CHAPTER 26

GEOTECHNICAL REPORTING AND DOCUMENTATION
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26.1 OVERVIEW AND GENERAL REQUIREMENTS

The Geotechnical Engineering Bureau, and some Consultants working on NYSDOT projects, produce geotechnical reports and design memorandums in support of project scope, project design, and final PS&E development (see NYSDOT GDM Chapter 1). Also produced are project specific Special Provisions, plan details, boring logs, Summary of Geotechnical Conditions, and the final project geotechnical documentation. Information developed to support these geotechnical documents are retained in the Geotechnical Engineering Bureau and Regional Geotechnical Engineers files.

The information includes project site data, subsurface exploration logs and/or seismic reports, test results, design calculations, and construction support documents. This chapter provides standards for the development and detailed checklists for review of these documents and records, with the exception of borings logs, which are covered in NYSGDM Chapter 5.

Documents and project geotechnical documentation/records produced by the Geotechnical Engineering Bureau, and Consultants working on NYSDOT projects, shall meet as applicable the informational requirements listed in the following FHWA manual:


This FHWA manual also includes a PS&E review checklist. The PS&E review checklist contained in this FHWA manual should be used to supplement the NYSDOT Geotechnical Engineering Bureau PS&E review checklist provided in NYSDOT GDM Appendix 26-A. These checklists should be used as the basis for evaluating the completeness of the PS&E regarding incorporation of the project geotechnical recommendations and geotechnical data included in the geotechnical report for the project.

26.2 REPORT CERTIFICATION AND GENERAL FORMAT

New York State Education Law, Article 145, Professional Engineering and Land Surveying, Section 7209 Special Provisions, 2. States “To all plans, specifications, plats and reports to which the seal of a professional engineer or land surveyor has been applied, there shall also be applied a stamp with appropriate wording warning that it is a violation of this law for any person, unless he is acting under the direction of a licensed professional engineer or land surveyor, to alter an item in any way. If an item bearing the seal of an engineer or land surveyor is altered, the altering engineer or land surveyor shall affix to the item his seal and the notation "altered by" followed by his signature and the date of such alteration, and a specific description of the alteration”.

NYSDOT Engineering Instruction on Policy on Professional Seals and Signatures (EI 08-001) states that Design Approval Documents shall be sealed and signed by the Group Director responsible for the project. Every Final Design Report, construction plan sheet, and field change
Sheets must bear the seal and signature of the professional responsible for its production. The NYSDOT will retain copies of the final sealed and signed documents and supporting material in the official project files for a period of not less than six years, or longer if required by the NYSDOT’s record retention policies. The supporting material may include, but is not limited to; drawings, specifications, reports, calculations and references to applicable codes and standards. Professionals sealing a design work will place supporting material in the project file, in accordance with the requirements of the Rules of the Board of Regents, Part 29 Unprofessional Conduct, §29.3.a.3, and will have access to the file for a minimum of six years.

Figure 26-1 Note Used in Conjunction with Professional Engineer Stamp

Table 26-1 provides a listing of reports and geotechnical design memoranda produced by the Geotechnical Engineering Bureau, the type of certification needed to be consistent with the above mentioned policies and the general format that would typically be used.

For reports that cover individual project elements, a geotechnical design memorandum may suffice, with the exception of large embankment construction or structure foundation construction, in which case a formal geotechnical report should be issued. For reports for NYSDOT projects, a formal geotechnical report should be issued to the Designer incorporating the geotechnical element into the project design. For geotechnical reports that are sent to agencies outside of NYSDOT, a letter report format will be used in place of the memorandum format. Alternatively, a formal report transmitted with a letter may be used.

E-mail may be used for geotechnical reporting in certain circumstances. E-mails may be used to transmit review of construction submittals, and Regional soil reports sent to the Geotechnical Engineering Bureau for concurrence. E-mails may also be used to transmit conceptual foundation or other conceptual geotechnical recommendations. In both cases, a print-out of the e-mail should be included in the project file. For time critical geotechnical designs sent by e-mail that are not
conceptual, the e-mail should be followed up with a stamped memorandum or report as soon as possible. A copy of the e-mail should also be included in the project file.

