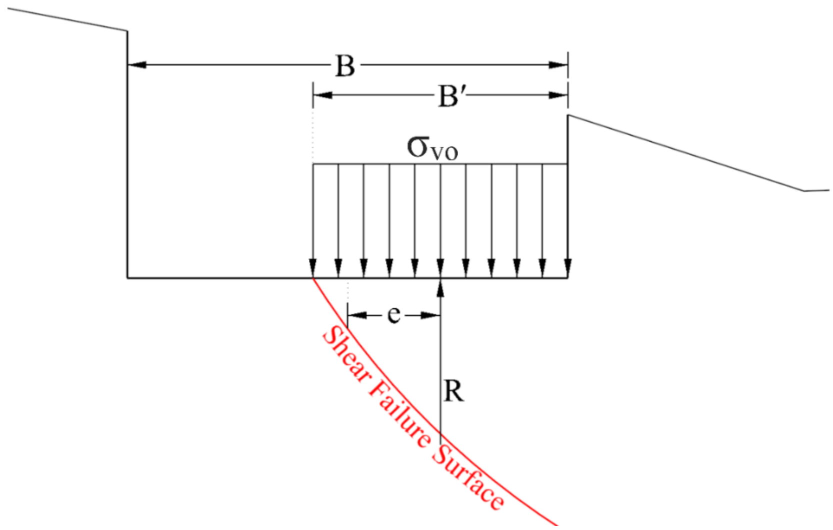
In general, do not calculate the load inclination factors, i_c, i_q, and i_γ, and for the cases of vertical or inclined eccentric loading on the bottom of the footing, assume the values of $i_c = i_q = i_\gamma = 1.00$. In an inclined eccentric loading, the inclination of the load is taken care of in the eccentricity and effective footing width calculations; therefore, load inclination factors less than one would be overly conservative. Only calculate the load inclination factors in the case of an inclined, centric loading (where the resultant load passes through the center of the footing), using AASHTO LRFD Equations 10.6.3.1.2a-5 through 10.6.3.1.2a-9.

1303.3.2 Limit Equilibrium Bearing Resistance

AASHTO LRFD Article 10.6.3.1.2c states, "Limit analysis, or limit equilibrium analysis, should be considered to estimate the nominal bearing resistance of footings on or adjacent to slopes composed of soils and/or site conditions that are not consistent with the parameters and conditions described in the reference documents (i.e., embedment >0, layered soils, steeper slopes)." However, there is no reason that bearing resistance calculation by limit equilibrium (LE) analysis need be limited to footings on or adjacent to slopes. This method particularly lends itself to layered soil systems, which are relatively difficult to analyze by other methods, such as those which are recommended in AASHTO LRFD Articles 10.6.3.1.2d through 10.6.3.1.2f.

The first step in performing limit equilibrium bearing resistance analysis is to calculate the Strength Limit State eccentricity and effective footing width by conventional means, in accordance with Section 1303.1.2. In the LE analysis model, place a void at the location of the footing, but limit the development of the shear failure surface to the dimensions of the effective footing width, B′; see Figure 1300-1. Do not limit the location of the "toe" of the shear failure surface.

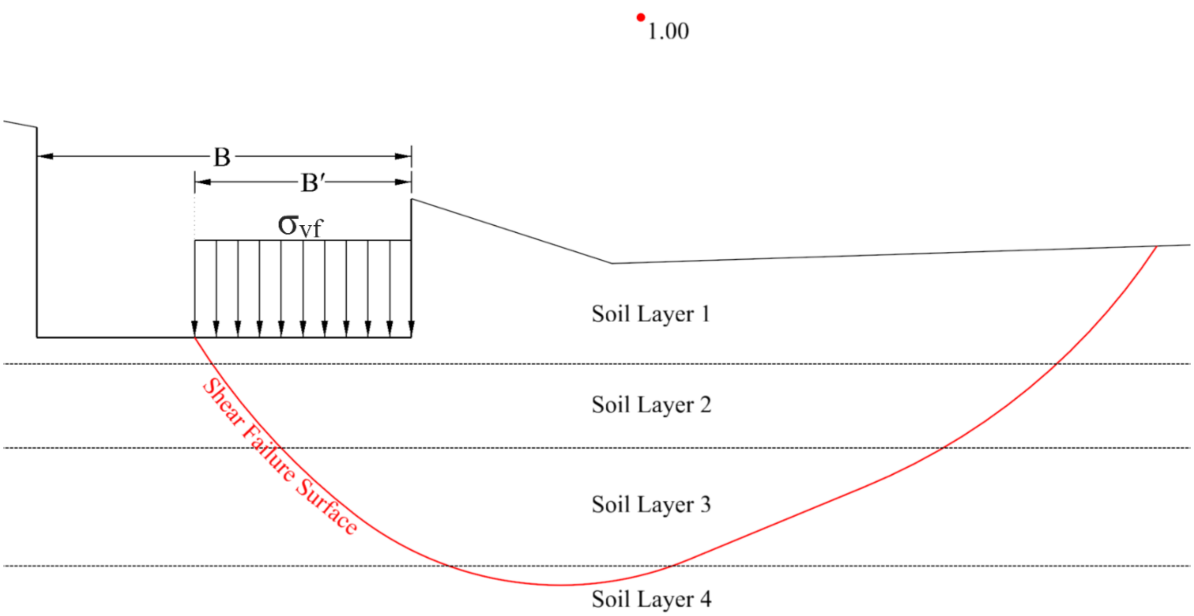
Figure 1300-1: Limit Equilibrium Bearing Resistance Setup



As a first trial, set the value of σ_{vo} in the limit equilibrium analysis model equal to the Strength Limit State factored uniform bearing stress distribution, σ_v, calculated in accordance with Section 1303.1.3. Search for the most critical shear failure surface exit location and shape. If the footing is properly sized, this should give a LE factor of safety greater than one.

Re-run the limit equilibrium analysis with different values of σ_v until a factor of safety of 1.00 is achieved at σ_{vf}; see Figure 1300-2. This final value will be the calculated nominal bearing resistance, $q_n = \sigma_{vf}$.

Figure 1300-2: Limit Equilibrium Bearing Resistance Final Value



Multiply this nominal bearing resistance by the appropriate resistance factor, φ_b, according to ASHTO LRFD Article 10.5.5.2.2 for bridge pier footings, or according to AASHTO LRFD Table 11.5.7-1 for retaining wall or abutment footings, for calculation of the factored bearing resistance, $q_R = \varphi_b\ q_n$, in accordance with AASHTO LRFD Equation 10.6.3.1.1-1.

Please note that this method will not work properly with a circular failure surface and will give an unconservative result. Failure surface optimization needs to be performed in the Limit Equilibrium analysis program (both GeoStudio Slope/W and Rocscience Slide2 have this capability). If using other Limit Equilibrium analysis software, check the result by performing a simple bearing resistance analysis in a deep, single layer system, and comparing the result to a Terzaghi/Vesic/Munfakh method analysis in accordance with AASHTO LRFD Article 10.6.3.1.2a; the results should be roughly comparable. If the calculated bearing resistance differs by 10% or more, the Limit Equilibrium analysis software cannot be used for this method.

In a layered system where bedrock is at a near depth below the footing, the shear failure surface in the soil may be limited by the bedrock surface, which is acceptable. However, do not use limit equilibrium analysis to determine the bearing resistance for foundations bearing on bedrock; bearing resistance of rock is controlled by crushing or by structural discontinuities in the rock.

1303.3.3 Bearing Resistance of Bedrock

When foundation bedrock meets ALL of the following three conditions:

- the bedrock surface under the footing is not steeply sloping such that discontinuities would control the bearing resistance (a bedrock slope of 2H:1V or less),
- the foundation bedrock has a Rock Mass Rating (RMR) ≤ 70, and
- the foundation bedrock is moderately strong or less in strength ($q_u \leq 7500$ psi),

then calculate drained shear strength properties (c′ and ϕ′) in accordance with Bieniawski (1989) and use the Terzaghi/Vesic/Munfakh method to calculate nominal bearing resistance of the bedrock in accordance with AASHTO LRFD Article 10.6.3.1.2a. The Bieniawski (1989) drained shear strength equations are as follows:

$c' = [0.104 \times RMR]$ (ksf);

$\phi' = [(RMR/2) + 5°]$ (degrees).

In determination of the RMR, keep in mind that the Spacing of Joints component of the RMR is not related to bedding, but is related to discontinuities. Furthermore, similarly to calculation of Rock Quality Designation (RQD), ignore mechanical breaks. For example, a 10-foot run of shale, with bedding of less than an inch could have only 3 or 4 natural, non-mechanical, discontinuities, such that the Spacing of Joints would fall in the 1-to-3-foot range, and not in the < 2-inch range.

Also, in determination of the RMR, if the natural discontinuities do not contain gouge or slickensides, do not rate them within the lowest two categories for Condition of Joints.

Lastly, in determination of the RMR, do not use the cases of "Water under moderate pressure" or "Severe water problems." The original intent of the RMR is for stability of the rock in roof-support for tunneling excavations, where moderate to severe water pressures could be encountered in the rock at some depth, and interstitial water pressure can make the rock significantly less stable. The Ground Water component of the RMR is not applicable to considerations of rock stability in shallow bearing considerations for spread footings (or even drilled shafts). In fact, it is recommended in the literature to ignore the Ground Water factor, and always assume "Completely Dry, 10" for surface excavations of the rock; however, we prefer to default to "Moist Only, 7" out of conservatism, unless the rock is indeed completely dry.

When foundation bedrock meets ANY of the following three conditions:

- the bedrock surface under the footing slopes steeper than 2H:1V,
- the foundation bedrock has a Rock Mass Rating (RMR) > 70, or
- the foundation bedrock is strong or greater in strength (q_u > 7500 psi),

then use the Carter and Kulhawy (1988) method, corrected in accordance with NCHRP Report 651, "LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures," Equation 82b, to calculate nominal bearing resistance of the bedrock in accordance with AASHTO LRFD Article 10.6.3.2.2, as follows:

$$q_n = \left(\sqrt{s} + \left(m\sqrt{s} + s\right)^{0.5}\right) q_u,$$

where:

q_n = nominal bearing resistance
q_u = uniaxial compressive strength of the intact rock
s = rock mass material constant defining intactness (quality) of the rock mass
m = rock mass material constant defining the shape of the Mohr's circle for uniaxial compression

Determine rock mass material constants *s* and *m* in the above equation in accordance with "The Hoek-Brown Failure Criterion - a 1988 Update" (Hoek and Brown, 1988) for undisturbed rock masses. The relevant equations are as follows:

$$\frac{m}{m_i} = exp\left(\frac{RMR-100}{28}\right),$$

$$s = exp\left(\frac{RMR-100}{9}\right).$$

where:

m_i = rock mass material constant *m* for intact rock (where *s* = 1.00).

Values of m_i for Ohio bedrocks are as follows:

- m_i = 15, Sandstone, breccia, and conglomerate
- m_i = 10, Claystone (mudstone), shale, clay-shale, and siltstone
- m_i = 7, Dolomite and limestone
- m_i = 1, Coal

Note that the Hoek-Brown GSI method (AASHTO LRFD Articles 10.4.6.4 and 10.8 in general, and Equation 10.8.3.5.4c-2) is not applicable to the problem of spread footings bearing on rock, though it uses similar-looking parameters, s, m_i, and m_b; it is only to be used for drilled shaft foundations. It may be that the Hoek-Brown GSI method and the Carter and Kulhawy RMR method could give comparable results, but this has not yet been verified by AASHTO, and AASHTO LRFD directs the use of the Carter and Kulhawy RMR method rather than the Hoek-Brown GSI method for the case of spread footings. Hoek et.al. (2013) Figure 4, and the following quote from the same, sheds some light on the use of GSI, "*A fundamental assumption of the Hoek-Brown criterion for the estimation of the mechanical properties of rock masses is that the deformation and the peak strength are controlled by sliding and rotation of intact blocks of rock defined by intersecting discontinuity systems. It is assumed that there are several discontinuity sets and that they are sufficiently closely spaced, relative to the size of the structure under consideration, that the rock mass can be considered homogeneous and isotropic*." Since the rock in Ohio is flat bedded, and it rarely includes multiple closely spaced joint sets (not such that could control the failure of the rock under vertical loading from a foundation), our rock is highly anisotropic, and GSI is not usually valid to represent the failure mechanisms that control structure foundations in rock in Ohio.

The bedrock surface in Ohio is generally flat, and except for on Appalachian hillsides or the sides of rock cut excavations, the rock would have nowhere to go in a bearing capacity failure. Therefore, the RMR method per Karter and Kulhawy (1988) reflected in AASHTO LRFD Article 10.6.3.2.2, and the Hoek-Brown GSI method reflected in AASHTO LRFD Articles 10.4 and 10.8, are extremely conservative and unrealistic in most instances in Ohio.

1303.3.4 Eccentric Load Limitations

Compare the eccentricity of load, e, calculated according to Section 1303.1.2, to the limiting eccentricity, e_{lim}, according to LRFD Article 10.6.3.3 for bridge pier footings, and according to LRFD Article 11.6.3.3 for retaining wall or abutment footings. Report the CDR as e_{lim}/e.

1303.3.5 Failure by Sliding

Calculate sliding resistance in accordance with BDM Section 305.2.1.3 and AASHTO LRFD Article 10.6.3.4 for bridge pier footings, and in accordance with BDM Section 307.1.3 and AASHTO LRFD Article 11.6.3.6 for retaining wall or abutment footings.

For drained sliding resistance, calculate the factored resistance in accordance with AASHTO LRFD Equation 10.6.3.4-2. For footings on bedrock, if there is no testing to verify the interface friction angle, assume the following values of ϕ_f:

- $\phi_f = 28°$, Claystone (mudstone) and clay-shale
- $\phi_f = 29°$, Shale
- $\phi_f = 30°$, Siltstone
- $\phi_f = 33°$, Sandstone, breccia, and conglomerate
- $\phi_f = 35°$, Dolomite and limestone

A sliding resistance check may be non-performed for footings keyed 3 inches or more into slightly strong or stronger bedrock ($q_u \geq 1500$ psi), as long as there is no evidence of features of the bedrock structure that may constitute planes of weakness such as laminations or interbedding. If such features exist, perform the sliding resistance check along the plane of weakness, using the above friction angles. If there is clay infilling, perform the sliding check in this material.

For a concrete-on-concrete sliding interface, assume $\phi_f = 35°$.

For bridge pier footings founded on weak or very weak bedrock ($q_u < 1500$ psi), use a sliding resistance factor of $\varphi_\tau = 0.90$; for bridge pier footings founded on slightly strong or stronger bedrock ($q_u \geq 1500$ psi), use a sliding resistance factor of $\varphi_\tau = 1.00$. Otherwise, use a sliding resistance factor for bridge pier footings in accordance with AASHTO LRFD Table 10.5.5.2.2-1. For retaining wall or abutment footings, use a sliding resistance factor of $\varphi_\tau = 1.00$ in accordance with AASHTO LRFD Table 11.5.7-1.

For undrained sliding resistance, calculate the factored resistance in accordance with AASHTO LRFD Figure 10.6.3.4-1. As this figure is not fully explained and poorly understood, we provide the following guidance for interpretation of the figure.

Do not use the effective footing width, B′, or uniform bearing stress for undrained sliding resistance analysis; for this calculation, use a linearly varying bearing stress over the full footing width, B, per the equations of AASHTO LRFD Article 11.6.3.2, and use a linearly varying distribution of unit shear resistance, q_s, equal to half the linearly varying bearing stress distribution in accordance with AASHTO LRFD Figure 10.6.3.4-1.

Limit the linearly varying unit shear resistance, q_s, by the value of undrained shear strength, S_u, for the soil immediately below the sliding interface. This will generally result in a pentagonal, trapezoidal, or triangular distribution of unit shear resistance, depending on if $\frac{1}{2}\sigma'_{vmax}$ is greater than or less than S_u. There are five possible cases, “Case 1” through “Case 5,” as outlined in Figure 1300-3 and detailed in Figures 1300-4 through 1300-8. For all five undrained sliding resistance cases, the unit shear resistance, $q_s = R_\tau/B$.

For passive resistance, do not calculate the passive earth pressure coefficient (k_p) through the Coulomb equation, as this is considered unconservative; use the Caquot and Kerisel method as defined in AASHTO LRFD Figures 3.11.5.4-1 and 3.11.5.4-2. Passive earth pressure (ep) is represented as a triangular distributed load with a value of zero at the ground surface and a maximum at the bottom of the footing or shear key; however, ignore passive resistance down to the frost depth or probable depth of disturbance, in accordance with BDM Section 305.2.1.2.a(E) and AASHTO LRFD Articles 10.6.3.4 and 11.6.3.5. This means that the remaining passive resistance below this depth will have a trapezoidal distribution. If utilities or pavement will exist within 6 feet on the passive resistance side of the foundation, assume a minimum depth of soil disturbance of 3 feet. Passive earth pressure will act against the front of the footing or shear key at the angle δ ***upwards*** from the normal.

For calculation of lateral earth pressure coefficients, always use the effective (drained) friction angle (ϕ') in the Coulomb, Rankine, or Caquot and Kerisel equations or charts. Do not use an undrained case with $\phi = 0$ for the calculation of lateral earth pressure coefficients.

Figure 1300-3: Undrained Sliding Resistance Cases

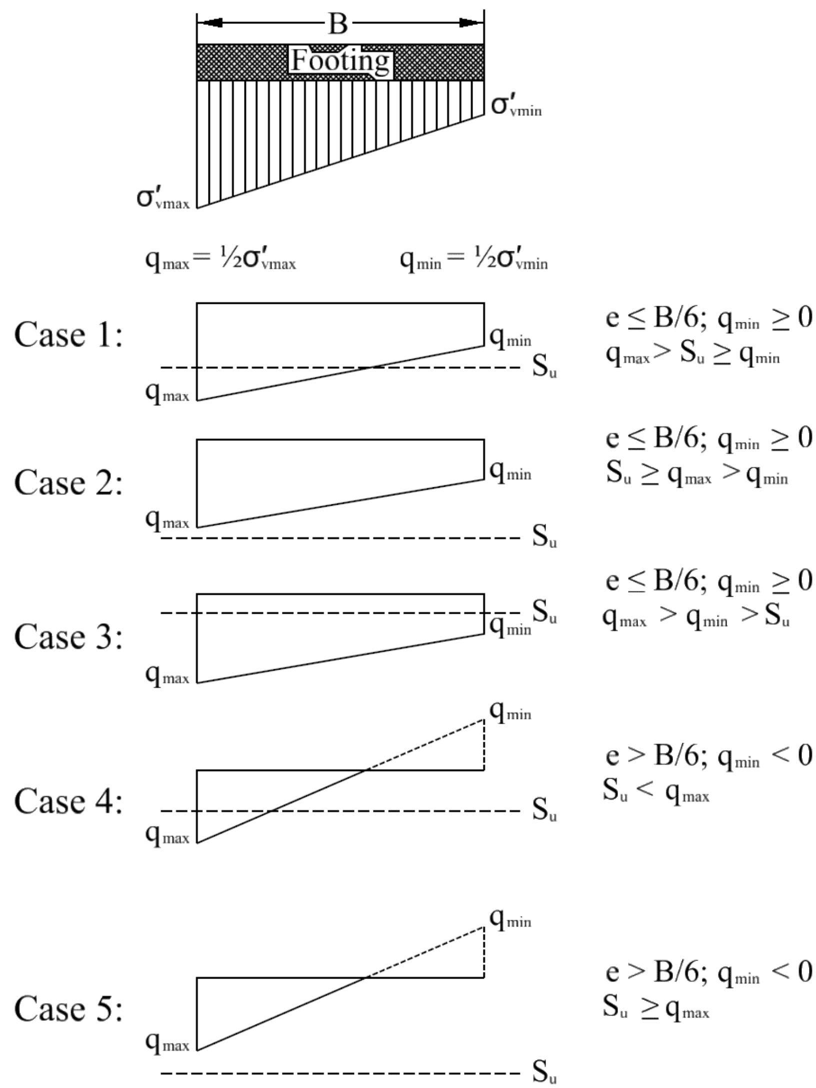
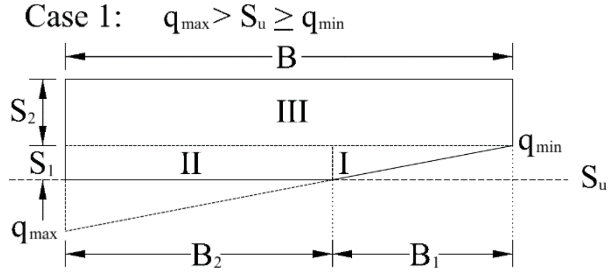


Figure 1300-4: Undrained Sliding Resistance Case 1

Case 1: $q_{max} > S_u \geq q_{min}$



$e \leq B/6$ $\quad q_{min} \geq 0$

$S_u = S_1 + S_2 \quad S_1 = S_u - q_{min} \quad S_2 = q_{min}$

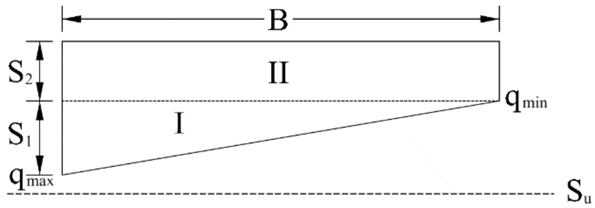
$$B_1 = \frac{B(S_u - q_{min})}{q_{max} - q_{min}} \qquad B_2 = \frac{B(q_{max} - S_u)}{q_{max} - q_{min}}$$

$I = ½\, S_1 B_1 \quad II = S_1 B_2 \quad III = S_2 B$

Sliding Resistance, $R_\tau = I + II + III$

Figure 1300-5: Undrained Sliding Resistance Case 2

Case 2: $S_u \geq q_{max} > q_{min}$



$e \leq B/6 \qquad q_{min} \geq 0$

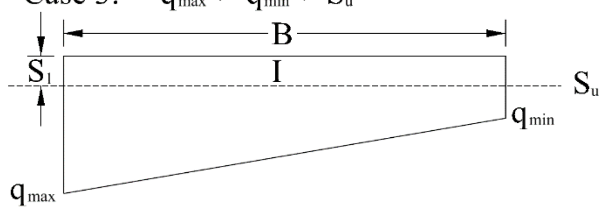
$S_u \geq S_1 + S_2 \qquad S_1 = q_{max} - q_{min} \qquad S_2 = q_{min}$

$I = ½\, S_1\, B \qquad II = S_2\, B$

Sliding Resistance, $R_\tau = I + II$

Figure 1300-6: Undrained Sliding Resistance Case 3

Case 3: $q_{max} > q_{min} > S_u$



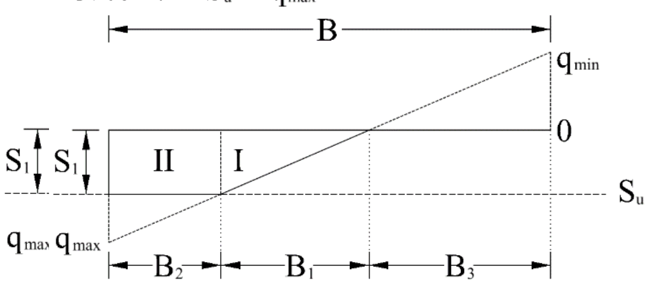
$e \leq B/6 \qquad q_{min} \geq 0$

$S_1 = S_u \qquad I = S_1\, B$

Sliding Resistance, $R_\tau = I$

Figure 1300-7: Undrained Sliding Resistance Case 4

Ca Case 4: $S_u < q_{max}$



$e > B/6 \qquad q_{min} < 0$

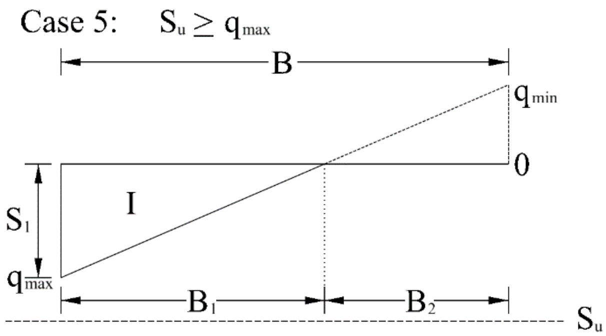
$S_1 = S_u \qquad B_3 = \frac{B(-q_{min})}{q_{max} - q_{min}}$

$B_1 = \frac{S_u(B - B_3)}{q_{max}} \qquad B_2 = B - (B_1 + B_3)$

$I = ½\, S_1\, B_1 \qquad II = S_1\, B_2$

Sliding Resistance, $R_\tau = I + II$

Figure 1300-8: Undrained Sliding Resistance
Case 5



$e > B/6 \quad q_{min} < 0$

$S_u \geq S_1 \quad S_1 = q_{max}$

$B_1 = \frac{B(q_{max})}{q_{max} - q_{min}} \quad B_2 = B - B_1$

$I = ½ S_1 B_1$

Sliding Resistance, $R_\tau = I$

1303.3.6 Overall Stability

Analyze overall (global) stability through limit equilibrium (LE) methods, in accordance with BDM Section 305.1.1 and AASHTO LRFD Article 10.6.3.5 for bridge pier footings, and in accordance with BDM Section 307.1.2 and AASHTO LRFD Article 11.6.3.7 for retaining wall or abutment footings. While overall stability is a Strength Limit State consideration, do not use factored loads for the analysis; factored loads are incompatible with LE analysis.

1303.4 Extreme Event Limit State Design

In Ohio, due to low seismicity, the Extreme Event I load combination is typically not investigated for foundations. However, there are several instances in which the Extreme Event II load combination is investigated, including vehicle or vessel collision (CT or CV), and most commonly, scour due to the scour check flood. In the Extreme Event Limit State, perform all the same checks as done in the Strength Limit State, with the exception of different load and resistance factors, in accordance with AASHTO LRFD Table 3.4.1-1 and Article 10.5.5.3.

1304 DRIVEN PILES

Design driven piles in accordance with the BDM Section 305.3 and AASHTO LRFD Article 10.7 and install in accordance with C&MS 507 and 523; provide appropriate installation plan notes in accordance with these design and construction references. Provide analysis, recommendations, and calculations supporting the pile design in the Structure Foundation Exploration (SFE) report in accordance with BDM Section 201.2.1.3 and the SGE Section 700. See BDM Section 305.3.5 for design considerations and limitations on the use of driven piles; pay particular attention to embedment into the pile cap in Section 305.3.5.1, Estimated Length and Order Length in Section 305.3.5.2, minimum pile penetration requirements in Section 305.3.5.7, limitations on the use of battered piles in Section 305.3.5.8, and pile setup in Section 305.3.5.9. Check lateral resistance of pile foundations in accordance with BDM Section 305.1.2.

The length of embedment into the pile cap is not accounted for in pile bearing resistance, but it is added in to determine the pile Estimated Length. Calculation of pile Estimated Length from the pile bearing resistance analysis is described in Section 1304.1.3, Analysis Implementation and Interpretation.

Typically, do not provide battered piles if the top of rock is within 15 feet below of the bottom of pile cap elevation (or the ground surface for piles without a buried cap). If these piles are proposed to bear on bedrock, they may skip along the bedrock surface during installation due to lack of overburden confining pressure. If battered piles must be used when the depth to the top of bedrock is less than 15 feet, pre-bore the piles and install them a minimum of 5 feet into bedrock; do not drive the piles. Provide plan notes to this effect.

When using friction piles, consider closed-end cast-in-place reinforced concrete pipe piles as the first option. Generally, H-piles should not be specified as friction piles. Experience demonstrates that H-piles tend to drive longer than pipe piles, generally from 10 to 20% longer, but sometimes more. In a full granular profile, H-piles may drive twice as long. However, if driving in very stiff soils where a minimum length must be achieved (e.g. for scour or uplift resistance), since H-piles do tend to drive longer, this may be a desirable trait.

H-piles should also generally not be specified as friction piles for driving in boulders and cobbles; the pile tips are often deformed and damaged by the cobbles and boulders. H-piles also tend to get cobbles and boulders jammed in the web, warping the flanges, and causing the pile to “corkscrew” or bend, or in some extreme cases, to form a U-shape turning back towards the surface. A thick-walled pipe pile with a conical steel driving point can shed cobbles and boulders and will generally drive through them with little effect. Typically, specify CIP pipe piles with conical points as friction piles for driving in boulders and cobbles.

For piles to be installed in hard driving conditions, consider including steel pile points or shoes in accordance with BDM Section 305.3.5.6. Generally, H-piles with pile points should not be specified as friction piles. With points, H-piles typically run deeper than estimated, because the pile points are slightly larger than the pile and pile side resistance is reduced.

When piles are to be driven to refusal on bedrock, consider steel H-piles as the first option. An H-pile generally has greater steel cross-sectional area than a comparable sized pipe pile, unless the pipe pile has a very thick wall. Therefore, the H-pile usually has greater survivability driving directly onto or into the top of bedrock when a refusal blow count of 20 blows per inch must be achieved. However, cast-in-place reinforced concrete pipe piles (including large-diameter open-ended pipe piles) may be considered for point bearing piles on bedrock, provided that all design and construction considerations are adequately addressed.

1304.1 Pile Bearing Resistance Analysis

For driven friction pile design, perform static analyses using the “FHWA Method,” in accordance with publication FHWA-SA-98-074, and AASHTO LRFD Articles 10.7.3.8.6b, 10.7.3.8.6e, and 10.7.3.8.6f, (the α-Method and the Nordlund/Thurman Method for cohesive and granular soils, respectively). Computerized solutions such as the FHWA DRIVEN program or Ensoft APILE, spreadsheet solutions, or hand calculations are all acceptable. In the case of spreadsheet solutions, provide the calculation file in native format to allow checking of the formulas; in the case of hand calculations, show and annotate all work written in a clear hand or with typed text. Note that the program GRLWEAP 14 by PDI now includes a module to perform pile bearing resistance analysis by the FHWA Method; this is an acceptable solution as long as all of the input and output data is provided for review.

For point bearing piles on rock, the bearing resistance is not limited by geotechnical resistance, but by the structural resistance of the pile; the bedrock, upon which the pile is driven to refusal, is assumed to have a greater bearing resistance than the pile can transmit to the structure. Factored axial resistance is calculated in accordance with BDM Section 305.3.3 and AASHTO LRFD Article 6.9.4.1. For piles driven to refusal on bedrock, typically use a structural resistance factor for piles subject to damage due to severe driving conditions in accordance with BDM Section 305.3.3 and AASHTO LRFD Article 6.5.4.2 (ϕ_c = 0.50 for H-piles and ϕ_c = 0.60 for pipe piles).

1304.1.1 Model Setup and Boundary Conditions

For pier piles that will have a pile cap cast in an excavation below the ground surface, start the analysis from the existing ground surface, with a zero-friction length from the ground surface elevation to the bottom of pile cap elevation. This will account for the overburden pressure of the soils surrounding the pile cap excavation, while discounting any pile bearing resistance from the soils removed in the pile cap excavation.

For conventional stub-abutment piles with a spill-through slope, start the analysis from the bottom of pile cap elevation. On the superstructure side, there will be no soil overburden, and at the time of construction, there will be no overburden fill soil placed on the approach side either. Therefore, the foundation soils will not see overburden pressure effects from soils above the bottom of pile cap elevation at the time of pile installation.

For capped-pile pier piles, start the analysis from the existing ground surface elevation (this could be the elevation of the bottom of a stream), with no zero-friction length and no soil overburden.

For piles to be driven through a proposed new embankment, BDM Section 305.3.5.7 requires that the length through the new embankment be prebored, so as to not impede drivability through the embankment into underlying foundation soils, and in order to reduce the potential effects of downdrag. The prebore length will have reduced side friction which cannot be relied upon for bearing resistance; for a bearing resistance analysis, conservatively count this as a zero-friction length, with the embankment soils providing an overburden pressure to the foundation soils. However, this is unconservative for drivability and downdrag analyses; for this case run a second analysis, counting the prebore length as a very loose granular soil with a friction angle of 28°.

For piles driven through other prebored holes drilled to avoid impacts to existing utilities or other structures, also perform two analyses as above, one with a zero-friction length within the pre-bore length, and one treating the pre-bore length as a very loose granular soil with a friction angle of 28°.

For piles to be installed within an MSE abutment, three analyses must be performed:

1) First, perform an analysis starting at the elevation of the bottom of the foundation preparation for the MSE wall, with no zero-friction length and no soil overburden above this elevation. This represents the case of piles fully driven to the required bearing resistance prior to construction of the MSE wall. This will be the most conservative case for bearing resistance.

2) Next, perform an analysis with a zero-friction length and overburden equal to the height from the foundation preparation elevation to the bottom of pile cap elevation. This represents the

case of piles driven to the required bearing resistance through the MSE wall completed to the pile cap elevation, with pile sleeves through the MSE wall fill. The MSE wall will provide an overburden pressure equal to approximately its height but will provide no friction due to the pile sleeves having no granular backfill. Use this analysis for drivability analyses.

3) Also perform an analysis starting at the bottom of pile cap elevation, assuming a very loose granular soil with a friction angle of 28° down to the foundation preparation elevation. This represents the case of piles driven to the required bearing resistance through the MSE wall completed to the pile cap elevation, with conventional granular-filled pipe pile sleeves through the MSE wall fill. While the pile sleeve fill cannot be counted on for bearing resistance to support the bridge load, it will provide some side friction, and it must be accounted for in the downdrag analyses.

In the case of piles subject to scour, run an additional ultimate resistance analysis, starting from the predicted channel scour elevation, with no zero-friction length and no soil overburden above this elevation. In this case, all surrounding soils are assumed to have been removed from the scour flood event, eliminating any soil overburden above the channel scour elevation. For piles with a local scour effect that is deeper than the surrounding channel scour (pier or abutment scour), start the analysis from the predicted channel scour elevation with a zero-friction length down to the local scour elevation. In this case, the soils surrounding the local scour hole will still provide overburden pressure to the foundation soils. This scour condition analysis will be used for the design bearing resistance of the piles; however, the installation of the piles will be performed in the existing, pre-scour condition. Driving resistance and drivability analyses will be performed in the existing condition; the difference in resistance necessary to achieve the pile tip elevation from the scour analysis must be added to the resistance necessary to support the bridge, as an additional driving resistance to be overcome at the time of installation (R_{Ssc}). The BDM provides plan notes for this case.

Keep in mind that the formulas provided in BDM Sections 305.3.2 and 305.3.2.4, while theoretically correct, are not directly applicable for practical design solutions. Calculate R_{Ssc} using the following approach:

1) In the ultimate resistance scour condition analysis (starting the soil model at the scour elevation), calculate the pile tip elevation using the total factored axial pile load (Q_p) divided by φ_{dyn} (typically, $\varphi_{dyn} = 0.7$).

2) In the ultimate resistance analysis without scour, use the same pile tip elevation to estimate the nominal resistance (by linear interpolation if necessary); this value is the Ultimate Bearing Value (UBV) to be included in the project plans.

3) Subtract the value of (Q_p/φ_{dyn}) from the UBV to determine R_{Ssc}.

For example, for a resistance from step 1 of ($Q_p/\varphi_{dyn} = 213.5/0.7 = 305$ kips), the pile tip elevation is found to be 986.3 feet. From step 2, the nominal resistance at this elevation is 373 kips (UBV). From step 3, R_{Ssc} is (373 – 305 = 68 kips). These values from step 2 and step 3 (373 kips and 68 kips) would be the values entered into BDM Plan Note 606.2-2.

For bridge piles installed at a water crossing, place the water level in the analyses at the same as the normal water elevation of the body of water or higher. For pier piles that will have a pile cap cast in an excavation below the ground surface, place the water level in the analyses at the bottom of the pile cap elevation or higher (if encountered higher). For piles to be installed in flat existing ground, place the water level in the analyses as indicated by the exploratory soil borings, but no deeper than 3 feet below the existing ground elevation. For piles to be driven through an existing or a proposed new embankment, place the water level in the analyses as indicated by the exploratory soil borings, but no deeper than half the height of the embankment below the pile cap elevation. For piles installed within an MSE abutment, place the water level in the analyses at the invert of the drainage pipe in accordance with SS840.04.A.6.

Typically, bedrock is not modeled in a pile bearing resistance analysis; however, for a pile driven to refusal on bedrock, realistic values should be provided to model the rock so that a wave equation pile drivability model can be developed to determine if the pile can be driven to refusal on bedrock without overstressing the pile (see Section 1304.2.1). In pile bearing resistance analyses, model rock as a cohesive soil with undrained shear strength (S_u) equal to one-half the unconfined compressive strength (Q_u).

1304.1.2 Cohesive Soil Analysis Models

When performing static analysis methods for friction piles, do not use the Tomlinson 1979 model in the analyses. The Tomlinson 1979 model was superseded by the Tomlinson 1980 models; furthermore, AASHTO LRFD recognizes only the 1980 models. By default, for cohesive soil layers, use the Tomlinson 1980 "same" or "firm to stiff clay" model, for the condition assuming the overlying layer to be similar in character to the layer being analyzed; this model will always apply to a cohesive soil layer located at the ground surface or bottom of pile cap elevation.

Only use the Tomlinson 1980 overlying sand model if the overlying layer has (Combined Gravel and Coarse Sand Sizes ≥ 35%) AND (P200 ≤ 30%) AND (PI ≤ 8%). If a granular layer does not meet the above qualifications, or if it is intermittent, consider modeling the layer as cohesive. The Tomlinson 1980 overlying sand model represents the frictional resistance of a cohesive layer as increased by a significant amount by granular material being freed from an overlying sand or sandy gravel layer. Do not use this model for cohesive soil layers below granular layers that consist primarily of fine sands or silts, or which contain a significant portion of fines or a high plasticity, such that a significant amount of coarse sand or gravel cannot be freed from the matrix of the soil. In these cases, the predicted effect is considered unconservative. The effect of the Tomlinson 1980 overlying sand model diminishes with depth to a maximum depth of 40 times the pile diameter below the granular layer producing the effect. If using the FHWA DRIVEN program, there is a known bug in the application of the Tomlinson 1980 overlying sand model, wherein the program will start over with each new layer to which the model is applied, assuming the depth below the granular layer to be zero at the top of each layer. This will produce an unconservative result; therefore, never apply this model to more than one consecutive layer in the FHWA DRIVEN program. Furthermore, recent research (Hird, Emmett, and Davies, 2006) has shown that the sand drawdown effect is generally less pronounced than previously thought, and it is based on the consistency of the cohesive material. According to this research, for a "weaker clay" this is about three 'pile' diameters regularly, and more than this irregularly. Therefore, for a relatively thick cohesive layer below a granular layer, divide the layer in two, with a short section at the top using the Tomlinson 1980 overlying sand model, and the remainder of the layer using the Tomlinson 1980 "same" model. Base the thickness of the upper layer on the consistency of the cohesive material as follows: Very Soft: 5 feet, Soft: 4 feet, Medium Stiff: 3 feet, Stiff: 2 feet, Very Stiff or greater: do not use the Tomlinson 1980 overlying sand model.

Only use the Tomlinson 1980 overlying soft clay model if the overlying layer has an undrained shear strength less than or equal to 750 psf, and an undrained shear strength less than or equal to ¼ of the undrained shear strength of the underlying layer. The Tomlinson 1980 overlying soft clay model represents the frictional resistance of a cohesive layer as reduced by softer overlying clay being drawn down during driving to lubricate the pile. The overlying clay needs to be soft enough to draw down, and significantly weaker than the underlying layer to have an effect. The effect of the Tomlinson 1980 overlying soft clay model diminishes with depth to a maximum depth of 20 times the pile diameter below the soft clay layer producing the effect. If using the FHWA DRIVEN program, there is a known bug in the application of the Tomlinson 1980 overlying soft clay model, wherein the program will start over with each new layer to which the model is applied, assuming the depth below the soft clay layer to be zero at the top of each layer. This will produce an overly conservative result; therefore, never apply this model to more than one consecutive layer in the FHWA DRIVEN program. Furthermore, also based on the Hird, Emmett, and Davies (2006) research, for thick underlying cohesive layers, divide the layer in two, with a short section at the top using the Tomlinson 1980 overlying soft clay model, and the remainder of the layer using the Tomlinson 1980 "same" model. Base the thickness of the upper layer on the consistency of the cohesive material as follows: Very Soft: 5 feet, Soft: 4 feet, Medium Stiff: 3 feet, Stiff: 2 feet, Very Stiff or greater: do not use the Tomlinson 1980 overlying soft clay model.

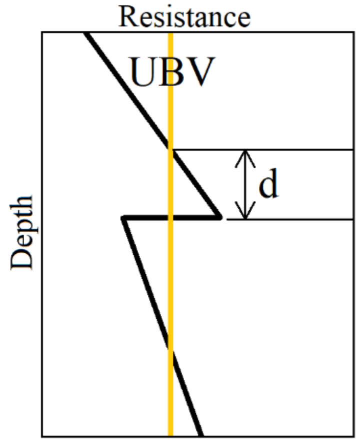
1304.1.3 Analysis Implementation and Interpretation

Perform both a "driving resistance" analysis, taking into account the driving losses to side frictional resistance experienced by the pile during driving and prior to setup, and an "ultimate resistance" analysis for the long-term condition after pile setup has occurred. The reduction factor (used in Ensoft APILE) is the inverse of the setup factor (f_{su}), while the driving loss (used in the FHWA DRIVEN program) is one minus the inverse of f_{su}. Use BDM Table 305-2, "Pile Driving Recommended Setup Factor (f_{su}) for Side Friction," to determine the setup factor for each soil layer based on Soil Class.

If analysis of a stronger upper soil layer indicates adequate bearing resistance to support the UBV, while analysis of a weaker lower layer indicates inadequate bearing resistance, a "nose" in the bearing resistance graph will form, looking like Figure 1300-9.

If the distance "d" is equal to or less than five feet or 2 times the pile diameter (whichever is greater), then punching shear through the bottom of the stronger layer is likely. Do not count the resistance of the "nose." Pass the pile through the bottom of the stronger upper layer to bear in the lower weaker layer or deeper.

Figure 1300-9: Stronger Layer over a Weaker Layer Forming a "Nose" in the Bearing Resistance Graph



To determine the Estimated Length of the pile, linearly interpolate the depth of penetration between the tabulated values of calculated total bearing resistance (APILE Ultimate Capacity or DRIVEN Total Capacity) for the UBV of the pile. Add to this length any stickup (free-standing) length and embedment into the pile cap, and round this up to the nearest 5 feet. Per BDM Section 305.3.5.2, if rounding up to the nearest 5 feet for Estimated Length adds less than a foot, increase to the next 5-foot interval.

An example for determining the estimated length of the pile in this "nose" scenario is presented here. For a 16-inch diameter pipe pile with a UBV = 584 kips, 12.76 feet of free length, and 1.5 feet of embedment into the pier cap (a capped-pile pier), we have the following:

Depth (ft)	Total Resistance (kips)
72.5	542.32
74.99	588.64
75.01	546.47
85	578.49
90	668.74

We see that the UBV is exceeded at a depth of 74.99 feet; however, this is a "nose" with a distance "d" of only 0.249 feet (less than 2 times the pile diameter, which equals 2.667 feet). Therefore, this "nose" cannot be counted on for resistance, and the pile must pass through this layer to bear in a lower layer. Between a depth of 85 feet and 90 feet, we interpolate to find the depth at which 584 kips is reached, 85.31 feet. In order to calculate the pile Estimated Length, add soil embedment + free length + pile cap embedment = 85.31 + 12.76 + 1.5 feet = 99.57 feet. Since 100 – 99.57 = 0.43 feet is less than one foot, round up by increasing to the next 5-foot interval, 105 feet = Estimated Length.