For reports produced by others outside of NYSDOT, the certification requirements described herein are applicable, but the specific report format will be as mutually agreed upon by the Geotechnical Engineering Bureau and those who are producing the report.
### A. Highway Design & Construction Section

<table>
<thead>
<tr>
<th>Subject</th>
<th>General Format</th>
<th>Who Certifies</th>
<th>Type of Certification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geotechnical Summary Report</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Plan Review</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Retaining Wall Analysis Report (Wall Alternatives: Cut Walls or Fill Walls)</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Construction Submittal Review</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Claim Review</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Case History</td>
<td>Report</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Technical Report</td>
<td>Report</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Lessons Learned</td>
<td>Report/ Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Geotechnical Baseline Report (DB Projects)</td>
<td>Report</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
</tbody>
</table>

### B. Subsurface Explorations Section

<table>
<thead>
<tr>
<th>Subject</th>
<th>General Format</th>
<th>Who Certifies</th>
<th>Type of Certification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Terrain Reconnaissance Report</td>
<td>Report</td>
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<td>Signature</td>
</tr>
<tr>
<td>Building Foundation Report</td>
<td>Report</td>
<td>Section Head</td>
<td>PE Seal, Dated and Signed</td>
</tr>
<tr>
<td>Canal Impounding Embankment Inspection Report</td>
<td>Report</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Canal Dredge Sampling Report</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Monitoring Well Sampling and Testing Report</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Earth Dam Inspection Report</td>
<td>Report</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
</tbody>
</table>

**Table 26-1 NYSDOT Geotechnical Report Certification and Format Requirements**
C. Structure Foundations Section

<table>
<thead>
<tr>
<th>Subject</th>
<th>General Format</th>
<th>Who Certifies</th>
<th>Type of Certification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preliminary Foundation Recommendations</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Foundation Design Report</td>
<td>Report</td>
<td>Section Head &amp; Office of Structures</td>
<td>PE Seal, Dated and Signed</td>
</tr>
<tr>
<td>Retaining Wall Analysis Report (Sheeting, Soldier Pile &amp; Lagging, etc.)</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>1. Wall Alternatives: Signature</td>
</tr>
<tr>
<td>Plan Review</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Construction Submittal Review</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
</tbody>
</table>

D. Roadway Foundation Section

<table>
<thead>
<tr>
<th>Subject</th>
<th>General Format</th>
<th>Who Certifies</th>
<th>Type of Certification</th>
</tr>
</thead>
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<tr>
<td>Preliminary Embankment Foundation Report</td>
<td>Memorandum</td>
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<tr>
<td>Embankment Foundation Report</td>
<td>Report</td>
<td>Section Head</td>
<td>PE Seal, Dated and Signed</td>
</tr>
<tr>
<td>Retaining Wall Analysis Report (Geosynthetically Reinforced Soil System (GRSS, Geocells, etc.)</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>1. Wall Alternatives: Signature</td>
</tr>
<tr>
<td>Plan Review</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
<tr>
<td>Construction Submittal Review</td>
<td>Memorandum</td>
<td>Section Head</td>
<td>Signature</td>
</tr>
</tbody>
</table>

Table 26-1 NYSDOT Geotechnical Report Certification and Format Requirements (cont.)
E. Engineering Geology Section

<table>
<thead>
<tr>
<th>Subject</th>
<th>General Format</th>
<th>Who Certifies</th>
<th>Type of Certification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Survey</td>
<td>Memorandum</td>
<td>Squad Leader</td>
<td>Signature</td>
</tr>
<tr>
<td>Rock Slope Recommendations (Recut, Scaling, etc.)</td>
<td>Memorandum</td>
<td>Squad Leader</td>
<td>Signature</td>
</tr>
<tr>
<td>Water Well Report</td>
<td>Report</td>
<td>Squad Leader</td>
<td>Signature</td>
</tr>
<tr>
<td>Bearing Resistance of Footings on Rock (Construction Inspection)</td>
<td>Memorandum</td>
<td>Squad Leader</td>
<td>Signature</td>
</tr>
</tbody>
</table>

Table 26-1 NYSDOT Geotechnical Report Certification and Format Requirements (cont.)
26.3 GEOTECHNICAL ENGINEERING BUREAU REPORT CONTENT REQUIREMENTS

A detailed description of the geotechnical information and types of recommendations that should be provided in geotechnical reports is provided in the sections that follow. Both conceptual and preliminary level reports and final reports are addressed.

26.3.1 Conceptual or Preliminary Level Geotechnical Reports

Conceptual level geotechnical reports are typically used to provide geotechnical input for the following:

- developing the project scope,
- development of preliminary bridge substructure layouts,
- Conceptual geotechnical studies for environmental permit development activities,
- Reconnaissance level corridor studies,
- development of Environmental Impact Statement (EIS) technical analyses to examine and evaluate issues that are significant and propose mitigation for significant impacts that are identified, and
- Geotechnical Baseline Reports (GBR) for design-build projects (see NYSDOT GDM Chapter 25).

Preliminary level geotechnical reports are typically used to provide geotechnical input for the following:

- the determination of preliminary location and size of infiltration facilities,
- alternative analyses (e.g., temporary retaining walls for structure replacements, preliminary grading analyses, preliminary foundation analysis, etc.),
- rapid assessment of emergency repair needs (e.g., landslides, rockfall, structure foundation scour, etc.).

Conceptual level geotechnical reports are in general developed based on a minimum of an office review of existing geotechnical data for the site, and generally consist of feasibility assessment and identification of geologic hazards. Geotechnical design for conceptual level reports is typically based on engineering judgment and experience at the site or similar sites.

For preliminary level design, a geological reconnaissance of the project site and a limited subsurface exploration program are usually conducted, as well as some detailed geotechnical analysis to characterize key elements of the design, adequate to assess potential alternatives and estimate preliminary costs. For conceptual level design of more complex projects with potentially unusual subsurface conditions, or potential instability, a geotechnical reconnaissance of the site should be conducted in addition to the office review to assess the site conditions. Note that for preliminary design of infiltration facilities, the seasonal ground water depth should be established early in the project to assess feasibility (i.e., during project scoping), since it usually takes a minimum of one season to characterize groundwater conditions. A minimum of one to two test holes, with piezometers installed, are usually required to establish the water table depth for this
These conceptual or preliminary level reports should contain the following elements:

1. A general description of the project, project elements, and project background.
2. A brief summary of the regional and site geology. The amount of detail included here will depend on whether the report is at the conceptual or preliminary level, and on the type of report. For example, an EIS technical analyses will tend to need a more detailed discussion on site and regional geology than would a conceptual bridge foundation report, an emergency landslide, or a scour repair evaluation report.
3. A summary of the site data available from which the conceptual or preliminary recommendations were made.
4. A summary of the field exploration conducted, if applicable.
5. A summary of the laboratory testing conducted, if applicable.
6. A description of the project soil and rock conditions. The amount of detail included here will depend on whether the report is at the conceptual or preliminary level, and on the type of report. For preliminary design reports in which new borings have been obtained, soil profiles for key project features (e.g., bridges, major walls, etc.) may need to be developed and tied to this description of project soil and rock conditions.
7. A summary of geological hazards identified that may affect the project design (e.g., landslides, rockfall, debris flows, liquefaction, soft ground or otherwise unstable soils, seismic hazards, etc.), if any.
8. A summary of the conceptual or preliminary geotechnical recommendations.
9. Appendices that include any boring logs and laboratory test data obtained, soil profiles developed, any field data obtained, and any photographs.

26.3.2 Final Geotechnical Design Reports

Final (PS&E level) geotechnical reports are in general developed based on an office review of existing geotechnical data for the site, a detailed geologic review of the site, and a complete subsurface investigation program meeting AASHTO and FHWA standards, or as augmented in this (NYSDOT GDM) manual. Final geotechnical reports should contain the following elements:

1. A general description of the project, project elements, and project background.
2. Project site surface conditions and current use.
3. Regional and site geology. This section should describe the site stress history and depositional/erosional history, bedrock and soil geologic units, etc.
4. Regional and site seismicity. This section should identify potential source zones, potential magnitude of shaking, frequency, historical activity, and location of nearby faults. This section is generally only included in reports addressing structural elements (e.g., bridges, walls, etc.) and major earthwork projects.
5. A summary of the site data available from project or site records (e.g., final construction records for previous construction activity at the site, as-built bridge or other structure layouts, existing subsurface exploration logs, geologic maps, previous or current geologic reconnaissance results, etc.).
6. A summary of the field exploration conducted, if applicable. Here, a description of the methods and standards used is provided, as well as a summary of the number and types of explorations that were conducted. Include also a description of any field instrumentation installed and its purpose. Refer to the detailed logs located in the report appendices.