For point bearing piles on bedrock, consider the depth of penetration as the elevation on the nearest soil boring where either bedrock coring begins or where SPT refusal blow count occurs with a recovered sample visually classified as bedrock. If in-between borings, locate this depth at the substructure unit by interpolation between borings located to either side of the unit. Round up to the nearest 5 feet in the same manner as for a friction pile.

1304.2 Drivability Analysis

Perform a wave equation pile drivability analysis using the PDI GRLWEAP program, or similar software, according to BDM Section 305.3.1.2 to determine whether the pile can be driven to the UBV or to refusal on bedrock without overstressing the pile utilizing commonly available pile driving hammers. While we know of the existence of several alternative programs, we are familiar with GRLWEAP, and know it to be by far the most common software used for this purpose. Therefore, this section will address the use of this software for the performance of wave equation pile drivability analysis; if using other software, apply the guidance provided here in principle to the analysis.

Start all analyses with the hammer type and size recommended in BDM C305.3.1.2, which corresponds roughly to a Delmag D 19-42 hammer. This is considered to be the largest commonly available hammer in Ohio. Other types and larger sized pile driving hammers may have higher mobilization costs, and should only be considered in design if a Delmag D 19-42 hammer is not capable of driving the pile to the required bearing depth or resistance without overstressing the pile according to AASHTO LRFD Article 10.7.8 using $\varphi_{da} = 1.00$. If analysis with a Delmag D 19-42 hammer results in overstress to the pile, either 1) increase the yield strength of the pile steel and provide a plan note specifying the new minimum yield strength; or 2) choose a thicker pile wall and provide a plan note specifying the minimum pile wall thickness; or 3) try a smaller hammer and provide a plan note specifying the pile driving hammer *maximum* rated energy. If analysis with a Delmag D 19-42 hammer indicates that the required bearing depth or resistance cannot be reached without driving refusal, try a larger hammer and provide a plan note specifying the pile driving hammer *minimum* rated energy. For pile driving prior to reaching the required bearing depth or resistance, blows per foot higher than 100 bpf can be considered practically to be refusal. Refusal on bedrock is defined in BDM Section 305.3.1.2 as when the pile penetrates into bedrock 1 inch or less after receiving at least 20 blows from the pile hammer; this is equivalent to or greater than 20 blows per inch (240 bpf).

Do not use the ST “Soil Type-Based,” SA “SPT N-Value,” or most other soil model generation input forms built into GRLWEAP to generate a soil model. Using GRLWEAP to automatically generate shaft and tip resistances is typically over-simplified and will result in unit resistance values that are different from the ones calculated in an FHWA-method pile bearing resistance analysis. However, use of the built-in FHWA method Soil Profile Input in GRLWEAP 14 is acceptable if this module was used for the pile bearing resistance analysis. Use the same unit side and tip resistance values utilized in the pile bearing resistance analysis for the “driving resistance” analysis. Enter and manually check and edit the full “S1” Soil Profile table. Note that both FHWA DRIVEN and Ensoft APILE can generate GRLWEAP input files from the program inputs for the pile bearing resistance analysis. We recommend taking advantage of this feature, and not generating an entire GRLWEAP input file manually. Manual generation of the file invites greater possibility of user input errors. However, please note that the GRLWEAP input files generated by the DRIVEN and APILE programs are incomplete; the engineer will have to fill in the remaining data, and in some cases correct data that is too generalized. See the following sections for particulars.

1304.2.1 GRLWEAP "S1" Soil Profile Table

Ensure that the GRLWEAP Soil Profile table has enough lines of information to replicate the model used for the pile bearing resistance analysis. Ensure that the depths on each line of the GRLWEAP Soil Profile table correspond to the depths used in the pile bearing resistance analysis model. Ensure that the GRLWEAP Soil Profile Unit Shaft Resistance (Rskn) and Unit Toe Resistance (Rtoe) for each line of data and each soil layer correspond to the unit side and tip resistances used in the pile bearing resistance analysis. Note that the software will linearly interpolate between the soil property values entered at consecutive depths; however, a value of zero (0) entered for a property at a specific depth will repeat the value for the preceding (overlying) depth.

"Quake" is a coefficient defining "soil spring" stiffness for elasto-plastic dynamic analysis along either the pile side (Skin Quake, Qs) or the pile tip (Toe Quake, Qt). For all soil types, set the shaft quake to a value of 0.10 inch. For displacement piles: for very dense or hard soils, set the toe quake equal to the diameter of the pile in inches divided by 120; for loose or soft soils, set the toe quake equal to the diameter of the pile in inches divided by 60. For soil strengths in-between these, use intermediate toe quake values, according to Table 1300-1. For non-displacement piles (driving unplugged), set the toe quake equal to 0.10 for all soils and soft rock. For all piles driving into stronger rocks, see Table 1300-2 or Table 1300-5 for the recommended value of Qt.

Table 1300-2: Displacement Pile Recommended Value of Toe Quake (Qt)

Cohesive Soils

SPT N_{60} (bpf)	**D/Qt (in/in)**
0 - 3	60
4 - 35	$52 \times N_{60}^{0.233}$
36+	120

Granular Soils

SPT $N1_{60}$ (bpf)	**D/Qt (in/in)**
0 - 6	60
7 - 54	$35 \times N1_{60}^{0.307}$
55+	120

Bedrock

Qu (psi)	**Qt (in)**
0 - 100	D/120
100 - 7000	$CSCH(Q_u^{0.16})/4 + 0.032$
7000+	0.04

"CSCH" is the hyperbolic cosecant function.

Displacement piles include closed-ended pipe piles, timber piles, solid concrete piles, and H-piles or open-ended pipe piles which are small enough to plug during driving. Analyze H-piles and open-ended pipe piles with diameters 20 inches or less as displacement piles. Non-displacement piles include sheet pile and H-piles or open-ended pipe piles which are large enough not to plug during driving. It is typically assumed that pipe piles with diameters of 30 inches or more do not plug during driving while H-piles and pipe piles of diameter 20 inches or less will plug during driving into a bearing layer. Between 20 and 30 inches, open-ended pipe piles may or may not plug; in this case analyze the pile both ways and use the more conservative result.

"Damping" is a coefficient defining viscous behavior of the soil for elasto-plastic dynamic analysis along either the pile side (Skin Damping, Js) or the pile tip (Toe Damping, Jt). Skin Damping varies, based on the soil type, from 0.05 for non-cohesive granular soils to 0.20 for cohesive clay soils. Use Table 1300-3 to assign skin damping values based on ODOT soil classification.

Table 1300-3: Skin Damping (Js) by ODOT Soil Classification

Soil Type	ODOT Soil Classes	Skin Damping (seconds/ft)
Non-cohesive granular	A-1-a, A-1-b, A-3, A-3a	0.05
Sandy silt or clayey sand	A-2-4, A-2-5, A-2-6, A-2-7, A-4a (NP), A-4b (NP)	0.10
Cohesive silt or a sandy clay	A-4a (plastic), A-4b (plastic), A-5, A-6a, A-8a	0.15
Cohesive clay	A-6b, A-7-5, A-7-6, A-8b	0.20

In Table 1300-3, (NP) denotes a non-plastic sandy silt (A-4a) or silt (A-4b) soil. See Section 402 for a discussion on whether to consider a low-plasticity soil as non-plastic (granular) or plastic (cohesive). For all soil types, set the toe damping to a value of 0.15 s/ft.

Use BDM Table 305-2, "Pile Driving Recommended Setup Factor (f_{su}) for Side Friction," to determine the Setup Factor (SU F) for each soil layer based on soil classification. These setup factors should be the inverse of the reduction factor (used in Ensoft APILE) or the inverse of one minus the driving loss (used in the FHWA DRIVEN program), used for the "driving resistance" analysis. A drivability analysis uses the driving resistance of the pile, therefore the proper setup factor must be chosen for each soil layer.

Set Limit Distance (Lim D) in the GRLWEAP analysis to 6.0 feet for all soil layers and soil types. This is the distance of continuous pile movement through a soil layer which causes the soil resistance to completely lose any setup effect. This is not a well-known soil property, and typically 6.0 feet is assumed.

Setup Time (SU T) varies, based on the soil type, from 1 hour for non-cohesive granular soils to 168 hours (7 days) for cohesive clay soils. Use Table 1300-4 to assign setup time values based on ODOT soil classification.

Table 1300-4: Set-up Time (SU T) by ODOT Soil Classification

ODOT Soil Classes	Setup Time (hours)
A-1-a, A-1-b, A-3a	1
A-2-4, A-2-5, A-2-6, A-2-7, A-3, A-4a (NP), A-4b	24
A-4a (plastic), A-5, A-6a, A-6b, A-7-5, A-7-6, A-8a, A-8b	168

Toe Area need not be entered in the GRLWEAP Soil Profile table if it is defined in the Pile Information area of the Main Input screen. However, if toe area is entered in the Soil Profile table, it will override the value entered on the Main Input screen. To simulate driving of open ended piles, for which plugging is anticipated at some depth, it is necessary to enter two different toe areas on the Soil Profile table, first, the steel end area prior to plugging, then the total pile cross-sectional area at the plugging depth. It is our practice to always fill in the toe area values on the Soil Profile table, to avoid any possible confusion.

If piles are to penetrate more than 15 feet of new embankment, they must be prebored in accordance with BDM Section 305.3.5.7. However, for lesser thicknesses of new embankment, or if there is some site constraint that necessitates driving of piles through a thicker proposed embankment fill, model the fill as a cohesive soil with an undrained shear strength of 4000 psf.

Typically, bedrock is not modeled in a wave equation pile drivability analyses; however, for a pile driven to refusal on bedrock, realistic values should be provided to model the rock, to determine if the pile can be driven to refusal on bedrock without overstressing the pile, in accordance with BDM Section 305.3.3. When modeling bedrock for drivability analyses, input values in the GRLWEAP Soil Profile according to the following table, based on the Geologic Relative Strength Descriptor of the rock. The column containing unconfined compressive strength (Qu) is for reference, but provides a typical value corresponding to the Geologic Relative Strength Descriptor. The toe area for closed-end pipe piles is the total area of the pile tip, for other types of piles (e.g., H-piles or open-end pipe piles) the toe area in rock is always the cross-sectional area of the pile.

Table 1300-5: Bedrock Drivability Properties by Geologic Relative Strength Descriptor

Geologic Relative Strength Descriptor	Qu (psi)	Rskn (ksf)	Rtoe (ksf)	Qs (in)	Qt (in)	Js (s/ft)	Jt (s/ft)	SU F (ratio)	Lim D (ft)	SU T (hr)
Very Weak	200	7.00	70	0.10	0.08	0.20	0.15	1.43	1.00	24.0
Very Weak to Weak	360	12.5	125	0.10	0.07	0.20	0.15	1.25	1.00	24.0
Weak	750	25.0	250	0.10	0.06	0.20	0.15	1.11	1.00	24.0
Weak to Slightly Strong	1125	40.0	400	0.10	0.055	0.20	0.15	1.00	0.00	0.0
Slightly Strong	1500	50.0	500	0.10	0.052	0.20	0.15	1.00	0.00	0.0
Slightly Strong to Moderately Strong	2250	80.0	800	0.10	0.048	0.20	0.15	1.00	0.00	0.0
Moderately Strong	3600	125	1,250	0.10	0.044	0.20	0.15	1.00	0.00	0.0
Moderately Strong to Strong	5000	180	1,800	0.10	0.042	0.20	0.15	1.00	0.00	0.0
Strong	7500	250	2,500	0.10	0.04	0.20	0.15	1.00	0.00	0.0
Strong to Very Strong	10,000	360	3,600	0.10	0.04	0.20	0.15	1.00	0.00	0.0
Very Strong	15,000	500	5,000	0.10	0.04	0.20	0.15	1.00	0.00	0.0
Very Strong to Extremely Strong	20,000	700	7,000	0.10	0.04	0.20	0.15	1.00	0.00	0.0
Extremely Strong	30,000	1000	10,000	0.10	0.04	0.20	0.15	1.00	0.00	0.0

1304.2.2 GRLWEAP Main Input Screen

For drivability analyses, there are five areas of the Main Input screen that have required data entries: Hammer Information, Pile Material, Cushion Information, Pile Information, and Resistance Gain/Loss Factors.

In the Hammer Information section, first select “Delmag D 19-42” from the Hammer Information /Hammer Maintenance table. Analyses with different hammer sizes is not necessary in design unless there are drivability problems, or if the designer knows that a different specific hammer system will be used. See Section 1304.2 for a discussion on selection of hammer type and size. Once a hammer is chosen, all the fields in the Hammer Parameters section will be filled in with default values; use default values in design unless specific details of the driving system are known.

In the Pile Material section, click on the radio button that corresponds to the type of pile being driven. Typically, this will be steel (select steel for H-piles, closed-end pipe piles, and open-end pipe piles). This should fill in default values for elastic modulus and specific weight in the Pile Information section.

In the Cushion Information section, click in the first open field, “Area,” and then push the F3 button for Help. Select “Manufacturer’s recommended driving system,” and then click the “OK” button. In the following “Drive System Help” dialogue, click the “Check All” button, and then the “OK” button. All the fields in the Cushion Information section will be filled in with default values; use default values in design unless specific details of the driving system are known.

In the Pile Information section, click in the third open field, “Section Area,” and then push the F3 button for Help. In the “Area Calculator” dialogue, choose the pile type (typically, Pipe or H-Pile) and size. For pipe piles, enter the Wall Thickness in inches, and then click in any other field; the remaining dimensions will all recalculate. When done, click the “OK” button. For closed-end pipe piles, choose “Total Area” on the following “Toe Area Determination of Pipe Pile” dialogue, and then click the “OK” button. For pile length, ensure that the length is equal to the Order Length of the pile, in accordance with BDM Section 305.3.5.2, including any stickup (free-standing) length, embedment into the pile cap, rounding up to the nearest 5 feet, and the additional 5 feet. For pile penetration, enter the estimated ground penetration to reach the required UBV or refusal on bedrock, including any additional length to penetrate soil that is predicted by a scour analysis to be subsequently scoured away. For elastic modulus (Elast Modulus) for steel piles, use 30,000 ksi (29,000 ksi is generally accepted for static analyses, but steel gives a slightly stiffer response for dynamic analyses). For specific weight (Spec Weight) for steel piles, use 492 pcf. For the Toe Area for closed-end pipe piles, use the total area of the pile tip; for H-piles, use the “box perimeter” area of the pile (section depth times flange width); for open-end pipe piles driving unplugged, use the steel cross-sectional area of the pile. If an open-end pipe pile is expected to plug at a certain depth, enter the total area of the pile for toe area at the appropriate depth in the Soil Profile table. For perimeter, enter the circumference for pipe piles, and enter the “box perimeter” for H-piles. For pile size, enter the outer diameter for pipe piles, and enter the flange width for H-piles.

For pipe piles, use a pile wall thickness that is available for the diameter of pile to be analyzed. ASTM A252 pile piles from 10 inches to 24 inches in outside diameter are available in wall thicknesses ranging from 0.250 inches to 1.000 inches, generally in increments of 1/8”, 1/16”, or

1/32”, with some exceptions. Based on a survey of pipes available from several manufacturers, the sizes in Table 1300-6 are available. Larger pipe sizes and some greater thicknesses are also available; query steel pipe manufacturers for more information.

Table 1300-6: Commonly Available ASTM A252 Pipe Pile Sizes and Wall Thicknesses

Size	12"	14"	16"	18"	20"	22"	24"
1/4"	0.250	0.250	0.250	0.250	0.250	0.250	0.250
9/32"	0.280	0.280	0.280	-	-	-	-
23/75"	-	-	-	-	-	-	-
5/16"	0.312	0.312	0.312	0.312			
33/100"	0.330	-	-	-	-	-	-
11/32"	0.344	0.344	0.344	-	-	-	-
27/74"	-	-	-	-	-	-	-
3/8"	0.375	0.375	0.375	0.375	0.375	0.375	0.375
13/32"	0.406	0.406	0.406	0.406	0.406	0.406	0.406
7/16"	0.438	0.438	0.438	0.438	0.438	0.438	0.438
15/32"	-	0.469	0.469	0.469	0.469	0.469	0.469
1/2"	0.500	0.500	0.500	0.500	0.500	0.500	0.500
9/16"	0.562	0.562	0.562	0.562	0.562	0.562	0.562
19/32"	-	0.594	-	-	0.594	-	-
5/8"	0.625	0.625	0.625	0.625	0.625	0.625	0.625
21/32"	-	-	0.656	-	-	-	-
11/16"	0.688	0.688	0.688	0.688	0.688	0.688	0.688
23/32"	-	-	-	-	-	-	-
3/4"	0.750	0.750	0.750	0.750	0.750	0.750	0.750
13/16"	-	0.812	0.812	0.812	0.812	0.812	0.812
27/32"	0.844	-	0.844	-	-	-	-
7/8"	-	-	-	-	-	0.875	0.875
15/16"	-	0.938	-	0.938	-	-	0.938
31/32"	-	-	-	-	-	-	0.969
1"	1.000	-	-	1.000	-	1.000	1.000

It is preferable to specify a single wall thickness across smaller structures (one to three spans) whenever the piles are of the same diameter, so that the piles cannot be mixed up and the wrong piles installed at a substructure unit. If a structure has different size piles at the abutments and the pier(s) – for example 12-inch and 16-inch respectively – then different pile wall thicknesses can be reasonably specified for the two sizes without any worry of mixing up the piles by the pile installer.

In the Resistance Gain/Loss Factors section, there are five lines for Shaft and Toe gain/loss factors. On the first line, for Shaft gain/loss factor, enter the inverse of the highest setup factor used in the Soil Profile table. For example, if setup factors of 1.0, 1.2, and 1.75 were all used for setup factors on various lines of the Soil Profile table, enter the inverse of 1.75 (the highest value) as 1/1.75 = 0.57 for Shaft gain/loss factor on the first line. On the first line, enter 1.0 for Toe gain/loss factor. On the lower four lines of this table, enter 0 for all values of Shaft and Toe gain/loss factors. The GRLWEAP program applies the defined shaft gain/loss factor to the soil layers with the highest setup factor, and scales the shaft gain/loss factor applied to other layers, based on the ratio between the highest setup factor and the setup factor applied to each layer.

1304.2.3 GRLWEAP "D" Driving System Modifications Table

Inspect the Driving System Modifications table and ensure that there is at a minimum one line of data, with the same depth, corresponding to each line of the Soil Profile table. The tabular output for the drivability analysis will include one output line of data for each line in the Driving System Modifications table. Additional intermediate lines may be added to the table if the engineer wants more detail within a certain range of driving depth (for example, to pinpoint the depth at which refusal occurs after the pile enters a particularly dense layer).

1304.2.4 GRLWEAP Output and Interpretation

For the output included in a Geotechnical Report, include three items: the GRLWEAP drivability graphical and tabular output sheets, and the first two pages of the GRLWEAP output (*.gwo) file, which include all the information entered on the Main Input screen and the Soil Profile table. The remainder of the output file merely provides iterations of wave equation calculations performed for simulated hammer impacts at each analyzed depth. These are not generally of interest to the Office of Geotechnical Engineering; we only want to know the inputs to the analysis and the summary of the output.

In the output, check that the pile is not overstressed according to Section 1304.2. For an ASTM A709 50 ksi steel pile, overstress constitutes any compressive stress greater than 0.9 φ_{da} f_y = (0.9)(1.00)(50 ksi) = 45 ksi. For a 35 ksi ASTM A252 Grade 2 steel pipe pile, overstress constitutes any compressive stress greater than 31.5 ksi. For a 45 ksi ASTM A252 Grade 3 steel pipe pile, overstress constitutes any compressive stress greater than 40.5 ksi. For concrete piles, prestressed concrete piles, or timber piles, see AASHTO LRFD Article 10.7.8; for these piles there is a limit to the tensile stress in addition to the compressive stress. Also check that refusal blow counts (greater than 100 bpf) are not calculated prior to the pile reaching the required depth. If the pile is calculated to be overstressed, or refuses prior to reaching the required depth, modify the pile or pile driving hammer in accordance with Section 1304.2.

1304.3 Pile Setup and Relaxation

Setup and relaxation are gains or losses, respectively, in nominal pile resistance over time, caused by changes in the stresses in the foundation soil between pile driving or end of initial drive (EOID) and at some time later. Refer to publication FHWA-NHI-16-009/010, Geotechnical Engineering Circular 12 (GEC 12) "Design and Construction of Driven Pile Foundations" for guidance on the potential for pile setup or relaxation for various soil types. Whenever setup or relaxation are realized in the design, specify restrike testing of the same piles that are dynamically load tested (specify one restrike item for each dynamic load testing item on the structure).

1304.3.1 Pile Setup

Dissipation of excess pore water pressure in foundation soils after driving can result in increased soil shear strength and additional side friction commonly referred to as setup. Typically, driven piles are not designed to account for setup, therefore, the additional amount of resistance to account for setup of the side friction after EOID, $R_{Ssu} = 0$. This means the EOID resistance equals UBV resistance. However, if soft cohesive soils are encountered at the site, and pile bearing resistance analysis for the “driving resistance” indicates long estimated pile lengths at EOID, it may prove beneficial to consider setup in the design.

Perform both a “driving resistance” analysis, taking into account the driving losses to side frictional resistance experienced by the pile during driving and prior to setup, and an “ultimate resistance” analysis for the long-term condition after pile setup has occurred. Use BDM Table 305-2, “Pile Driving Recommended Setup Factor (f_{su}) for Side Friction,” to determine the setup factor for each soil layer based on soil classification. If the estimated driving resistance indicates driving losses that would increase the length of the pile during driving by more than 10 feet with EOID compared to the UBV, then account for pile setup in the design.

Pile setup can potentially result in upwards of a six-fold increase in the UBV compared to EOID. Conversely, if the loss of resistance during driving is not realized in design, this can result in substantial pile quantity overruns during construction. Realization of pile setup in design can therefore result in substantial pile quantity savings. A site-specific pre-construction evaluation utilizing test piles (Alternative C) should be considered if over 10,000 feet of piles of the same type, size, and nominal resistance are specified.

Utilize one of the following three alternatives for the design:

1) Drive all piles to the full Estimated Length, perform dynamic load testing on the first two piles driven and then restrike testing on the same two piles after a specified waiting period, to verify setup and the UBV resistance as shown in the plans, or

2) Specify the performance of a site-specific construction evaluation utilizing variable length driving and dynamic load testing of the first four production piles, and restrike testing on the same four piles after a specified waiting period, to determine a specific pile tip elevation to be used during construction, or

3) Specify the performance of a site-specific pre-construction evaluation utilizing test piles with dynamic load testing of each pile and a multiple number of restrike tests at a variable waiting period to determine values of EOID and UBV along with pile driving criteria to be used during construction.

Plan notes for all three of the above alternatives are available in BDM Section 600.

For Alternatives 1 and 2 restrike the load test piles after a specified waiting period, based on the calculated composite setup factor for the pile as f_{su} = UBV / EOID resistance, and Table 1300-7 as a guide for the wait time.

Table 1300-7: Setup Waiting Period

Setup Factor (f_{su})	Percent Setup (%)	Wait Time (Days)
$1.0 \leq f_{su} \leq 1.1$	0 – 10	1
$1.1 < f_{su} \leq 1.2$	10 – 20	3
$1.2 < f_{su} \leq 1.4$	20 – 40	7
$1.4 < f_{su} \leq 1.6$	40 – 60	14
$1.6 < f_{su} \leq 2.0$	60 – 100	30
$f_{su} > 2.0$	> 100	45

For example, for a 14-inch diameter CIP pipe pile with an Estimated Length of 60 feet, the UBV is 367.50 kips and the EOID resistance is 318.22 kips. Therefore, f_{su} = UBV / EOID resistance = 367.50 kips / 318.22 kips = 1.15. According to Table 1300-7, the recommended wait time for a setup factor of from 1.1 to 1.2 is 3 days.

For Alternative 3, there can be a variable wait period for the restrike testing; the default in BDM Plan Note 606.7-5 is 7 days and 45 days for two restrike tests. The designer can select different waiting periods and more restrike tests, and modify the plan note accordingly, if a more complete setup curve is desired. This alternative is typically reserved for very large bridge projects or for design build projects, therefore customization by the designer can be expected.

For Alternative 1, all production piles for the substructure are driven at the same time, starting with the two test piles, to the plan Estimated Length. The assumption is that UBV will be met upon restrike. The risk that this will not be so is considered low, and it will typically result in a wait to restrike a second time at an additional wait time equal to the initial wait time. Thus, the cost to the project would be in a scheduling delay. In rare instances, a second restrike will not suffice, and then the choice needs to be made to wait longer for a third restrike or to order more length of pile. Most of the time, the cost of additional pile is less than the cost of additional wait time.

For Alternative 2, the idea is to generate a curve of depth versus nominal resistance, with points at 75%, 85%, and 100% plan Estimated Length. The point at which the curve crosses the required UBV defines the depth to which the remaining piles in the foundation unit are to be driven, without further dynamic load testing. With this alternative, the test piles are ordered and furnished first, and the remaining piles in the foundation unit are not ordered and furnished until the driving depth is defined.

For Alternative 3, a test pile is driven and tested at the location of the foundation unit, prior to construction. The testing will define a profile of cumulative setup with depth, and a setup curve, with nominal resistance versus time. The Engineer will use this data to specify either an EOID resistance and driving criterion for the remaining piles in the foundation unit (along with a predicted quantity of pile setup), or a depth to which the remaining piles in the foundation unit are to be driven, without further dynamic load testing. If the test pile is properly located, it may be utilized as a production pile for the foundation unit.

If pile setup is utilized in the design, specify restrike testing of the same piles that are dynamically load tested during driving (specify a minimum of one restrike item for each dynamic load testing item). For Alternative 3, specify two restrike items for each dynamic load testing item (or more, if the plan note is modified to include more than two restrikes).

If the potential for pile setup is indicated for point bearing piles on bedrock, make no change to the design. For point bearing piles on bedrock, while pile setup will doubtless increase the pile bearing resistance some unknown amount, it is ignored in design, since soil friction resistance is not included in the pile bearing resistance due to the large difference in displacement necessary to mobilize side resistance in soil compared to tip resistance on rock.

1304.3.2 Pile Relaxation

Relaxation can affect pile resistance at the toe in dense, saturated, relatively fine grained cohesionless soils (such as non-plastic silts or fine sands), in heavily over-consolidated clays, or in weak rocks such as shale and claystone. In the case of dense fine sands and silts or clays, the soils can dilate near the pile tip, causing negative pore water pressure (suction) that results in a temporary increase in pile tip resistance. In the case of weak rocks, relaxation is believed to be attributable to a release of locked-in horizontal stresses as the pile tip breaks through the rock.

Pile relaxation can significantly reduce UBV compared to EOID in heavily over-consolidated clays and in dense saturated silts and fine sands. Research has shown relaxation related losses as much as 40%. However, the amount of displacement to re-mobilize the full UBV is considered small, and it is routinely neglected in design. For point bearing piles on bedrock, while relaxation in the bedrock at the pile tip will doubtless decrease the pile bearing resistance, this is also ignored, as the additional displacement necessary to re-mobilize the tip resistance is considered negligible.

Therefore, if the subsurface geotechnical conditions at the site indicate the potential for pile relaxation, make no change to the design regarding the nominal resistance provided at the pile tip. Setup may still be considered for the nominal resistance provided along the side of friction piles.

1304.4 Pile Uplift

When a structure goes into uplift, the uplift force must be resisted by the weight and side frictional resistance of the foundations. Uplift is typically induced by wind forces, loading on alternating bridge spans, or an unbalanced loading that results in overturning.

Where the estimated pile length is controlled by the required uplift resistance along the side of the pile, refer to BDM Section 305.3.2.3 and specify a pile tip elevation to provide a specific length of side resistance. A plan note is available in BDM Section 600. In this case, the piles are typically driven to the pile tip elevation and dynamic load testing of the pile is not performed. However, in design, perform a wave equation pile drivability analysis according to BDM Section 305.3.1.2 to determine whether the pile can be driven to the tip elevation without overstress.

When a pile must resist uplift loads, calculate the nominal uplift resistance with the same static analysis methods as for bearing resistance (see Section 1304.1); however, do not include tip resistance (it cannot function in tension), but do include pile self-weight. For self-weight, assume a unit weight of 150 pcf, as for reinforced concrete, and use a minimum dead weight of components load factor of γ_{DC} = 0.90. For side (frictional) resistance, use a resistance factor φ_{up} or φ_{ug} in accordance with BDM Table 305-1.

For group uplift in cohesionless soils, do not include the weight of the external block of soil with a slope of 1:4 as shown in AASHTO LRFD Figure 10.7.3.11-1; we consider it unrealistic that the piles could lift this large plug of soil out of the ground. In this case, include only the weight of the block of soil encompassed within the perimeter of the group as shown in AASHTO LRFD Figure 10.7.3.11-2.

Driven piles under permanent tensile loads are generally not permitted. If site constraints or other factors make permanent tensile loads on driven piles unavoidable, use a redundancy load modifier of η_R = 1.05 for the permanent tensile loads on the foundation elements.

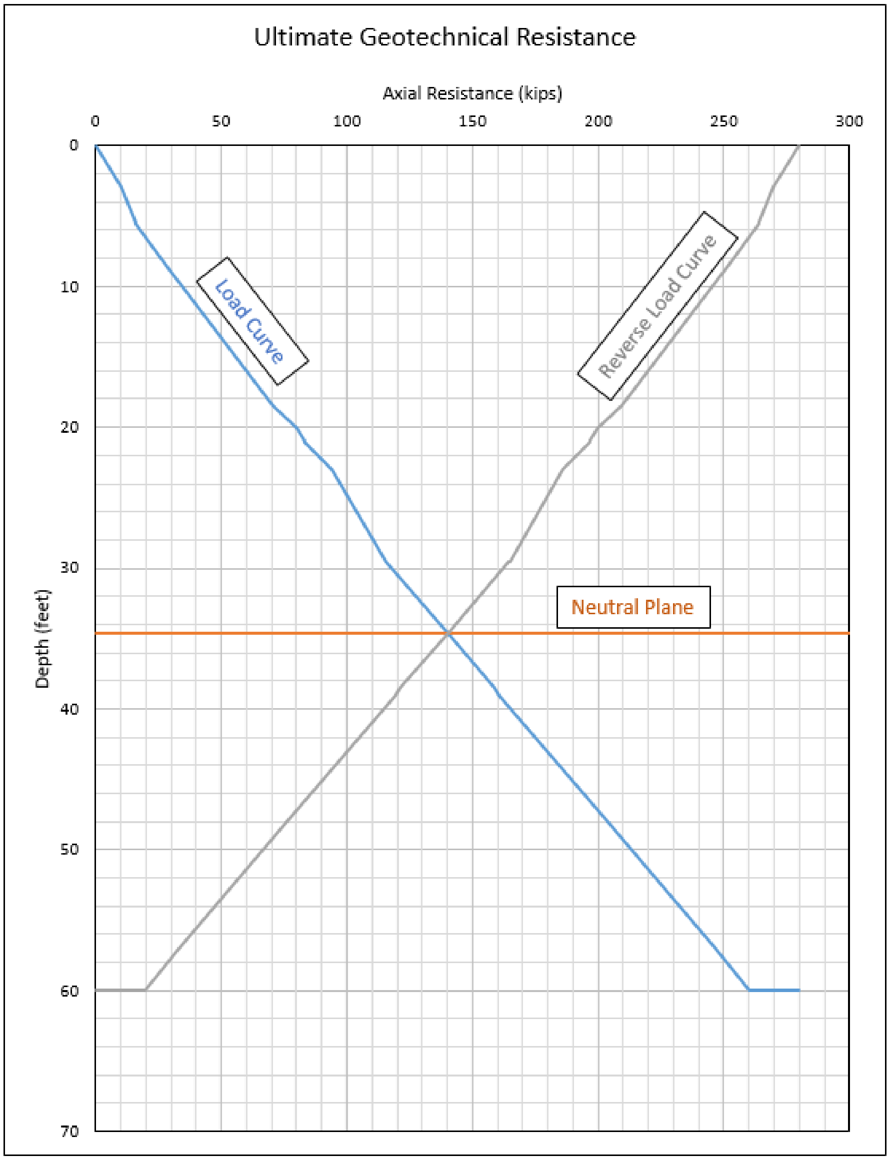
1304.5 Pile Downdrag and Drag Load

When soil moves downward relative to the pile, it creates a drag load on, and therefore within, the pile. The downward soil movement also creates the potential for downward pile movement. This downward pile movement is referred to as downdrag. The subsurface conditions, pile installation methods, pile loading sequences, as well as the pile and ground surface configuration determine the magnitude of the drag load and the downdrag movement.

Assume downdrag will occur if ground settlement of greater than or equal to 0.4 inches will occur after pile installation at the substructure unit in question. For all friction piles subject to downdrag, calculate the location of the neutral plane per the Goudreault and Fellenius (1994) method as described in FHWA-NHI-16-009/010, Geotechnical Engineering Circular 12 (GEC 12) "Design and Construction of Driven Pile Foundations," Section 7.3.5.7.

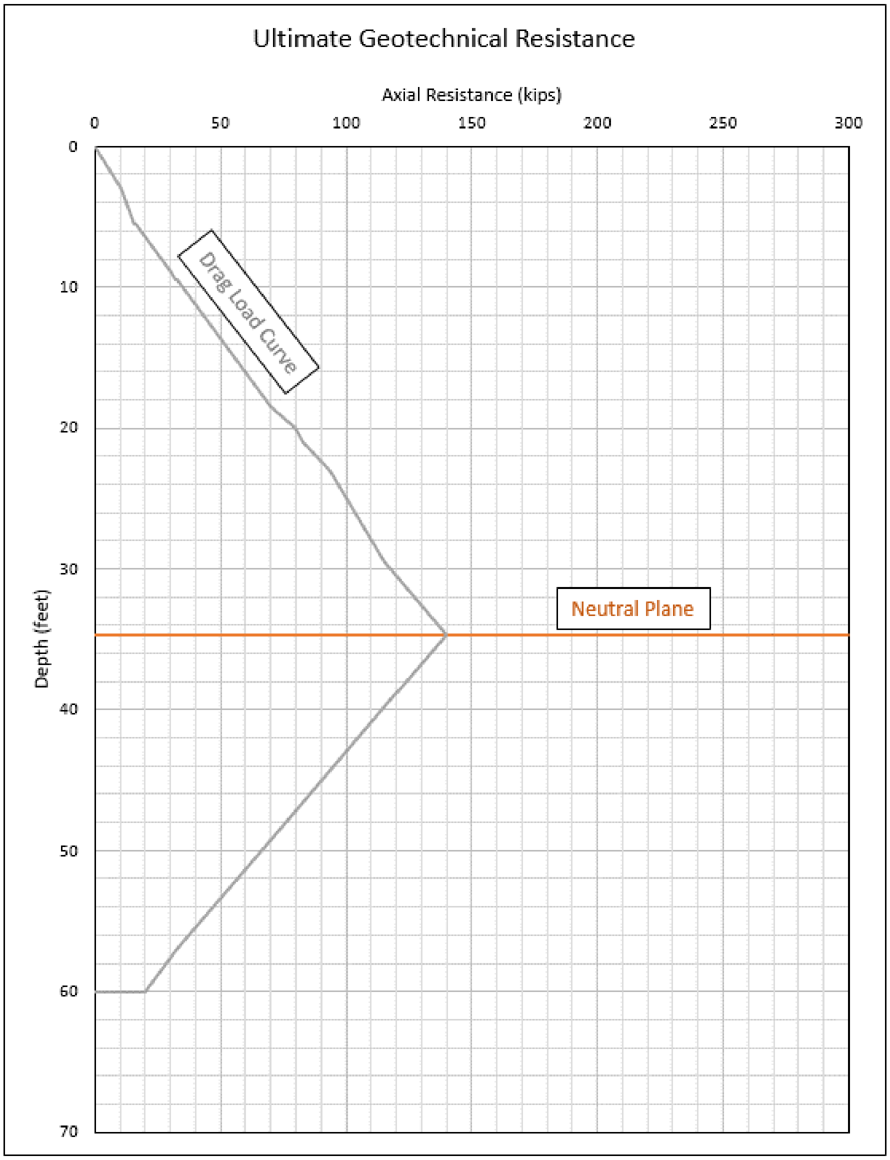
To locate the neutral plane for a friction pile by the Goudreault and Fellenius (1994) method, see Figure 1300-10. Perform a static axial pile bearing resistance analysis for the estimated length of the pile and plot the nominal resistance curve for the "ultimate resistance" including the cumulative side frictional resistance and tip resistance. This can be seen with the blue "Load Curve" in Figure 1300-10, which has a cumulative side frictional resistance starting at 0 at the head of the pile and ending at 260 kips at 60 feet depth, and a tip resistance of an additional 20 kips for a total resistance of 280 kips. Next, plot the reverse of the load curve, starting at the estimated length of the pile at zero resistance, adding in the tip resistance, and then adding the cumulative side frictional resistance starting at the tip of the pile and adding up to the head of the pile. This can be seen with the gray "Reverse Load Curve" in Figure 1300-10, which has a tip resistance of 20 kips at 60 feet depth, and a cumulative side frictional resistance starting at 20 kips at the tip of the pile and ending at a total resistance of 280 kips at the head of the pile. Where these two curves intersect is the estimated depth of the neutral plane. This can be seen with the orange "Neutral Plane" line in Figure 1300-10, at a depth of 34.67 feet. The maximum nominal axial load in the pile occurs at the neutral plane. At this time, before adding any load at the head of the pile, the load is entirely due to drag load. The maximum load at the depth of the neutral plane in Figure 1300-10 is 140 kips (nominal drag load = 140 kips). The drag load curve for the example case, with the loading portion above the neutral plane, a maximum axial load of 140 kips at the neutral plane depth of 34.67 feet, and the unloading portion below the neutral plane, can be seen in Figure 1300-11.

Figure 1300-10: Neutral Plane by Goudreault and Fellenius (1994) Method



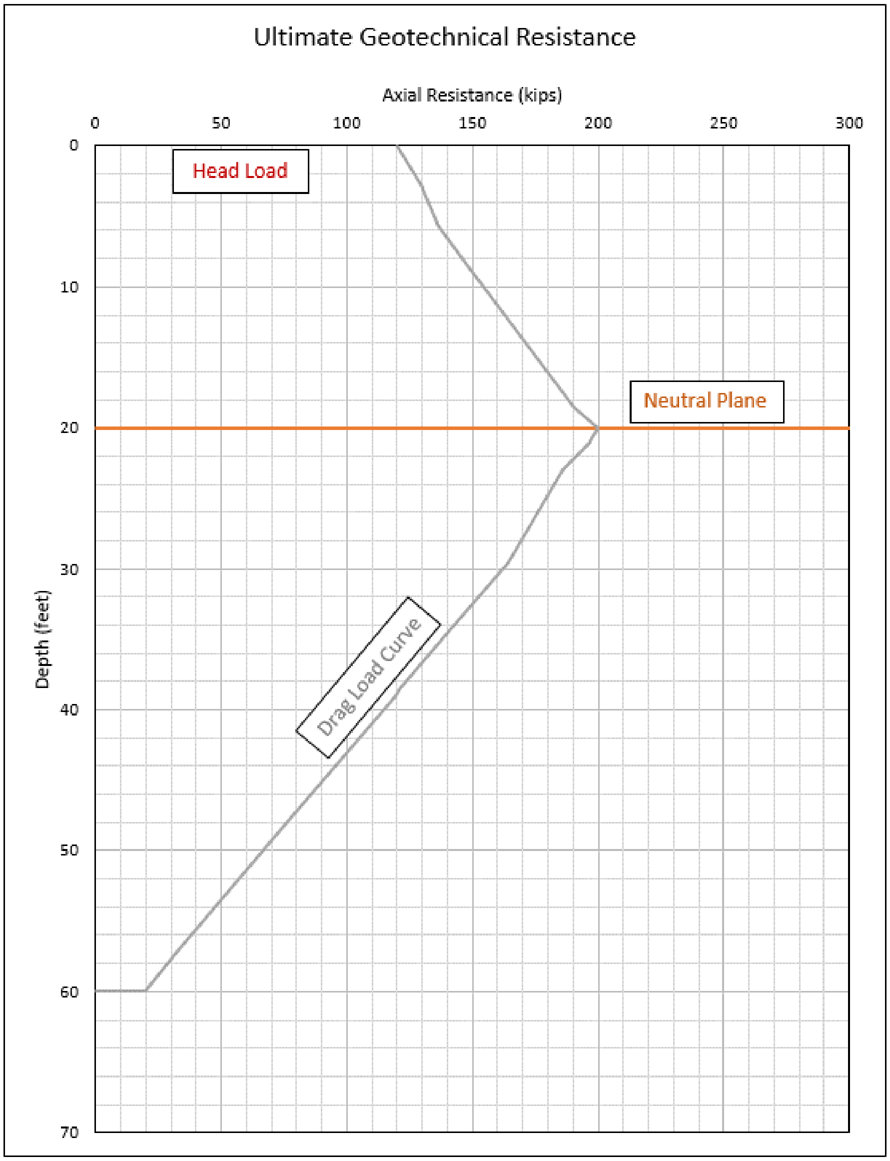
The neutral plane is the point at which the soil and pile move the same amount. As the pile head is loaded, and the pile deflects downwards, the neutral plane will move upwards, converting some of the drag load into side friction resistance. The pile will stop moving when the neutral plane climbs to a point where the geotechnical resistance (below the neutral plane) equals the pile head load plus drag load. If the pile head load continues to increase, ultimate resistance is reached (UBV) when the neutral plane reaches the top of the pile or the ground surface, and all drag load is converted into side friction resistance. At this point, any increase in load will begin to plunge the pile.

Figure 1300-11: Drag Load Curve, no Pile Head Load



In Figure 1300-12, 120 kips of head load has been added to the pile, and the neutral plane has moved up to a depth of 20 feet, as the pile has moved slightly downwards in relation to the soil. The maximum axial load in the pile is now 200 kips at the neutral plane, and nominal drag load = maximum axial load – head load = 200 kips – 120 kips = 80 kips.