7. A summary of the laboratory testing conducted, if applicable. Again, a description of the methods and standards used is provided, as well as a summary of the number and types of tests that were conducted. Refer to the detailed laboratory test results in the report appendices.

8. Project Soil/Rock Conditions. This section should include not only a description of the soil/rock units encountered, but also how the units tie into the site geology. Ground water conditions should also be described here, including the identification of any confined aquifers, artesian pressures, perched water tables, potential seasonal variations, if known, any influences on the ground water levels observed, and direction and gradient of ground water, if known. If rock slopes are present, discuss rock structure, including the results of any field structure mapping (use photographs as needed), joint condition, rock strength, potential for seepage, etc.

These descriptions of soil and rock conditions should in general be illustrated with subsurface profiles (i.e., cut parallel to roadway centerline) and cross-sections (i.e., cut perpendicular to roadway centerline) of the key project features. A subsurface profile or cross-section is defined as an illustration that assists the reader of the geotechnical report to visualize the spatial distribution of the soil and rock units encountered in the borings and probes for a given project feature (e.g., structure, cut, fill, landslide, etc.). As such, the profile or cross-section will contain the existing and proposed ground line, the structure profile or cross-section if one is present, the boring logs (including SPT values, soil/rock units, etc.), and the location of any water table(s). Interpretive information contained in these illustrations should be kept to a minimum. What appears to be the same soil or rock unit in adjacent borings should not be connected together with stratification lines unless that stratification is reasonably certain. The potential for variability in the stratification must be conveyed in the report, if a detailed stratification is provided. In general, geologic interpretations should not be included in the profile or cross-section, but should be discussed more generally in the report.

A subsurface profile must always be provided for structure replacement or widening (in conjunction with the Foundation Design Report). For other situations, judgment may be applied to decide whether or not a subsurface profile is needed. In these instances, the Departmental Geotechnical Engineer may opt to describe the subsurface profile in a paragraph(s) outlining the subsurface exploration program performed for the project, the type of soil testing utilized in examination of the subsurface conditions and identification of the concluding subsurface conditions including subsurface strata and groundwater elevation.

9. Summary of geological hazards identified and their impact on the project design (e.g., landslides, rockfall, debris flows, liquefaction, soft ground or otherwise unstable soils, seismic hazards, etc.), if any. Describe the location and extent of the geologic hazard.

10. For analysis of unstable slopes (including existing settlement areas), cuts, and fills, background regarding the following:
   - analysis approach,
• assessment of failure mechanisms,
• determination of design parameters, and
• any agreements with Region or other customers regarding the definition of acceptable level of risk.

Included in this section would be a description of any back-analyses conducted, the results of those analyses, comparison of those results to any laboratory test data obtained, and the conclusions made regarding the parameters that should be used for final design.

11. Geotechnical recommendations for earthwork (fill design, cut design, etc.). This section should provide embankment design recommendations, if any are present, such as the slope required for stability, any other measures that need to be taken to provide a stable embankment (e.g., geosynthetic reinforcement, wick drains, controlled rate of embankment construction, lightweight materials, etc.), embankment settlement magnitude and rate, and the need and extent of removal of any unsuitable materials beneath the proposed fills. Cut design recommendations, if any are present, are also provided in this section, such as the slope required for stability, seepage and piping control, erosion control measures needed (concept only – other NYSDOT offices will provide the details on this issue), and any special measures required to provide a stable slope.

12. Geotechnical recommendations for rock slopes and rock excavation. Such recommendations should include, but are not limited to, stable rock slope, rock bolting, and other stabilization requirements, including recommendations to prevent erosion/undermining of intact blocks of rock, internal and external slope drainage requirements, feasible methods of rock removal, etc.

13. Geotechnical recommendations for stabilization of unstable slopes (e.g., landslides, rockfall areas, debris flows, etc.). This section should provide a discussion of the mitigation options available, and detailed recommendations regarding the most feasible options for mitigating the unstable slope, including a discussion of the advantages, disadvantages, and risks associated with each feasible option.

14. Geotechnical recommendations for bridges, tunnels, hydraulic structures, and other structures. This section should provide a discussion of foundation options considered, the recommended foundation options, and the reason(s) for the selection of the recommended foundation option(s), foundation design requirements (for strength limit state - ultimate bearing resistance and depth, lateral and uplift resistance, for service limit state - settlement limited bearing, and any special design requirements), seismic design parameters and recommendations (e.g., design acceleration coefficient, soil profile type for standard AASHTO response spectra development, or develop non-standard response spectra, liquefaction mitigation requirements, extreme event limit state bearing, uplift, and lateral resistance, and soil spring values), design considerations for scour when applicable, earth pressures on abutments and walls in buried structures, and recommendations regarding bridge approach slabs.

15. Geotechnical recommendations for retaining walls and reinforced slopes. This section should provide a discussion of wall/reinforced slope options considered, the recommended wall/reinforced slope options, and the reason(s) for the selection of the recommended option(s), foundation type and design requirements (for strength limit state - ultimate bearing resistance, lateral and uplift resistance if deep foundations selected, for service limit state - settlement limited bearing, and any special design requirements), seismic
design parameters and recommendations (e.g., design acceleration coefficient, extreme event limit state bearing, uplift and lateral resistance if deep foundations selected) for all walls except standard concrete cantilever walls, design considerations for scour when applicable, and lateral earth pressure parameters (provide full earth pressure diagram for non-gravity cantilever walls and anchored walls). For nonproprietary walls/reinforced slopes requiring internal stability design (e.g., geosynthetic walls, soil nail walls, all reinforced slopes), provide minimum width for external and overall stability, embedment depth, bearing resistance, and settlement, and also provide soil reinforcement spacing, strength, and length requirements in addition to dimensions to meet external stability requirements. For proprietary walls, provide minimum volume for installation (see NYSDOT Standard Sheet 554-01) including minimum width for overall stability and bearing resistance. For anchored walls, provide achievable anchor capacity, no load zone dimensions, and design earth pressure distribution.

16. Geotechnical recommendations for infiltration/detention facilities. This section should provide recommendations regarding infiltration rate, impact of infiltration on adjacent facilities, effect of infiltration on slope stability, if the facility is located on a slope, stability of slopes within the pond, and foundation bearing resistance and lateral earth pressures (vaults only).

17. Long-term or construction monitoring needs. In this section, provide recommendations on the types of instrumentation needed to evaluate long-term performance or to control construction, the reading schedule required, how the data should be used to control construction or to evaluate long-term performance, and the zone of influence for each instrument.

18. Construction considerations. Address issues of construction staging, shoring needs and potential installation difficulties, temporary slopes, potential foundation installation problems, earthwork constructability issues, dewatering, etc.

19. Appendices. Typical appendices include design charts for foundation bearing and uplift, P-Y curve input data, design detail figures, layouts showing boring locations relative to the project features and stationing, subsurface profiles and typical cross-sections that illustrate subsurface stratigraphy at key locations, all boring logs used for the project design (includes older borings as well as new borings), including a boring log legend for each type of log, laboratory test data obtained, instrumentation measurement results, and special provisions needed.

The detail contained in each of these sections will depend on the size and complexity of the project or project elements and subsurface conditions. In some cases, design memoranda that do not contain all of the elements described above may be developed prior to developing a final geotechnical report for the project. The following sections outline specific final geotechnical reports:
26.3.2.1 Terrain Reconnaissance (TR) Report

The choice of an alignment for roadway construction may depend on the terrain and depositional conditions in the area. A physiographic and subsurface analysis of a site is provided to the Highway Designer in a Terrain Reconnaissance (TR) Report. A TR report describes the soils anticipated to be encountered along a length of proposed roadway. Subject areas are delineated into Areas of Interest (AOI’s). The AOI’s are depicted in maps and Engineering Properties, Water Features, Particle Size & Coarse Fragments, and Physical Soil Properties are presented for the soil types mapped in each AOI. Descriptions of the mapped types are typically presented at the end of the report.