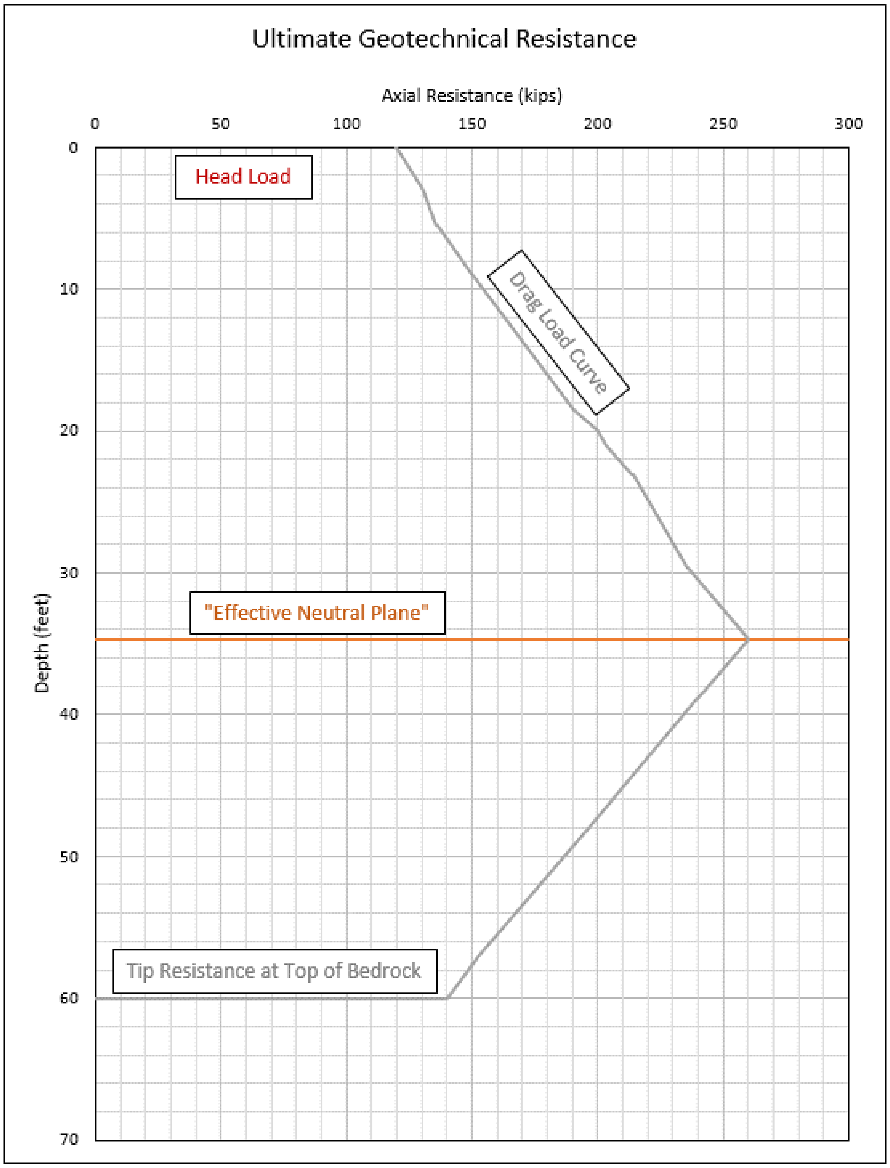
Figure 1300-12: Drag Load Curve with Pile Head Load



As the head load is increased, the neutral plane will continue to move upwards, and the point of maximum axial load in the pile will continue to move to the right. The neutral plane will reach the head of the pile when the head load (and the maximum axial load in the pile) reaches the ultimate geotechnical bearing resistance of the pile, or ultimate bearing value (UBV), and the curve will become the same as the "Reverse Load Curve" in Figure 1300-10. This represents the Geotechnical Strength Limit State of the pile (note that this is different than the Structural Strength Limit State of the pile, which is typically greater). At this point all drag load has been converted to resisting load (drag load = 0). At the Geotechnical Strength Limit State of the pile, there is no longer any drag load. The UBV is the same whether there is downdrag or not. Therefore, we do not have to embed piles deeper to overcome the drag load with additional bearing resistance. If the load applied to the pile is increased, the pile will move again, mobilizing the full geotechnical resistance. If the total load exceeds the full geotechnical resistance, the pile will begin to plunge.

For all point bearing piles on bedrock, consider the neutral plane to be at the top of bedrock; this essentially indicates that the entire pile length is above the neutral plane and subject to drag load. However, do not consider drag load at soil movements of less than 0.4 inches relative to the pile, where the relative movement is not enough to fully mobilize the downdrag load.

Figure 1300-13: Drag Load Curve for Point Bearing Pile on Bedrock



In Figure 1300-13, see an example of this procedure. This pile is similar to the previous example in Figure 1300-10 through Figure 1300-12, with the exception that it is driven to refusal on bedrock at a 60-foot depth. A static axial pile bearing resistance analysis has been run for the estimated length of the pile, developing a nominal resistance curve for the cumulative side frictional resistance. To this has been added the pile head load as in Figure 1300-12, to develop a drag load curve. A separate settlement analysis has been run, showing cumulative settlement starting at 0 at the top of rock and reaching 0.4 inches at a depth of 34.67 feet. We will call this point the "effective neutral plane," above which side friction accumulates in drag load on the pile, and below which the side friction dissipates down to the tip resistance at the top of bedrock.

In this example, the pile head load is 120 kips as in the previous example, but since the pile cannot shift downwards to raise the neutral plane and shed some of the drag load, the maximum axial load in the pile is higher at 260 kips at the "effective neutral plane." Below the "effective neutral plane," the side friction acts in the positive direction, dissipating the load in the pile until it reaches a tip resistance of 140 kips at the top of bedrock. In this case, nominal drag load = maximum axial load – head load = 260 kips – 120 kips = 140 kips.

For a more refined procedure, consider the elastic shortening of the pile, and locate the "effective neutral plane" at the point where the relative consolidation movement of the soil against the shortened pile is 0.4 inches.

Analyze downdrag and drag load per the Siegel et al. (2013) method as described in GEC 12, Section 7.3.6.1, except as modified below.

Evaluate drag load versus structural axial resistance of the pile – structurally, i.e., only inside the pile – at the Strength Limit State or Extreme Limit State. Evaluate the total factored axial load per pile including drag load versus the factored structural axial resistance per pile as follows:

$$Q_p = \Sigma \eta_i \gamma_i Q_i + \eta_i \gamma_p DD \leq P_r = \varphi_c P_n$$

Where:

Q_p = Total factored axial load per pile for highest loaded pile at each substructure unit (kips)

$\Sigma \eta_i \gamma_i Q_i$= Sum of factored loads for highest loaded pile at each substructure unit, excluding drag load (kips)

DD = Nominal drag load (downdrag load) per pile (kips)

γ_p = Load factor for drag load = 1.05 in Strength Limit State, 1.00 in Extreme Limit State

P_r = Factored structural axial compressive resistance per pile (kips), calculated per BDM Section 305.3.3 and AASHTO LRFD Article 6.15.3.1.

φ_c = resistance factor for axial resistance of piles in compression under good driving conditions, per BDM Sections 305.3.2, 305.3.2.2, and 305.3.3 and AASHTO LRFD Article 6.5.4.2. For H-piles, $\varphi_c = 0.60$; for pipe piles, $\varphi_c = 0.70$.

P_n = Nominal structural axial compressive resistance per pile (kips), calculated per BDM Section 305.3.3 and AASHTO LRFD Article 6.9.4.1.

When calculating the factored structural axial compressive resistance of the pile (P_r), if including drag load, use the resistance factor for steel piles in compression under good driving conditions. Otherwise (when not including drag load), use the resistance factor for steel piles in compression and subject to damage due to severe driving conditions in accordance with AASHTO LRFD Article 6.5.4.2.

The reasoning for using the resistance factors for piles in compression under good driving conditions when considering drag load is that the maximum load in the pile when including drag load will occur somewhere in the middle of the pile, not at the head or toe of the pile. The highest driving stresses occur at the extreme ends of the pile – either at the head or toe – where driving damage is more likely. Therefore, the lesser resistance factors for piles in compression subject to damage due to severe driving conditions are to only be used for consideration of factored structural axial resistance per pile versus the total factored axial load per pile excluding drag load.

Fellenius (1984) recommends using a "factor of safety" of 1.5 for the structural material properties. When using a load factor of 1.05 and a corresponding structural resistance factor of 0.60 for H-piles in compression under good driving conditions, this would yield an equivalent "factor of safety" of 1.05/0.60 = 1.75. For pipe piles, using a load factor of 1.05 and a corresponding structural resistance factor of 0.70 yields a "factor of safety" equivalent to 1.05/0.70 = 1.50. In both cases, these meet or exceed the recommended "factor of safety."

Drag load is the total sum of skin friction in the downwards direction (negative skin friction) for the full length of the pile in contact with the soil above the neutral plane. Calculate skin friction and drag load using static analysis methods, per AASHTO LRFD Section 10.7.3.8.6. In this calculation, typically assume 100% mobilization of tip (toe) resistance. Alternately, resistance versus displacement curves (t-z and q-w curves), along with a comparison to the predicted settlement of the foundation soil, may be used in a more refined iterative approach.

Do not specify battered piles when drag load is anticipated. The additional bending forces imposed on the piles by the drag load can cause pile deformation and damage.

Evaluate the downdrag movement, as it contributes to pile head settlement, at the geotechnical Service Limit State. Consider the pile downdrag movement (S_{dd} = settlement due to downdrag) equal to the settlement of the foundation soil at the location of the neutral plane, plus the elastic compression of the pile above the location of the neutral plane due to the effects of the permanent load at the pile head plus the drag load. Calculate settlement of the foundation soils per AASHTO LRFD Section 10.6.2.4. Calculate elastic compression of the pile per GEC 12, Section 7.3.5.1. Exclude transient loads from the calculation of settlement and elastic compression. See BDM Section 305.1.3 for limits on vertical movement with respect to a supported bridge structure.

For point bearing piles on bedrock, include both transient loads and drag load in the computation of total factored axial load per pile, and provide this value in the structure general notes.

For friction piles, do not include both transient loads and drag load in the computation of total factored axial load per pile. If transient loads exceed the value of drag load, include only transient loads. If transient loads are less than the value of drag load, include only the drag load.

If greater axial structural resistance is necessary, consider using larger piles, thicker pile walls, stronger pile steel, or increasing the number of piles and reducing the applied load per pile. To reduce or eliminate downdrag, consider preloading the soil so settlement occurs before pile installation. Also consider installing wick drains and an additional earth surcharge load to decrease the amount of time required for settlement to occur. A Special Provision for Installation of Wick Drains is available from the Office of Geotechnical Engineering.

1304.6 Pile Scour Considerations

The maximum estimated scour depth may occur with the design flood, check flood or an overtopping flood of lesser recurrence interval. See Section 1304.1.1 for how to set up the model and boundary conditions for an analysis of bearing resistance for piles subject to scour. Ensure that driven piles extend a minimum of 15 feet below the maximum estimated scour depth. Use BDM Plan Note 606.2-2 for friction piles subject to scour conditions. This plan note specifies driving to the UBV or a tip elevation, whichever is deeper.

If the design flood scour reaches top of rock, do not use driven piles. Piles prebored into rock may still be used; in this event, the piles must penetrate a minimum distance below the maximum estimated scour depth in accordance with Section 1305.4. This scour depth includes any scour of non-scour resistant rock. For determination on whether rock is scour resistant or not, see BDM Section 305.2.1.2.b.B.

Where scour is predicted, neglect the pile resistance provided by soil in the scour zone. Use the depth of scour resulting from the design flood, per AASHTO LRFD Section 2.6.4.4.2, with Strength and Service Limit State checks. Use the depth of scour resulting from the check flood with Extreme Event II Limit State checks.

For construction installation of friction piles, include the side resistance from the soil within the scour zone (Rs_{sc}) in the Ultimate Bearing Value. Determine the UBV according to BDM Section 305.3.2. A plan note is available in BDM Section 600.

Because the pile will lose support along the scour depth, investigate the structural capacity of the pile considering the depth of the scour as an unbraced length. Analyze the piles for buckling and lateral stability as unsupported columns above the point of fixity with scour depths included in accordance with BDM Section 305.3.5.5. Also perform a p-y analysis on the piles according to BDM Section 305.1.2 to demonstrate:

1) Lateral stability against overturning failure in the Strength Limit State with the maximum estimated scour depth at the design flood.

2) Lateral stability against overturning failure in the Extreme Event II Limit State with the maximum estimated scour depth at the check flood.

3) Lateral stability against excessive deflection in the Service Limit State at the design flood.

1305 PILES PLACED IN PREBORED HOLES

BDM Sections 305.3.2.1 and 305.3.5.7 limit the use of driven piles in situations where bedrock is shallow, particularly when scour is a consideration; in accordance with BDM Section 305.3.5.7 use piles placed in prebored holes extending a minimum of 5 feet into bedrock if bedrock exists within 10 feet of the bottom of pile cap. When scour ***is not*** a consideration, extend the prebored holes a minimum of 10 feet into bedrock if the bedrock unconfined compressive strength is less than 1500 psi. When scour ***is*** a consideration, see the required embedment depths in Section 1305.4.

Unlike piles prebored under MSE walls, which requires two rows of piles, this provision accommodates the possibility of a single row of piles with significant lateral loads and bending moments. The extension into rock should create a point of fixity for pile stability. Analyze lateral resistance of the piles by performing a p-y analysis according to BDM Section 305.1.2. If necessary for lateral stability, increase the length of the prebored hole into rock.

If using piles placed in prebored holes in bedrock, use the bottom of the prebored hole elevation as the pile tip elevation. Round up to nearest 5 feet for Estimated Length and add an additional 5 feet for the pile Order Length, as for driven piles in accordance with BDM Section 305.3.5.2.

Do not design piles placed in prebored holes as battered piles; the prebored holes are difficult or impossible to drill on a batter.

Place the piles in the prebored holes without driving the piles. The prebored hole diameter shall be larger than the diameter or diagonal dimension of the pile. For integral abutment piles, backfill the bottom 5 feet of the prebored hole with Class QC Misc. concrete; otherwise place the concrete in the prebored holes up to the top of bedrock. If the abutment is integral, backfill from the top of concrete to the bottom of pile cap elevation with granular material conforming to 703.11, Structural Backfill Type 2, except 100 percent of the material shall pass through a ¾-inch sieve. If the abutment is an MSE wall abutment, sleeve the pile and backfill the prebored hole with Class QC Misc. to the bottom of Foundation Preparation elevation at the bottom of the pile sleeve; backfill within the pile sleeve in accordance with SS840. Otherwise, if the abutment is conventional or semi-integral, continue backfill with Class QC Misc. concrete to the bottom of pile cap elevation.

Include the drilled holes with C&MS Item 507 Prebored Holes, As Per Plan for payment. The Department will include the backfill materials with the prebored holes for payment. The Department will pay for the piles under Item 507 Steel Piles HP_x_, Furnished, As Per Plan with the placement into the prebored holes incidental to the pay item. Do not include Item 505 Pile Driving Equipment Mobilization or Item 507 Piles Driven when placing piles in prebored holes.

For modeling of piles placed in prebored holes, place the groundwater level in accordance with Section 1304.1.1.

1305.1 Pile Bearing Resistance

For piles placed in prebored holes in bedrock, similar to point bearing piles on rock, the bearing resistance is not limited by geotechnical resistance but by the structural resistance of the pile, and the factored axial resistance is calculated in the same way, in accordance with Section 1304.1. However, for piles placed in prebored holes, since the piles are not driven, they are not subject to driving stresses or potential driving damage. They are then essentially steel-only columns in axial compression, and they use a structural resistance factor, $\phi_c = 0.95$, in accordance with BDM Section 305.3.5.7 and AASHTO LRFD Article 6.5.4.2. Otherwise, determine the nominal axial structural resistance of the piles according to BDM Section 305.3.3.

1305.2 Pile Uplift

The amount of relative movement necessary to mobilize the side resistance in soil is much greater than that to mobilize the side resistance in rock, therefore, only count side frictional resistance against uplift for the length of the pile placed in bedrock with concrete backfill. Do not include drag load in soil as resistance against uplift.

The concrete backfill of the prebored hole is assumed to bond more effectively to the steel pile than to the surrounding soil and bedrock, therefore the interface with the concrete and bedrock is the critical surface for calculation of side frictional resistance. Calculate the nominal side resistance as for a drilled shaft with a rock socket, in accordance with BDM Section 305.4.1.3 and AASHTO LRFD Article 10.8. For this calculation, assume the drilled shaft diameter is the same as the nominal prebored hole diameter; if unknown, add 2 inches to the diagonal dimension to the pile cross section, and round to the nearest 2-inch interval. For example, for a HP12x53 pile, the diagonal dimension is 16.85 inches. If we add 2 inches, we get 18.85 inches; rounding to the nearest 2-inch interval, we get 18 inches for the assumed nominal prebored hole diameter.

Use a resistance factor against uplift of $\phi_{up} = 0.40$ for the side friction in rock, as for a drilled shaft rock socket, in accordance with BDM Table 305-1.

Include the weight of the steel pile and the weight of any concrete backfill of the prebored hole in the factored resistance. If the prebored hole is backfilled with concrete above top of bedrock, include this weight, even though the side frictional resistance is ignored over this length. If the prebored hole is backfilled with granular material above top of rock, ignore the weight of this backfill. Use a unit weight of 145 pcf for the plain concrete backfill. Use a unit weight of 490 pcf for the pile steel. Use a resistance factor for the weight equal to the minimum load factor for permanent dead load of components, $\gamma_{DC} = 0.90$.

1305.3 Pile Downdrag and Drag Load

Analyze pile downdrag and drag load for piles placed in prebored holes in bedrock, similar to point bearing piles on rock, locating the “effective neutral plane” at the depth at which cumulative settlement, starting at 0 at the top of rock, reaches 0.4 inches.

If the prebored hole is backfilled with granular material above top of rock, calculate side friction for this length as for a driven pile in soil, assuming the soil to be sand with an internal friction angle of $\phi_f = 28°$. If the prebored hole is backfilled with concrete above top of bedrock, calculate side friction as for a drilled shaft with the same nominal prebored hole diameter; if unknown, estimate the diameter as described in Section 1305.2. The length embedded in rock is always backfilled with concrete, and therefore side friction should be calculated as for a drilled shaft rock socket. Note that, due to greater load shedding in the portion embedded in rock (higher side friction), the load could dissipate to zero before reaching the pile tip.

Include both transient loads and drag load in the computation of total factored axial load per pile and provide this value in the structure general notes. Additionally, as noted in Section 1305.1, the structural resistance factor is $\phi_c = 0.95$ for steel-only columns in axial compression.

1305.4 Pile Scour Consideration

Piles placed in prebored holes must extend a minimum of 10 feet below the maximum estimated scour depth if bedrock $Q_u < 2500$ psi; they must extend a minimum of 5 feet below the maximum estimated scour depth if bedrock $Q_u \geq 2500$ psi. Pile bearing resistance is unaffected by scour, except for limitations imposed by buckling and lateral stability. Analyze the piles for buckling and lateral stability as for driven piles, according to Section 1304.6. However, if the prebored hole is to be backfilled with concrete through the cantilever length, consider the pile flanges to be continuously braced through this length.

1305.5 Plan Notes for Piles Placed in Prebored Holes

When using piles placed in prebored holes in bedrock, include BDM Plan Notes 606.7-6 and 606.7-7 in the structure General Notes. Modify the plan notes as necessary to fit the type and size of pile used, and the type of abutment.

1306 DRILLED SHAFTS

See BDM Section 305.4 for specifications regarding the design of drilled shaft foundations for the support of structures, and BDM Section 307.6 for specifications regarding the design of drilled shaft walls. Design drilled shaft foundations for vertical (axial) geotechnical resistance in accordance with ASHTO LRFD Article 10.8, using resistance factors in accordance with BDM Table 305-1. Design drilled shaft foundations for lateral geotechnical resistance and deflection in accordance with BDM Section 305.1.2. Design drilled shaft walls for lateral geotechnical resistance and deflection with p-y analysis or moment equilibrium analysis methods in accordance with BDM Sections 307.1.3, 307.1.4, and 307.1.6.

For modeling of drilled shaft foundations, place the groundwater level in accordance with Section 1304.1.1.

1306.1 Design Considerations

Drilled shafts are deep foundations and can penetrate through surficial weak soils to reach deeper bearing soils without the need for over-excavation. However, they are also useful to limit the depth of foundations if there is some concern with going too deep, such as penetration of a confined artesian aquifer or natural gas deposit. They can be constructed at nearly any diameter, the only practical limit being economics inherent in mobilization of equipment of adequate size.

Drilled shaft construction creates little disturbance with vibration compared to driven piles, produces little soil excavation compared to spread footings, typically poses no need for temporary shoring, and does not necessarily require a footing or pile cap.

Drilled shafts have strong bending resistance, shear resistance, and lateral resistance to deflection. They can stand for a long distance unsupported, similarly to a bridge column, making them useful in conditions of substantial predicted depth of scour, or even in construction underwater without the need of cofferdams.

Compared to other foundation systems, drilled shafts are highly reliable with respect to geotechnical and structural resistance; they utilize known materials, the geotechnical design equations are conservative, the excavations can be inspected, their integrity can be tested, and bearing resistance can be proved.

1306.1.1 Drilled Shaft Detailing

Drilled shaft rock sockets do not begin at auger refusal, top of bedrock core, or beginning of "competent" bedrock; rock sockets always begin at top of rock (TR), regardless of the character of the rock. Designers are free to assign reduced or even zero axial resistance to the upper weathered zone in the bedrock, as they see fit according to engineering judgement, but the rock socket pay quantities and reduced rock socket shaft diameter will begin at the TR encountered in the field at the time of construction. Assume this is the same as the TR revealed by the borings drilled for the design of the substructure unit.

Although an assumed flat or linearly sloping TR across a drilled shaft unit may be used for design of drilled shafts or estimation of quantities, do not display this on the plan sheets (do not show a linear representation of the assumed TR to either side of, across, or connecting between drilled shafts). TR is likely more variable away from or between boring locations, and showing an assumed definite TR may lead to construction claims if the actual TR differs. Only show TR at each boring location in the profile view on the Site Plan with a short line using the line style "ground" in accordance with BDM Section 201.2.1.1.b(E). Next to each TR, provide a callout stating "Approx. Top of Rock, xxx.x ft. Elev., <Exploration ID>," replacing the x's in each callout with the appropriate TR elevation from the boring and <Exploration ID> with the actual exploration identification number for the boring.

Do not provide the elevation of the top of drilled shaft rock sockets in the plans; provide only the rock socket diameter and length.

If drilled shafts are short (around 10 to 15 feet long), and if dry construction methods are anticipated, use a single diameter drilled shaft above bedrock and into bedrock (for the rock socket), and use the same concrete cover and size of reinforcing cage for the full length of the drilled shaft.

Likewise, if the full length of the drilled shaft will be drilled in bedrock (either weak and weathered or more competent), do not step down the drilled shaft diameter for a "rock socket" at the top of competent rock. In this case, the entire length and all quantities will be drilled shaft into bedrock (rock socket).

Pay close attention to how many longitudinal reinforcing bars can fit inside a drilled shaft given the minimum concrete cover requirements of BDM Section 305.4.4.2 and the requirement of minimum clear distance between reinforcing bars of five times the maximum aggregate size in accordance with BDM Section 305.4.4.3. Please note that in C&MS Item 499, Class QC 5 concrete allows for as small as a 3/8-inch *Nominal Maximum* Aggregate size, which is No. 8 Coarse Aggregate, which has a ½-inch *Maximum* Aggregate size. This means that the minimum clear gap between bars is 0.5 inch × 5 = 2.5 inches.

1306.1.2 Drilled Shaft General Notes

Provide the recommended plan notes for the structure general notes, modified as necessary, and provide the values of loads and resistances to fill into the blanks in the BDM plan notes. For drilled shafts supporting an axial load, always include BDM plan note 606.8-1 for drilled shafts socketed into rock or BDM plan note 606.8-2 for friction drilled shafts (not socketed into rock). If only tip resistance or side resistance is used in the rock socket, modify BDM plan note 606.8-1 accordingly, and revise or remove the phrase(s) referring to the other component of resistance, e.g., "*The maximum factored load at each substructure is fully supported by the drilled shafts in tip resistance, ignoring any contribution from side resistance*."

For rock-socketed drilled shafts, generally provide the tip resistance or side resistance value in the plans, but not both. Most of the time this will be only the tip resistance, as it is usually adequate to support the drilled shaft load except in the weakest of rocks. If both tip and side resistance are used, modify each according to deflection and mobilization of resistance in accordance with Section 1306.3.2.

Do not limit geotechnical resistance in BDM plan notes 606.8-1 or 606.8-2 by the structural strength of the concrete, but note separately if the concrete strength limits resistance structurally. In this event, the drilled shafts could be modified in construction to use a higher strength concrete to take advantage of more of the geotechnical resistance.

For drilled shafts supporting a lateral load, include BDM plan note 606.8-3.

When uplift loading controls the embedment length of a drilled shaft, provide BDM plan note 606.8-4.

When it is necessary to install and test a demonstration drilled shaft in accordance with BDM Section 305.4.4.6, provide BDM plan note 606.8-5. In this event, also include BDM plan notes 606.8-6, 606.8-7, and 606.8-8 specifically referencing the demonstration drilled shaft. Include quantities for the testing of the demonstration drilled shaft in the general summary and estimated quantities, as testing is paid separately from construction of the demonstration drilled shaft.

1306.1.3 TIP Integrity Testing

At all substructure units supported on drilled shafts, perform integrity testing by Thermal Integrity Profiling (TIP), in accordance with BDM Section 305.4.5. Provide BDM plan note 606.8-6.

It is not necessary to specify TIP integrity testing for drilled shaft walls (not supporting bridge structures) nor landslide drilled shafts unless the shafts include a rebar cage. In this event, test a minimum of 10% of all drilled shafts including at least one shaft per wall. If landslide drilled shafts are installed to protect a bridge substructure unit, test every drilled shaft.

Do not specify integrity testing by Crosshole Sonic Logging (CSL) unless the drilled shaft is a demonstration drilled shaft or nominal shaft diameter is ≥ 84 inches.

1306.1.4 High Strain Dynamic Testing

BDM Section 305.7.2 specifies static load tests for friction drilled shafts, but states that high-strain dynamic testing is an acceptable alternative. In general, because of the high cost and difficulty of performing a static load test on a drilled shaft foundation, specify high-strain dynamic testing instead, and do not specify static load testing of drilled shafts except for structures of high importance (e.g., cable-stay structures, high tower structures, or any structure with Operational Importance $\eta_I > 1.0$ according to AASHTO LRFD 1.3.5).

When using high-strain dynamic testing in place of static load testing, specify high-strain dynamic testing for a minimum of 2% of all friction drilled shafts including at least one shaft per substructure unit. In accordance with BDM Table 305-1, this will allow a construction control resistance factor of $\varphi_{dyn} = 0.60$. Always test the drilled shaft in a substructure unit with the lowest capacity-demand ratio (CDR), typically the shaft with the highest individual load or the shaft with the least calculated resistance. Provide Special Provision: High-Strain Dynamic Testing of Drilled Shafts attached to the project plans, which can be found on the OGE web site.

1306.2 Load Effects and Special Design Considerations

In addition to load effects induced from supporting structures, uneven heights of fill (retaining walls), or landslides, drilled shafts may be subject to additional load effects from special conditions such as scour, downdrag, and uplift. See BDM Section 305.4.1 regarding calculation of loads on drilled shafts.

1306.2.1 Scour

See BDM Section 305.4.1.1 for additional detailing and design requirements for drilled shafts subject to scour.

1306.2.2 Downdrag and Drag Load

See Section 1304.5, Pile Downdrag and Drag Load, for a description and discussion of downdrag and drag load on deep foundations.

Analyze downdrag and drag load for rock-socketed drilled shafts similarly to point bearing piles on rock, in Section 1304.5, locating the “effective neutral plane” at the depth at which cumulative deflection, starting at 0 at the top of rock, reaches 0.4 inches.

To analyze downdrag and drag load for friction drilled shafts, use a refined iterative approach with load versus deflection curves (t-z and q-w curves), along with a comparison to the predicted deflection of the foundation soil, to locate the neutral plane. Drag load is the total sum of skin friction in the downwards direction (negative skin friction) for the full length of the drilled shaft in contact with the soil above the neutral plane. The full factored load per drilled shaft including drag load is resisted by the axial structural resistance of the drilled shaft.

For friction drilled shafts at the geotechnical Strength Limit State, the full upwards soil resistance is mobilized by the drilled shaft moving downwards with respect to the soil, eliminating all drag load from the foundation. Therefore, do not include drag load for comparison of axial geotechnical resistance at the Strength or Extreme Event Limit States for friction drilled shafts.

See BDM Section 305.4.1.2 for additional design requirements for drilled shafts subject to downdrag and drag load.

1306.2.3 Uplift

For drilled shafts subject to uplift, provide a drilled shaft tip elevation in the plans, so that the drilled shafts will develop adequate side resistance. For nominal uplift resistance, use the same static analysis methods as for bearing resistance (see Sections 1306.4.1 and 1306.4.2); however, do not include tip resistance (it cannot function in tension), but do include drilled shaft self-weight. For self-weight, assume a unit weight of 150 pcf, as for reinforced concrete, and use a minimum dead weight of components load factor of γ_{DC} = 0.90. Also include the effect of buoyancy for any portion of the drilled shaft located below the water table, factored by γ_{WA} = 1.00. For side (frictional) resistance, use a resistance factor φ_{up} or φ_{ug} in accordance with BDM Table 305-1. If the drilled shaft is high-strain dynamic load tested, and the side and tip resistance are separated out through CAPWAP analysis, a resistance factor of φ_{up} = 0.50 may be used for the side resistance in uplift.

For uplift resistance of rock-socketed drilled shafts, do not use side resistance in the soil portion of the shaft; count only the side resistance in the rock socket. The amount of relative movement necessary to mobilize the side resistance in the soil is much larger, and not compatible with the amount of movement to mobilize the side resistance in the bedrock socket, therefore only one or the other can act at one time. For the side resistance in the soil to act, the side resistance in the rock socket would have to fail, and the drilled shaft would have to mobilize an additional distance. As the amount of upward movement for this to occur is unacceptable, discount the side resistance in the soil.

For friction drilled shafts, calculate group uplift resistance as for piles in Section 1304.4.

See BDM Section 305.4.1.3 for additional detailing and design requirements for drilled shafts subject to uplift.

1306.3 Service Limit State Design

See BDM Section 305.1.3 for differential vertical deflection and horizontal deflection limits for drilled shaft foundations for bridges. See BDM Section 307.1.6 for differential vertical deflection and horizontal deflection limits for drilled shaft walls. For drilled shaft walls with a cast-in-place facing, limit differential vertical deflection to 1/500. For drilled shaft foundations for all other structures, limit total and differential vertical deflections to the values necessary to prevent unacceptable deflections or overstress of the supported structure, in accordance with AASHTO LRFD Articles 2.5.2.6, the Service and Strength Limit State requirements of AASHTO LRFD Sections 5 through 9, or other requirements according to the structural designer. Otherwise, the Service Limit State total vertical deflection limit for drilled shaft foundations is five percent (5%) of the nominal drilled shaft diameter.

1306.3.1 Vertical and Horizontal Deflection

Calculate Service Limit State vertical and horizontal deflections in accordance with BDM Sections 305.1.2 and 305.4.4.5 for drilled shaft structure foundations, and in accordance with BDM Sections 307.1.3, 307.1.4, and 307.1.6 for drilled shaft walls.

1306.3.2 Deflection and Mobilization of Resistance

Drilled shafts are constructed in an at-rest condition with respect to earth pressure, and initially have no mobilized resistance in side friction or bearing pressure. Drilled shafts depend on vertical and horizontal deflections to develop soil reactions and generate geotechnical resistance. This means that unlike driven piles (which have locked-in driving stresses), friction drilled shafts must vertically deflect typically around 2% to 4% of their nominal diameter to develop their maximum (ultimate or nominal) resistance. Downdrag can result in additional vertical deflection even before the geotechnical resistance begins to develop. For this reason, considerations of settlement and deflection of friction drilled shafts is often critical to their use as structure foundations.

Rock-socketed drilled shafts do not deflect much to develop their maximum vertical resistance; the deflection in the rock is typically only a fraction of an inch. Therefore, rock-socketed drilled shaft foundations are preferred to friction drilled shafts.

For both kinds of drilled shaft, it takes considerably more vertical deflection to develop the maximum tip resistance than it does to develop the maximum side resistance (generally on the order of 10 times). In fact, the side resistance will exceed the maximum, and a shear failure will occur on the sides of the shaft before the maximum tip resistance can be achieved. Therefore, for friction drilled shafts, it is important to consider the relative deflection versus load to mobilize the maximum combined value in accordance with AASHTO LRFD Article 10.8.2.2.2 and Figures 10.8.2.2.2-1 through 10.8.2.2.2-4. Use the maximum combined value (at the same value of deflection) as the nominal drilled shaft vertical (axial) geotechnical resistance in the Strength Limit State, and report the nominal and factored resistances of the two components of resistance (side and tip) for the drilled shaft at that value of deflection for inclusion in the plans.

A vertical deflection beyond 5% of the drilled shaft diameter is considered geotechnical failure. If the combined nominal resistance is still increasing at a deflection of 5% of the drilled shaft diameter (e.g., tip resistance in coarse granular soils), limit the value of nominal resistance to that at 5% deflection.

For rock-socketed drilled shafts, typically utilize only the tip resistance or side resistance, but not both, in accordance with Section 1306.1.2. If using both components of resistance, use the full calculated side resistance in accordance with section 1306.4.2, but do one of the following three options:

1) Use the following formulas to estimate the load transfer to the drilled shaft tip based on rock socket diameter and length, rock intact modulus, and RQD. The load transfer ranges from 35% to 0% of the total load applied to the drilled shaft. The remaining percentage taken from 100% (65% to 100%) will be the load transfer to side resistance.

$Q_b/Q_t = 40(0.90^{(L/D)})(L/D)^{-0.333}$, where $E_i \times RQD < 50{,}000$ psi

$Q_b/Q_t = 40(0.81^{(L/D)})(L/D)^{-0.333}$, where $50{,}000 \text{ psi} \leq E_i \times RQD < 500{,}000$ psi

$Q_b/Q_t = 40(0.56^{(L/D)})(L/D)^{-0.333}$, where $E_i \times RQD \geq 500{,}000$ psi

Where:

Q_b/Q_t = Ratio of base load to total load (%)
L/D = Length/Diameter Ratio, where $1.0 \leq L/D \leq 10$
L = Length of drilled shaft socket (ft)
D = Diameter of drilled shaft socket (ft)
E_i = Rock Intact Modulus (psi)
RQD = Rock Quality Designation (%)

If L/D is 1.5 or less, assume 100% load transfer to the base (drilled shaft tip) in accordance with BDM Section 305.4.2; if L/D is greater than 10, assume 0% load transfer to the base.

2) Consult publication NCHRP Synthesis 360, Rock-Socketed Shafts for Highway Structure Foundations, Chapter 3, to estimate the load transfer to the drilled shaft tip based on rock socket diameter and length, drilled shaft modulus, and rock mass modulus. This will provide a similar, but more refined solution to the equations provided in option 1.

3) Perform bi-directional load testing in accordance with BDM Section 305.7.3 on a test drilled shaft to measure the relative side versus tip load transfer, along with an increased resistance factor of $\varphi_{load} = 0.70$ for static load testing in accordance with BDM Table 305-1. If the load test manages to achieve failure in either the side or tip resistance (or both), use the measured unit stress as the nominal resistance for the respective component of resistance. If the measured resistance short of failure exceeds the static calculation of the resistance, this value may also be used for the nominal resistance. Otherwise, use only the relative load transfer of the two components applied to the static calculation of the resistance.

For example, if using option 1 above, where rock socket diameter is D = 4 feet, rock socket length is L = 12 feet, E_i = 68,000 psi, and RQD = 45%:

$E_i \times RQD$ = 68,000 psi × 45% = 30,600 psi < 50,000 psi; therefore,

$Q_b/Q_t = 40(0.90^{(12/4)})(12/4)^{-0.333} = 20.2\%$ load transfer to the base (drilled shaft tip).

This leaves 100% - 20.2% = 79.8% load transfer to the side.

For this example, if we assume static calculations of side and tip resistance result in 15 ksf for nominal side resistance and 270 ksf for nominal tip resistance, total calculated nominal side resistance is 1885 kips (neglecting the top 2 feet in accordance with BDM Section 305.4.2) and nominal tip resistance is 270 ksf × 12.57 sf = 3393 kips. Considering load transfer, the nominal tip resistance is 20.2/79.8 × 1885 kips = 477.2 kips; and the total nominal resistance is $R_P + R_S$ = 477.2 kips + 1885 kips = 2362.2 kips. Total factored resistance is $R_R = \varphi_{qp}R_P + \varphi_{qs}R_S$ = 0.50 × 477.2 kips + 0.55 × 1885 kips = 1275.4 kips. Alternately, given total shear failure of the side of the rock socket, the full tip resistance would be engaged; a rock socket can have no less resistance than this. This makes the total factored resistance $R_R = \varphi_{qp}R_P + 0$ = 0.50 × 3393 kips + 0 = 1696.5 kips.

As can be seen, other than cases with relatively long sockets, or strong rock, it is generally not worthwhile to consider a combination of rock socket side and tip resistance.

1306.4 Strength Limit State Design

Provide a discussion of the design methodology and calculations for side and tip resistance, vertical and horizontal deflections, resistance to overturning, and structural resistance in accordance with BDM 2020 Sections 201.2.1.3.c, 305.1.2, and 305.4 and SGE Section 705.7. Provide the resistance factors and their sources. Provide the factored loads and resistances for inclusion in plan notes, in accordance with Section 1306.1.2.

Do not use presumptive values of bearing resistance from AASHTO LRFD Section 10.6.2.5.1 for the bearing resistance of drilled shaft foundations.

Ignore the effects of self-weight of the drilled shaft foundation and of buoyancy for considerations of bearing resistance. The equations for bearing resistance of deep foundations inherently already include self-weight of the foundation element and buoyancy, as they are based on load tests applied to the top of piles or shafts and are measuring the net resistance of the system including the effects of self-weight and buoyancy.

1306.4.1 Soil Bearing Resistance

Calculate bearing resistance for friction drilled shafts in accordance with AASHTO LRFD Article 10.8.3.5.

For side resistance in cohesive soils, in accordance with AASHTO LRFD Article 10.8.3.5.1, the top 5 feet of drilled shafts are not taken to contribute to the development of resistance through skin friction. However, keep in mind that these 5 feet are measured from the ground surface. Therefore, do not neglect the upper 5 feet if the top of the drilled shaft is buried greater than 5 feet, nor if there is a buried drilled shaft cap above the drilled shaft, if the bottom of the cap extends below the frost penetration depth according to BDM Figure 305-3.

If a permanent steel casing is used, note that the interface shear resistance for steel against cohesionless soil can vary from 50 to 80 percent of the interface shear resistance of poured in place concrete against soil, depending on whether the steel is clean or rusty and whether the soil is dry or saturated. See AASHTO LRFD Articles C10.8.3.5.1b and C10.8.3.5.2b for commentary. Use reduction factors per soil classification in accordance with Table 1300-8 based on Potyondy (1961).

Table 1300-8: Permanent Steel Casing Shear Resistance Reduction Factors

Class	Factor	Class	Factor	Class	Factor	Class	Factor
A-1-a	0.70	A-2-7	0.55	A-4a PL	0.55	A-7-5	0.65
A-1-b	0.75	A-3	0.75	A-4b PL	0.55	A-7-6	0.65
A-2-4	0.55	A-3a	0.80	A-5	0.55	A-8a NP	0.55
A-2-5	0.55	A-4a NP	0.60	A-6a	0.65	A-8a PL	0.55
A-2-6	0.55	A-4b NP	0.60	A-6b	0.65	A-8b	0.55

If a permanent steel casing is driven into the ground by an impact hammer, the bearing resistance can be calculated as for a large-diameter open-ended pipe pile. In this event, consult with the OGE.

1306.4.2 Rock Bearing Resistance

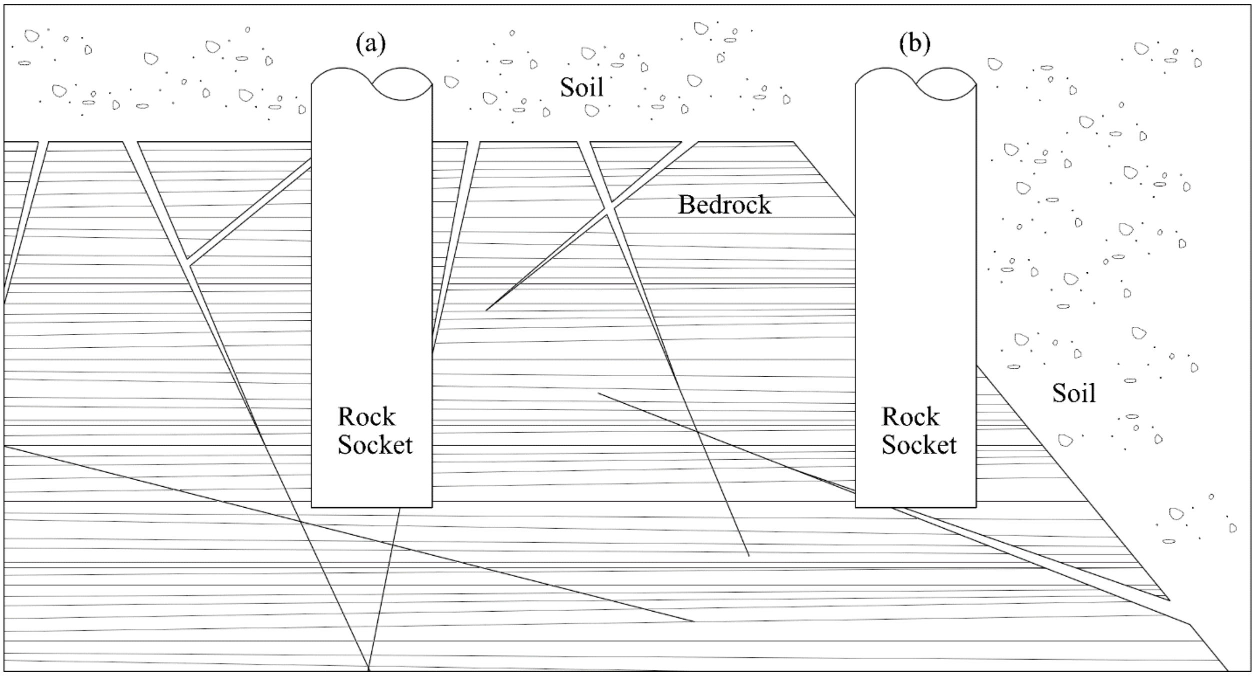
Calculate bearing resistance for rock-socketed drilled shafts in accordance with AASHTO LRFD Article 10.8.3.5.4; typically utilize only the tip resistance or side resistance, but not both, in accordance with Section 1306.1.2. Most of the time this will be only the tip resistance, as it is usually adequate to support the drilled shaft load except in the weakest of rocks. If both tip and side resistance are used, modify each according to deflection and mobilization of resistance in accordance with Section 1306.3.2.

For the calculation of rock socket nominal side resistance, typically use AASHTO LRFD Equation 10.8.3.5.4b-1, where C = 1.0 for normal conditions. Do not use AASHTO LRFD Equation 10.8.3.5.4b-2 for the calculation of nominal side resistance, unless the rock "caves and cannot be drilled without some type of artificial support," in accordance with AASHTO LRFD Article 10.8.3.5.4b. This will typically not be the case in Ohio bedrocks, unless the rock is highly weathered and broken, or decomposed. However, keep in mind that BDM Section 305.4.2 directs to neglect the contribution to skin friction provided by the top 2 feet of the rock socket, which may fully encompass the upper highly weathered zone in most Ohio bedrocks.

For the calculation of rock socket nominal tip resistance, typically use AASHTO LRFD Equation 10.8.3.5.4c-1 ($q_p = 2.5q_u$). Do not use AASHTO LRFD Equation 10.8.3.5.4c-2 (the Hoek-Brown Geologic Strength Index [GSI] method) for the calculation of nominal tip resistance, unless (1) a rock socket length of less than 1.5B is used, where B is the diameter of the drilled shaft; (2) voids or karst or clay filled seams are identified within the bearing stratum; or (3) bedrock with adverse jointing is identified, such that the structure of the joints will control the mode of bearing failure. Case (2) should not occur by design, and case (3) is rare except for when the bedrock surface is steeply sloping. The bedrock in Ohio is generally flat, and except for on Appalachian hillsides, steep preglacial valleys, or the sides of rock cut excavations, the rock at the bottom of a rock socket has nowhere to go in a bearing capacity failure. The Hoek-Brown GSI method is not appropriate for rock socket tip resistance in Ohio bedrock unless there is some structural discontinuity that results in a critical feature controlling the bearing resistance. In most instances, the GSI method is extremely conservative and unrealistic. If using AASHTO LRFD Equation 10.8.3.5.4c-2, justify its use in accordance with BDM Section 305.4.2.

In Figure 1300-14 two cases are presented; in case (a) discontinuities do not control bearing resistance, and in case (b) discontinuities do control bearing resistance. In case (a) the rock is fully confined, and none of the discontinuities leads to a potential bearing failure surface. In case (b) the rock is not confined to the right, and an adverse joint at the base of the drilled shaft intersects the bedrock surface to the right.

Figure 1300-14: Rock Sockets and Rock Discontinuities



Hoek et.al. (2013), Figure 4, and this quote shed additional light on the use of the GSI method, "A fundamental assumption of the Hoek-Brown criterion for the estimation of the mechanical properties of rock masses is that the deformation and the peak strength are controlled by sliding and rotation of intact blocks of rock defined by intersecting discontinuity systems. It is assumed that there are several discontinuity sets and that they are sufficiently closely spaced, relative to the size of the structure under consideration, that the rock mass can be considered homogeneous and isotropic." Since Ohio bedrock is relatively flat bedded, and it rarely includes multiple closely-spaced joint sets – generally not such that controls the failure of the rock under vertical loading from a foundation – the rock is highly anisotropic, and GSI is not usually valid to represent the failure mechanisms that control structure foundations in rock in Ohio.

However, design drilled shafts bearing on top of bedrock, or with a rock socket length of less than 1.5B, using Equation 10.8.3.5.4c-2, which provides a reduced resistance due to lessened bedrock confinement. Whenever a rock socket has a length of less than 1.5B, also exclude all side resistance in accordance with BDM Section 305.4.2. However, a weak rock over a strong rock still counts for confinement and rock socket length (e.g., a 3-foot diameter rock socket with 8 feet in very weak shale and 1 foot in strong limestone still has a socket length of 9 feet, which is greater than 1.5B, and uses Equation 10.8.3.5.4c-1 for tip resistance).

Conversely, it is typically not necessary for a rock socket length greater than 1.5B for resistance to axial loads; however, resistance to lateral loads may dictate a longer rock socket length. If lateral load controls rock socket length, provide calculations demonstrating the minimum drilled shaft length necessary to resist the lateral load in accordance with Section 1306.4.3.

Do not use the previous RMR method, per Karter and Kulhawy (1988), reflected in AASHTO LRFD 6th Edition (2012) and in previous editions, for the calculation of drilled shaft nominal tip resistance. This superseded method uses strength parameters derived from Rock Mass Rating (RMR), while the current Equation 10.8.3.5.4c-2, used since the 7th Edition (2014), uses strength parameters derived from GSI. While it is possible for both methods to produce similar results, the current method is more refined, and the GSI parameters less ambiguous; while the previous method and RMR parameters were more ambiguous and open to mistakes in engineering interpretation.

1306.4.3 Lateral Resistance

When lateral loads are applied to drilled shafts, this can control their length, and require greater embedment than is necessary for bearing resistance. Strength Limit State resistance to lateral loading is termed "overturning resistance," and is checked in accordance with BDM Section 305.1.2 and AASHTO LRFD Article 10.8.3.8 for drilled shaft structure foundations and in accordance with BDM Sections 307.1.3 and 307.1.4 for drilled shaft walls. See FHWA GEC 10, Section 11.5.1, Table 11-1 regarding p-multipliers with a more comprehensive table of values for drilled shafts. Provide calculations demonstrating lateral resistance against overturning and the necessary drilled shaft length and diameter to achieve the necessary resistance. Also provide necessary plan notes and values in accordance with Section 1306.1.2.

1306.4.4 Structural Resistance

Although 4.5 ksi (Class QC 5) concrete is used for drilled shafts, use 4.0 ksi for design purposes per BDM Section 305.4.4. If a stronger concrete is used according to plan, reduce the strength by 10% (assume a strength of 0.90 f'_c in the design). The reason for the strength reduction in design is to account for potential poor placement control during construction, and the inability to inspect the in-place concrete. For structural stiffness, also use 90% of f'_c for the calculation of the drilled shaft modulus of elasticity.

For drilled shafts with an embedded steel section, use the unreduced strength of the steel. For calculations of steel beam shear resistance, the transverse stiffener spacing is zero for a beam embedded in a concrete drilled shaft (the beam is continuously braced).

For soldier pile walls with a steel beam section that extends out of the drilled shaft, assume this portion is unbraced, with an unbraced length equal to the retained height (H). Check flexural resistance in accordance with BDM Section 307.6.3.

If a permanent steel casing is used for a drilled shaft, design the shaft structurally as a concrete-filled steel tube (CFST), in accordance with AASHTO LRFD Articles 6.9.6, 6.12.2.3.3, and 6.12.3.2.2.

Otherwise, analyze a drilled shaft structurally as a tension-controlled or compression-controlled section with spirals or ties in accordance with AASHTO LRFD Article 5.6.2.1, with resistance factors in accordance with AASHTO LRFD Article 5.5.4.2.

1306.5 Extreme Event Limit State Design

In Ohio, due to low seismicity, the Extreme Event I load combination is typically not investigated for foundations. However, there are several instances in which the Extreme Event II load combination is investigated, including vehicle or vessel collision (CT or CV), and most commonly, scour due to the scour check flood. For design for scour, see Section 1306.2.1. In the Extreme Event Limit State, perform all the same checks as done in the Strength Limit State, with the exception of different load and resistance factors, in accordance with AASHTO LRFD Table 3.4.1-1 and Article 10.5.5.3.

1306.6 Computerized Analysis Programs

Much of the analysis for drilled shafts can be done by hand calculation or by specialty-built spreadsheet calculations. However, there are computerized solutions that make the analyses simpler and quicker. While we know of the existence of several alternative programs, we are familiar with SHAFT and LPILE, produced by Ensoft, and know these are the most common software used for this purpose. Therefore, this section will address the use of these programs for the performance of vertical bearing and deflection analyses and p-y lateral resistance and deflection analyses, respectively; if using other software, apply the guidance provided here in principle to the analysis.

1306.6.1 SHAFT Program

The SHAFT program is primarily used to calculate Strength Limit State vertical (axial) geotechnical resistance for drilled shaft foundations in accordance with Sections 1306.4.1 and 1306.4.2.

The SHAFT program also analyzes drilled shaft Service Limit State vertical load versus deflection (with t-z and q-w curves) in accordance with Section 1306.3.1. This can be used to predict the drilled shaft deflection at ultimate load, and it can be used with an iterative analysis to calculate drag load and deflection resulting from downdrag in accordance with Section 1306.2.2.

Note that the SHAFT program has a limit that restricts analysis within 2 drilled shaft diameters of the bottom of the soil profile; therefore, the soil profile must extend at least 2 diameters below the bottom of the drilled shaft being analyzed. If the drilled shaft diameter is increased, ensure the soil model extends to the new applicable limit.

1306.6.2 LPILE Program

The LPILE program is a p-y analysis program, used to calculate Service Limit State lateral load versus deflection for drilled shaft and pile foundations in accordance with Section 1306.3.1. It can also be used to analyze Strength Limit State geotechnical resistance to lateral (overturning) load, in accordance with Section 1306.4.3.

Perform p-y calculations for lateral geotechnical resistance and deflection of drilled shafts both with and without vertical loads from the supported structure. Calculations without the vertical loads applied are often the more critical condition. Application of the vertical load increases the stiffness of the drilled shaft and decreases deflection. When removing vertical loads from the supported structure, also remember to remove lateral load effects from the same source (such as braking or thermal contraction loads).

In a scour analysis, assess the lateral geotechnical resistance and deflection of the drilled shaft foundations with a cantilever length starting at the predicted scour elevation. Keep in mind that if the drilled shafts are not tangent or if the scour progresses beyond a certain depth, lateral loads from retained soil will also disappear as the soil is scoured away. Also, if a bridge approach is scoured away, discount loads relating to live load on the bridge.

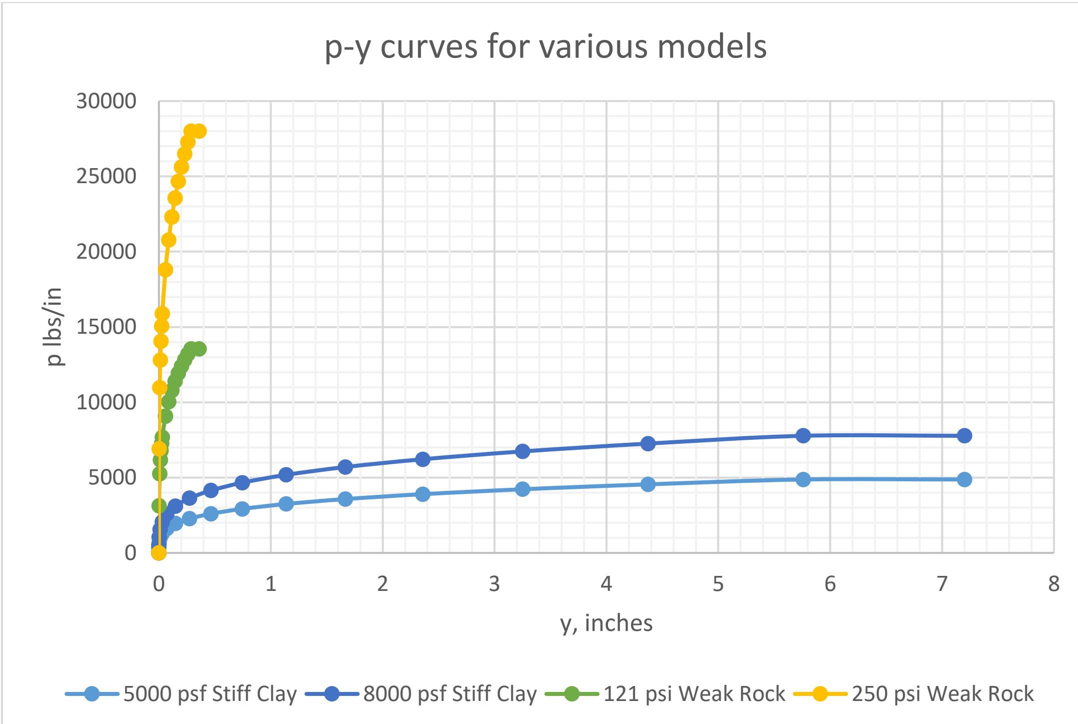
Typically, analyze a drilled shaft as a “Round Concrete Shaft (Bored Pile)” or “Round Concrete Shaft with Permanent Casing” depending on whether it has steel casing left in place as a structural element.

For a drilled shaft with an embedded steel pipe, HP-section, or W-section, analyze the foundation element as a “Round Shaft with Casing and Core/Insert” under “Section Type” in LPILE. Set the casing outside diameter to the nominal drilled shaft diameter, set the casing wall thickness to 0 inches unless a permanent steel casing is used, and set the number of bars to “0” for the rebars unless a supplemental reinforcing bar cage is used. For the core/insert type, choose either “Steel Pipe Section,” “Steel H Section Strong Axis,” “Steel H Section Weak Axis,” “Steel AISC Section Strong Axis,” or “Steel AISC Section Weak Axis,” depending on the type of steel section being used, and depending on the axis of flexure for an HP-section, or W-section. This analysis method accounts for the additional stiffness from the composite action of the concrete contained within the web and flanges of the steel section. It will also produce a non-linear non-elastic analysis that accounts for the loss of stiffness from a cracked section with deflections beyond the tensile strain limit for the concrete.

For a drilled shaft with an embedded steel section other than a pipe, HP-section, or W-section, typically analyze the foundation element as an “Elastic Section (Non-yielding)” under “Section Type” in LPILE. This section type will result in an elastic analysis which uses a constant beam stiffness, which is unaffected by deflection of the deep foundation element. Select the structural shape “Circular without Void” under “Dimensions and Properties” and set the “Elastic Section Diameter” equal to the nominal drilled shaft diameter in order to develop the proper reaction from the soil in LPILE. (Representing the section as a “Steel H Section Strong Axis” or something similar will not develop the proper reaction at the interface between the drilled shaft perimeter and the resisting materials). However, set the moment of inertia and area under “Elastic Section Properties” equal to the actual values of I_X and A_S for the steel section(s). Set the modulus of elasticity equal to that for a steel section alone (approximately 29,000,000 psi), not for a composite section. This analysis method is admittedly conservative for a steel section embedded in concrete, but not enough research has been made into the composite action of concrete and various steel sections to properly represent the behavior as a composite section.

For weak or very weak bedrock, do not model the material as a stiff clay with a high undrained cohesion. As an illustration, see Figure 1300-15 for p-y curves of stiff clay and weak rock models at several different strengths generated in LPILE. The response of even a very weak bedrock is substantially different from that of a stiff clay of any strength.

Figure 1300-15: p-y Curves for Various Models



Do not represent stronger bedrock in LPILE with the "Strong Rock (Vuggy Limestone)" model. This is inappropriate for all bedrock types in Ohio. This model is meant for a particular vuggy limestone formation encountered in south Florida, which exhibits brittle behavior, fracturing at a portion of its full unconfined compressive strength. No rock in Ohio behaves in this manner. If using a version of LPILE prior to v2015, use the Weak Rock model to represent all Ohio bedrocks, per publication FHWA-NHI-18-024, GEC 10. Strong rocks beyond the acceptable limits of the weak rock model will generate non-critical errors in LPILE, which will not invalidate the analysis. If using LPILE v2015 or later, we also recommend the "Massive Rock" model for weaker Appalachian bedrocks, based on the research of Dr. Jamal Nusairat, Ph.D., P.E. (ODOT Research Project 134137, "Design of Rock Socketed Drilled Shafts"), specifically done in Ohio rocks.

The Massive Rock model is generally appropriate for Appalachian shale, claystone, or sandstone bedrocks with Very Weak to Moderately Strong bedrock strength. Per our discussions with Dr. Nusairat, this model does not apply to carbonate bedrocks (Limestone and Dolomite), nor to bedrocks of Strong to Extremely Strong bedrock strength. Per publication FHWA-NHI-10-016, GEC 10, use the "Weak Rock" model for Ohio Bedrocks when the "Massive Rock" model cannot be used. In essence, the "Massive Rock" model produces too "soft" a curve in Strong to Extremely Strong rocks, weakening the elastic response of the rock. The "Massive Rock" model utilizes a hyperbolic curve for the p-y relationship, where the "Weak Rock" model has more of a straight elastic zone with a sudden failure, which would be expected in a strong rock.

SECTION 1400 CULVERTS

1401 GENERAL

Culverts are buried structures, designed to carry the roadway over a linear feature: this is generally a stream, but sometimes over a path or another roadway or railway. Culverts are not bridges; as buried structures, culvert foundations have a higher reliability (modes of failure are fewer and probability of failure is lower than for bridges), and potential foundation movement or failure has a lesser impact on the overlying roadway. Therefore, culverts have different requirements for exploration and design: see SGE Section 303.7.2, BDM Sections 201.1.2.5, 305.2.1.2.b(A), 305.2.3, and 307.1, and ODOT Location and Design Manual, Volume 2, Drainage Design (L&D2) Section 1008.9. Spread footing foundations are permitted for water-crossing culverts in some conditions where they are not for bridges.

However, for a culvert with a span of greater than 20 feet, perform geotechnical exploration as for a bridge in accordance with SGE Sections 303.7.1 and 303.7.2.d. Such culverts are inventoried as bridges per FHWA, and the alternative could change to a bridge during project detailed design.

Multiple three-sided culverts placed side-by-side to produce a multiple span structure over a waterway are considered as bridges and are subject to the limitations on spread footing foundations for bridges in accordance with BDM Section 305.2.1.2.b.

1402 FOUNDATION EXTERNAL STABILITY

Do not perform external stability analyses for culvert pipes or 4-sided culverts; these structures replace a quantity of soil or rock with a structural annulus and a void space, making failure less critical than the previously existing condition. However, if the pipe is to be placed through or under a proposed fill, check total settlement and differential settlement along the length of the culvert run, as this may be necessary for design of the culvert with a cambered profile in accordance with L&D2 Sections 1008.1.3 and 1008.2.1.

For three-sided culverts (arched or flat-topped) check external stability (settlement, bearing, and overall stability) for the linear foundations along both sides of the culvert. Sliding and overturning are not valid failure modes for three-sided culverts; however, calculate the horizontal (at-rest) earth pressure against the sides of the culvert, and the vertical pressure on the top of the culvert for use in structural design.

If a half-height headwall is used, do not check external stability. Exploration and soil or bedrock information is not necessary for half-height headwalls in accordance with SGE Section 303.7.2.b.

There is no minimum limit to foundation soil shear strength for the use of spread footings to support full-height headwalls. However, only use standard full-height headwall designs in accordance with Design Data Sheets HW-1.1 or HWDD-1 if the foundation soil shear strength is greater than or equal to the minimum requirements on these sheets: a minimum of either an undrained shear strength of $S_u = 1500$ psf or a drained friction angle of $\phi_f = 28°$. If a standard full-height headwall design is used in accordance with HW-1.1 or HWDD-1, do not check external stability other than overall (global) stability; if the height of the embankment above the headwall is less than the height of the headwall (dimension H on HW-1.1 or HWDD-1), then the requirement for overall (global) stability may also be assumed met by inspection. Otherwise, if the foundation soil shear strength is less than the minimum requirements on HW-1.1 or HWDD-1, or a custom headwall design is used, check all aspects of external stability for full-height headwalls as for retaining walls, in accordance with BDM Section 307 and AASHTO LRFD Section 11.

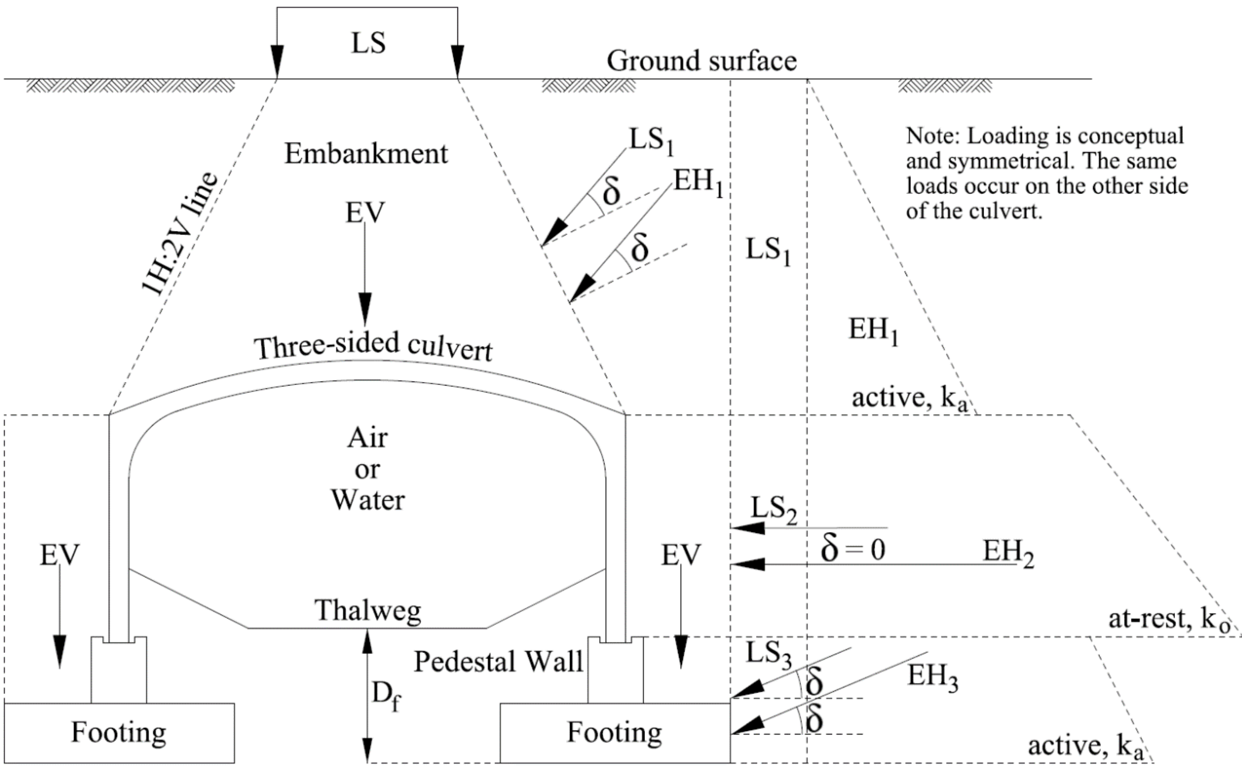
If the foundation soil shear strength is less than the minimum values on HW-1.1 or HWDD-1, consider over-excavation and replacement of the foundation soil under the footing with Granular Material. Specify a lateral limit of the over-excavation and replacement to 1 foot wider than the footing on all sides. The depth of the over-excavation is dependent on the requirements of external stability.

1402.1 Three-Sided Culvert Foundation Loads

For vertical bearing loads on a three-sided culvert foundation, include the dead load weight of components (DC) of the culvert, pedestal wall, and footing. For vertical earth pressure load (EV) on the top of the culvert, draw 1H:2V lines up and inwards from the outside edges of the culvert, and include only that triangle or trapezoidal area of soil above the culvert. If the area is trapezoidal (if the 1H:2V lines intersect the ground surface before forming a vertex), include vertical live load surcharge (LS) within the width of the intersection; otherwise, exclude LS as a vertical load on top of the culvert. Also include the vertical component of the lateral earth pressure (EH and LS) loads acting against this triangle or trapezoid. For vertical earth pressure load (EV) on the top of footings, use the rectangle of soil on top of the footing heel, up to the top of the culvert. Also include the vertical component of the lateral earth pressure (EH and LS) loads acting against the footing and pedestal wall. The lateral earth pressure (EH and LS) loads are inclined against the 1H:2V lines and the footing and pedestal wall at soil-structure interface angle δ downward from the normal to each surface. Use $\delta = \frac{2}{3}\phi$ for soil-on-soil or for soil-on-cast-in-place concrete in accordance with BDM Section 307.1.1 and AASHTO LRFD Article C3.11.5.3. Do not incline the horizontal (at-rest) earth pressure against the sides of the culvert ($\delta = 0$). If the top of the culvert is 30 feet or greater below the overlying road surface, exclude the LS load acting against the sides of the 1H:2V lines and the footing and pedestal wall. See Figure 1400-1 for a graphical representation of the loading.

For a three-sided culvert on spread footing foundations, analyze bearing resistance in accordance with BDM Section 307.1.5. Consider lateral earth pressure only against the footing and pedestal wall in the calculation of eccentricity of load; do not consider lateral earth pressure against the three-sided culvert. The footing embedment depth (D_f) is the depth of the footing below the Thalweg. In the case of scour, the effective D_f will become the depth of the footing below the total scour elevation. See Section 1403 for more details.

Figure 1400-1: Three-Sided Culvert Foundation Loading



The lateral thrust force outward from the legs of three-sided culverts can be insignificant for flat-topped or precast arch structures with vertical legs. However, it can be more significant for plate arch structures, two-part precast arches with a central joint, or precast shapes with a flatter profile. Nevertheless, the passive earth pressure outside the arch and foundations is more than adequate to resist the lateral thrust force from any of these structures. See Section 1404 for a discussion. Conversely, the active earth pressure load external to the arch is not enough to overcome the combination of the structural strength of the arch, the lateral thrust force, passive resistance of the interior soil, and friction forces at the bottom of the footing. For a significant differential lateral load to transmit to the foundation would require a structural failure of the three-sided culvert or significant scour.

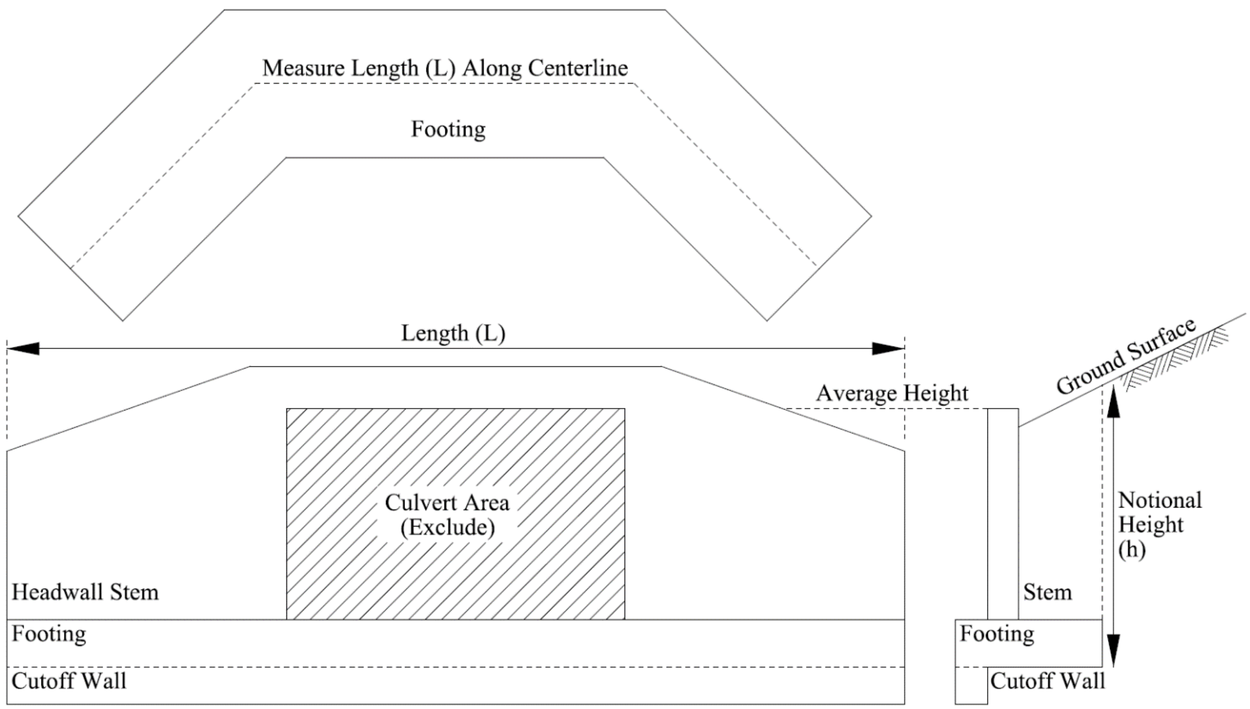
Accordingly, do not specify battered piles for the support of a three-sided culvert. Do not perform a lateral load analysis for the foundation elements for a three-sided culvert on deep foundations unless scour progresses below the bottom of the footing. See Section 1403 for more details.

1402.2 Full-Height Headwall Loads

For full-height headwalls, loading is generally in accordance with BDM Section 307.1.1 and AASHTO LRFD Article 11.6.1.2 as for retaining walls. However, headwalls are generally shorter in length than a grade-separation wall or a bridge abutment and are analyzed as a single unit. As the height of a headwall often changes over the length of the wingwalls, use the average height of the wall over its length for the calculation of lateral earth pressure loads. If there is a sloped fill behind the headwall, use this average height as the height of the stem, and extend the vertical line for the notional height of earth pressure diagram (h) from the back of the heel to the ground surface at the location of the average wall height. Do not include the cutoff wall in the notional height. Exclude any earth pressure loading within the area (the outside dimensions) of the culvert.

For considerations of loading and external stability, measure the length (L) of the wall and foundation along the centerline of the foundation. If there are bends in the headwall, measure the length around the bends, but conservatively ignore the bends in the loading and external stability analyses (treat the headwall as a straight wall with the length L). See Figure 1400-2 for a definition of the full-height headwall dimensions to be used for loading.

Figure 1400-2: Full-Height Headwall Dimensions for Loading



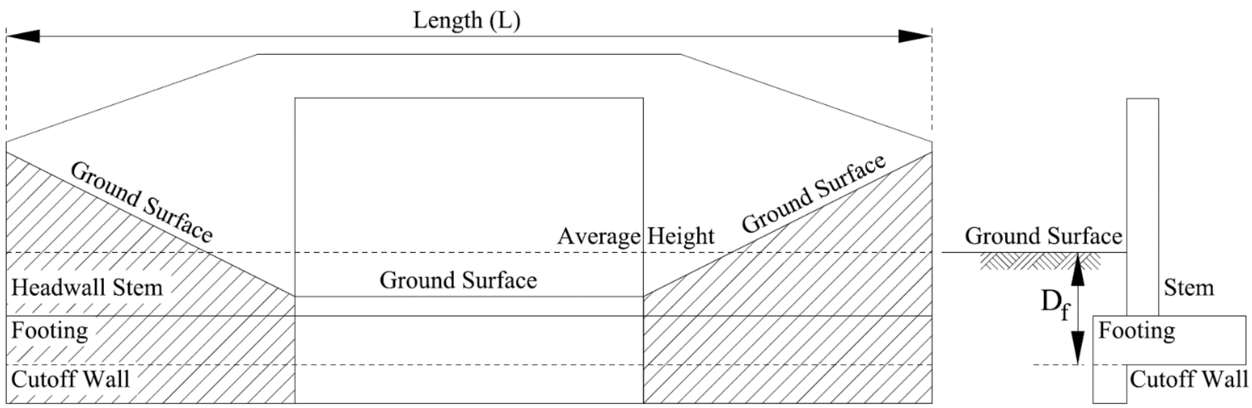
If the wingwalls and foreslope wall are cast separately, and are not structurally connected, consider their external stability individually. In this case, use the average height of each individual wall element for calculation of lateral earth pressures.

If any part of the headwall (including the heel of the footing) is within a horizontal distance of one-half the notional height (h) of traffic, include the live load traffic surcharge (LS) load on the back of the vertical surface at the location of the notional height

1402.3 Full-Height Headwall Sliding Resistance

For full-height headwalls on spread footings, analyze resistance to lateral loads with a combination of foundation sliding resistance and passive resistance, in accordance with BDM Sections 305.2.1.3 and 307.1.3 and AASHTO LRFD Article 10.6.3.4. However, for passive resistance, use the full depth of soil from the ground surface to the bottom of the cutoff wall only in the areas outside the channel bottom (to either side of the culvert conduit); do not exclude the frost depth for passive resistance. The areas for passive resistance are shown graphically as the hatched regions in Figure 1400-3. The area in the center, under the channel bottom, is conservatively excluded for passive resistance since scour may remove the resisting soil.

Figure 1400-3: Full-Height Headwall Passive Resistance and Footing Depth



The passive earth pressure (ep) load is inclined against the front face of the footing and cutoff wall at soil-structure interface angle δ upward from the normal to the front face of the footing. Use $\delta = \frac{2}{3}\phi$ for soil-on-soil or for soil-on-cast-in-place concrete in accordance with BDM Section 307.1.1 and AASHTO LRFD Article C3.11.5.3. Calculate the passive earth pressure coefficient in accordance with AASHTO LRFD Article 3.11.5.4 and use resistance factor $\varphi_{ep} = 0.50$ for the passive resistance.

When analyzing sliding resistance for a spread footing with over-excavation and replacement with granular material, analyze sliding resistance first at the top of the granular material, with the footing against the granular material. Use $\phi_f = 32°$ for Granular Material Type B and $\phi_f = 34°$ for Granular Material Type C in accordance with BDM Table 307-1; include passive resistance of the existing foundation soil to the bottom of the shear key. Also analyze sliding resistance at the bottom of the granular material, with the granular material against the existing foundation soil; include passive resistance of the existing foundation soil to the bottom of the granular material.

For full-height headwalls supported on deep foundations, analyze resistance to lateral loads for the deep foundation elements in accordance with BDM Sections 305.1.2 and 305.2.1.3. The service limit deflection criterion is 1% of the maximum retained height (H) of the headwall in accordance with BDM Section 307.1.6. Battered piles may be used for the support of full-height headwalls.

1402.4 Full-Height Headwall Eccentricity and Overturning Resistance

For full-height headwalls on spread footings, calculate eccentricity of load and effective footing width for the headwall footing in accordance with AASHTO LRFD Article 10.6.1.3, and check limiting eccentricity in accordance with BDM Section 307.1.4 and AASHTO LRFD Article 11.6.3.3. Calculate eccentricity from the center of the bottom of the footing, ignoring the cutoff wall. However, include the weight of the cutoff wall and the passive resistance load in the calculation of eccentricity. Angle the passive resistance load upwards in accordance with Section 1402.3 and use the resistance factor for passive resistance ($\varphi_{ep} = 0.50$) as the load factor for this load. However, for this calculation, do not use a factored passive resistance that is more than the factored active earth pressure (EH and LS combined); if the factored passive resistance calculates to more than the factored active earth pressure, scale it down to be equal.

For full-height headwalls supported on deep foundations, analyze resistance to overturning as for retaining walls with deep foundations in accordance with BDM Sections 305.1.2 and 307.1.4.

1402.5 Full-Height Headwall Bearing Resistance

For full-height headwalls on spread footings, calculate bearing resistance for the wall footing in accordance with BDM Sections 305.2.1 and 307.1.5. The footing embedment depth (D_f) is the depth of the footing below the average ground surface elevation over the length of the wall; see Figure 1402-3 for a definition of D_f. Include the weight of the cutoff wall and the passive resistance load in the calculation of eccentricity and effective footing width in accordance with Section 1402.4.

When analyzing bearing resistance for a spread footing with over-excavation and granular material replacement, use one of the following three methods:

1) Perform a bearing resistance analysis, in accordance with BDM Sections 305.2.1 and 307.1.5, at the bottom of the granular material, treating the granular material as an additional thickness of the footing. Use a unit weight of 120 pcf for Granular Material Type B, or use a unit weight of 130 pcf for Granular Material Type C, in accordance with BDM Table 307-1.

2) Perform a bearing resistance calculation for a two-layer soil system, in accordance with AASHTO LRFD Article 10.6.3.1.2, treating the granular material as the upper layer, and the existing foundation soil as the lower layer.

3) Perform a bearing resistance analysis using limit equilibrium methods, in accordance with AASHTO LRFD Article C10.6.3.1.2c, or with finite element methods. Model all soil strata, including the granular material, with individual shear strength and unit weight properties.

For full-height headwalls supported on deep foundations, analyze bearing resistance for the deep foundation elements in accordance with BDM Sections 305.3 and 305.4.

1403 CULVERT SCOUR

Scour does not occur inside closed-conduit culverts (pipes or four-sided boxes). Scour can compromise the stability of the headwalls at the inlets and outlets; however, no scour analyses are run for the inlet and outlet structures for closed-conduit culverts. Instead scour countermeasures are specified for installation based on the flow velocity at the conduit outlet in accordance with L&D2 Section 1002.2.3 and Figure 1002-4.

Check scour for three-sided culverts as for a single-span bridge, with long-term degradation, contraction scour, and local abutment scour considered in accordance with L&D2 Section 1008.10.3. Increased turbulence at the corners of the structure (where the conduit connects with the headwall or the approach channel) often results in more critical scour occurring at these locations. Publication FHWA-HIF-12-003 (HEC 18) "Evaluating Scour at Bridges," Section 6.9 presents equations for calculation of depth of combined contraction plus local scour at the corners of the structure. Long-term degradation scour must still be added to get the total scour depth.

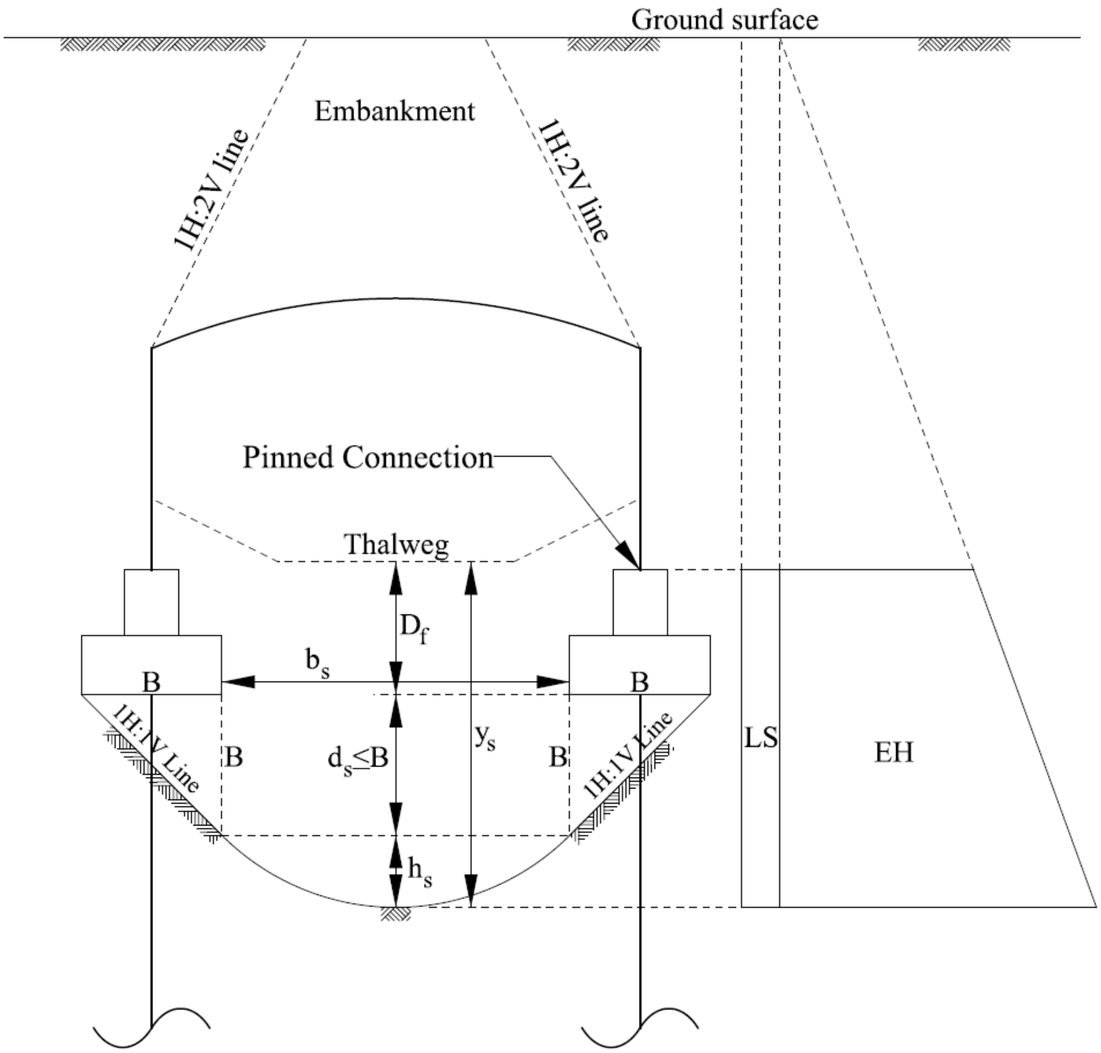
Spread footings bearing on soil for the support of three-sided culverts are allowed within the limits of the 100-year flood plain, in accordance with BDM Section 305.2.1.2.b(A). However, place the bottom of the spread footing a minimum of 2 feet below the predicted scour elevation (either below the total scour or below the predicted scour from the time-rate of scour analysis, for either the scour design flood or scour check flood, whichever is deeper). If a scour depth of greater than 4 feet below the thalweg is predicted, or if placement of the spread footings below the predicted scour elevation is not feasible, then place the structure on deep foundations. In this case, design the deep foundation elements for the scour conditions in accordance with BDM Sections 305.1.6, 305.3.2.1, and 305.4.1.1. Consider the length of the deep foundations exposed down to the scour elevation as an unsupported length; bending and buckling will be the failure modes of concern.

For spread footings bearing on bedrock supporting three-sided culverts, follow the requirements of Section 1302.3 and BDM Section 305.2.1.2.b.

While there is no net lateral load on the deep foundation elements in the as-constructed condition, a lateral load can develop on the foundations as the result of scour. In the first stage of scour, if the predicted scour elevation remains above the bottom of footing elevation, continue to assume no net lateral load on the deep foundations. However, in the second stage of scour, once the predicted scour elevation advances below the bottom of the footing, begin to analyze the deep foundation elements for lateral load.

In the second stage of scour, soil is still retained behind the foundations, and there will be a net lateral load from active earth pressure directed inwards, toward the middle of the culvert; see Figure 1400-4.

Figure 1400-4: Second Stage of Scour, $0 < d_s \leq B$



For scour to be in the second stage, the following equation must be satisfied:

$D_f < y_s \leq D_f + B + h_s$

Where:

D_f	= depth of the footing (below the original Thalweg)
y_s	= depth of scour (below the original Thalweg)
d_s	= depth of substrate/stream bottom below bottom of footing elevation
B	= footing width
h_s	= height of substrate/stream bottom = $0.2071\ b_s = b_s\,(1/\sqrt{2} - ½)$
b_s	= width between footings

The full vertical load on the foundations, in accordance with Section 1402.1, is still present in the second stage of scour. If the top of the pedestal (or foundation, if there is no pedestal) is less than

30 feet below the overlying road surface, include live load surcharge (LS) in the active lateral earth pressure load against the deep foundations, otherwise exclude LS. For the active earth pressure loading against the deep foundations, do not incline earth pressure loads at angle δ (assume $\delta = 0$). Applying a vertical load component of earth pressure to deep foundations will tend to increase the stiffness response of the cantilever members and is considered unconservative. In this condition, consider the top of the foundation to have a pinned connection (free to rotate, but not free to translate); the combination of lateral earth pressure, structural strength of the arch, and the lateral thrust force prevents lateral displacement of the top of the foundation. Consider the deep foundation elements to be standing unsupported to the bottom of y_s.

In the third stage of scour, where $d_s > B$, the soil is eroded from behind the deep foundations, and the three-sided culvert becomes a free-standing structure. The Designer should focus on avoiding this scenario rather than designing for it. Such a design is a special case scenario beyond the scope of this manual. Do not design the foundations to accommodate this condition without consultation with OGE.

1404 THREE-SIDED CULVERT LATERAL THRUST FORCE

Assuming scour will not affect the soil behind the culvert wall and foundation, there is no need to design the foundation to resist the lateral thrust force because the passive resistance will be more than adequate. If scour reaches the third stage, as described in Section 1403, there will be no passive resistance to counteract the thrust force, and the foundation will need to be designed to resist it. However, this design scenario is beyond the scope of this Manual.

SECTION 1500 RETAINING WALLS

1501 GENERAL DISCUSSION

A retaining wall is a structure that supports a differential height of earth on either side or retains earth laterally. Design retaining walls, including bridge abutments, wing walls, and culvert headwalls, in accordance with BDM Section 307 and AASHTO LRFD Section 11. See BDM Sections 200 and 300 for additional design requirements of retaining walls.

Shallow foundation walls are defined as the retaining wall types listed in Sections 1502 (rigid gravity and semigravity walls), 1503 (prefabricated modular walls), 1504 (MSE walls), 1506.2 (cellular sheet pile walls), 1508 (soil nail walls), 1509.1 (wire faced MSE walls), and 1509.2 (fabric wrapped walls). Nongravity cantilever walls are defined as the retaining wall types listed in Sections 1505 (drilled shaft walls) and 1506.1 (cantilever sheet pile walls). Deep foundation walls include the retaining wall types listed in Sections 1505 (drilled shaft walls), 1506.1 (cantilever sheet pile walls), and 1507 (anchored walls); the retaining wall types in Section 1502 (rigid gravity and semigravity walls) can also be converted to deep foundation walls by the inclusion of piles or drilled shafts.

Use BDM Table 307-1 for retaining wall fill soil design parameters for internal and external stability. Determine soil parameters for the foundation soil or bedrock materials based on the in-situ conditions encountered by the soil borings. This guidance is for fill soils that do not exist at the time of wall design. Walls supporting existing materials should be designed to support the existing conditions.

- MSE reinforced soil, also known as Select Granular Backfill (SGB) in accordance with SS840.03.E, is placed around and for 2 feet behind the soil reinforcements for the retaining wall types listed in Sections 1504 (MSE walls) and 1509.1 (wire faced MSE walls). The limits of SGB are shown on the plans and define the limits of wall quantities versus embankment quantities. In design, the SGB is assumed to end at the limit of the soil reinforcements (the reinforced soil zone), and this defines the back pressure surface of the wall.

- Retained soil is the soil behind the wall heel for the retaining wall types listed in Section 1502 (rigid gravity and semigravity walls); behind the wall modules or the wall drainage for the retaining wall types listed in Section 1503 (prefabricated modular walls); behind the reinforced soil zone for the retaining wall types listed in Sections 1504 (MSE walls) and 1509.1 (wire faced MSE walls), and 1509.2 (fabric wrapped walls); behind the back face of the vertical wall elements or the wall drainage for the retaining wall types listed in Sections 1505 (drilled shaft walls), 1506 (steel sheet pile walls), and 1507 (anchored walls); or behind the end of the soil nails for the retaining wall types listed in Section 1508 (soil nail walls). Retained soil is natural in situ soil for cut walls or Item 203 embankment for fill walls, generally consisting of on-site soils varying from sandy lean clay to silty sand, per C&MS 703.16.A, unless a different backfill is specified in the project plans. Retained soil is the soil that produces active earth pressure against the back pressure surface of the wall.

- CIP or precast semigravity wall infill is the material placed behind the wall stem, and above the footing heel for the retaining wall types listed in Section 1502 (rigid gravity and semigravity walls). For CIP rigid gravity and semigravity walls (Sections 1502.1, 1502.2, and 1502.3), use of specific wall infill is optional, and walls may be infilled/backfilled with any material meeting the requirements of C&MS 703.16.A, with the same design parameters and material properties as retained soil (above). For precast semigravity walls (Section 1502.4), use of specific wall infill, in accordance with SS851.03.B, is required. When used, the limits of specific wall infill are shown on the plans and define the limits of wall quantities versus embankment quantities. In some cases, precast semigravity walls may require that the specific wall infill extends back from the heel at an angle encompassing the active wedge, to provide additional external stability.

- Prefabricated modular wall infill, in accordance with SS870.03.C, is the material placed between or within the prefabricated wall modules (inside the bins, cribs, or hollow blocks) for the retaining wall types listed in Section 1503 (prefabricated modular walls).

- Backfill behind abutments and below the approach slabs, conforming to 703.16.C, Granular Material, Type B, is placed in accordance with C&MS 503.08.

- Headwall granular base, conforming to 703.16.C, Granular Material, Type B, is placed to backfill excavations of unsuitable material under culvert headwall footings in accordance with C&MS 602.02.

- MSE foundation preparation, in accordance with SS840.03.G, is placed below the MSE reinforced soil zone and MSE leveling pad for the for the retaining wall types listed in Sections 1504 (MSE walls) and 1509.1 (wire faced MSE walls).

1501.1 Loading

The predominant, and generally the most significant, source of loading on retaining walls is horizontal (lateral) earth pressure, EH, which serves to make the wall more critical for failure in sliding or overturning. This includes earth pressure surcharge loads, such as LS and ES, that serve to uniformly increase the magnitude of the earth pressure load. The increase in load eccentricity associated with the overturning load can also serve to make the wall more critical for failure in bearing.

Vertical earth pressure, EV, dead load of components, DC, and vertical live load surcharge on top of the retaining wall, LS, can act to increase the vertical load on the wall foundations. This will make the retaining wall more critical for failure in bearing but will often serve to make the wall more stable against failure in sliding or overturning.

Other external loads, such as wind load, WS, vehicular collision load, CV, directed against traffic barriers on the retaining wall, and loads from supported structures (especially for bridge abutments), can act to either destabilize or stabilize the wall. When transient loads act to stabilize the wall, ignore these load effects (set their magnitude to zero). If it is uncertain whether a single, transient load will either serve to stabilize or destabilize the retaining wall, perform the calculations both ways (with the maximum magnitude and load factor, and with zero magnitude).

1501.1.1 Earth Pressure Coefficient

Generally, use the Coulomb earth pressure theory to calculate the active earth pressure coefficient (k_a) for horizontal (lateral) earth pressure loads, in accordance with AASHTO LRFD Article 3.11.5.3. However, when using the loading diagrams in AASHTO LRFD Article 3.11.5.6, use Rankine earth pressure theory, as the Broms and Teng loading diagrams were derived using Rankine theory. When using the loading diagrams in AASHTO LRFD Article 3.11.5.7, use Apparent Earth Pressure (AEP) theory.

The inputs into the Coulomb active earth pressure equation are soil internal friction angle (ϕ), soil-structure interface angle (δ), backslope angle from the horizontal (β), and batter angle of the back pressure surface of the wall from the horizontal (θ) or from the vertical (α). The backslope angle could be flat ($\beta = 0$), an (equivalent) infinite backslope ($\beta > 0$), or a complex backslope, which uses an effective backslope angle ($\beta' \geq 0$) calculated by the methods described in Section 1501.1.5. The broken backslope, where there is a short slope with a level surface above, is a special case of the complex backslope also described in Section 1501.1.5.

Use the at-rest earth pressure coefficient (k_o), in accordance with AASHTO LRFD Article 3.11.5.2, to compute the horizontal earth pressure loads for retaining walls that are restrained from deflecting freely. Examples of retaining walls restrained from deflecting freely include abutments of rigid frame bridges, abutment walls keyed to the superstructure, back-to-back walls sharing the same footing, and some types of U-abutments. Refer to AASHTO LRFD Commentary C3.11.1 for the magnitude of movements to reach active or passive earth pressures.

Generally, calculate the at-rest earth pressure coefficient in accordance with AASHTO LRFD Article 3.11.5.2. However, for sloped backfills (backslope angle, $\beta \geq 0$), calculate the at-rest earth pressure coefficient as described in Engineer Manual, EM 1110-2-2502, Retaining and Flood Walls, Section 3-10, in accordance with the Danish Code (Danish Geotechnical Institute 1978):

$$k_{o\beta} = (1 - \sin \phi')(1 + \sin \beta)$$

Do not calculate the passive earth pressure coefficient (k_p) using the Coulomb equation, as this is considered unconservative; use the Caquot and Kerisel method as defined in AASHTO LRFD Figures 3.11.5.4-1 and 3.11.5.4-2.

For calculation of lateral earth pressure coefficients, always use the effective (drained) friction angle (ϕ') in the Coulomb, Rankine, or Caquot and Kerisel equations or charts. Do not use an undrained case with $\phi = 0$ for the calculation of lateral earth pressure coefficients.

1501.1.2 Earth Pressure Soil-Structure Interface Angle (δ)

Incline the active and passive earth pressure loads on retaining walls at an angle from a perpendicular to the pressure surfaces of the wall equal to the soil-structure interface angle (δ). Use the following values of δ for the following wall interfaces; see Figure 1500-1:

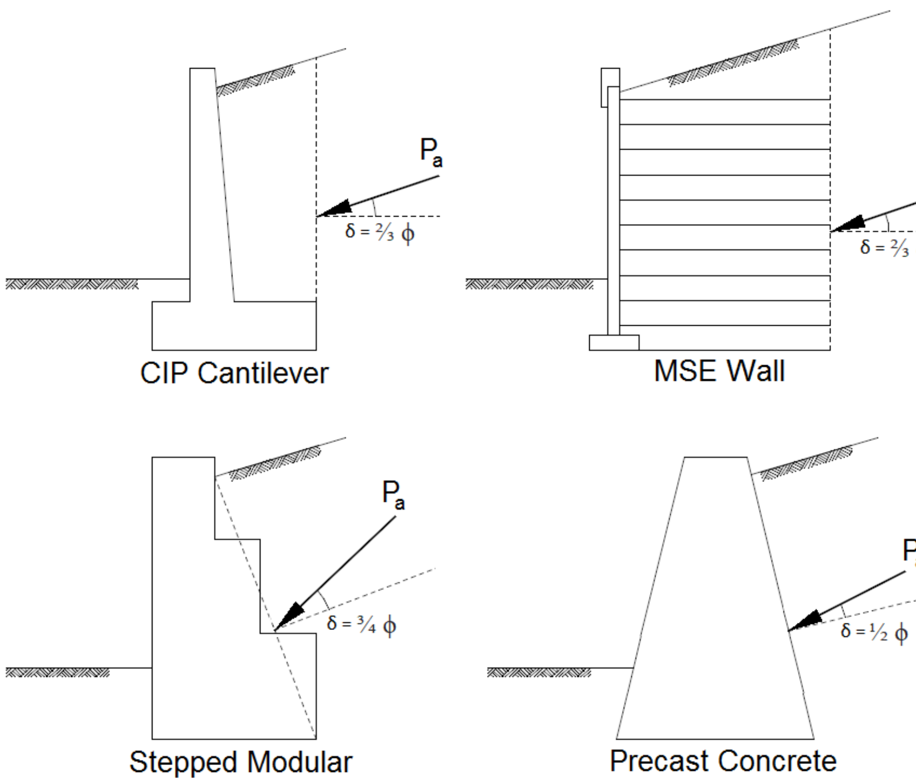
- $\delta = \frac{2}{3}\phi$ for soil-on-soil or for soil-on-cast-in-place concrete;
- $\delta = \frac{1}{2}\phi$ for soil-on-precast concrete;

- $\delta = \frac{1}{3}\phi$ for soil-on-steel; and

- $\delta = \frac{3}{4}\phi$ for soil-on-prefabricated modular wall stepped modules, where ϕ is the internal friction angle of the retained soil.

See AASHTO LRFD Commentary C3.11.5.3 and Table C3.11.5.9-1 for the source of the specified values for δ. A soil-on-soil interface is typically assumed for external stability of semigravity walls regardless of whether the wall stem is composed of cast-in-place or precast concrete, since there is a soil backfill placed above the footing heel which defines the location of the back pressure surface of the wall. See Figure 1500-1 for graphical examples of how to apply the angle δ on the back pressure surface of different wall types. See Figures 1500-1, 1500-3 (c), and 1500-9 for examples of prefabricated modular wall stepped modules.

Figure 1500-1: Interface Angle (δ) for Various Wall Types



Active earth pressure will act against the back pressure surface of the wall at the angle δ ***downwards*** from the normal to the back pressure surface.

Passive earth pressure will act against the front of the wall at the angle δ ***upwards*** from the normal to the front pressure surface. The front pressure surface will generally be a vertical surface at the front face of the wall for most walls, and at the toe of the footing for walls with footings. For walls with a permanent front batter (e.g., rigid gravity walls with a trapezoidal cross section), the front pressure surface will be battered at the same angle as the front of the wall.

At rest earth pressure will act against the back or front pressure surface of the wall at the normal to the pressure surface ($\delta = 0$).

For analysis of earth pressure loading on nongravity cantilever walls and anchored walls, do not incline earth pressure loads at angle δ (assume $\delta = 0$). This is a conservative assumption, providing the maximum lateral loading. Applying vertical load components to these types of walls will tend to increase the stiffness response of the vertical wall elements, increasing their bending resistance, and is considered unconservative. These types of walls are flexible over their length (deflecting back-and-forth) or will rotate backwards at the toe. Research has shown that there is little vertical movement of the retained soil against the back of the wall, except for near the ground surface with large wall movements, and the soil movement does not produce significant vertical loads.

When factoring any inclined load into horizontal and vertical components for mathematical convenience in the calculations, do not use separate maximum and minimum load factors for the respective components of the load. Treat each inclined load as a single load, which either serves to stabilize or destabilize the structure, then apply the same load factor (either minimum if the load stabilizes, or maximum if the load destabilizes) to both the horizontal and vertical components of the load. Inclined loads do not have actual horizontal and vertical components; these are a mathematical convenience to make the stability computations easier to calculate. If it is uncertain whether a single, inclined load will either serve to stabilize or destabilize the structure, perform the calculations both ways (with the minimum load factor, and with the maximum load factor).

1501.1.3 Earth Pressure Surcharge Loads

Earth pressure surcharge loads increase the magnitude of the earth pressure load uniformly, adding a rectangular load distribution to the typical triangular earth pressure load distribution for the height of the back pressure surface of the wall.

Use Earth Surcharge (ES) for earthen surcharge loads (such as a temporary earthen surcharge for consolidation) or for additional walls or earthen structures supported above the height of the wall (such as tiered walls). For tiered wall systems, use a magnitude of ES load equal to the unit weight of the backfill soil times the height of the upper wall (or walls) above the top of the lower wall, as long as the upper wall is within a horizontal distance of one-half the height (H) of the lower wall behind the back pressure surface of the lower wall. See Figures 1500-3, 1500-4, and 1500-9 for a definition of H.

Do not treat a block of soil at the top of a wall (e.g., a sloped soil backfill on top of an MSE wall) as an earth surcharge (ES) load with load factor γ_{ES} if the wedge is located within the notional height of the back pressure surface of the wall (h), in front of the back pressure surface (W_2 as shown in Figure 1500-3). Instead, use vertical earth pressure (EV) for this load. The uncertainty attached to load factor γ_{ES} is higher than for γ_{EV} because the mechanism of load transfer to the wall is less certain.

If the horizontal distance between the back pressure surface of the wall and traffic loading is less than or equal to ***half*** the notional height of the back pressure surface of the wall (h/2), apply a vehicular live load surcharge (LS) loading on the wall in accordance with AASHTO LRFD Article 3.11.6.4, with an equivalent unit weight γ_{eq} = 125 pcf, and with an equivalent height h_{eq} in accordance with Table 3.11.6.4-1 or Table 3.11.6.4-2, respectively, depending on whether the direction of traffic is perpendicular or parallel to the face of the wall.

If an anchored wall is within a horizontal distance from the back pressure surface of the wall of ***half*** the notional height of the back pressure surface of the wall (h/2), include LS in addition to the apparent earth pressure (AEP) calculated according to AASHTO LRFD Article 3.11.5.7. Add additional surcharge loads and water loads to the AEP, as applicable.

If the wall will have construction equipment supported on the retained soil within a horizontal distance from the back pressure surface of the wall of ***half*** the notional height of the back pressure surface of the wall (h/2), then provide a LS load with a magnitude according to the weight of the supported equipment. Do not use an equivalent unit weight less than γ_{eq} = 125 pcf.

When a retaining wall is located within the influence zone of a railroad track, within a horizontal distance from the back pressure surface of the wall of the ***full*** notional height of the back pressure surface of the wall (h), also apply a LS load, but consult with the applicable railroad company regarding the specific loading requirements. See Chapter 15, Part 1 of the AREMA Manual for Railway Engineering in reference to the Cooper E 80 load.

For a railroad track running parallel to the face of a wall, a Cooper E 80 load on a single track equates to a Live Load Surcharge of an 1882-psf strip load, 8.5-ft wide. If there are multiple supported tracks, then overlap the individual track strip loads by superposition. Note that the lateral earth pressure distribution for a strip load running parallel to the face of a wall is calculated in accordance with AASHTO LRFD Figure 3.11.6.2-1.

If a railroad track runs perpendicular to the face of the wall (e.g., a railroad bridge abutment wall), distribute the single-track load as a rectangular LS load over a width of the back pressure surface of the wall equal to the notional height of the back pressure surface of the wall (h) plus 8.5 feet. If there are multiple supported tracks, distribute the sum of the individual track loads over the full width of the back pressure surface of the wall.

1501.1.4 Earth Pressure Load Distribution

Represent active earth pressure (EH) loading on the wall as a triangular distributed load over the notional height of the back pressure surface of the wall (h) with a value of zero at the ground surface and a maximum at the bottom depth of the notional height of the back pressure surface of the wall. The actual load distribution is complex, and impossible to determine without direct measurement or back-calculation through measurement of displacements; however, a triangular load distribution is a close enough approximation of the actual condition to develop a realistic calculation of distributed shear, moment, and displacement of the wall.

The notional height of the back pressure surface of the wall (h) will extend from the ground surface to the bearing elevation for shallow foundation walls; for these walls, the bearing elevation can be considered synonymous with the design grade. The notional height of the back pressure surface of the wall (h) will extend from the ground surface to the elevation of the design grade in front of the wall for deep foundation walls.

Passive earth pressure (ep) is also represented as a triangular distributed load over the front pressure surface of the wall, with a value of zero at the ground surface and a maximum at the bottom depth of the front pressure surface; however, ignore passive resistance down to the frost depth or probable depth of disturbance, in accordance with BDM Section 305.2.1.2.a(E) and AASHTO LRFD Articles 10.6.3.4 and 11.6.3.5. This means that the remaining passive resistance below this depth will have a trapezoidal distribution. If utilities or pavement will exist within 6 feet of the front face of the wall, assume a minimum depth of soil disturbance of 3 feet.

Similarly, for integral or semi-integral abutment bridges, the shallow portion of the active earth pressure will act against the diaphragm and the horizontal component will be carried by the superstructure across to the end of the span. Therefore, neglect the horizontal component of active earth pressure loading within the height of the diaphragm. The remaining active earth pressure will have a trapezoidal distribution from the beam seat to the bottom of the back pressure surface of the wall. However, the vertical component of the earth pressure above the beam seat, acting on the diaphragm, will be transmitted down through the bearing into the abutment wall.

For nongravity cantilever walls, do not represent the load on the vertical wall elements as a single resultant point load, and "cut off" the top of the vertical wall elements at the point of application of this resultant load. This method does not realistically predict either the shape or magnitude of moment and shear distributions and cannot predict the displacement at the head of the vertical wall elements.

In AASHTO LRFD Figures 3.11.5.6-1, 3.11.5.6-4, or 3.11.5.6-5 the equations for the passive earth pressure (P_p) do not apply if the discrete vertical wall elements are spaced at closer than three diameters (3b) center-to-center; these equations assume the passive earth pressure to be distributed over a longitudinal distance of 3b. The zones of passive resistance for each discrete vertical wall element cannot overlap. Therefore, for closer-spaced elements, the load must be distributed over a longitudinal distance equal to the center-to-center spacing between the discrete vertical wall elements (l). If the foundation soil is very loose to loose granular or very soft to soft cohesive material, use the passive resistance only over a width of b. In soft or loose soils, full soil arching is assumed to not develop. See AASHTO LRFD Commentary C3.11.5.6-1 for other restrictions.

In AASHTO LRFD Figures 3.11.5.6-4 through 3.11.5.6-7, if $\gamma_s H < 2S_u$, use a value of zero (0) for the active earth pressure in the portion of the wall below the retained height (P_{a2}). The active earth pressure in the portion of the wall below the retained height (P_{a2}) could result in a negative value if $\gamma_s H - 2S_u < 0$; however, it is not possible to have a negative active earth pressure.

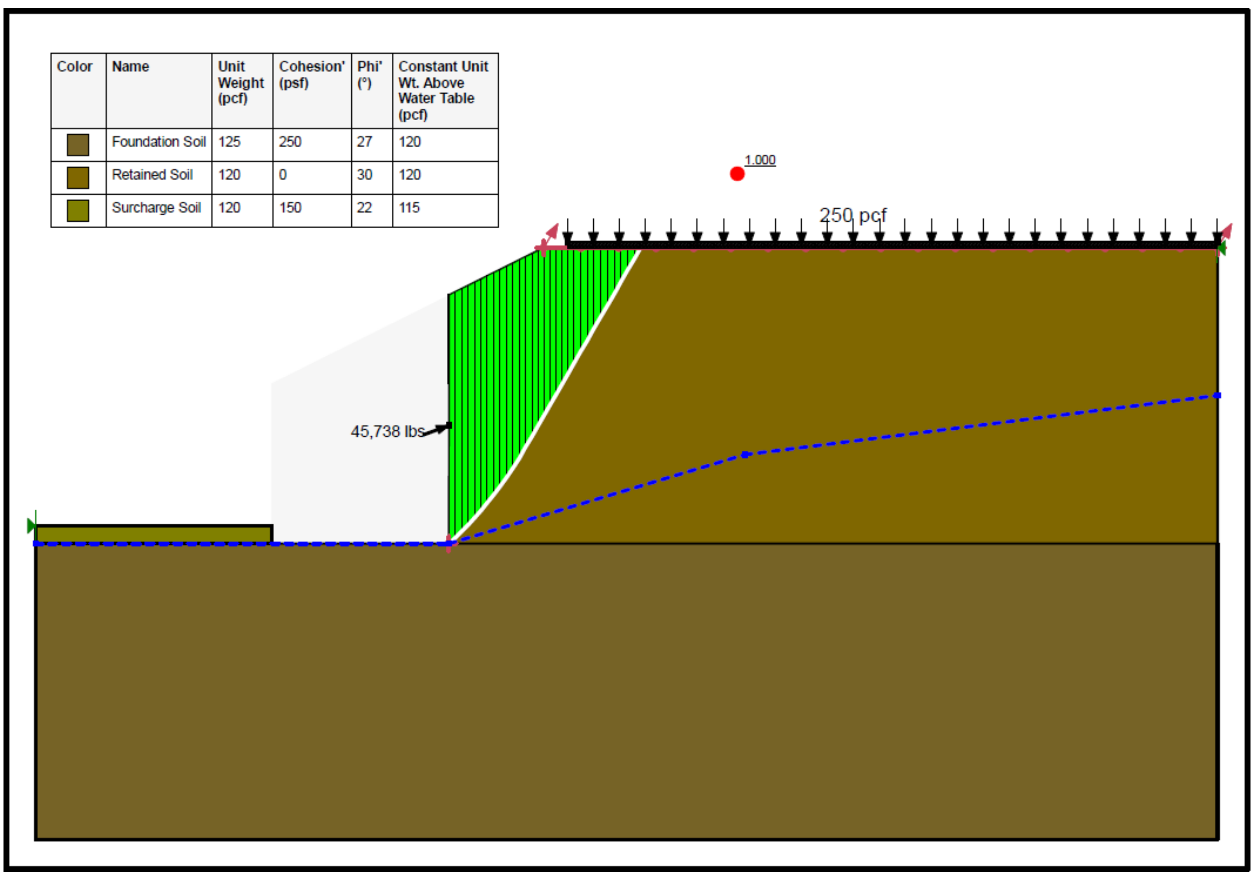
For a nongravity cantilever wall that directly supports a cut rock face, there will typically be no earth pressure load from the rock, as it will stand vertically after it is cut. However, weaker rock will relax some with the confining pressure of the neighboring material removed. Therefore, to make the design appropriately conservative, we recommend using a uniform distributed load (not a triangular distributed load) over the depth of the rock cut. For very weak rock, use 1/3 of the overburden pressure over the rock (or a minimum of 150 psf); for weak rock use 1/10 of the overburden pressure over the rock (or a minimum of 50 psf); for stronger rock, typically use no lateral earth pressure. However, if there is adverse jointing within the cut rock mass such that the mass may slide on a joint and impose an additional load on the retaining structure, perform a kinematic analysis of the stability of the cut rock face using limit equilibrium methods to determine if there is a potential for instability of the jointed rock mass, and what load the failing rock mass will impose on the cut face. Excessive load transfer may be mitigated through methods such as rock bolts or ground anchors. References such as EM 1110-1-2907 Rock Reinforcement, EM 1110-1-2908 Rock Foundations, Rock Slope Stability Analysis (Norrish and Wyllie, 1996), and Practical Rock Engineering (Hoek, 2006) may assist in performing this analysis.

1501.1.5 Earth Pressure Load with a Complex Backslope

For a complex or irregular backslope, use a Generalized Limit Equilibrium (GLE) analysis, Coulomb trial wedge analysis, or Culmann Graphical Method to determine the resultant active earth pressure load (P_a) on the back pressure surface of the wall. GLE analysis and Coulomb trial wedge analysis are described in AASHTO LRFD Article 3.11.5.8.1, and Appendices A11.3.1 through A11.3.3. The Culmann Graphical Method is described in Soil Mechanics in Engineering Practice, 3rd Edition (Terzaghi, Peck, Mesri, 1996) and Theoretical Soil Mechanics (Terzaghi, 1943).

By far the simplest of these three methods is GLE analysis. GLE analysis can be performed with any 2-dimensional limit equilibrium slope stability software. See Figure 1500-2 for an example of this method.

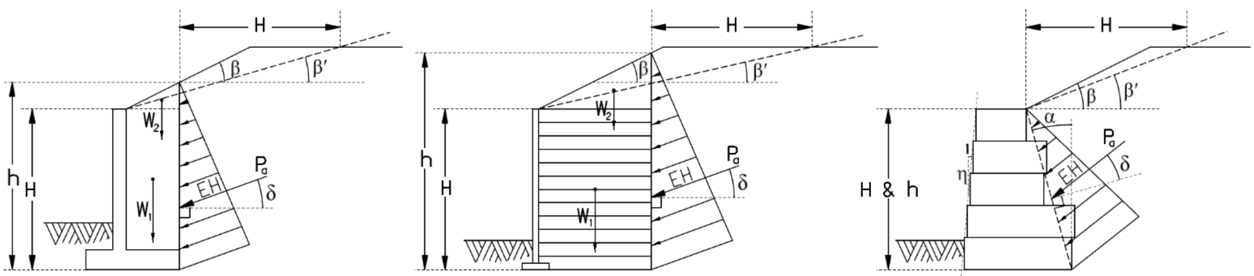
Figure 1500-2: Generalized Limit Equilibrium (GLE) Analysis



In the GLE analysis, remove the wall from the cross section from the toe of the foundation to the back pressure surface. Place a reaction force on the back pressure surface, angled upwards at the soil-structure interface angle (δ), representing the earth pressure load at a height of approximately 0.4 h. Allow a free initiation of the shear failure surface at the ground surface of the backslope, and force the exit of the shear failure surface to the bottom of the back pressure surface of the wall. Adjust the reaction force until a factor of safety of 1.00 is achieved. The reaction force will then equal the nominal value of the resultant active earth pressure load (P_a) on the back pressure surface. In order to separate out the portion of the reaction force corresponding to the earth pressure surcharge load (LS or ES), perform the analysis with and without the surcharge load.

A broken backslope, where there is a short slope with a level surface above, is a special case of the complex backslope. In this case, the "simplified approach," may be used as an alternative method to determine the resultant active earth pressure (P_a) load. The simplified approach is described in AASHTO LRFD Commentary C3.11.5.8.1. See Figure 1500-3 (a), (b), and (c) for examples of this procedure for a semigravity wall, MSE wall, or prefabricated modular wall with stepped modules, respectively.

Figure 1500-3: Simplified Approach for Broken Backslope Effective Slope Angle, β′



(a) (b) (c)

If the level part of a broken backslope is present within a horizontal distance equal to the height (H) of the wall behind the back pressure surface of the wall, utilize the effective slope of backfill (β′) in place of the slope of the backfill surface behind the retaining wall (β) in the Coulomb equation for the calculation of the active earth pressure coefficient (k_a). Continue to use β to determine the notional height of the back pressure surface of the wall (h). The active earth pressure (EH) is angled downwards at angle δ from the normal to the back pressure surface. Note that the prefabricated modular wall with stepped modules uses an imaginary line for the back pressure surface, battered at an angle (α) from the vertical, so EH is angled downwards at angle α + δ from the horizontal; see Section 1503 for further details.

If the level part of a broken backslope is beyond a horizontal distance equal to the height (H) of the wall behind the back pressure surface of the wall, then calculate the active earth pressure as for an (equivalent) infinite slope and use the slope of the backfill surface behind the retaining wall (β) in the Coulomb equation for the calculation of the active earth pressure coefficient (k_a).

1501.1.6 Other External Loads

Include wind-induced loads on noise barriers mounted on top of retaining walls in foundation stability analyses in the Strength III Limit State. See Section 1604.3 and AASHTO LRFD Article 3.8 for calculation of the wind load.

Include vehicular collision forces (CT) in foundation stability analyses in the Extreme Event II Limit State for walls with integral traffic barriers. Distribute vehicular collision forces along the length of the wall for a distance as dictated by a structural analysis of the concrete barrier, to a maximum of the distance between two expansion joints. Use the loads specified in AASHTO LRFD Table A13.2-1 for the appropriate Test Level (TL) of the barrier.

1501.1.7 Load and Resistance Factors

When performing Strength or Extreme Event Limit State analyses for shallow foundation walls, perform both an “a” and “b” analysis (e.g., Strength Ia and Strength Ib). For the “a” analysis, use minimum load factors for vertical loads and maximum load factors for horizontal loads. Sliding resistance and limiting eccentricity (overturning) will typically be critical for the “a” analysis. For the “b” analysis, use maximum load factors for vertical loads and maximum load factors for horizontal loads. Bearing resistance will typically be critical for the “b” analysis. For inclined loads, do not use different load factors for the separate components of the loads; see Section 1501.1.2 for additional details.

In the Strength and Extreme Event Limit States, see AASHTO LRFD Tables 3.4.1-1 and 3.4.1-2 for load factors for all transient loads, and minimum and maximum load factors for all permanent loads.

In the Strength Limit State for shallow foundation walls, use a passive earth pressure load resistance factor of $\varphi_{ep} = 0.50$ in accordance with AASHTO LRFD Table 10.5.5.2.2-1 and Article C10.6.3.4. In the Strength Limit State for deep foundation walls, if not using p-y analysis methods, use a passive earth pressure resistance factor of $\varphi_{ep} = 0.75$ in accordance with AASHTO LRFD Table 11.5.7-1. If performing a p-y analysis, do not apply resistance factors to the passive resistance; see Section 1501.7 for additional details.

See Table 1500-1 for resistance factors against sliding and bearing failure for all wall types. Note that sliding is not a valid failure mode for deep foundation walls; see Section 1501.3.

Table 1500-1: Resistance Factors for Sliding and Bearing

Retaining Wall Type	Sliding	Bearing
Rigid Gravity and Semigravity Walls	1.00	0.55
Prefabricated Modular Walls	1.00	0.55
MSE Walls	1.00	0.65
Nongravity Cantilever Walls	N/A	*
Cellular Sheet Pile Walls	1.00	0.55
Anchored Walls	N/A	*
Soil Nail Walls	1.00	φ_{bh}

Wire Faced MSE Walls	1.00	0.65
Fabric Wrapped Walls	1.00	0.55

* See BDM Table 305-1 for resistance factors for deep foundation elements.

See section 1508 for a description of the resistance factor φ_{bh} for basal heave of soil nail walls.

1501.2 Overall Stability

Analyze overall (global) stability through limit equilibrium (LE) methods, in accordance with BDM Section 307.1.2 and AASHTO LRFD Article 11.6.3.7 for retaining walls and abutments.

While overall stability is a Strength Limit State consideration, do not use factored loads for the analysis; factored loads are incompatible with LE analysis. With factored loads, the shear failure surface will move to a different location. Reliability is not the same; it is dependent on the load factor combination. Small bridge loads in comparison to the size of a slope will have less factored effect, which will negatively affect large bridges.

When considering overall stability of a tiered wall system, analyze the stability of the tiered wall system collectively, and for each tier separately.

1501.3 Resistance to Horizontal Forces

Evaluate resistance to horizontal forces in the Strength Limit State.

For shallow foundation walls, perform a check for sliding resistance according to AASHTO LRFD Article 10.6.3.4. See Section 1303.3 for additional guidance on sliding resistance analysis of shallow foundations. If the sliding resistance of the foundation material is inadequate to withstand the horizontal forces, do as follows:

- For the retaining wall types listed in Section 1502 (rigid gravity and semigravity walls), see BDM Section 305.2.1.3 for methods of remediation. Increasing the footing width is preferred over the use of a shear key, due to increased cost and difficulty in constructing the key; if using a shear key, include the weight of the shear key in the external stability calculations.

- For the retaining wall types listed in Sections 1503 (prefabricated modular walls) and 1506.2 (cellular sheet pile walls), increase the foundation width or depth, or specify a wall infill material with a higher unit weight.

- For the retaining wall types listed in Sections 1504 (MSE walls), 1508 (soil nail walls), 1509.1 (wire faced MSE walls), and 1509.2 (fabric wrapped walls), increase the soil reinforcement length.

For deep foundation walls, see Section 1501.7 regarding p-y analysis.

For the retaining wall types listed in Section 1507 (anchored walls), sliding and overturning are not valid failure modes. If the wall does not have adequate resistance to the horizontal load, increase the Factored Design Load (FDL) of the anchors.

1501.4 Limiting Eccentricity and Overturning Resistance

Evaluate limiting eccentricity and overturning resistance in the Strength Limit State.

For shallow foundation walls, perform a check for limiting eccentricity according to AASHTO LRFD Article 11.6.3.3. See Sections 1303.1.2 and 1303.3.4 for additional guidance on limiting eccentricity analysis of shallow foundations.

For the retaining wall types listed in Sections 1502.4 (precast gravity and semigravity walls) and 1503 (prefabricated modular walls), if they use a separate wall unit and footing without a moment or tension connection, or if they are placed on a concrete leveling pad in accordance with SS851.06.D, perform a check of overturning stability about the toe of the wall units. Use Strength Ia Limit State factored loads (maximum horizontal and minimum vertical) in accordance with Sections 1303.3 and 1501.1.7 for the sum of the horizontal load driving moments and the sum of the vertical load resisting moments. The wall is considered stable in overturning if the sum of the factored resisting moments exceeds the sum of the factored driving moments.

For deep foundation walls with more than one row of deep foundation elements, overturning is typically not a valid failure mode. Check the bearing resistance and pullout resistance of the deep foundation elements that are working as a moment couple. See Section 1300 and AASHTO LRFD Section 10.

For nongravity cantilever walls (or any deep foundation wall with only one row of deep foundation elements), either:

- see Section 1501.7 regarding p-y analysis, or

- demonstrate moment equilibrium about the toe of the vertical wall element, in accordance with AASHTO LRFD Articles 3.11.5.6, 11.6.3.5, and 11.8.4.1, with reference to AASHTO LRFD Figures 3.11.5.6-1 through 3.11.5.6-7, and utilizing the methodology as outlined in AASHTO LRFD Commentary C11.8.4.1. Please note that Figures 3.11.5.6-1 through 3.11.5.6-7 do not include the effects of vehicular live load surcharge (LS), which will have to be added by the engineer. Also note that the figures do not utilize load or resistance factors (all loads and resistances shown are nominal); apply appropriate load and resistance factors as described by AASHTO LRFD Commentary C11.8.4.1. For the determination of Service Limit State horizontal deflection using this method, it will be necessary to perform multiple integrations of the loading diagram, using the assumed stiffness, EI, of the vertical wall element, which is quite complex and should be performed as a last resort.

If a nongravity cantilever wall exhibits excessive horizontal deflection or cannot achieve moment equilibrium at the analyzed length, this is considered failure. In this case, deeper embedment of the vertical wall elements or a larger diameter drilled shaft foundation may be required to meet the requirements of geotechnical resistance against overturning.

The following definitions apply for all nongravity cantilever walls and to all analyses using AASHTO LRFD Figures 3.11.5.6-1 through 3.11.5.6-7:

b = width of a discrete vertical wall element (drilled shaft diameter)

ℓ = center-to-center spacing of vertical wall elements (drilled shaft center-to-center spacing)
ℓ/b = spacing-to-diameter ratio
D = Foundation Depth (embedment depth of vertical wall element below design grade)
H = Design Retained Height (height of the wall above design grade)
L = Length of vertical wall elements = D + H
D/L = embedment-to-length ratio

Measure the design retained height (H) of nongravity cantilever walls from the top of the retained earth to the design grade as shown in AASHTO LRFD Figures 3.11.5.6-1 through 3.11.5.6-7. The minimum embedment (D) for nongravity cantilever walls founded entirely in soil is equal to the retained height (H) such that, the embedment-to-length ratio (D/L) is not less than 0.5.

The design grade is typically taken as either the frost penetration depth, the depth to pavement subgrade located in front of the wall, or the assumed depth of future utility excavations which may occur immediately in front of the wall.

Do not utilize the "Geotechnical Strength Limit State" per FHWA Geotechnical Engineering Circular No. 10 (GEC 10), Publication FHWA-NHI-18-024, Drilled Shafts: Construction Procedures and Design Methods, Section 9.3.3.3.1 to check overturning resistance of drilled shaft walls. The "Geotechnical Strength Limit State" is not intended for retaining walls and will produce overly conservative results.

For the retaining wall types listed in Section 1507 (anchored walls), overturning is not a valid failure mode.

1501.5 Bearing Resistance
Evaluate bearing resistance in the Strength Limit State.

For shallow foundation walls, perform a check for bearing resistance in accordance with AASHTO LRFD Article 10.6.3.1 or AASHTO LRFD Article 10.6.3.2, as applicable. See AASHTO LRFD Table 11.5.7-1 for bearing resistance factors for retaining walls. See Section 1303.3 for additional guidance on bearing resistance analysis of shallow foundations.

For nongravity cantilever walls, do not check bearing resistance unless the wall is being used to support a structure (e.g., as a bridge abutment).

For the retaining wall types listed in Section 1507 (anchored walls), consider bearing resistance for the vertical wall elements as deep foundations, assuming all vertical components of loads are transferred to the embedded portion, according to AASHTO LRFD Article 11.9.4.1.

For other deep foundation walls, including those used as bridge abutments, check the bearing resistance of the deep foundation elements in accordance with Section 1300.

For the retaining wall types listed in Section 1508 (soil nail walls), refer to publication FHWA-NHI-14-007, Geotechnical Engineering Circular 7 (GEC 7), Section 5.6.6, regarding basal heave.

1501.6 Vertical and Horizontal Movements

Evaluate vertical and horizontal movement in the Service Limit State. Measure differential settlements for walls in the longitudinal direction.

Limit differential settlements for rigid gravity and semigravity walls, precast concrete panel walls, and all other walls with full-height concrete faces (whether cast-in-place or precast) to 1/500. This limit will minimize cracking of the concrete.

Limit differential settlements for prefabricated modular walls with stacked modules to 1/200.

Regardless of the size of facing elements, limit differential settlements for MSE walls to 1/100 without slip joints, and to 1/50 with slip joints. If differential settlements will exceed 1/50 or with total settlement of 6 inches or more, design a two-stage wall system in accordance with BDM Section 307.4.1 or design a remediation to limit the settlement.

Limit differential settlements for gabion walls, temporary walls, or for walls with facings applied after wall settlement has completed to 1/50.

For anchored walls, account for the relaxation or elongation of the anchors (and resultant unloading or loading of the anchors) due to settlement of the soil material behind the wall face or settlement of the vertical wall elements. Relaxation and unloading of anchors may result in unacceptable lateral deflection of the anchored wall face.

Limit total settlements to the serviceability requirements of the infrastructure supported by the wall. Additional limitations on total and differential settlement for retaining walls supporting bridge superstructures are provided in BDM Section 305.1.3.

The horizontal deflection limit for all retaining walls is one percent (1%) of the height (H) of the wall. Additional limits may control depending on other elements supported by or located behind the wall. If a retaining wall is to be installed within 10 feet of the edge of pavement, and the pavement is not to be replaced along with the same project, limit the horizontal deflection to 2 inches or less. For retaining walls that support bridge structures, ensure superstructure movements are not impeded by horizontal deflection of the retaining structure. Use whichever serviceability limit requires the least deflection.

For deep foundation walls, see Section 1501.7 regarding p-y analysis.

1501.7 p-y Analysis

For deep foundation walls, perform p-y analysis to check the resistance to horizontal forces and overturning resistance in the Strength Limit State, and to predict horizontal movement in the Service Limit State. Analysis of these walls considers discrete vertical wall elements (single piles, single drilled shafts, or a one-foot slice of continuous steel sheet pile).

Calculate the reaction of the vertical wall elements to the load, the lateral displacement at the head of the vertical wall elements, and the moment and shear distributions in the vertical wall elements. Check the factored resistance of the vertical wall elements versus the calculated factored maximum moment and maximum shear. Any capable p-y analysis software, such as LPILE, or FBPIER may be used. ODOT currently uses the program LPILE, developed by Ensoft, Inc., therefore, the examples in this section refer to this program.

Note that there is no such thing as a drained (as opposed to an undrained) p-y analysis; the analysis is always a static, long-term analysis unless a cyclic loading analysis is performed. The fact that the undrained shear strength is the design parameter used to define the shape of the p-y curve for cohesive soils does not indicate that analyses including the Soft Clay or Stiff Clay models are undrained or short-term analyses. Further note that the response of sand compared to clay soils in p-y analyses is much "softer" and more critical (laterally loaded piles in sand will deflect much more than piles in clayey soils). Therefore, it is not good practice to represent a cohesive soil as an "equivalent" shear strength sand in a p-y analysis; unrealistically large deflections of the pile will occur.

1501.7.1 Distributed Lateral Loading

If using LPILE, convert the distributed load into units of pounds per inch (lb/in) of length along the vertical wall element.

Model the vertical wall element in the p-y analyses full length from proposed top of wall elevation to the estimated tip elevation. Do not analyze the vertical wall elements by cutting them off at the bottom of the retained height, and applying a resultant shear, moment, and axial load at the foundation elevation. Analyze the vertical wall elements by using the theoretical distributed load over the entire retained height to determine the reactions above the foundation elevation. While "cutting off" the vertical wall elements may be sufficient for analyzing reactions below the foundation elevation and produces realistic moment and shear distributions below the cut-off point, it does not include the moment and shear above the cut-off point and it does not adequately account for the deflection at the head of the vertical wall elements (the top of the wall).

Run two p-y analyses for each loading case; one analysis with unfactored loading for the Service Limit State, to determine head deflection of the vertical wall element; and one analysis with factored loading for the Strength Limit State, to check the structural and geotechnical resistance of the vertical wall element.

1501.7.2 p-y Modification Factors for Group Action

If vertical wall elements are placed at a center-to-center spacing closer than 3.75 diameters, use a p-multiplier reduction in the soil resistance p, for the p-y curve behavior of the soil. The loss in resistance is due to soil-structure-soil interaction, and an overlap in the region of the soil that provides passive resistance to the deflection of the vertical wall elements when placed in a closely spaced group. This effect does not occur where vertical wall elements are embedded in a relatively much stiffer material, such as bedrock, where the stress field effects are very limited, and the material does not deform substantially under the design loadings. Therefore, only apply the p-multiplier from the design grade to the top of bedrock or to the bottom of the vertical wall elements, whichever is shallower.

For determination of the p-multiplier, use the equation $p_m = 0.64(S/D)^{0.34}$, for $1 \leq S/D < 3.75$, where $0.640 \leq p_m \leq 1.00$, published by Reese, Isenhower, and Wang, "Analysis and Design of Shallow and Deep Foundations" (2006) for a single row of piles placed side by side. This is an empirical relationship based on testing by several researchers in multiple different soil types. For continuous vertical wall elements (e.g., tangent drilled shafts or steel sheet pile), $p_m = 0.640$.

1501.7.3 Soil Layering and p-y Models

Set the ground surface in LPILE equal to the design grade and model all soil layers below the design grade as in the proposed condition.

1501.7.4 Vertical Wall Element Length

Embed the vertical wall elements in a solid stratum such that deflection at the head of the vertical wall elements is constrained to appropriate serviceability limits (see Section 1501.6). Also select a total vertical wall element length such that the vertical wall element is geotechnically stable (see Section 1501.7.9).

1501.7.5 Section Type, Dimensions, and Cross-section Properties

When analyzing a steel re-bar reinforced concrete shaft, analyze the vertical wall element as a "Round Concrete Shaft (Bored Pile)" or "Round Concrete Shaft with Permanent Casing" under "Section Type" in LPILE, depending on whether a permanent steel casing is to be used or not. These section types will result in a non-linear non-elastic analysis that accounts for the loss of stiffness from a cracked section with deflections beyond the tensile strain limit for the concrete. Set the "Section Diameter" or "Casing Outside Diam." under "Shaft Dimensions" equal to the nominal borehole diameter for the drilled shaft. Under "Rebars," select the appropriate options to define the proposed steel re-bar arrangement. Set the Yield Stress and Elastic Modulus equal to the values for the type of longitudinal steel reinforcing bars to be used. Under Concrete, set the Compressive Strength equal to that for the structural concrete of the shaft (typically 4.0 ksi compressive strength). While Class QC 5 4.5 ksi compressive strength concrete is typically specified for drilled shafts, 4.0 ksi compressive strength is assumed for design, due to quality control limitations on drilled shaft concrete placement. If a concrete compressive strength greater than 4.5 ksi is to be specified, assume 0.9 f'_c for the design.

For a HP-section or W-section embedded in a concrete drilled shaft, including the portion of a soldier pile embedded in the drilled shaft foundation, analyze the vertical wall element as a "Round Shaft with Casing and Core/Insert" under "Section Type" in LPILE. Set the casing outside diameter to the nominal drilled shaft diameter, set the casing wall thickness to 0 inches unless a permanent steel casing is used, and set the number of bars to "0" for the rebars unless a supplemental reinforcing bar cage is used. For the core/insert type, choose either "Steel H Section Strong Axis" or "Steel AISC Section Strong Axis," depending on whether a HP-section or W-section is being used. This analysis method accounts for the additional stiffness from the composite action of the concrete contained within the web and flanges of the steel section. It will also produce a non-linear non-elastic analysis that accounts for the loss of stiffness from a cracked section with deflections beyond the tensile strain limit for the concrete.

For the portion of a soldier pile above the drilled shaft concrete, or if a steel section other than a single HP-section, W-section, or pipe is embedded in a concrete drilled shaft, typically analyze the vertical wall element as an "Elastic Section (Non-yielding)" under "Section Type" in LPILE. This section type will result in an elastic analysis which uses a constant beam stiffness, which is unaffected by deflection of the beam. For a HP-section or W-section, select the structural shape

"H-Pile Strong Axis" or "H-Pile Weak Axis" (depending on the axis of flexure). For other steel sections, select the structural shape "Circular without Void" under "Dimensions and Properties." If it is a soldier pile above the drilled shaft concrete, set the "Elastic Section Diameter" equal to the width of the steel section. If the steel section is embedded in a drilled shaft, set the "Elastic Section Diameter" equal to the nominal drilled shaft diameter to develop the proper reaction from the soil in LPILE. However, set the moment of inertia and area under "Elastic Section Properties" equal to the actual values of I_X and A_S for the steel section(s). Set the modulus of elasticity equal to that for a steel section alone (approximately 29,000,000 psi), not for a composite section. This analysis method is admittedly conservative for a steel section embedded in concrete, but not enough research has been made into the composite action of concrete and various steel sections to properly represent the behavior as a composite section.

For a steel sheet pile section, also analyze the vertical wall element as an "Elastic Section (Non-yielding)" under "Section Type" in LPILE. However, select the structural shape "Rectangular" under "Dimensions and Properties." To develop the proper reaction from the soil in LPILE, set the "Elastic Section Width" equal to 12 inches (a unit one-foot slice of continuous steel sheet pile), and the "Elastic Section Depth" to the thickness of the sheet pile flange. However, set the moment of inertia and area under "Elastic Section Properties" equal to the Per foot of Wall values of I_X and A_S for the steel sheet pile section. Set the modulus of elasticity equal to that for the steel sheet pile section (approximately 29,000,000 psi).

Set the ground surface as level or inclined per the proposed slope of the design grade.

For a drilled shaft retaining wall, unless there is a constructability concern which dictates a smaller rock socket diameter, specify the same diameter for the drilled shaft over its entire length. Use 50 ksi or higher yield strength steel for soldier piles or structural steel sections embedded in drilled shafts. Assume a drilled shaft concrete strength for design purposes in accordance with BDM Section 304.2.1 and commentary C304.2.1. This means f'_c = 3.5 ksi for Class QC 1 concrete, f'_c = 4.0 ksi for Class QC 5 concrete, and typically, f'_c = 3.5 ksi for Class QC 4 Mass Concrete unless a higher strength is specified as per plan.

For a steel section embedded in a concrete drilled shaft, a steel soldier pile, or a steel sheet pile wall, use an assumed steel section for the p-y analysis to determine the Service Limit State head deflection and Strength Limit State distributed and maximum moment in the vertical wall element. Check that the head deflection of the vertical wall element is less than the required serviceability limit (see Section 1501.6) and check the factored resistance of the vertical wall element versus the calculated factored maximum moment and maximum shear (see BDM Sections 307.6, 307.6.3, and 307.7.1). If these requirements are not met, select a steel section with greater resistance. If an embedded steel section with the minimum section properties will not fit within a selected nominal drilled shaft foundation diameter with the minimum required concrete cover, specify a larger diameter drilled shaft. If the deflection, flexure, and shear requirements are exceeded, consider selecting a lighter steel section (and possibly a smaller diameter drilled shaft) to save cost. Decreasing or increasing the center-to-center spacing of discrete vertical wall elements to reduce or increase the applied load per element is another option for optimization of the wall.

Every time there is a change in the spacing or diameter of the discrete vertical wall elements, or there is a change in concrete reinforcement, soldier pile size or type, steel sheet pile size or type, or embedded steel section size or type, recalculate the reactions with a new p-y analysis. The change in stiffness or soil reaction will result in a different deflection and shear and moment distributions in the vertical wall elements.

1501.7.6 Pile-Head Loading and Options

At the head, a vertical wall element is free to move both laterally and rotationally. In LPILE, there are multiple Pile-Head Loading Condition options to define boundary conditions and loading at the head of the vertical wall element. Select the option “1 Shear [F] & 2 Moment [F-L]”, with a value of zero (0) input for both the moment and shear loading. This defines a vertical wall element which is free at the head, with a moment and shear which will start at zero at the head. All other options define rotational or displacement fixity of the head or define a known deformation at the head.

Set the option “Compute Top Y vs. L?” to “Yes,” as this will generate a plot of “Top Deflection versus Length,” which will aid in determining the required length of the vertical wall elements to resist the lateral loading (see Sections 1501.7.7 and 1501.7.9).

1501.7.7 LPILE Output

After the p-y analysis calculations are completed, inspect several items immediately. LPILE can produce a plot of “Top Deflection versus Length” (see Section 1501.7.6). For both the Service Limit State analysis and the Strength Limit State analysis, the length(s) at which either of these plots climbs to infinity or becomes indeterminate is the point at which the vertical wall element length becomes too short; choose a length beyond this point. If it appears that several iterations may be required to determine the optimal vertical wall element length through incremental increases, it may be more efficient to analyze a vertical wall element which is known to be too long, and then cut down the length to the optimal point. Note that we do not recommend an embedment of less than 10 feet below the design grade, regardless of the results of the deflection plots.

LPILE also generates a plot of Lateral Deflection versus Depth and calculates a (maximum) Pile-head deflection. For the Service Limit State analysis, limit the maximum Pile-head deflection to the horizontal deflection limits specified in Section 1501.6. If the wall deflects more than the required serviceability limit, we consider this to represent failure, and a stiffer vertical wall element or larger diameter drilled shaft foundation must be selected and re-analyzed. Achieving fixity at the tip (bottom) of the vertical wall elements is not required, provided the Service Limit State deflection limit is met at the top.

LPILE also provides maximum values for moment and shear in the vertical wall element. Use these values from the Strength Limit State analysis, in the structural analysis of the vertical wall element, to check if it has adequate structural resistance without failing in either bending or shear (check both the factored flexural resistance and factored shear resistance versus the maximum factored moment and shear in the discrete vertical wall element). For cantilever sheet pile walls (with continuous vertical wall elements), check the factored flexural and shear resistance versus the maximum factored moment and shear per unit width of the wall.

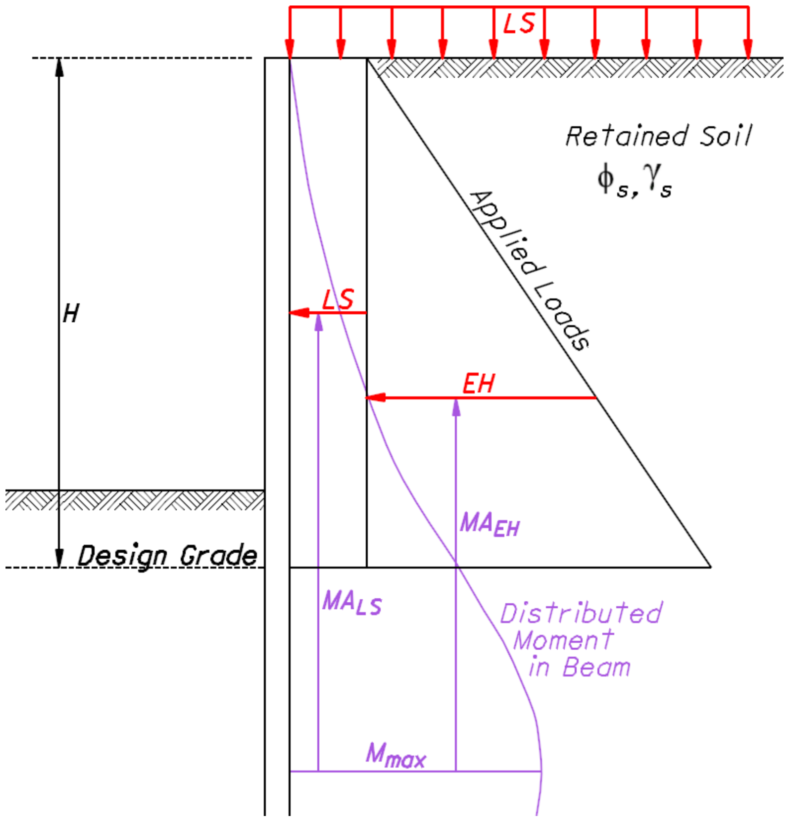
1501.7.8 Artificial Plastic Hinging

Although the Strength Limit State p-y analysis will work in most instances, it should be noted that, strictly speaking, p-y analyses are load-deflection analyses that are intended to use unfactored service loads. While it is possible to factor the load going into the vertical wall element, it is not possible to accordingly factor the stiffness of the vertical wall element or the soil response. In some instances, due to lack of stiffness in the system, factored loads will result in excessive soil deflection that transmits through the p-y response into a much higher load in the vertical wall element or in the vertical wall element deflecting beyond the elastic range of the wall materials and developing a plastic hinge. In this case, it will appear that a much larger and stiffer section will be needed to resist the load. In other words, while the vertical wall element should be able to take the applied factored loads according to the section modulus and shear area of the section, the factored load is causing enough deflection to induce buckling in the section or to produce an artificially-magnified resultant moment and shear (often by a factor of from 2 to 10 times what would be calculated from a purely static beam analysis or moment equilibrium analysis). In this event, for the Strength Limit State structural resistance checks, use only ***unfactored*** service loads in the p-y analysis, but then factor the ***resulting*** maximum moment and shear by the composite Strength Limit State load factors to check the bending and shear resistance of the vertical wall element.

To determine the composite Strength Limit State load factors for both moment and shear, use the following procedure:

1) Construct a load diagram with the triangular EH load distribution and the rectangular LS load distribution; see Figure 1500-4.

Figure 1500-4: Nongravity Cantilever Wall Load Diagram



2) Calculate the unfactored Service Limit State resultant loads:

   $EH = \frac{1}{2} k_a \gamma_s H^2$

   $LS = k_a(2.0')(125\ pcf)H$

   Where:

   k_a = Coulomb earth pressure coefficient
   γ_s = unit weight of retained soil (typically $\gamma_s = 120$ pcf)
   H = retained height (the depth between the pile head and design grade).

3) Multiply the unfactored Service Limit State resultant EH and LS loads by the Strength Limit State load factors, to arrive at the Strength Limit State factored resultants, $\gamma_{EH}EH$ and $\gamma_{LS}LS$.

4) The <u>composite Strength Limit State</u> load factor for shear is:

   $\gamma_{shear} = (\gamma_{EH}EH + \gamma_{LS}LS)/(EH + LS) \geq 1.50$

5) Examine the Service Limit State p-y analysis results, and determine the location of the maximum value of the internal distributed moment in the vertical wall element, M_{max}.

6) The location of the EH and γ_{EH}EH resultant loads will be ⅓H up from the design grade; the location of the LS and γ_{LS}LS resultant loads will be ½H up from the design grade. Calculate the moment arms for both sets of resultant loads, as the distance between these resultant loads and M_{max}. These will be:

   MA_{EH} = moment arm from M_{max} to the resultant loads EH and γ_{EH}EH

   MA_{LS} = moment arm from M_{max} to the resultant loads LS and γ_{LS}LS

7) The composite Strength Limit State load factor for moment flexure is:

   $\gamma_{moment} = (MA_{EH}\gamma_{EH}EH + MA_{LS}\gamma_{LS}LS)/(MA_{EH}EH + MA_{LS}LS) \geq 1.50$

1501.7.9 Artificial Shear Spike

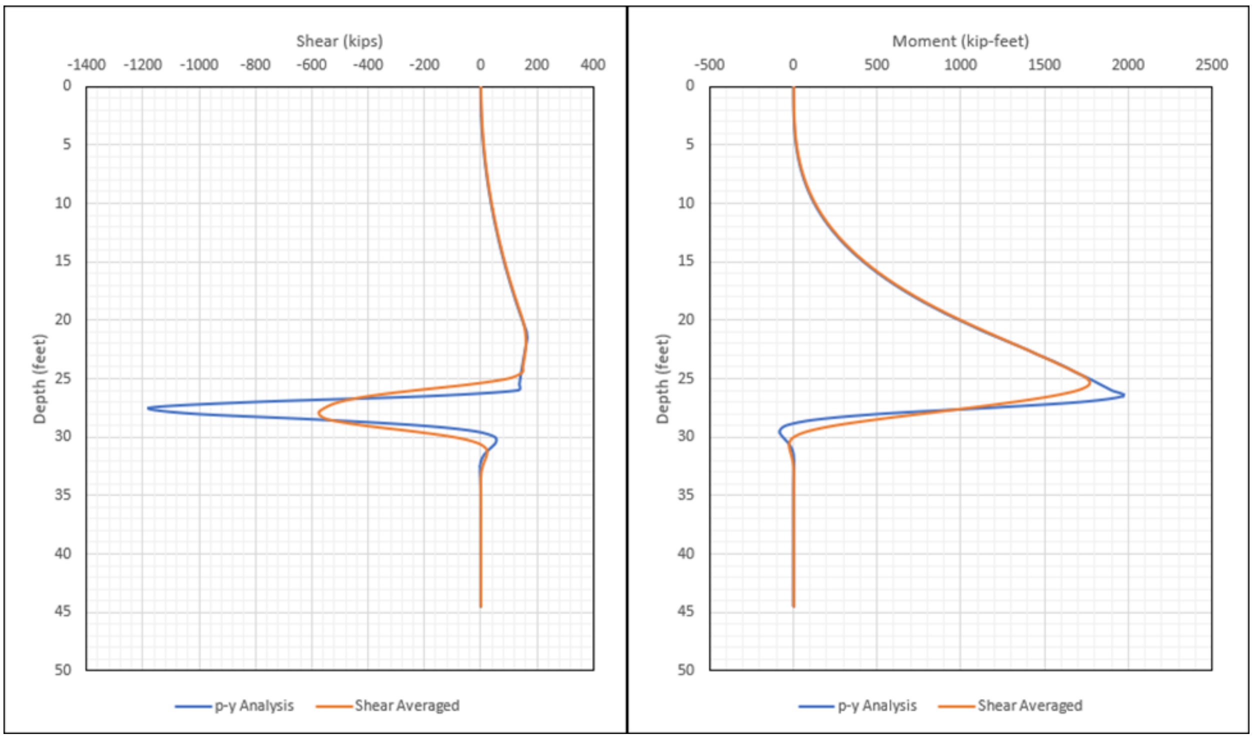
P-y analyses utilize closely-spaced nodes connected with springs of certain stiffness to represent the piles or drilled shafts. This means that each node can react independently in the analyses, while this is not possible in reality. For example, node n could deflect 100 inches and node n+1 could deflect 0.1 inches; this could not occur in a real pile or shaft, since the nodes are part of the same structural element. Similarly, the calculated internal shear in a pile element can go from a small value to a design-controlling value from one node to the next; this is termed an "artificial shear spike," and it typically occurs when there is a large change in the stiffness of the surrounding material, such as at the top of bedrock. This behavior is also not realistic in a real structural element, since shear will build more gradually within the element. A shear rupture would occur at approximately a 45-degree angle, meaning that the vertical distance or length over which the shear acts is approximately equal to the depth or width of the pile or shaft element that is resisting the load.

Typically, shear does not control the design in a p-y analysis; it is typically controlled by either flexure/bending in the Strength Limit State or deflection in the Service Limit State. However, if an analysis indicates that shear will control the Strength Limit State design, then the shear will need to be moderated to generate a more realistic distribution by averaging over a length of the pile or drilled shaft equal to the width – or shear depth – of the element. For example, if a 3-foot diameter drilled shaft is being analyzed, and if the nodes are spaced at 6 inches, the shear should be averaged over 7 nodes (six times 6-inch lengths, with one node on each end). For ease of calculation, we recommend setting the increment of length to 6 inches; to do this in LPILE, go to the menu "Program Options and Settings" and set the "Number of Pile Increments" equal to the length of the pile or shaft element in feet times 2. For example, for a 44.5-foot-long drilled shaft, set the number of pile increments to 44.5 × 2 = 89.

The averaged value applies to the middle node in the set, e.g., when averaging over 7 nodes, the shear value for node n will average the values for nodes n-3 to n+3. Near the ends (top and bottom) of the element, fewer nodes will be averaged (since fewer nodes are available), until there are enough nodes available to make up the full length. For example, the value for node 1 is unchanged (the average of only node 1). The value for node 2 will average the values for nodes 1 to 3; the value for node 3 will average the values for nodes 1 to 5; and the value for node 4 will average the values for nodes 1 to 7.

An example of shear averaging can be seen in Figure 1500-5. The artificial shear spike can be seen in the p-y analysis results at a depth of 27.5 feet, corresponding approximately to the top of bedrock in the analysis model. In this analysis, the drilled shaft is 3 feet in diameter, and the pile increments are 6 inches, therefore 7 nodes are being averaged for each node in the Shear Averaged results. As can be seen, the shape of the shear distribution is approximately the same, except that in the vicinity of the shear spike, the shape of the spike has widened, and the magnitude has lessened to approximately one half of the maximum shear represented in the artificial spike. In the graph on the right, the averaged shear has also been used to recalculate the distributed moment for comparison; the shape of this curve is also approximately the same as from the p-y analysis results, except that the extrema have lessened slightly.

Figure 1500-5: Comparison of p-y Analysis to Shear Averaged Results



1501.7.10 Geotechnical Overturning Resistance

Perform a Strength Limit State check of geotechnical resistance against overturning of the vertical wall element. The check of geotechnical resistance is not a check of the structural resistance of the vertical wall element, but of the geotechnical resistance of the soil and bedrock to resist excessive overturning movement of the vertical wall element.

Consider the pile-head deflection calculated in the Strength Limit State p-y analysis. If the deflection does not indicate failure – either failure of the program to converge at a solution, an infinite deflection, or a very large deflection (typically around 100 inches) – then the vertical wall element is stable, with adequate geotechnical resistance against overturning. It is acceptable for the Strength Limit State analysis deflection to be quite large, as long as the Service Limit State analysis deflection meets the required serviceability limits (see Section 1501.6). The LPILE plot of Top Deflection versus Length (see Section 1501.7.7) can be helpful to find the point of optimized vertical wall element length.

If the Strength Limit State loading results in a large deflection that creates a plastic hinge in the pile (see Section 1501.7.8), then this analysis cannot be completed conventionally. We would be showing a failure of structural resistance in the pile, and not a failure of geotechnical resistance against overturning. In this case, magnify the stiffness of the vertical wall element in the Strength Limit State p-y analysis by multiplying the moment of inertia (I_X) by a factor of 2 and re-run the analysis. This should make the vertical wall element stiff enough to not buckle internally due to the load and will result in most of the deflection coming from the soil p-y reaction.

1501.8 Seismic Design
A retaining wall does not need to be designed for seismic effects unless it supports a structure which is required to be designed for seismic loading. See BDM Section 307.1.7.

1501.9 Prefabricated Retaining Wall Systems (PRWS)
For all Prefabricated Retaining Wall Systems (PRWS) ODOT instituted the PRWS Approval Process in 2016. This was an expansion of the previous "Accreditation Procedure for MSE Walls," formerly performed by the Office of Structural Engineering and controlled by the Bridge Design Manual. The Department will only accept prefabricated retaining wall systems approved through the Prefabricated Retaining Wall System Approval Process. Select an approved wall system listed on the Department's Approved Products List. The current PRWS Approval Process and a list of the approved wall systems can be accessed here.

Under this process, three classifications of PRWS are recognized:

- Precast Gravity And Semigravity Retaining Wall Systems (see Section 1502.4),
- Prefabricated Modular Retaining Wall Systems (see Section 1503), and
- Mechanically Stabilized Earth (MSE) Wall Systems (see Section 1504).

The PRWS Approval Process and associated Approved List applies to all walls supporting roadway infrastructure (a roadway, roadway embankment, or other transportation-related structure, or used as a wingwall for a bridge or culvert) or any wall greater than 3 feet in exposed height. If the wall does not meet either of these criteria, then a prefabricated retaining wall system that is not on the Approved List may be used. An example would be a less than 3-foot-tall landscaping wall supporting a lawn adjacent to the roadway.

Please note that Item 610, "Cellular Retaining Wall," has been removed from the C&MS and from the Item Master. This item was formerly used for "constructing retaining walls composed of a series of cells formed by assembling galvanized metal or precast reinforced concrete units," as stated in the previous editions of the C&MS. These types of walls, typically metal bin walls and concrete crib walls, were the first common type of prefabricated walls, and were popular in the 1950s through 1970s.

Item 610 was added to the C&MS long ago due to the popularity of these cellular wall systems at the time. In more recent years – in the 1980s and beyond – these types of walls waned in popularity and have been supplanted with Mechanically Stabilized Earth (MSE) walls, modular block walls, and other types of prefabricated wall systems. Many of these old crib and bin (cellular) walls, constructed for ODOT 50 to 70 years ago, are nearing the end of their useful life and are in advanced states of decay.

Item 610, "Cellular Retaining Wall," was never intended for any other type of retaining wall and did not contain appropriate specifics to control the construction of other types of walls. It was often inappropriately used in recent years for the construction of other wall types, leaving construction control and inspection in question. These other wall types should have been more appropriately covered by Special Items, corresponding Special Plan Notes, and Special Provisions to properly control the construction. In the past, often used items have been Item 610 Retaining Wall, Misc., Item 690 Special, or Item 530 Special – Structures, or a combination of Items 203, 503, 507, 509, 511, 513, 518, 520, 524, and 613. This created a great deal of inconsistency in how these walls have been bid, constructed, and tracked. See sections 1502 through 1509 for a list of the current Pay Items to be used for each type of retaining wall.

1502 RIGID GRAVITY AND SEMIGRAVITY WALLS

Rigid gravity and semigravity walls include rigid gravity walls (which use only self-weight for stability) and semigravity walls (which use a combination of self-weight and backfill soil weight on a mechanically attached heel for stability). These walls are often referred to as "conventional walls" or "cast-in-place walls." Refer to AASHTO LRFD Figure C11.6.1.1-1 for a graphical illustration of the different types of rigid gravity and semigravity walls. Design rigid gravity and semigravity walls in accordance with AASHTO LRFD Article 11.6, as modified by GDM Section 1500. See the BDM for additional design requirements.

Some traffic barrier shapes can support a differential height of earth on either side; traffic barriers used in this way are rigid gravity walls. Do not perform a stability analysis for standard traffic barrier Type C or C1 supporting a differential height of earth of 2 feet or less. If traffic barrier rigid gravity walls are bearing on pavement layers (including the aggregate base course) then do not embed the barrier further into the ground unless required for foundation stability analyses. Do not key traffic barrier rigid gravity walls bearing directly on bedrock that are found to have adequate resistance in foundation stability analyses, and the base of the wall is a minimum of 2 feet below the proposed ground surface, including pavement.

For the CIP or precast reinforced concrete components of rigid gravity and semigravity walls, use a unit weight of 150 pcf. For the soil backfill over the heel of semigravity walls, see Section 1501(C) and BDM Table 307-1. In accordance with AASHTO LRFD Table 3.4.1-2, use a DC load factor for the concrete components, and an EV load factor for the soil backfill over the toe or heel of the wall footing.

Use the following pay items for rigid gravity and semigravity retaining wall systems:

- 503 (CY) Unclassified Excavation

- 509 (LB) Epoxy Coated Reinforcing Steel
- 511 (CY) Class QC1 Concrete With QC/QA, Retaining/Wingwall Not Including Footing
- 511 (CY) Class QC1 Concrete With QC/QA, Retaining/Wingwall Including Footing
- 511 (CY) Class QC1 Concrete With QC/QA, Footing
- 518 (SY) Prefabricated Geocomposite Drain
- 518 (FT) Corrugated Polyethylene Smooth Lined Pipe

If a CIP wall is allowed to be replaced with a precast gravity or semigravity wall, provide the 511 Class QC1 Concrete and 509 Epoxy Coated Reinforcing Steel items as separate quantities for the wall and footing, since many Precast Gravity And Semigravity Retaining Wall Systems still use a CIP footing. Otherwise, make no separation, and use Item 511 Class QC1 Concrete With QC/QA, Retaining/Wingwall Including Footing. Item 518 Prefabricated Geocomposite Drain and Item 518 Corrugated Polyethylene Smooth Lined Pipe are for the wall drainage and drainage outlet pipe, respectively. See Figure 1500-6 for examples of rigid gravity and semigravity walls.

Figure 1500-6: Rigid Gravity and Semigravity Walls

1502.1 Rigid Gravity Walls

Rigid gravity walls use only self-weight for stability. Rigid gravity walls can be steel reinforced or unreinforced plain concrete. Unreinforced rigid gravity walls are often referred to as "concrete mass gravity walls." For unreinforced rigid gravity walls, use a concrete unit weight of 145 pcf; for steel reinforced rigid gravity walls, use a concrete unit weight of 150 pcf.

1502.2 Cantilever Walls

Cantilever walls are semigravity walls, which have a cantilever stem attached rigidly to the footing and internally reinforced to resist the moment and shear loads.

In accordance with BDM Section 307.2.2, for semigravity walls greater than 15 feet in height (H), batter the front face 1/16 in/ft of height of the wall stem. The specified batter allows for slight tilting of the wall after the backfill has been placed, in order to achieve an active earth pressure condition.

1502.3 Counterfort Walls

Counterfort walls are semigravity walls, which have a stem restrained from sliding and overturning by counterforts attached to the back of the stem and heel of the footing in permanent tension. Counterforts are reinforced concrete slabs, projecting perpendicular from the stem and footing. The stem of a counterfort wall is generally thinner and more lightly reinforced than that of a cantilever semigravity wall of similar height.

In accordance with BDM Section 307.2.3, provide counterforts for semigravity walls exceeding 30 feet in height (H). Counterforts may be used on shorter walls if found to be economically and structurally efficient. Counterforts are common on precast semigravity walls.

Geotechnically, the counterforts will increase the gravity load on the heel of the footing. Calculate the load and resultant location of the weight of the counterforts as for reinforced concrete components in accordance with Section 1502. Subtract a corresponding volume from the backfill soil providing vertical earth pressure on the heel of the footing.

1502.4 Precast Gravity and Semigravity Walls

Precast gravity and semigravity walls can be either rigid gravity, cantilever, or counterfort types, in accordance with the respective Sections 1502.1, 1502.2, and 1502.3. All precast gravity and semigravity wall systems must be approved by the Department through the Prefabricated Retaining Wall System Approval Process, as described in Section 1501.9. See BDM Section 307.2.4 and SS851 for additional design requirements. See Figure 1500-7 for examples of precast gravity and semigravity walls.

Figure 1500-7: Precast Gravity and Semigravity Walls

These walls are often built with precast upright elements (stem and counterforts) but with cast-in-place footings; however, precast footings may also be used. If the wall includes a precast footing, place a bed of grout under the footing (between the foundation soil and the footing). This grout is to ensure an intimate contact on the bottom of the precast units and prevent rocking or differential settlement due to uneven contact with the ground. If the wall does not include a precast footing, construct a cast-in-place footing, and place grout bedding between the cast-in-place footing and the bottom of the precast wall units. Specify the grout bedding as a minimum of one inch thick. Grout shall conform to 851.03.G.

Use the following pay items for precast gravity and semigravity retaining wall systems:

- 203 (CY) Granular Embankment
- 503 (CY) Unclassified Excavation
- 509 (LB) Epoxy Coated Reinforcing Steel
- 511 (CY) Class QC1 Concrete With QC/QA, Footing
- 851 (SF) Precast Gravity And Semigravity Retaining Wall
- 851 (LS) Wall Drainage System
- 851 (CY) Natural Soil
- 851 (FT) Concrete Coping
- 851 (Days) On-Site Assistance
- 851 (LS) PGSRW Inspection and Compaction Testing

Reference SS851 Precast Gravity And Semigravity Retaining Wall in the plans for all precast gravity or semigravity walls. Depending on the wall system, a granular backfill may be required in the active wedge zone of the wall backfill; in this case, use Item 203 Granular Embankment and extend the wall quantities to encompass the active wedge zone with a 1H:1V slope up from the heel of the wall. If the wall is constructed with a cast-in-place footing, use Item 509 and Item 511 for the footing. Item 851 Precast Gravity And Semigravity Retaining Wall is inclusive of all accessories and appurtenances necessary to construct the wall, including wall units, bearing pads, joint covering, wall infill materials, leveling pads, precast footings, aesthetic surface treatment, and other items which do not have separate pay items but are necessary to complete the wall; if the wall includes a precast footing, this is inclusive of the footing. Item 851 Wall Drainage System is inclusive of all wall drainage items, including Prefabricated Geocomposite Drainage (PGD) or porous backfill and geotextile fabric, perforated drainage pipe, non-perforated drainage pipe, and the drainage outlet, and takes the place of corresponding Item 518 items. Item 851 Natural Soil is used to provide a cohesive soil cap over the precast semigravity wall infill, and over the Item 203 Granular Embankment, when used. The natural soil cap is a minimum of 3.0 feet thick.

1503 PREFABRICATED MODULAR WALLS

Prefabricated modular retaining walls use the weight of stacked prefabricated units as a gravity wall. These units often make cells containing granular backfill. The units are stacked vertically at the face, placed battered at an angle $1/\eta$ from the vertical, or stacked vertically with a setback (an effective batter). These wall systems include modular block walls, metal bin walls, concrete bin walls, and concrete crib walls. All prefabricated modular retaining wall systems must be approved by the Department through the Prefabricated Retaining Wall System Approval Process, as described in Section 1501.9. Design prefabricated modular walls in accordance with AASHTO LRFD Article 11.11, as modified by GDM Section 1500. See BDM Section 307.3 and SS870 for additional design requirements. See Figure 1500-8 and AASHTO LRFD Figure C11.11.1-1 for examples of prefabricated modular walls.

In Figure 1500-8, (a) is an example of a modular block wall, (b) is an example of a metal bin wall, (c) is an example of a concrete bin wall, (d) is an example of a concrete crib wall, (e) is an example of an open back-faced concrete bin wall. Modular block walls depend almost entirely on the weight of the concrete modular units for stability, with some additional weight of soil or increased soil-structure interface angle (δ) for the reaction of the soil above the heel of stepped modules. Bin walls and crib walls rely more on a composite behavior of the modular units with the modular wall infill placed within the modules.

Figure 1500-8: Prefabricated Modular Walls

Prefabricated modular walls cannot be used for bridge abutment walls or as bridge substructures.

Use the following pay items for prefabricated modular wall systems:

- 203 (CY) Embankment
- 203 (CY) Granular Embankment
- 870 (SF) Prefabricated Modular Retaining Wall

- 870 (CY) Wall Excavation
- 870 (CY) Natural Soil
- 870 (FT) 6" Drainage Pipe, Perforated
- 870 (FT) 6" Drainage Pipe, Non-Perforated
- 870 (FT) Concrete Coping
- 870 (Days) On-Site Assistance
- 870 (LS) PMRW Inspection and Compaction Testing

Reference SS870 Prefabricated Modular Retaining Wall in the plans for all prefabricated modular walls. Item 203 Embankment or Granular Embankment is for backfill behind the wall, within the limits of the wall excavation. Depending on the wall system, a granular backfill may be required in the active wedge zone of the wall backfill; in this case, use Item 203 Granular Embankment and extend the wall quantities to encompass the active wedge zone with a 1H:1V slope up from the heel of the wall; otherwise, use Item 203 Embankment. Item 870 Prefabricated Modular Retaining Wall is inclusive of all accessories and appurtenances necessary to construct the wall, including the wall infill and leveling pad. Item 870 Wall Excavation is inclusive of all excavation necessary to install the wall, including the foundation preparation, and replaces Item 503 Unclassified Excavation. Item 870 Natural Soil is used to provide a cohesive soil cap over the modular wall infill, and over the Item 203 Granular Embankment, when used. The natural soil cap is a minimum of 3.0 feet thick and is separated from the modular wall infill by a geotextile fabric, Type A; Item 870 Natural Soil is inclusive of the geotextile fabric, Type A. Item 870 6" Drainage Pipe, Perforated is for collection of water behind the wall, and is inclusive of porous backfill and geotextile fabric, and takes the place of corresponding Item 518 items. Item 870 6" Drainage Pipe, Non-Perforated is for conveyance of the groundwater from the drainage collection and is inclusive of the outlet.

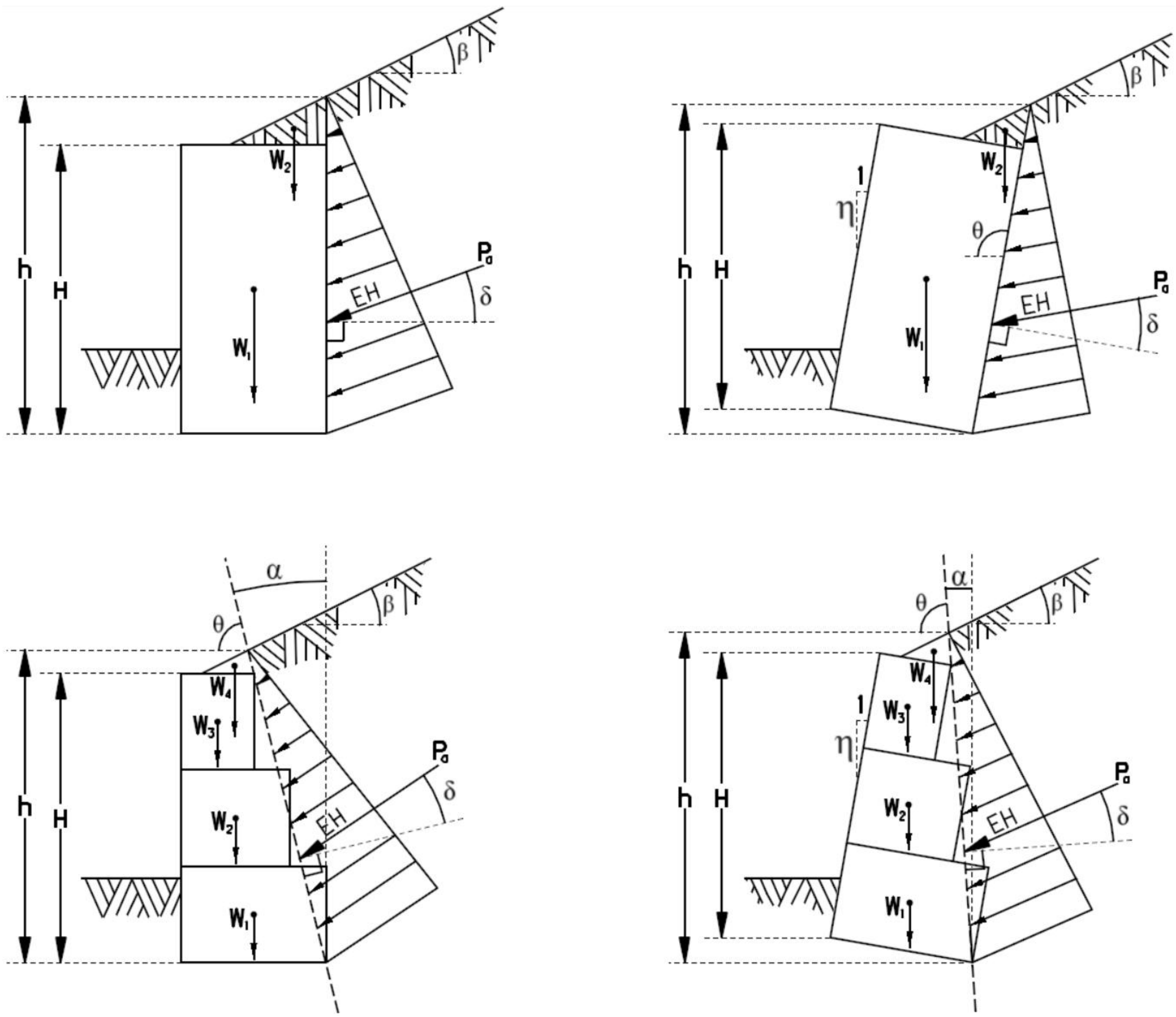
Include foundation preparation consisting of excavation to below the wall base elevation, and placement and compaction of a minimum 12 inches of granular material leveling pad according to SS870.06.D and C&MS 203.07. The thickness of the granular material leveling pad may be increased according to the requirements of the wall system supplier, or based on design necessity to remove unsuitable soils, provide increased bearing resistance, or reduce settlement. For walls founded on bedrock, eliminate the foundation preparation.

If the foundation of the wall requires more excavation, then increase the quantity of wall excavation and specify one of the following suitable materials: modular wall infill according to SS870.03.C; Item 203 Granular Embankment; Item 203 Granular Material, Type B; or Item 203 Granular Material, Type C. Show the deeper excavation limits on the plans; include Item 203 Granular Embankment, As Per Plan; and a plan note permitting any of the four material types listed above. Extend the limits of the granular material leveling pad over the entire wall foundation area, and 12 inches horizontally in front of the wall face.

If the proprietary wall system calls for a concrete leveling pad or for a prefabricated modular wall with a total wall height (H) greater than 6 feet, design and construct a concrete leveling pad on top of the granular material according to SS870.06.D.

The minimum foundation embedment is a vertical distance of 2 feet, measured from any point of the ground surface within 4 feet from the front face of the wall to the top of the granular material leveling pad (or to the bottom of the concrete leveling pad, if provided). The design height of the wall (H) includes this foundation embedment as shown in Figure 1500-9. Measure the notional height of the back pressure surface of the wall (h) from the lowest point at the heel of the wall foundation to the point where an imaginary line on the back pressure surface intersects the ground surface of the backslope, as shown in Figure 1500-9.

Figure 1500-9: Back Pressure Surface for Prefabricated Modular Walls



Determine sliding resistance of prefabricated modular wall foundations both at the top and bottom of the granular material leveling pad. For sliding resistance at the bottom of the granular material leveling pad, if the foundation preparation excavation extends below the frost depth or probable depth of disturbance, include passive resistance with a trapezoidal distribution on the granular material leveling pad in accordance with Section 1501.1.4. If the wall uses a concrete leveling pad over a granular material leveling pad, determine sliding both at the top of the concrete leveling pad and at the bottom of the granular material leveling pad.

For sliding of precast concrete modular units on a concrete leveling pad or in-between levels of modular units, use an interface friction angle of $\phi = 35°$ for the concrete-on-concrete interface, and use a friction coefficient reduction factor of C = 0.80 for precast concrete in accordance with AASHTO LRFD Article 10.6.3.4. For bin or crib systems, use a soil-on-soil interface either for the modular units sliding on the granular material leveling pad or in-between levels of modular units, and for bin or crib modular units sliding on a concrete leveling pad, use a foundation friction angle of $\phi_f = 35°$ for the cast-in-place concrete. Where there are two dissimilar materials along the interface, use the lower of the two frictional coefficients (e.g., for a bin wall with modular bin wall infill soil ($\phi = 32°$ in accordance with Table 870.04-1) sliding on a concrete leveling pad ($\phi_f = 35°$), use $\phi = 32°$ for the modular bin wall infill).

Determine bearing resistance and settlement of prefabricated modular wall foundations at the bottom of the granular material leveling pad. Depth of the foundation (D_f) is the depth to the bottom of the granular material leveling pad and the foundation width (B) is the width of the granular material leveling pad.

Internal stability of prefabricated modular retaining walls is the responsibility of the wall supplier and consists of the following checks: stability of prefabricated modular walls at every module level according to AASHTO LRFD Article 11.11.4.1, including overall stability, resistance to horizontal forces (sliding resistance), and overturning resistance; and a check of the individual modules for structural resistance against earth pressure according to AASHTO LRFD Article 11.11.5.1.

1503.1 Modular Block Walls

Modular block walls are made up of segmental concrete masonry units (blocks) of precast concrete (either dry-cast or wet-cast), typically laid in a running bond “brick” pattern, dry stacked (without mortar), in level courses (or layers) of blocks. Modular block walls are also commonly known as Segmental Retaining Walls (SRW), modular segmental retaining walls, and modular concrete block retaining walls. See Figures 1500-3 (c) and 1500-8 (a) for examples of modular block walls.

Do not use dry-cast modular block walls to support a roadway, roadway embankment, or other transportation-related structure, or as a wingwall for a bridge or culvert. Road salt and freeze-thaw cycle conditions have been shown to cause deterioration in dry-cast concrete, and severely limiting service life.

For sliding resistance of modular block systems, use a friction coefficient reduction factor of C = 0.80 for precast concrete footings for sliding on the granular material leveling pad or concrete leveling pad.

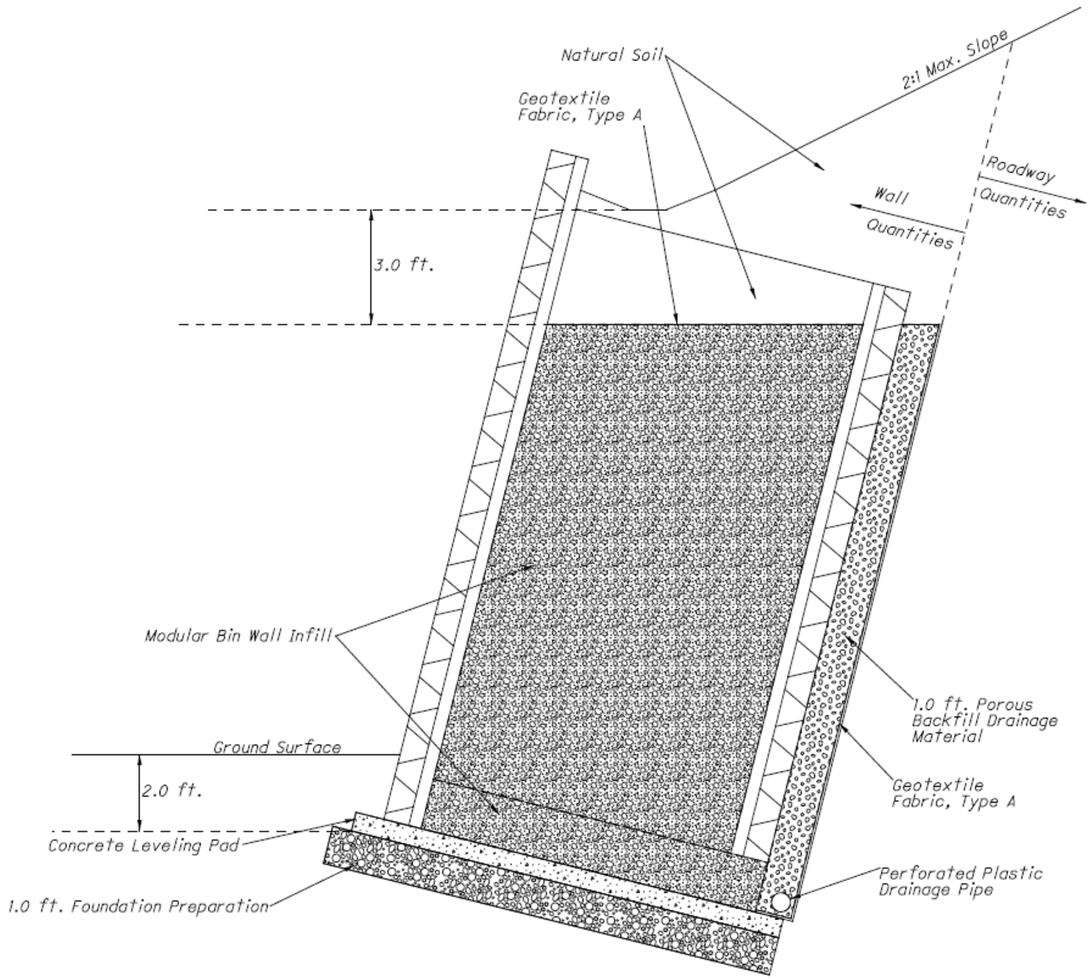
If a modular block wall has layers of geosynthetic reinforcement running from the precast units into the backfill, design and construct it as an MSE wall with a block facing in accordance with Section 1504 and SS840.

1503.2 Bin Walls

Bin walls are generally constructed with metal bins (metal cellular walls). However, some bin walls are made up of precast concrete bin units. See Figure 1500-10 for examples of metal bin walls.

Figure 1500-10: Metal Bin Walls

Figure 1500-11: Metal Bin Wall Typical Section

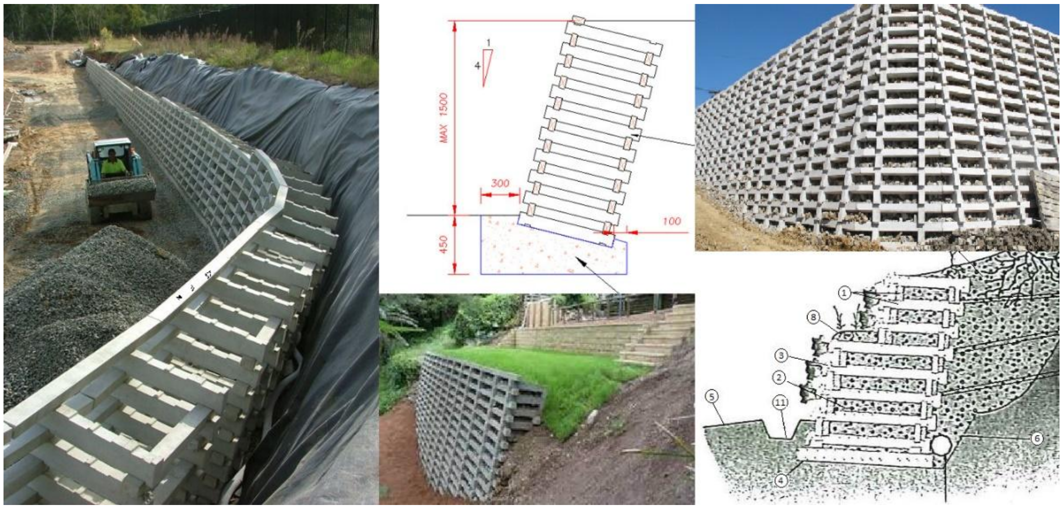


Bin walls are backfilled with modular bin wall infill, which is composed of Item 203.02.H Granular Embankment in accordance with 703.16.B, from the bottom of the wall in contact with the leveling pad up to the bottom of the Item 870 Natural Soil cohesive soil cap. The backfill of metal bin walls needs to meet additional chemical requirements in accordance with SS870.03.C. See Figure 1500-11 for an example of a typical section of a metal bin wall on a concrete leveling pad.

1503.3 Concrete Crib Walls

A concrete crib wall is a cellular wall constructed of interlocking prefabricated reinforced or unreinforced concrete elements. Each crib is comprised of alternating transverse and longitudinal horizontal beams. The wall face can be either open or closed. In closed-face cribs, the horizontal elements are in contact with each other. In open-faced cribs, there is a small gap between the horizontal elements. Each crib unit is filled with granular, free draining soil, which is compacted inside each unit. See Figure 1500-12 for examples of concrete crib walls.

Figure 1500-12: Concrete Crib Walls



Concrete crib walls are backfilled with modular wall infill, in accordance with SS870.03.C, from the bottom of the wall in contact with the leveling pad up to the bottom of the Item 870 Natural Soil cohesive soil cap. They are nearly identical to bin walls in their typical section.

1503.4 Gabion Walls

Design gabion walls in accordance with AASHTO LRFD Article 11.11, as modified by GDM Section 1500. See BDM Section 307.3 and SS838 for additional design requirements.

Gabion walls are composed of metal baskets of prefabricated welded or twisted wire mesh, backfilled with stone. While the wire mesh is prefabricated in accordance with SS838.02.A, this wall type does not fall under the Prefabricated Retaining Wall System Approval Process, and the wire mesh baskets are not listed on the Department's Approved Products List. Design of gabion walls is performed by the project designer, and all gabion walls are custom. The wire mesh baskets are typically fabricated by the contractor at time of construction and the backfill primarily placed by hand in accordance with SS838.03. See Figure 1500-13 for examples of gabion walls.

Figure 1500-13: Gabion Walls

Gabion baskets used for walls shall have a minimum dimension of 36 inches in all directions. Cover the back face of a gabion wall with Geotextile Fabric, Type A as a filter fabric between the retained soil and the gabion stone. Show the geotextile fabric on the plans. The gabions are inclusive of the geotextile fabric.

Use the following pay items for gabion walls:

- 503 (CY) Unclassified Excavation
- 838 (CY) Gabions
- 838 (CY) Gabions With Additional Coating

Reference SS838 Gabions in the plans for all gabion walls. Item 203 Granular Material, Type C is for the foundation preparation layer (granular material leveling pad). Regardless of the height of a gabion wall, always use a granular material leveling pad foundation preparation, and never use a concrete leveling pad. Typically specify Item 838 Gabions, which are galvanized in accordance with SS838.02.A. Specify Item 838 Gabions With Additional Coating when soil or site conditions are identified that are considered indicative of potential deterioration or corrosion from environmental conditions according to AASHTO LRFD Article 10.7.5.

For the weight of the gabions in design, assume a unit weight of 100 pcf and apply a load factor for vertical earth pressure (EV). For considerations of overall stability and sliding resistance, assume the gabions to have an internal and external friction angle of $\phi = 35°$. For a gabion wall, do not consider the stability at every module level, as the gabion baskets are wired together into a single acting mass. However, analyze overall stability for a gabion wall both externally with the gabion wall as a rigid body, and internally as if the wall is a rock fill through which compound overall stability shear failure surfaces may pass. Check sliding only at the foundation of the wall, both at the top and bottom of the granular material leveling pad, assuming the gabion wall to be a rigid mass.

1504 MSE WALLS

Mechanically Stabilized Earth (MSE) Wall Systems use the weight of granular backfill on thin reinforcements to restrain a modular face. The entire reinforced soil mass acts as a gravity structure.

All MSE retaining wall systems must be approved by the Department through the Prefabricated Retaining Wall System Approval Process, as described in Section 1501.9. Design MSE walls in accordance with AASHTO LRFD Article 11.10, as modified by GDM Section 1500. See BDM Section 307.4 and SS840 for additional design requirements. The wall system supplier is responsible for designing the internal stability of the wall in accordance with the project plans and SS840. See Figures 1500-3 (b) and 1500-14 and BDM Figures 201-2 through 201-7 for examples of MSE walls.

Figure 1500-14: Mechanically Stabilized Earth (MSE) Walls

Use the following pay items for MSE wall systems:

- 840 (SF) Mechanically Stabilized Earth Wall
- 840 (CY) Wall Excavation
- 840 (SY) Foundation Preparation
- 840 (CY) Select Granular Backfill
- 840 (CY) Natural Soil
- 840 (FT) 6" Drainage Pipe, Perforated

- 840 (FT) 6" Drainage Pipe, Non-Perforated
- 840 (FT) Concrete Coping
- 840 (SF) Aesthetic Surface Treatment
- 840 (Days) On-Site Assistance
- 840 (LS) SGB Inspection and Compaction Testing

Reference SS840 Mechanically Stabilized Earth Wall in the plans for all MSE walls. Item 840 Mechanically Stabilized Earth Wall is inclusive of all accessories and appurtenances necessary to construct the wall, including facing panels, soil reinforcements, connection devices, bearing pads, joint covering, pile sleeves, leveling pads, and other items which do not have separate pay items but are necessary to complete the MSE wall. Item 840 Select Granular Backfill is inclusive of the granular backfill in the reinforced zone and to 2 feet behind the reinforced zone. Item 840 Wall Excavation is inclusive of all excavation necessary to install the wall, including the foundation preparation, and replaces Item 503 Unclassified Excavation. Item 840 Natural Soil is used to provide a cohesive soil cap over the Select Granular Backfill (SGB) when a vegetated slope is to be constructed above an MSE wall (See BDM Figure 201-x). Item 712.09, Geotextile Fabric, Type A is placed as a separation layer between the SGB and the Natural Soil; Natural Soil is inclusive of the Geotextile Fabric. Item 840 6" Drainage Pipe, Perforated is used for collection of water behind the wall, and is inclusive of porous backfill and geotextile fabric, and takes the place of corresponding Item 518 items. Item 840 6" Drainage Pipe, Non-Perforated is used for conveyance of the groundwater from the drainage collection and is inclusive of the outlet.

Include foundation preparation consisting of excavation to below the leveling pad elevation, and placement and compaction of a minimum 12 inches of Granular Material, Type C according to SS840.06.D and C&MS 204.07. Despite the foundation preparation being placed and compacted in accordance with 204.07, for an MSE wall ***do not*** place Item 204 Geotextile Fabric at the bottom of the undercut (the base of the foundation preparation) as per C&MS 204.07.A. The purpose of the foundation preparation is not the same as for a stabilized roadway subgrade, and geotextile fabric would reduce sliding resistance of the wall foundation. Extend the limits of the foundation preparation over the entire wall foundation area, and 12 inches horizontally in front of the leveling pad.

The thickness of the foundation preparation layer may need to be increased to remove unsuitable soils, to provide increased bearing resistance, or to reduce settlement. If the foundation preparation of the wall requires more excavation, then specify Item 840 Foundation Preparation, As Per Plan, increase the quantity of Item 840 Wall Excavation, and show the deeper excavation limits on the plans. For a wall founded on bedrock, eliminate the foundation preparation.

See Figure 1500-3 (b) and AASHTO LRFD Figures 3.11.5.8.1-1 and 3.11.5.8.1-2 for a definition of how to measure notional height of the back pressure surface of the wall (h). Measure h at the limit of the soil reinforcements (at the back pressure surface of the wall) from the top or bottom of the foundation preparation layer (depending on at which level stability analysis is being performed). For an MSE abutment, measure h up to the profile grade elevation (the roadway surface for the bridge approach pavement). For all other MSE walls, measure h up to the ground surface immediately at the top of the back pressure surface of the wall. See SS840.04.A.2 for a definition of how to measure the wall height (H); note that this is not the same as the notional height (h) used for stability analyses.

Specify a soil reinforcement length not less than 70 percent of the wall height (H) or 8 feet, whichever is greater. Only increase this minimum soil reinforcement length as necessary to meet external stability requirements: bearing resistance, sliding resistance, limiting eccentricity (overturning), and overall (global) stability.

Determine sliding resistance of MSE wall foundations both at the top and bottom of the foundation preparation layer. For sliding resistance at the bottom of the foundation preparation layer, if the foundation preparation extends below the frost depth or probable depth of disturbance, include passive resistance with a trapezoidal distribution on the foundation preparation granular material in accordance with Section 1501.1.4.

For an MSE wall, use a soil-on-soil interface at both the top and bottom of the foundation preparation layer. At the top of the foundation preparation layer, use an interface friction angle of $\phi = 34°$, as this is the friction angle for both the Select Granular Backfill (SGB) and the foundation preparation granular material in accordance with BDM Table 307-1. Where there are two dissimilar materials along the interface at the bottom of the foundation preparation layer, use the lower of the two frictional coefficients (e.g., for the foundation preparation layer with $\phi = 34°$ sliding on a foundation soil with $\phi_f = 30°$, use $\phi_f = 30°$ in accordance with AASHTO LRFD Article 10.6.3.4). When the foundation soil has a lower interface friction angle than the foundation preparation layer, then the sliding analysis at the top of the foundation preparation layer can be excluded by inspection. When this is the case, make a comment to this effect.

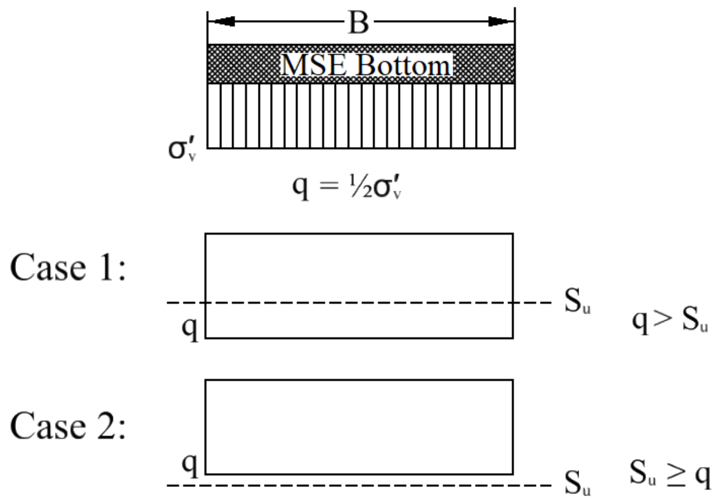
Undrained sliding resistance of an MSE wall is a special case. See Section 1303.3.5 and Figures 1300-3 through 1300-8, except that an MSE wall being a flexible mass composed of soil, cannot develop an eccentric load at the base, and cannot therefore develop a linearly varying unit shear resistance (i.e., there will be no pentagonal, trapezoidal, or triangular distribution of unit shear resistance). An MSE wall will always develop a uniform unit vertical stress, q, across the entire base width, B. Therefore, for undrained sliding resistance of MSE walls, the unit shear resistance, q_s, is simply the minimum of either the foundation soil undrained shear strength, S_u, or the uniform unit vertical stress, q, where:

$$q = ½\, \sigma'_v = ½\, \Sigma V/(B \times L)$$

See Figure 1500-15 for a graphical depiction of the two possible cases, where $q>S_u$ or $S_u>q$:

- for Case 1, $q_s = S_u$
- for Case 2, $q_s = q$

Figure 1500-15: MSE Undrained Sliding Resistance Cases



Determine bearing resistance of MSE wall foundations both at the top and bottom of the foundation preparation layer. At the top of the foundation preparation layer, the foundation width (B) is the same as the length of the soil reinforcements. In this case, either average the foundation soil properties over a depth of 1.5 B below the bottom of the foundation elevation, or perform limit equilibrium bearing resistance analysis (see Section 1303.3.2); either way, include the properties of the foundation preparation granular material in the foundation soil properties. At the bottom of the foundation preparation layer, treat the foundation preparation granular material as the foundation; the depth of the foundation (D_f) is the depth to the bottom of the foundation preparation layer and the foundation width (B) is the width of the foundation preparation layer.

Calculate settlement of MSE walls at the bottom of the foundation preparation layer.

1504.1 Two-Stage MSE Walls

Two-stage MSE walls are typically used when the amount of predicted settlement of the wall precludes the use of conventional MSE wall construction, due to likely damage to the precast concrete MSE wall facing. This wall type consists of constructing a wire faced MSE wall in the first stage, and then constructing and attaching permanent precast concrete facing panels in the second stage after settlement has been mitigated. The precast panels are typically conventional MSE wall panels, set on a leveling pad and individually attached to the wire faced wall. The space between the precast concrete facing panels and the wire faced wall is then filled with MSE wall select granular fill. See BDM Section 307.4.1 for additional design requirements.

This type of wall is designed the same as a conventional MSE wall, except that there are two completion stages: the temporary wire faced MSE wall, and the permanent conventional MSE wall. A full design, including all internal (performed by supplier) and external stability checks, must be performed for both conditions. Since the first stage wire faced MSE wall is required to have a minimum reinforcement length equal to 70 percent of h or 8 feet, whichever is greater, in accordance with SS867.04.A.2, the second stage permanent wall will have a width B equal to the reinforcement length plus the spacing between the temporary wire-facing and the permanent conventional MSE facing.

A two-stage MSE wall uses all the same pay items as for a conventional MSE wall (see Section 1504), but also adds the pay item, 867 (LS) Temporary Wire Faced Mechanically Stabilized Earth Wall, As Per Plan. This pay item is inclusive of only the wire facing units, soil retention fabric, and special connection devices. The wire facing units and connection devices must be galvanized in accordance with SS867.03.A and SS840.03.B. Use SS840 to cover all other pay items. Reference SS867 Temporary Wire Faced Mechanically Stabilized Earth Wall in the plans for all MSE walls utilizing a temporary wire face. Reference SS840 Mechanically Stabilized Earth Wall in the plans for all MSE walls with a permanent face. Include the following plan note:

Item 867 Temporary Wire Faced Mechanically Stabilized Earth Wall, As Per Plan

This plan note is for the construction of a two-stage MSE wall. This pay item includes only the wire facing units, soil retention fabric, and special connection devices between the wire facing units and the permanent precast concrete facing of the second stage of the wall. Galvanize the furnished wire facing units and connection devices according to SS867.03.A. The wall is to be otherwise constructed in accordance with SS840, and all other items for the MSE wall will be measured and paid for under SS840 pay items.

Construct the first stage of the wall with wire facing units and the embankment behind the wall for a minimum distance of (1) behind the wall. Do not begin the construction of the second stage of the wall with permanent precast concrete facing until after the above required wire faced MSE wall and embankment have been constructed and a (2) calendar day waiting period has elapsed. The Engineer may adjust the length of the waiting period based on settlement platform readings. The space between the wire faced wall and the precast concrete facing is to be filled with Item 840 Select Granular Backfill in accordance with SS840.

Note to Designer:

(1) Generally, 200-ft. Optionally, this distance may be defined by station-to-station dimensions.

(2) Estimate the length of the waiting period by determining the time required for 90% of primary settlement to occur. If the designer determines that a waiting period is not necessary, this portion of the plan note may be removed.

1504.2 GRS-IBS Abutments

GRS-IBS stands for Geosynthetic Reinforced Soil Integrated Bridge System. See publications FHWA-HRT-11-026 and FHWA-HRT-11-027 for details of the system and a design methodology for GRS-IBS. See BDM Section 307.4.2 for additional design requirements.

1505 DRILLED SHAFT WALLS

Drilled shafts can be constructed as a wall typically either tangent (shafts side-by-side, touching each other) or spaced (typically with a spacing of 2 to 3 times the diameter of the shafts, center-to-center). Drilled shaft concrete reinforcement often consists of a conventional steel reinforcement bar "cage," constructed with relatively large longitudinal bars, with relatively small bars bent into a spiral or hoops around the longitudinal bars. Sometimes, a steel section (typically a W-section) will be embedded inside the reinforcing bar cage of a drilled shaft wall to increase flexural resistance and stiffness, decrease the amount of steel in the cage, and potentially decrease the size of the drilled shafts. Sometimes a steel section (typically an HP-section or W-section) will be used ***in place of*** a reinforcing bar cage; this is typically the case in soldier pile walls or landslide drilled shafts.

Design drilled shaft walls as nongravity cantilever walls with discrete vertical wall elements in accordance with AASHTO LRFD Article 11.8, as modified by GDM Section 1500. See BDM Section 307.6 for additional design requirements. See Figure 1500-16 for examples of drilled shaft walls.

Figure 1500-16: Drilled Shaft Walls

Use the following pay items for tangent or spaced drilled shaft walls:

- 503 (CY) Unclassified Excavation, As Per Plan
- 509 (LB) Epoxy Coated Reinforcing Steel
- 511 (SY) Class QC1 Concrete, Retaining/Wingwall Not Including Footing
- 513 (Each) Welded Stud Shear Connectors, As Per Plan
- 518 (SY) Prefabricated Geocomposite Drain
- 518 (FT) Corrugated Polyethylene Smooth Lined Pipe

- 524 (FT) Drilled Shafts, __" Diameter, Above Bedrock
- 524 (FT) Drilled Shafts, __" Diameter, Into Bedrock
- 530E51000 (SF) Special - Retaining Wall, Precast Wall Façade Panel

Item 503 Unclassified Excavation, As Per Plan is for excavation of the drilled shaft wall face; detail the pay limits between roadway excavation and structure excavation. Item 524 covers the drilling, placement of the steel reinforcing cage, and concrete placement for the structural (drilled shaft) portion of the wall, all elements inclusive. For a CIP facing, use Items 509 and 511. For connection of the CIP facing to the drilled shafts, 513 Welded Stud Shear Connectors, As Per Plan, may be used; in this case, a prefabricated, curved steel plate on the outside of the drilled shaft should be used, with shear studs welded to the inside (for attachment to the shaft) and outside (for attachment to the facing). Other alternatives for connection of the facing, such as bent or hooked bars, dowels set with cement grout, mechanical expansion anchors, or pre-installed female-threaded anchors, are also acceptable; make these items incidental to Item 524 and change to an As Per Plan item with plan notes and details for the installation. For connection of a CIP facing to soldier piles, 513 Welded Stud Shear Connectors, As Per Plan, are typically used, and are welded directly onto the soldier piles. Post-installed adhesive anchors are not acceptable. Item 530E51000 (SF) Special - Retaining Wall, Precast Wall Façade Panel is used for a precast facing. Item 518 Prefabricated Geocomposite Drain and Item 518 Corrugated Polyethylene Smooth Lined Pipe are used for the wall drainage and drainage outlet pipe, respectively. Wall drainage is placed between the structural elements of the wall (the drilled shafts or soldier piles) and the facing.

Consideration may be given to embedding a steel section (typically a W-section) inside the reinforcing bar cage of a drilled shaft wall to increase flexural resistance and stiffness, decrease the amount of steel in the cage, and potentially decrease the size of the drilled shafts. Sometimes an HP-section or W-section will be used inside a drilled shaft without a reinforcing bar cage. In either case, use composite behavior when analyzing for the shear and flexural resistance of the discrete vertical wall elements (the drilled shafts). See Section 1501.7.5 for details of setting up the composite section in a p-y analysis.

Check the factored structural resistance of the discrete vertical wall element (drilled shaft) versus the calculated factored maximum moment and maximum shear. Use a resistance factor $\varphi_f = 0.90$ for flexural resistance in accordance with AASHTO LRFD Table 11.5.7-1. For a concrete drilled shaft (with or without an embedded steel section), use a resistance factor $\varphi_v = 0.90$ for shear resistance in accordance with AASHTO LRFD Article 5.5.4.2. For a non-composite steel section (typically a soldier pile above the drilled shaft concrete) use a resistance factor $\varphi_v = 1.00$ for shear resistance in accordance with AASHTO LRFD Article 6.5.4.2. Calculate the flexure resistance of a concrete drilled shaft according to AASHTO LRFD Article 5.6.2. Calculate the shear resistance of a concrete drilled shaft according to AASHTO LRFD Article 5.7.3.3. Calculate the flexure resistance of a non-composite steel section according to AASHTO LRFD Article 6.10.8. Calculate the shear resistance of a non-composite steel section according to AASHTO LRFD Article 6.10.9. For the portion of the steel section embedded in the concrete drilled shaft, assume that it has continuous lateral bracing and transverse stiffening and is restrained from both local and lateral buckling by the concrete embedment. For the portion of a steel soldier pile beam section that extends above the drilled shaft foundation, assume it is unbraced and is not restrained against buckling, therefore, check the steel section for both lateral torsional bucking and flange local buckling with an unbraced length equal to the retained height (H), in accordance with AASHTO LRFD Article 6.9.4.1.2.

Ignore the effects of self-weight of the drilled shaft foundation and of buoyancy for considerations of bearing resistance. The equations for bearing resistance of deep foundations inherently already include self-weight of the foundation element and buoyancy, as they are based on load tests applied to the top of piles or shafts and are measuring the net resistance of the system including the effects of self-weight and buoyancy.

For tangent or secant drilled shaft walls that support a vertical load, calculate the axial side resistance for the group of drilled shafts by taking the perimeter of the group of drilled shafts, according to AASHTO LRFD Articles 10.7.3.9 and 10.8.3.6 and publication FHWA-NHI-10-016, Geotechnical Engineering Circular 10 (GEC 10) "Drilled Shafts: Construction Procedures and LRFD Design Methods," Section 14.4. This condition will typically occur where a drilled shaft wall is acting as a bridge abutment. The effective individual side resistance for each drilled shaft may be determined by dividing the group side resistance by the number of drilled shafts.

For drilled shaft walls embedded into bedrock, typically specify a single drilled shaft diameter for the full length of each vertical wall element, and do not specify the drilled hole above bedrock as 6 inches larger in diameter than the rock socket unless construction requirements dictate this as a necessity (for example the need to use a cased hole above bedrock).

1505.1 Tangent Drilled Shaft Walls

Tangent drilled shaft walls have a spacing-to-diameter ratio (l/b) approximately equal to one. The wall is composed of a series of drilled shafts constructed adjacent to each other or with a small gap (less than 6-in) between the shafts. See BDM Section 307.6.1 for additional design requirements.

1505.2 Secant Drilled Shaft Walls

ODOT does not typically construct secant shaft walls, as they are generally specified to produce a water-tight cofferdam, and their design and construction are significantly different than tangent or spaced drilled shaft walls. See BDM Section 307.6.2 for design requirements.

1505.3 Soldier Pile Walls

Soldier pile walls are drilled shaft walls supporting cantilever structural sections – typically HP-sections or W-sections – with a spacing-to-diameter ratio (l/b) greater than one. The gaps between the discrete vertical wall elements (soldier pile beam sections) are filled with lagging, consisting of either permanent precast concrete panels or temporary lagging timbers. Permanent walls utilizing temporary timber lagging will have a permanent cast-in-place facing. Timber lagging is assumed to deteriorate in the permanent condition, therefore the permanent cast-in-place facing is designed to resist the lateral earth pressure. In cases where the wall does not face a roadway or public space, soldier pile walls are often designed with permanent precast lagging without a second façade. However, an aesthetic surface treatment can be applied to the precast lagging panels, as well as painting of the soldier pile beams, for the wall to present an aesthetic appearance in publicly viewed locations. Walls with permanent precast lagging may also have a CIP facing or use full-height precast concrete facing panels as a second façade. See Figure 1500-17 for examples of soldier pile walls.

Design solder pile walls as nongravity cantilever walls with discrete vertical wall elements in accordance with AASHTO LRFD Article 11.8, as modified by GDM Section 1500. Design lagging to resist the maximum lateral earth pressure at the bottom of the retained height. See BDM Section 307.6.3 for additional design requirements.

If the soldier pile beam section is embedded in a drilled shaft, the width of a discrete vertical wall element (b) is the nominal diameter of the drilled shaft. If the soldier piles are driven rather than placed in a drilled shaft, the width of a discrete vertical wall element (b) is the flange width of the soldier pile beam section.

Figure 1500-17: Soldier Pile Walls

Use the following pay items for soldier pile walls:

- 503 (CY) Unclassified Excavation, As Per Plan
- 507 (FT) Steel Piles, Misc.: Soldier Piles
- 509 (LB) Epoxy Coated Reinforcing Steel
- 511 (SY) Class QC1 Concrete, Retaining/Wingwall Not Including Footing
- 513 (Each) Welded Stud Shear Connectors, As Per Plan

- 518 (SY) Prefabricated Geocomposite Drain
- 518 (FT) Corrugated Polyethylene Smooth Lined Pipe
- 524 (FT) Drilled Shafts, __" Diameter, Above Bedrock, As Per Plan
- 524 (FT) Drilled Shafts, __" Diameter, Into Bedrock, As Per Plan
- 530E51010 (SF) Special - Retaining Wall, Precast Concrete Lagging
- 530E51020 (SF) Special - Retaining Wall, Timber Lagging
- 530E51000 (SF) Special - Retaining Wall, Precast Wall Facade Panel

Soldier Pile Walls require Item 524, As Per Plan for the drilled shaft foundation of the wall (without placement of a steel reinforcement cage), and Item 507 for the soldier piles. Item 503 Unclassified Excavation, As Per Plan is used for excavation of the soldier pile wall face; detail the pay limits between roadway excavation and structure excavation. For the lagging placed in-between the upright soldier piles, the following Items have been developed: Item 530E51010 (SF) Special - Retaining Wall, Precast Concrete Lagging and Item 530E51020 (SF) Special - Retaining Wall, Timber Lagging (use one or the other). For a CIP facing, use Items 509, 511, and 513 Welded Stud Shear Connectors, As Per Plan. For a precast facing, the following Item has been developed: Item 530E51000 (SF) Special - Retaining Wall, Precast Wall Façade Panel. Item 518 Prefabricated Geocomposite Drain and Item 518 Corrugated Polyethylene Smooth Lined Pipe are used for the wall drainage and drainage outlet pipe, respectively. Wall drainage is placed between the structural elements of the wall (the soldier piles and lagging) and the facing.

As Per Plan, Item Special, and Item Misc plan notes are required for the drilling and concrete placement in the drilled shafts, fabrication and placement of the steel soldier piles, fabrication and placement of the precast concrete or timber lagging, and attachment or the permanent CIP face with shear stud connectors. Include the plan notes presented in the following sections as applicable.

1505.3.1 Item 507, Steel Piles, Misc.: Soldier Piles

This work consists of furnishing and placing steel soldier piles into drilled holes. Furnish soldier piles consisting of structural steel members that meet the plan requirements and conform to ASTM A572, Grade 50 in accordance with C&MS 711.01. Galvanize soldier piles as shown on the plans and in accordance with C&MS 711.02. Do not field weld or splice steel soldier piles.

The Department will measure soldier piles along the axis of the soldier pile from the top of wall elevation to the bottom of the drilled shaft, as determined by the Engineer. The Department will pay for Soldier Piles at the contract unit price per foot for Item 507, Steel Piles, Misc.: Soldier Piles.

Note to Designer:

- Indicate on the plans the steel section required for the soldier pile (e.g., HP 12x53) and the length of the soldier pile that will be galvanized (if any). Typically, provide a sheet with a schedule of beam and drilled shaft dimensions for the wall.

Also provide the following line in the **General Notes**, under **Design Data**:

Steel soldier piles - ASTM A572 - yield strength 50 ksi

If a different grade of steel is used in the design for the soldier piles, modify the plan note and Design Data entry accordingly.

1505.3.2 Item 524, Drilled Shafts, (1)" Diameter, Above Bedrock, As Per Plan
Item 524, Drilled Shafts, (1)" Diameter, Into Bedrock, As Per Plan

This work consists of furnishing and installing drilled shafts for soldier pile and lagging walls. The drilled shafts are reinforced with soldier piles instead of reinforcing steel cages (2). The soldier piles extend above the top of the drilled shaft. Furnish and install the drilled shafts in accordance with C&MS 524 except as modified and supplemented below.

A maximum depth to bedrock of (3) feet is assumed for the design of the soldier pile wall from station (3) to (3). If bedrock is encountered at a deeper depth, inform the Engineer immediately.

Excavate the hole for the drilled shaft within 3 inches of the plan location. Place the soldier pile within the hole so it is vertical and not inclined more than 1 inch between top to bottom. Place the soldier pile so that the flanges are parallel to the centerline of the row of drilled shafts. Do not allow the orientation of the flanges to vary by more than 10 degrees. Support the soldier pile so that it does not move during concrete placement.

(2) Use Class QC 1 concrete according to C&MS 511. Place concrete to the elevation for the top of the drilled shaft. The Contractor may place concrete using the free fall method provided the depth of water is less than 6 inches and the concrete falls without striking the sides of the hole. Pouring concrete along the web of the soldier pile is acceptable.

Check the position, the vertical alignment and orientation of the soldier pile immediately after concrete placement. Make corrections as necessary to meet the above tolerances. (4) If shown on the plans, fill the hole above the bottom of the lagging to the existing ground surface with Item C&MS 613 Low Strength Mortar Backfill (LSM).

Remove concrete and LSM as necessary from around the soldier pile to place the lagging. Place lagging so that the soldier pile flange overlaps the ends of the lagging by at least (5) inches at both ends of the lagging. Wait at least 12 hours after placing concrete in the drilled shafts before placing lagging.

Sequence of Installation: Install the drilled shafts in a sequence such that no drilled shaft is installed adjacent to either an open drilled shaft excavation or a drilled shaft in which the concrete has less than a 48-hour cure. Installing the shafts in an alternating sequence or any other sequence that meets these criteria is permissible.

Protection of Unattended Open Shafts: Cover unattended open shafts. Use temporary covers of adequate strength to prevent a person or animal from falling in. Leave no drilled shaft excavation un-poured overnight.

The Contractor is responsible for the means and methods used to construct the drilled shafts and place lagging. Any temporary grading, excavation, embankment, aggregate, drainage, sheeting, etc. needed to complete the work is included in the bid price for the drilled shafts. The cost of any excavation and subsequent replacement of embankment (in accordance with Item 203 Embankment) is included in the various bid items for the drilled shafts and lagging, unless separately itemized. No separate payment will be made.

Method of Measurement: The Department will measure Drilled Shafts Above Bedrock, As Per Plan, along the axis of the drilled shaft from the existing ground surface to the top of bedrock, as determined by the Engineer. The Department will measure Drilled Shafts Into Bedrock, As Per Plan, along the axis of the drilled shaft from top of bedrock to the bottom of the drilled shaft, as determined by the Engineer.

Payment is full compensation for constructing the drilled shafts, including furnishing and placing concrete and LSM, removal of concrete or LSM from around the soldier pile in order to place lagging.

Note to Designer:

(1) Edit the note title to indicate the nominal drilled shaft diameter. Provide a separate line for each diameter specified, both Above Bedrock and Into Bedrock.

(2) If a supplemental reinforcing bar cage is to be used, replace “instead of reinforcing steel cages” with “and reinforcing steel cages.” In this case, also replace Class QC 1 concrete with Class QC 5 concrete.

(3) Edit the note to indicate the maximum depth to bedrock assumed for the design and the station limits to which this applies. If the wall is designed without bedrock embedment, remove all references to bedrock from the plan note.

(4) Detail the LSM filling the drilled hole above the concrete in the plans. It may not be necessary in all situations; if it is unnecessary, edit the note to remove references to LSM.

(5) Edit the note to indicate the overlap of the flanges with the lagging. See BDM Section 307.6.3 and notes “Item 530 Special - Retaining Wall, Timber Lagging” and “Item 530 Special - Retaining Wall, Precast Concrete Lagging” for details.

1505.3.3 Item 530 Special - Retaining Wall, Timber Lagging

This work consists of furnishing and placing timber lagging between the soldier piles as temporary support for the retained soil. Furnish timber lagging consisting of construction grade, untreated hardwood with a minimum thickness of (1) inches. To permit drainage, provide 1/4 to 1/2-inch spaces between lagging boards using 3/8-inch-thick spacer blocks or other means acceptable to the Engineer. Place the lagging boards between the flanges of the soldier piles and bearing against the flanges on the exposed side of the wall so that the soldier pile flange overlaps the end of the lagging by at least 2 inches at both ends of the lagging boards. Perform excavation for placement of the lagging in such a manner that the lagging is tight against the excavation cut face. Backfill any voids behind the lagging with a suitable compacted Granular Material conforming to C&MS 703.16.C acceptable to the Engineer. The cost of any such backfilling required, including material, placement, and compaction, is incidental to the cost of the lagging.

The Department will pay for Timber Lagging at the contract unit price per square foot for Item 530 Special - Retaining Wall, Timber Lagging.

Note to Designer:

(1) Edit the note to indicate the thickness of the timber lagging. Design timber lagging in accordance with BDM Section 307.6.3. The minimum thickness is 3 inches.

1505.3.4 Item 530 Special - Retaining Wall, Precast Concrete Lagging

This work consists of furnishing and placing precast reinforced concrete panels between the soldier piles to function as lagging for the retaining wall. Provide precast concrete lagging from a precast concrete manufacturer certified according to Supplement 1073. Provide Class QC 1 concrete according to C&MS 499. Provide epoxy coated reinforcing steel according to C&MS 709.00. In lieu of epoxy coating, a corrosion inhibiting concrete admixture may be used at the specified dosage rate. A qualified product list of corrosion inhibiting admixtures is on file at the Laboratory. Manufacturers should recognize that the corrosion inhibitor may affect the strength, entrained air content, workability, etc. of their concrete mixes. The manufacturer’s choice to use one of these corrosion inhibitors does not alleviate meeting all design requirements. Do not allow the dimensions of the lagging or location of the reinforcing steel to vary by more than 1/4-inch. Cast threaded inserts into the top of each panel for lifting and placing.

Fill all cavities produced by form ties and other single defects or defected areas and with a prequalified trowelable mortar in accordance with Supplemental Specification 843.02 and 843.06. Likewise fill cavities for lifting inserts in the top row of precast concrete lagging panels. Provide a broom/brush finish to all trowelable mortar patches. Cure the trowelable mortar according to Supplemental Specification 843.07. Air dry for at least 10 days after completion of the manufacturer’s recommended cure time for trowelable mortar. Brush abrasive blast, followed by air brooming or power sweeping, to remove dust and sand from the surface and opened pores.

(1) Finish the faces of the precast concrete lagging panels that will not be exposed to a uniform surface, free of open pockets of aggregate. Finish the exposed face of the panels to a smooth surface. (2) Seal the front (exposed) face, sides, top, bottom, and 3-inch minimum of the back face of the concrete panel with Item 512, Sealing of Concrete Surfaces (Epoxy-urethane). The color of the urethane shall be Federal Color Number 17778 (Light Neutral). Cost of sealing shall be incidental and included with the precast concrete lagging panels for payment.

Permanently mark each precast concrete lagging panel to indicate which face will be placed against the soil. Place the panel between the flanges of the soldier piles and bearing against the flanges on the exposed side of the wall so that the soldier pile flange overlaps the end of the lagging by at least (3) inches at both ends of the lagging panel.

Handle, store, and ship the precast concrete lagging panels to avoid chipping, cracking and fracturing the panels. Support the panels on firm blocking while storing and shipping. Do not ship panels until concrete has attained a minimum 3000 psi compressive strength. Submit shipment documentation to the Engineer as the panels are delivered to the project, including the Precaster's record of final inspection, the measurements and tolerances, strength, and dimensions of each panel, along with the TE-24 shipping document.

Inspect all precast concrete lagging panels and reject panels having any of the following:

- Defects that indicate imperfect molding.
- Defects that indicate honeycombed or open texture concrete.
- Defects in the physical characteristics of the concrete, or damage to the sealing of concrete surface treatment or to aesthetic surface treatments.
- Concrete chips or spalls that are larger than 4 inches wide or 2 inches deep. Repair all chips and spalls that are smaller.
- Stained form faces, due to form oil, curing, or other contaminants.
- Signs of aggregate segregation.
- Cracks wider than 0.01 inches, penetrating more than 1 inch or longer than 20 percent of the length of the face containing the crack. Repair all cracks that are smaller.
- Panels that do not meet the specified dimensional tolerances.
- Unusable lifting inserts.
- Exposed reinforcing steel.
- Insufficient concrete compressive strength.

Either replace damaged precast concrete lagging panels or document the damage and propose to the Engineer a repair method for the damaged panel; perform repairs with the acceptance of the Engineer. Provide acceptable replacement panels for any that are rejected.

When installing the precast concrete lagging panels, place hardwood wedges near the top and bottom on each side to hold the lagging panels against the front inside flange of the steel piles.

Payment for all labor, equipment, and material required to fabricate, transport, and install the precast concrete lagging panels shall be made at the contract unit price per square foot for Item 530 Special - Retaining Wall, Precast Concrete Lagging.

Note to Designer:

(1) If an aesthetic surface treatment is to be applied to the precast concrete lagging panels, specify to "cast the aesthetic surface treatment into the front (exposed) face of the panels." For panels that will be covered over with a permanent cast-in-place concrete facing, "finish all faces to a uniform surface, free of open pockets of aggregate."

(2) For panels that will be covered over with a permanent cast-in-place concrete facing, eliminate sealing with epoxy-urethane. Federal Color Light Neutral may be replaced with any other color according to the aesthetics of the project.

(3) Specify the minimum distance for the soldier pile flanges to overlap each end of the lagging panels. This must be at least one inch more than the concrete cover over the reinforcing steel at both ends of the lagging, or a minimum of 3 inches at each end of the lagging, whichever is greater. Show the design of the precast lagging in the plans, including dimensions, rebar sizes, and locations. Modify the required concrete strength for the lagging if necessary. Design precast concrete lagging in accordance with BDM Section 307.6.3.

1505.3.5 Item 513 - Welded Stud Shear Connectors, As Per Plan

Weld headed steel studs to the flanges of the soldier pile to connect the cast-in-place concrete wall facing to the soldier pile. Attach headed studs according to C&MS 513.22 and as shown in the plans. The contractor may attach the studs either before placing the soldier pile in the drilled hole or after excavating in front of the wall. Protect the headed studs from damage until the concrete wall facing is poured. Repair or replace damaged headed studs at no expense to the department.

Note to Designer:

- Welded stud shear connectors are used only when the solder pile wall will be covered with a permanent cast-in-place concrete facing. Detail the locations of the studs in the plans.

1505.4 Landslide Drilled Shafts

Drilled shafts may be utilized for retaining structures, either tangent or spaced at greater than one shaft diameter, for remediation of landslides. Refer to Section 900 for design of drilled shafts for landslide stabilization.

1506 STEEL SHEET PILE WALLS

Steel sheet pile walls consist of interlocking steel sheet pile sections driven into the ground to provide an earth retaining structure. They can be installed as a nongravity cantilever wall or as a cellular structure to form a type of shallow foundation wall. Steel sheet pile is most often used for temporary support of excavations, but it can be used for permanent applications if the steel is corrosion protected or an additional sacrificial thickness of steel is provided. Corrosion protection sometimes consists of coating with zinc (galvanization) or painting, but permanent walls are often covered in a permanent CIP facing, attached to the sheet pile with shear studs. See Figure 1500-18 for examples of steel sheet pile walls.

Figure 1500-18: Steel Sheet Pile Walls

For steel sheep pile used for temporary shoring, see Section 1509.3.

For permanent steel sheet pile walls use the following pay items:

- 503 (CY) Unclassified Excavation, As Per Plan
- 504 (SF) Steel Sheet Piling Left In Place, Minimum Section Modulus of____ cubic inches per foot of Wall
- 509 (LB) Epoxy Coated Reinforcing Steel
- 511 (SY) Class QC1 Concrete, Retaining/Wingwall Not Including Footing
- 513 (Each) Welded Stud Shear Connectors, As Per Plan
- 514 (SF) Surface Preparation Of Existing Structural Steel
- 514 (SF) Field Painting Of Existing Structural Steel, Prime Coat
- 514 (SF) Field Painting Structural Steel, Intermediate Coat
- 514 (SF) Field Painting Structural Steel, Finish Coat

- 514 (MNHR) Grinding Fins, Tears, Slivers On Existing Structural Steel
- 514 (Each) Final Inspection Repair
- 518 (SY) Prefabricated Geocomposite Drain
- 518 (FT) Corrugated Polyethylene Smooth Lined Pipe

Item 504 Steel Sheet Piling Left In Place constitutes the structural vertical wall elements for permanent sheet pile walls. Typically, specify the minimum section modulus per foot (S_x in in^3/foot) for steel sheet pile. If a particular section is required for dimensional reasons, use Item 504, Steel Sheet Piling Left In Place, As Per Plan with a Plan Note specifying the section to be furnished. Use Item 503 Unclassified Excavation, As Per Plan for excavation of the soldier pile wall face; detail the pay limits between roadway excavation and structure excavation. If aesthetics are not of concern, the steel sheet pile can be left bare; however, provide an additional sacrificial thickness of steel in accordance with BDM Section 305.3.5.3 based on the exposure conditions of the wall for a 75-year design life. Otherwise, provide a CIP facing (typically) or a coating over the bare steel. For a CIP facing, use Items 509, 511, and 513 Welded Stud Shear Connectors, As Per Plan. For coatings, either use Item 504, Steel Sheet Piling Left In Place, As Per Plan with a Plan Note referencing hot-dip galvanization in accordance with 711.02 or use field painting with Item 514. Use Item 518 Prefabricated Geocomposite Drain and Item 518 Corrugated Polyethylene Smooth Lined Pipe for the wall drainage and drainage outlet pipe, respectively. Wall drainage is placed between the structural elements of the wall (the sheet pile) and the facing. See BDM Section 307.7 for additional design requirements.

Design steel sheet pile with ASTM A572, Grade 50 steel, minimum. Specify the steel grade in the general notes, design data; see BDM Section 602.3-1.

In contrast to driven pile foundations, it is acceptable to drive steel sheet pile wall sections to full length with a vibratory hammer instead of using an impact hammer. However, in dense or very stiff soils driving sheet pile with vibratory methods may prove impractical. The vibratory pile driving process works by inducing excess pore pressure into the pore water and causing liquefaction of the soils. This procedure works best with saturated sands and does not work well with hard clays (particularly with low water contents).

A simplified “Drivability Index” calculation may be performed to determine if a vibratory hammer can drive a pile to a given depth, as follows:

$I_d = \Sigma\ t_s\ N_{60}\ C_d\ P_p/3$

Where:

I_d = Drivability Index (unitless)
t_s = thickness of soil stratum (feet)
N_{60} = SPT N_{60} value of soil stratum (unitless)
C_d = Drivability Coefficient (unitless), from Table 1500-2
P_p = Pile Perimeter (feet).

When I_d > Rated Hammer Power (kW), this indicates driving refusal. If the Drivability Index indicates that the hammer cannot drive the pile to the required depth, perform a more detailed wave equation pile drivability analysis, using the setup factors for vibratory hammers (f_s) provided in Table 1500-2. See Section 1304.2 regarding drivability analyses, except for vibratory hammers, double all quake and damping values.

Table 1500-2: Vibratory Hammer Setup Factors and Drivability Coefficients

Class	Class Description	C_d	f_s
A-1-a	Gravel	0.2	5.0
A-1-b	Sandy Gravel	0.2	5.0
A-2-4	Sandy, Silty Gravel	0.333	3.0
A-2-5	Silty Gravel	0.4	2.5
A-2-6	Silty Clayey Gravel	0.5	2.0
A-2-7	Clayey Gravel	0.667	1.5
A-3	Fine Sand	0.2	5.0
A-3a	Sand	0.2	5.0
A-4b	Non-Plastic Silt	0.667	1.5
A-4a	Non-Plastic Sandy Silt	0.667	1.5
A-4b	Plastic Silt	0.833	1.2
A-4a	Plastic Sandy Silt	0.833	1.2
A-5	Elastic Silt and Clay	0.909	1.1
A-6a	Silt and Clay	1	1.0
A-6b	Silty Clay	1	1.0
A-7-5	Elastic Silt and Clay	1	1.0
A-7-6	Clay	1	1.0
A-8a	Organic Silt	1	1.0
A-8b	Organic Clay	1	1.0

Table 1500-2 is based on the GRLWEAP Table "Suggestions for shaft setup factors, f_s, for Vibratory pile driving," found in the GRLWEAP help files. The values of C_d in this table are in inverse of the values of f_s, reflecting the fact that the driving resistance is in inverse to the soil setup after driving.

Rated Hammer Power for the most commonly available vibratory hammer is 375 kW, roughly corresponding to an ICE Model 815 hammer. If vibratory driving drivability analysis indicates a commonly available vibratory hammer cannot drive a pile to the required depth, provide a plan note specifying a vibratory hammer of a higher rated energy or specify the use of an impact hammer.

1506.1 Cantilever Sheet Pile Walls

Cantilever sheet pile walls consist of interlocking steel sheet pile sections (typically PZ- or PZC-sections) driven into the ground to form a continuous nongravity cantilever wall, similar to a tangent drilled shaft wall or soldier pile wall. Design cantilever sheet pile walls as nongravity cantilever walls with continuous vertical wall elements in accordance with AASHTO LRFD Article 11.8, as modified by GDM Section 1500. Cantilever sheet pile walls are designed on a per-unit-width basis of vertical face. For cantilever sheet pile walls with permanent cast-in-place facing, designate a specific steel sheet pile section or equivalent. For all other cantilever sheet pile walls, specify a minimum section modulus and moment of inertia per unit width of the sheet pile.

1506.2 Cellular Sheet Pile Walls

Cellular sheet pile walls are composed of rectangular, circular, or semi-circular sheet pile cells typically composed of PS-Section elements, containing retained earth, which together act as a shallow foundation modular gravity wall similar to a bin wall. The sheet pile sections in this case do not retain the soil in cantilever action, but merely in tension across the face of the cell. Publication EM 1110-2-2503 provides a useful design reference.

The minimum foundation embedment of cellular sheet pile walls is the larger of 3 feet or the frost depth at the front (exposed) face; it is acceptable for the steel sheet pile sections to be embedded no deeper than this. Check the external stability of the sheet pile cells in bearing resistance, sliding resistance, and limiting eccentricity (overturning) at the foundation elevation as gravity walls in accordance with AASHTO LRFD Article 11.6.3.

For closed sheet pile cells, driving the rear sheet pile sections deeper to provide uplift resistance against overturning or passive resistance against sliding is acceptable. Check resistance versus uplift according to AASHTO LRFD Article 10.7.3.11 as for a group of driven steel piles. If providing resistance to sliding, ensure that the steel sheet pile sections can resist the horizontal load in flexure and shear.

Check the connection in interlock tension between individual steel sheet pile elements versus the maximum interlock strength between the elements. For semi-circular cells (which are not closed at the back) also check pullout resistance of the cell side walls versus the retained earth pressure. Assume the interface friction angle between the steel sheet pile and soil to be $\delta = \frac{1}{3}\,\phi$ for a soil-steel interface, in accordance with AASHTO LRFD Commentary C3.11.5.3. Resistance factors for the failure modes of connection interlock tension and horizontal pullout are as follows:

- Connection Interlock Tension: $\varphi_{interlock} = 0.75$
- Horizontal Pullout Resistance: $\varphi_{pullout} = 1.00$

These resistance factors are calibrated to ASD factors of safety for interlock tension = 2.0 and for pullout resistance = 1.5, assuming the primary LRFD factored load to be EH, horizontal earth pressure, with load factor $\gamma_p = 1.50$.

1507 ANCHORED WALLS

Anchored walls are typically one of the nongravity cantilever wall types listed under GDM Sections 1505 or 1506 (drilled shaft walls, or steel sheet pile walls), with the addition of ground anchors to limit deflection and resist overturning or structural failure of the vertical wall elements. Tieback anchors are also a possible solution to increase resistance to horizontal forces (sliding) for a rigid gravity or semigravity wall with spread footings, per BDM Section 305.2.1.3. The terms "ground anchor" and "tieback anchor" are considered synonymous. See Figure 1500-19 for examples of anchored walls.

Design anchored walls in accordance with AASHTO LRFD Article 11.9, SS866, and publication FHWA-IF-99-015, Geotechnical Engineering Circular 4 (GEC 4) "Ground Anchors and Anchored Systems," as modified by GDM Section 1500. Other than exceptions given in this section, conform these walls to GDM Sections 1502, 1505, or 1506, as applicable.

Specify anchor locations, anchor inclination, minimum anchor lengths, minimum unbonded lengths, Factored Design Load (FDL), and lock-off load on the Project Plans. For permanent ground anchors, specify the number of proof, performance, creep, and investigative anchor pullout tests to be performed. Both proof and performance tests in SS866 are sometimes called "proof tests" in other publications; investigative anchor pullout tests are sometimes referred to as "verification tests" in other publications. Also specify an appropriate level of corrosion protection for the anchors. Temporary ground anchors do not require specified testing or corrosion protection.

Figure 1500-19: Anchored Walls

When an anchored wall is subject to live load surcharge (LS) loading, the horizontal earth pressure load on the wall consists of a combination of an Apparent Earth Pressure (AEP) diagram load in accordance with AASHTO LRFD Article 3.11.5.7 and a live load surcharge load in accordance with AASHTO LRFD Article 3.11.6.4. For factored loading, separate these two components of the load and apply the appropriate load factors according to AASHTO LRFD Tables 3.4.1-1 and 3.4.1-2. Note that the AEP strength limit state load factor is typically 1.35 (not 1.50 as is the case for active earth pressure).

Ground anchors do not contribute to overall (global) stability unless the anchors develop their pullout resistance beyond the zone of potential soil shear failure.

For anchored walls, a p-y analysis of the foundation is not required. Overturning and serviceability lateral deflection of the foundation are not considered valid failure modes for an anchored wall. Design the embedment depth below design grade of anchored nongravity walls according to GEC 4, Section 5.5. For anchored gravity walls, add the factored pullout resistance of the ground anchor(s) to the factored sliding resistance of the foundation.

Unless the vertical wall elements are founded in bedrock, include the effect of settlement of vertical wall elements in design of anchored walls according to AASHTO LRFD Article 11.9.3.1, and GEC 4 Sections 5.6, and 5.11.1. Settlement can cause reduction of anchor loads. There is additional risk associated in placing a bridge abutment on top of an anchored wall, as the imposition of the bridge loads after construction of the wall can cause vertical displacement and relaxation of the anchors.

Optimize the spacing of the ground anchors in the design. The spacing between the anchors and to the top and bottom of the retained height are typically close to equal. Placing all the anchors too high or too low on the wall, or too far apart, will unnecessarily increase the loads in the unbraced section(s), requiring larger, less efficient steel sections. Placing all the anchors too high on the wall will also require an unnecessarily large embedment depth for stability. The expected embedment is in the range of 0.3 H to 0.4 H. See Figure 1500-4 for a definition of H.

Anchored soldier pile walls utilizing laced-together, back-to-back MC or C channel steel sections for the discrete vertical wall elements are not preferred. Lacing two channel sections together requires a large amount of fabrication and is inefficient for weight of steel per flexural resistance for each soldier pile beam. The preferred approach is to use HP or W steel sections with shop-prefabricated “windows” for the placement of the tieback anchors.

For anchored walls use the following pay items:

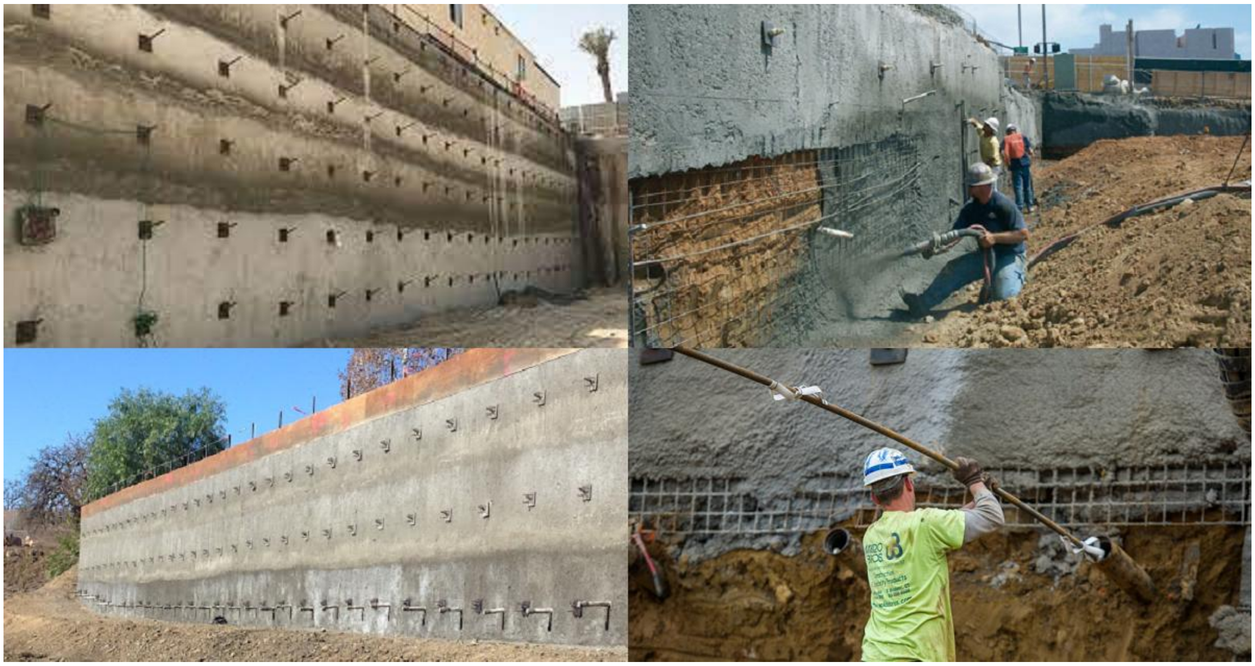
- Same Pay Items as for drilled shaft walls or steel sheet pile walls
- 513 (LB) Structural Steel Members, Level UF
- 866 (Each) Ground Anchor, ___ kip Max. Test Load
- 866 (Each) Performance Test
- 866 (Each) Extended Creep Test

Reference SS866 Ground Anchors in the plans for all walls utilizing ground anchors. As noted, anchored walls typically use the same C&MS Items as the nongravity cantilever wall types but will additionally require SS866 for the ground anchors. Item 513 may also be necessary if the anchors are to pass through walers composed of structural steel members.

1508 SOIL NAILS

Soil nail walls are similar to anchored walls (in that they use drilled-in soil reinforcement elements), and similar to MSE walls (in that the soil reinforcement elements are passive – not tensioned – and they depend on soil overburden weight and confinement stresses to engage the soil nails). Soil nail walls can be thought of as MSE walls constructed top-down (excavating from the top). See Figure 1500-20 for examples of soil nail walls.

Figure 1500-20: Soil Nail Walls



For soil nail walls use the following pay items:

- 503 (CY) Unclassified Excavation, As Per Plan
- 509 (LB) Epoxy Coated Reinforcing Steel, As Per Plan
- 511 (SY) Class QC1 Concrete, Retaining/Wingwall Not Including Footing, As Per Plan
- 518 (SY) Prefabricated Geocomposite Drain
- 518 (FT) Corrugated Polyethylene Smooth Lined Pipe
- 520 (SF) Pneumatically Placed Concrete Shotcrete, As Per Plan

- 530E51100 (Each) Special - Retaining Wall, Soil Nail
- 530E51110 (Each) Special - Retaining Wall, Soil Nail Verification Test
- 530E51120 (Each) Special - Retaining Wall, Soil Nail Proof Test

Use Item 503 Unclassified Excavation, As Per Plan for excavation of the soil nail wall face; detail the pay limits between roadway excavation and structure excavation. Use Items 509 and 511 for the permanent CIP facing. Use Item 518 Prefabricated Geocomposite Drain and Item 518 Corrugated Polyethylene Smooth Lined Pipe for the wall drainage and drainage outlet pipe, respectively. Use Item 520 Pneumatically Placed Concrete Shotcrete for the shotcrete facing. Use Item 530E51100 (Each) Special - Retaining Wall, Soil Nail for the installation of production soil nails, Item 530E51110 (Each) Special - Retaining Wall, Soil Nail Verification Test for verification testing of sacrificial pre-production soil nails (including installation of the sacrificial nails), and Item 530E51120 (Each) Special - Retaining Wall, Soil Nail Proof Test is for the proof testing of production soil nails.

Include the plan note presented in the following section to specify the design parameters for a soil nail retaining wall. Use this note in conjunction with, and include with the plans, the Special Provision: Soil Nail Retaining Wall, available on the OGE website Special Provisions and Plan Notes page.

1508.1 Soil Nail Retaining Wall (Plan Note)
Description of Work:

This work consists of constructing a permanent soil nail wall as specified herein, as shown on the contract drawings, and per the project Special Provisions. Furnish all labor, materials, equipment, and incidentals to complete the work. Design the soil nail wall to meet the minimum requirements specified herein, shown on the contract drawings, or specified in the project Special Provisions.

Design:

Reference: AASHTO LRFD Bridge Design Specifications Article 11.12, Soil Nail Walls. The batter of the designed wall is __(1)__. Provide __(2)__ soil nails with a Factored Design Load of __(3)__ kips per nail. The nails shall consist of a minimum of __(4)__ ksi steel, with a maximum __(5)__-foot vertical spacing and a maximum __(6)__-foot horizontal spacing. The nails shall be installed at an inclination of __(7)__ degrees from the horizontal. <u>Hollow Bar Soil Nails (HBSN) are not* allowed.</u> The shotcrete facing reinforcement shall consist of welded wire fabric per ASTM A1060. The shotcrete wall facing shall be per Item 520, with a minimum thickness of __(8)__". The cast-in-place permanent wall facing shall be per Item 511, Class QC1 Concrete, with a minimum thickness of __(9)__", and shall be reinforced per the wall details. <u>For all evaluations of overall stability, the department considers the proposed soil nail wall to be a "critical" structure as defined in FHWA-NHI-14-007.</u>**

Wall Drainage System:

Provide all elements of the soil nail wall drainage system consisting of geocomposite drain strips, PVC connection pipe, and weepholes, as shown in the contract drawings, that will provide a continuous path for water flow and prevent pore water pressure from building behind the wall. Provide geocomposite drain strips, weepholes, and outlet pipe per Item 518.

Basis of Payment:

The following estimated quantities have been carried to the General Summary to complete the above work:

Item 530, Special - Retaining Wall, Soil Nail, _(2)__, Each
Item 530, Special - Retaining Wall, Soil Nail Verification Test, _(10)__, Each
Item 530, Special - Retaining Wall, Soil Nail Proof Test, _(11)__, Each
Item 503, Unclassified Excavation, As Per Plan, _(12)__ CY
Item 509, Epoxy Coated Reinforcing Steel, As Per Plan, _(13)__ LB
Item 511, Class QC1 Concrete, Retaining/Wingwall Not Including Footing, As Per Plan, _(14) SY
Item 518, Prefabricated Geocomposite Drain, _(15)__ SY
Item 518, ___ inch Corrugated Polyethylene Smooth Lined Pipe, Including Specials, _(16)__ FT
Item 520, Pneumatically Placed Concrete Shotcrete, As Per Plan, _(17)__ SF

Note to Designer:

(1) Specify the batter of the wall (e.g., 80 degrees, 1H:4V, etc.). If this is clear from the detail drawings in the plans, this sentence may be eliminated from the plan note.

(2) Specify the number of soil nails for the wall. If there are multiple groups of nails with different design loads, repeat this sentence as necessary.

(3) Specify the Factored Design Load (FDL) for the soil nails, for soil nail walls designed under LRFD. The FDL is the load that the nails are to be designed to resist, the actual design of the nails being a design-build item.

(4) Specify the minimum yield strength of the soil nail bars. This is typically 60, 75, 80, 100, or 150 ksi, but may be any value that meets ASTM A615 or ASTM A722. Note that Hollow Bar Soil Nails, if allowed, typically have a yield tensile strength between 60 and 90 ksi.

(5) Specify the maximum vertical spacing of the soil nails. This is typically the same as the height of the soil excavation lifts for the wall face and is generally between 3 and 6 feet.

(6) Specify the maximum horizontal spacing of the soil nails. This is generally between 4 and 6 feet.

(7) Specify the inclination from the horizontal for the installation of the soil nails. This is typically 15 degrees, but it can be anywhere from 0 degrees to 45 degrees. However, inclinations flatter than 15 degrees are infeasible for gravity-grouted soil nails.

(8) Specify the minimum thickness of the shotcrete wall facing.

(9) Specify the minimum thickness of the cast-in-place permanent wall facing. If the wall is to have no cast-in-place permanent wall facing, this sentence may be eliminated from the plan note.

(10) Provide the specified number of verification tests for the plan estimated quantities.

(11) Provide the calculated number of proof tests for the plan estimated quantities.

(12) Provide the calculated quantity of structure excavation for the plan estimated quantities required to excavate for the wall face. Show the pay limit between structure excavation (Item 503) and general earthwork excavation (Item 203) in the plan details.

(13) Provide the calculated quantity of reinforcing steel for both the shotcrete facing and cast-in-place permanent facing for the Plan estimated quantities.

(14) Provide the calculated square yards of cast-in-place permanent facing for the plan estimated quantities.

(15) Provide the calculated square yards of geocomposite drain for the plan estimated quantities. Geocomposite drain shall provide 100% coverage of the wall face, between the cut soil face and the shotcrete facing, with the exception of small gaps necessary for the placement of the soil nails and anchorage.

(16) Provide the calculated quantity of corrugated polyethylene smooth lined pipe for the plan estimated quantities in order to construct drainage pipe to an outlet or weep holes. Drainage pipe shall be continuous and sloped at a minimum 1% grade to provide a positive gravity flow to an outlet. Show the approximate location of the outlet or weep holes on the plan view.

(17) Provide the calculated square yards of shotcrete facing for the plan estimated quantities.

* Typically, Hollow Bar Soil Nails (HBSN) are not allowed, due to concerns with quality control and difficulties in isolating bonded and unbonded segments for load testing; in this case, this sentence shall remain as-is in the plan note. However, HBSN are quick to install, and are generally preferable in granular soil conditions to avoid the necessity for a cased excavation; in this case, remove the word "*not*" from this sentence in the plan note.

** Any soil nail wall supporting a bridge or other type of structure shall be considered a critical structure for considerations of overall (global) stability, per GEC 7 Table 5.1, and shall meet a minimum Factor of Safety FS = 1.5 (a LRFD resistance factor of 0.65). Soil nail walls that do not support a structure may be considered non-critical and need only meet an overall stability FS = 1.3 (a LRFD resistance factor of 0.75). In this case, this sentence may be eliminated from the plan note.

Design soil nail walls in accordance with AASHTO LRFD Article 11.12, as modified by GDM Section 1500. See BDM Section 307.9, plan note Soil Nail Retaining Wall, and Special Provision Soil Nail Retaining Wall for additional design requirements.

Specify soil nail locations, soil nail inclination, minimum soil nail lengths, and Factored Design Load (FDL) in the Project Plans, in which:

$$FDL = \gamma_p T_{maxsn} \leq R_{po}\varphi_{po}$$

Where:

γ_p = maximum load factor for vertical earth pressure EV from AASHTO LRFD Table 3.4.1-2 = 1.35,
T_{maxsn} = maximum tensile force in the soil nail (kips),
R_{po} = nominal pullout resistance of the soil nail (kips), and
φ_{po} = resistance factor for soil nail pullout = 0.65.

1509 TEMORARY WALLS

Temporary walls have a design service life of no more than three years. If the design service life is longer than three years, then the wall must be designed as a permanent wall. See BDM Section 307.10 for additional design requirements.

Temporary walls can be of the same type as any permanent retaining wall. However, special retaining wall types that are only used for temporary applications include wire faced MSE walls, fabric wrapped walls, and temporary shoring (sheet pile temporary support of excavations). These wall types are covered in the following sections.

1509.1 Wire Faced MSE Walls

Design wire faced MSE walls in accordance with SS867. Perform the design for external stability as an MSE wall according to GDM Section 1504. Temporary wire faced MSE wall systems use the following pay item:

- 867 (LS) Temporary Wire Faced Mechanically Stabilized Earth Wall

Item 867 is lump sum, and includes all items necessary to complete the wall, in accordance with SS867. Reference SS867 Temporary Wire Faced Mechanically Stabilized Earth Wall in the plans for all MSE walls utilizing a temporary wire face.

This wall type can be used in permanent applications as a two-stage MSE wall, in accordance with Section 1504.1.

1509.2 Fabric Wrapped Walls

Fabric wrapped walls are also known as geotextile faced MSE walls. The face of a fabric wrapped wall is typically steeper than 1H:1V. Otherwise, these are essentially the same as reinforced soil slopes (RSS).

Design fabric wrapped walls as reinforced soil slopes in accordance with publications FHWA-NHI-10-024 and FHWA-NHI-10-025, Geotechnical Engineering Circular 11 (GEC 11); SS863; and GDM Section 502.2, except that geosynthetics (geotextile sheets) comprise both the soil reinforcement and the face of the wall. There is no need for a wrapped or shingled erosion control mat. Geotextiles used for fabric wrapped walls shall conform to Class 1 geotextile, woven, less than 50% elongation, as specified in AASHTO M288. Determine Ultimate Tensile Strength (strength per unit width) of the geotextile reinforcement from wide strip tests in accordance with ASTM D4595.

1509.3 Sheet Pile Temporary Support of Excavations

Specify sheet pile temporary support of excavations in accordance with the Excavation Bracing Plan Warrants in BDM Section 310.1.1.2. Design sheet pile temporary support of excavations as a cantilever sheet pile wall in accordance with Section 1506. See BDM Section 310.1 for additional design requirements.

For sheet pile temporary support of excavations, specify in the project plans the minimum length of the sheet pile sections and minimum section modulus per unit width of the sheet pile. Only specify a minimum moment of inertia per unit width of the sheet pile if the design is deflection critical (the sheet pile is supporting a utility, structure, or roadway that would be damaged by excessive deflection).

Provide a full detailed design in the project plans for all other aspects of a temporary support of excavation system, including walers, tiebacks, soldier piles, lagging, etc. Provide all notes and detail drawings necessary to specify all aspects of the design. Also provide BDM plan note 611.3-1 and include Item 503 (Lump Sum) Cofferdams and Excavation Bracing in the plans.

SECTION 1600 NOISE BARRIER FOUNDATIONS

1601 GENERAL

A noise barrier (also called a sound barrier, soundwall, noise wall, or acoustic barrier) is an exterior structure designed to block noise from traveling between adjacent sites. The wall height depends on the trajectory and level of the noise. Noise barriers can either be ground-mounted (supported on shallow or deep foundations) or structure-mounted (supported on bridges, crashworthy traffic railings, or retaining walls). Extensive use of noise barriers began in the United States after noise regulations were introduced in the early 1970s. See BDM Section 800 for design details of the anchor bolts, posts, and panels.

The procedure to select the drilled shaft length for the standard noise barrier system used by ODOT is outlined in Section 1603. Custom design for all other foundation types or sizes is performed in accordance with Section 1604.

1602 SUBSURFACE EXPLORATION

At each noise barrier alignment, develop a subsurface exploration program along the wall alignment which yields sufficient subsurface data to develop geotechnical profiles and models. The historic geotechnical data search may yield useful data for planning additional borings or reducing the number of necessary new borings. See SGE Section 303.7.5 for guidance on the minimum subsurface exploration requirements for a noise barrier (Boring Type E4). Present the noise barrier geotechnical exploration according to SGE Section 703.

1603 DRILLED SHAFT STANDARD DESIGN

The standard noise barrier foundation used by ODOT is a 30-inch diameter drilled shaft with a maximum length of 25 feet, the design of which is described through the procedure herein and is limited to the following conditions:

- Noise barrier heights up to 20 feet tall,
- Post spacings up to 24 feet center-to-center, and
- Ground slopes no steeper than 2H:1V.

Conditions marked with an "*" in Figures 1600-1 and 1600-2, indicate where a standard drilled shaft length would exceed the maximum drilling exploration depth of 25 feet. Obstructions such as underground utilities, drainage facilities, MSE wall components, etc. may preclude the use of standard 30-inch diameter drilled shafts as the foundation type. In these cases, the designer should consider reducing the barrier post spacing; relocating the barrier alignment to a different location; or performing a custom foundation design as described in Section 1604.

The standard drilled shaft design procedure is as follows:

1) At each noise barrier boring location, from 2.5 feet to 25 feet in 2.5-foot increments, use SPT N_{60} blow counts.

2) For cohesionless soils, correct the N_{60} values for overburden pressure ($N1_{60}$) in accordance with AASHTO LRFD Article 10.4.6.2.4. Alternatively, multiply the N_{60} values by the overburden correction factors in Table 1600-1. For cohesive soils, the correction factor, CN, is always 1.0 ($N1_{60} = N_{60}$). For proposed embankment fill, to be placed and compacted according to C&MS 203, assume $N1_{60} = 20$.

Table 1600-1: Overburden Correction Factors

Depth (ft.)	Correction Factor, C_N	Depth (ft.)	Correction Factor, C_N
2.5	1.64	15.0	1.04
5.0	1.40	17.5	0.98
7.5	1.27	20.0	0.94
10.0	1.17	22.5	0.90
12.5	1.10	25.0	0.86

3) The design N value used to establish the required minimum shaft length is the average of the $N1_{60}$ values along the length of the drilled shaft.

4) Establish the design soil type as granular or cohesive at each boring, based on the majority soil type within the design length of the drilled shaft foundation. Consider soil classifications A-1 and A-3 to be granular. Consider A-2 soil to be granular when the plasticity index is less than 7. Consider A-4 soil to be granular when the plasticity index is less than 4. Consider all other soil classifications to be cohesive.

5) According to the design soil type, select either the Soil Foundation Depth Table for granular soil (Figure 1600-1) or cohesive soil (Figure 1600-2) to determine the required drilled shaft length for the assumed post spacing and wall height at each boring location.

6) If bedrock is anticipated within the drilled shaft length required by the standard design procedure and the bedrock has an unconfined compressive strength of 7500 psi or greater, provide the shorter of either the required shaft length as calculated per the procedure described above or a shaft length including a 3-foot maximum rock socket length. For bedrock that has an unconfined compressive strength of less than 7500 psi, provide the shorter of either the required shaft length as calculated per the procedure described above or a shaft length including a 5-foot maximum rock socket length.

7) Avoid frequently varying plan specified shaft lengths throughout the project; use a minimum increment of plan specified shaft length of 2 feet.

The required minimum drilled shaft lengths were calculated through ODOT Research Project FHWA/OH-2022-19, “Division of Engineering Services Research On Call Agreement #34652 Task #6 Noise Barrier Foundation Design”, dated July 2022. OGE has developed a spreadsheet to perform the standard drilled shaft design, and it is available on the OGE website.

Figure 1600-1: Granular Soil Foundation Depths

Granular Soil Foundation Depth Table

	Post Spacing (PS) [ft]				Foundation Depth [ft]							
	PS ≤ 8'	8'< PS ≤ 12'	12'< PS ≤ 16'	16'< PS ≤ 24'	Soil Properties	N[(4)]	2-3	4-9	10-19	20-29	30-49	50-60
						φ[(5)]	25-32	27-35	30-38	32-40	34-40	36-44
Barrier Height (H) [ft]	H ≤ 12'	H ≤ 10'	H ≤ 8'	H ≤ 6'	Transverse Ground Slope	Level	9.0	7.5	6.0	6.0	6.0	6.0
						5:1	9.0	7.5	6.0	6.0	6.0	6.0
						4:1	9.0	7.5	6.5	6.0	6.0	6.0
						3:1	9.0	8.0	6.5	6.0	6.0	6.0
						2:1	9.0	8.5	7.5	7.0	6.0	6.0
	12'< H ≤ 16'	10'< H ≤ 14'	8'< H ≤ 12'	6'< H ≤ 10'		Level	10.5	9.0	8.0	7.5	6.5	6.0
						5:1	10.5	9.0	8.0	7.5	7.0	6.0
						4:1	10.5	9.5	8.5	8.0	7.0	6.5
						3:1	10.5	10.0	9.0	8.5	7.5	6.5
						2:1	12.5	11.5	10.0	9.5	8.5	7.5
	16'< H ≤ 20'	14'< H ≤ 20'	12'< H ≤ 16'	10'< H ≤ 14'		Level	12.0	10.5	9.5	9.0	8.0	7.0
						5:1	12.0	11.5	10.0	9.5	8.5	7.5
						4:1	12.5	11.5	10.5	10.0	9.0	8.0
						3:1	13.0	12.5	11.0	10.5	9.5	8.0
						2:1	15.0	14.0	12.5	11.5	10.5	9.0
			16'< H ≤ 20'	14'< H ≤ 20'		Level	13.5	13.0	12.0	11.0	10.0	9.0
						5:1	15.0	14.0	13.0	12.0	10.5	9.5
						4:1	15.5	14.5	13.5	12.5	11.0	10.0
						3:1	16.5	15.5	14.0	13.0	11.5	10.5
						2:1	24.5	17.5	15.5	14.5	13.0	11.5

Notes:

1. The foundation depth is the required embedment into in-situ soil.
2. Where new embankment soil will be placed and compacted according to C&MS 203, assume a corrected SPT $N1_{60}$ value of 20.
3. Barrier Height [H] is the distance from the top of the drilled shaft to the top of the higher barrier wall at the post rounded to the nearest foot.
4. N = Corrected Design SPT $N1_{60}$ value (see Section 1603).
5. φ = Estimated friction angle (degrees).

Figure 1600-2: Cohesive Soil Foundation Depth

Cohesive Soil Foundation Depth Table

	Post Spacing (PS) [ft]				Foundation Depth [ft]						
	PS ≤ 8'	8'< PS ≤ 12'	12'< PS ≤ 16'	16'< PS ≤ 24'	Soil Properties	N(4)	1	2-3	4-8	9-15	16-32
Barrier Height (H) [ft]	H ≤ 12'	H ≤ 10'	H ≤ 8'	H ≤ 6'	Transverse Ground Slope	Level	13.5	10.0	7.0	6.0	6.0
						5:1	14.5	10.5	7.0	6.0	6.0
						4:1	15.0	10.5	7.0	6.0	6.0
						3:1	15.0	11.0	7.0	6.0	6.0
						2:1	16.0	11.5	7.5	6.0	6.0
	12'< H ≤ 16'	10'< H ≤ 14'	8'< H ≤ 12'	6'< H ≤ 10'		Level	19.5	14.0	9.5	6.0	6.0
						5:1	21.0	15.5	10.0	6.0	6.0
						4:1	21.0	15.5	10.0	6.0	6.0
						3:1	21.5	16.0	10.5	6.5	6.0
						2:1	23.0	17.0	11.0	6.5	6.0
	16'< H ≤ 20'	14'< H ≤ 20'	12'< H ≤ 16'	10'< H ≤ 14'		Level	25.0	18.0	12.5	7.5	6.0
						5:1	*	20.0	13.0	8.0	6.5
						4:1	*	20.0	13.5	8.0	6.5
						3:1	*	21.0	13.5	8.0	6.5
						2:1	*	22.5	14.5	8.0	6.5
			16'< H ≤ 20'	14'< H ≤ 20'		Level	*	23.5	16.0	10.0	6.0
						5:1	*	*	17.0	10.0	7.5
						4:1	*	*	17.5	10.0	8.0
						3:1	*	*	18.0	10.0	8.5
						2:1	*	*	19.0	10.5	8.5

Notes:

1. The foundation depth is the required embedment into in-situ soil or rock.
2. Where new embankment soil will be placed and compacted according to C&MS 203, assume a corrected SPT $N1_{60}$ value of 20.
3. Barrier Height [H] is the distance from the top of the drilled shaft to the top of the higher barrier wall at the post rounded to the nearest foot.
4. N = Corrected Design SPT $N1_{60}$ value (see Section 1603).
5. * Drilled shaft length exceeds the maximum drilling exploration depth; perform a custom design or relocate the noise wall.

1604 NOISE BARRIER FOUNDATION CUSTOM DESIGN

For conditions other than those described in Section 1603, design the noise barrier foundations to withstand lateral wind and earth pressures and self-weight of the wall, in accordance with the procedure provided herein and the general principles specified in AASHTO LRFD Sections 3 and 15 and in Sections 10 or 11, depending on earth retention and/or the foundation type selected.

For additional information, see the white paper contained within the aforementioned ODOT Research Project Task #6, in which the loading, design procedure, and method of interpreting results are described. It is intended for ODOT design consultants to use when the standard design tables do not apply.

1604.1 Foundation Types

Ground-mounted post and panel noise barrier walls can either be founded on shallow foundations or deep foundations. Deep foundations for noise barrier walls can be drilled shafts, Continuous Flight Auger (CFA) piles, driven piles, or piles placed in prebored holes. Information on the valid failure mode checks and design details for these foundation types can be found in the corresponding design manual sections provided in Table 1600-2.

Table 1600-2: Corresponding Design Manual Information per Foundation Type

Foundation Type	GDM	BDM	AASHTO LRFD
Shallow (spread footing)	1303	305.2	10.6
Driven Piles	1304	305.3	10.7
Piles Placed in Prebored Holes	1305	305.3	10.7
Drilled Shafts	1306	305.4	10.8
Continuous Flight Auger (CFA) Piles	--	305.6	--

1604.2 Limit States and Special Design Considerations

The applicable limit states for noise barrier foundation design are Service I and Strength III. At the Service I Limit State, investigate noise barrier foundations for lateral and vertical deflection.

If using deep foundations, perform a Service Limit State p-y analysis to predict the lateral deflection of the foundation. Use the deflection and rotation at the top of the foundation to project deflection to the top of the noise barrier post, assuming the post to be rigid. Limit the resultant projected deflection at the top of the post to 1.5% of the height of the wall. The resultant projected deflection is the sum of the deflection at the top of the foundation plus the lateral rotation in radians at the top of the foundation times the height of the post above the foundation. It is not necessary to achieve a fully fixed tip of the foundation to “stabilize” the amount of deflection.

Do not use cyclic loading soil properties for the p-y analysis. Cyclic loading analyses are utilized in the case of repeated loads over time, such as wind, waves, collisions, etc. that will "loosen" the foundations, opening progressively larger gaps around the foundation element that will backfill with water and weaken the soils. This is appropriate in the case of a rigid-pole or non-fixed tip condition, where the pole will work back-and-forth with progressively larger deflections. However, in the case of noise barrier foundations, the relative lack of stiffness of the foundation elements, and the very small deflections in the foundation preclude the soils exhibiting cyclic loading behaviors.

Service Limit State lateral deflection is not checked for noise barriers founded on spread footings; instead check sliding resistance and limiting eccentricity in the Strength III Limit State.

Limit differential vertical settlement between two adjacent posts to 1/100.

At the Strength III Limit State, investigate noise barrier foundations for:

- bearing resistance, sliding resistance, and limiting eccentricity for shallow foundations only;

- Overturning resistance and structural resistance to shear and moment bending for deep foundations only;

- Overall (global) stability if the foundation is installed on or near a slope.

For deep foundations, check the geotechnical resistance against overturning of the noise barrier foundation with a p-y analysis in accordance with BDM Sections 305.1.2 and 307.1.4. For input into this analysis, calculate the resultant Strength III Limit State factored shear and moment loads at the top of the vertical foundation element. The check of overturning resistance is not a check of the structural capacity of the foundation, but of the geotechnical resistance of the soil to resist excessive overturning forces. The check of overturning resistance will give the minimum length and diameter of the vertical foundation element necessary to satisfy the requirements of stability.

For deep foundations, check the factored structural resistance against the maximum shear and moment developed in the foundation element from the p-y analysis under the Strength III Limit State loads.

In Ohio, the foundations of noise barrier walls are not investigated at the Extreme Event Limit States because low seismicity precludes Extreme Event I; and noise barriers are not designed to resist vehicle or vessel collision (CT or CV), precluding Extreme Event II.

1604.3 Load and Load Factors

Use Wind Exposure Category C for the design. While Wind Exposure Category D for flat, unobstructed areas will result in greater wind loads, it is unlikely that noise barriers will be located in such areas. In the unlikely event that a noise barrier is required in a flat, unobstructed area that meets the criteria for Wind Exposure Category D, then use category D as appropriate.

The design 3-second gust wind speed (V) is 70 mph for the Service I Limit State and 115 mph for the Strength III Limit State in accordance with AASHTO LRFD Table 3.8.1.1.2-1. The pressure exposure and elevation coefficient (K_z) is 1.0 for the Service I and Strength III Limit States in accordance with AASHTO LRFD Article 3.8.1.2.1. The gust effect factor (G) is 1.0 for the Service I Limit State and 0.85 for the Strength III Limit State in accordance with AASHTO LRFD Table 3.8.1.2.1-1. The drag coefficient (C_D) is 1.2 for the Service I and Strength III Limit States in accordance with AASHTO LRFD Table 3.8.1.2.1-2.

These specified wind speeds and coefficients result in a design wind pressure (P_Z) on the wall of 15.05 psf at the Service I Limit State and 34.53 psf at the Strength III Limit State, calculated using AASHTO LRFD Equation 3.8.1.2.1-1.

Apply the wind pressure uniformly over the area of the noise barrier panel. Calculate the shear and the moment at the bottom of the noise barrier post/top of the foundation due to the wind load. Calculate the moment using a moment arm equivalent to 55 percent of the noise barrier height in accordance with AASHTO LRFD Article 3.8.1.2.4.

For axial loads on the noise barrier foundation, use 75 psf multiplied by the height of the noise barrier and the post spacing. For shallow foundations in the Strength III Limit state, apply a minimum or maximum DC load factor to the axial load (γ_p = 1.25 or 0.90), as applicable, depending on if the load will increase or decrease the stability of the foundation.

For deep foundations in the Strength III Limit state, the axial load will tend to slightly decrease the stability of the foundation but will slightly improve the bending stiffness and moment resistance of the drilled shaft. However, neither effect is significant. Therefore, do not apply a minimum or maximum load factor to the axial loads in the lateral load analysis (γ_p = 1.00).

If the noise barrier panels are designed to retain soil, then include earth pressures as appropriate. Standard noise barriers can support up to 2 feet of differential earth pressure load from one side to the other; retaining a greater height of soil will require a custom design of the noise barrier anchor bolts, posts, and panels. Through research we concluded that ice loads have little to no effect on the foundations. Generally, no other loads are applicable to the design of noise barrier foundations.

Although it is not explicitly stated on the sheets, the Design Loads and Design Load Cases provided on sheet 1 of the NBS-1-09 sheets are to be used for structural design of the noise barrier anchor bolts, posts, and panels only, and do not apply to design of foundations.