A TR Report is an interpretive analysis of the available site information including pedologic and geologic maps, aerial photographs, subsurface conditions including soil data and information derived from the National Cooperative Soil Survey (accessed through the USDA National Resources Conservation Service), and field inspections. The physiographic and subsurface analysis of a site is outlined in a TR Report and is divided into sections, including:

- **Area Description**, which provides a location and description of the physiographic province and topographic features, drainage, geologic history and bedrock categories.
- **Route Description and Discussion**, which provides the proposed alignment(s) and a description of the soil deposits traversed.
- **Terrain Unit Descriptions**, which provides a map symbol corresponding to a terrain unit and a terrain unit description.
  
  Examples of Terrain Units include:
  
  o **Thick Till;** upland deposits comprised of soil and rock materials which were abraded, collected, mixed, transported, and finally deposited by glacial ice.
  o **Outwash Deposits;** material transported beyond the limits of the glacier by melt waters during withdrawal of the ice.
  o **Lacustrine Shore Deposits;** sediments deposited by moving water associated with the glacial lakes in the area.
  o **Alluvial Deposits;** deposits which are formed by sediments deposited by the overflow of streams.
  o **Organic Deposits;** the accumulation of plant and animal remains in depressional areas which have a water table at or close to the surface.
  o **Man-Made Features;** filled areas.
- **Generalized Terrain Map**, which provides a mapping of the proposed alignment(s) and the terrain units.
- **General Earth Engineering Considerations**, which provides the depositional unit along with the engineering significance in:
  
  o **Planning;** line and grade.
  o **Design;** line and grade, cut slope conditions, subgrade in cuts, embankment foundations, recharge and leaching basins, embankment materials, granular materials, granular materials, topsoil, pavement system.
  o **Construction;** trafficability, earthwork.
  o **Maintenance.**
26.3.2.2 Embankment Foundation Report

Embankment foundation recommendations are made to the Highway Designer in an Embankment Foundation Report. An Embankment Foundation Report describes the subsurface conditions along the proposed embankment alignment and provides the results of the geotechnical analyses performed to predict the consequence of the embankments construction.

The foundation design requirements outlined in an Embankment Foundation Report are divided into sections, including:

- Design Analysis, which provides the project description and basis of the design recommendations.
  
  The Design Analysis section is divided into subsections, including:
  - Site Information, which identifies the file and mapping information for the project.
  - Subsurface Information, which identifies the subsurface explorations progressed at the site and any additional subsurface information available (e.g. Natural Resources Conservation Services (NRCS) soil surveys).

- General Subsurface Conditions, which describes the subsurface exploration program performed for the project, the type of soil testing utilized in examination of the subsurface conditions and identifies the concluding subsurface conditions including subsurface strata and groundwater elevation.

- Stability Analysis, which provides the results of the stability analyses performed for the given embankment dimensions.

- Settlement Analysis, which may be divided into subsections, including:
  - Design Assumptions, which identifies the groundwater elevation, embankment material parameters and assumed construction practices.
  - Settlement Results, which provides the results of the settlement analyses performed for the given embankment dimensions.

- Design Recommendations, which addresses the requirements for the construction of the embankment. These requirements may identify actions such as:
  - Construction procedures for placement of material in embankment widening areas.
  - Special Notes for excavation of unsuitable material, with specific backfill requirements.
  - Specific measures required prior to placing embankment material.
  - Adhering to identified NYSDOT Standard Sheets.
  - Installation of appropriate erosion control and vegetative cover as recommended by the Regional Landscape Architect.

- Appendix, which provides information supporting the design recommendations including subsurface explorations, boring location plan, general subsurface profile, results of any soil testing, specifications, etc.
26.3.2.3 Retaining Wall Analysis Report

Retaining wall recommendations are made to the Highway Designer in a Retaining Wall Analysis Report. A Retaining Wall Analysis Report describes the subsurface conditions along the proposed retaining wall and provides the results of the geotechnical analyses performed to develop the retaining wall design details. In instances where the Highway Designer contacts the Departmental Geotechnical Engineer early in the design process, this report may include retaining wall alternatives with cost estimates.

The design requirements outlined in a Retaining Wall Analysis Report are divided into sections, including:

- **Design Analysis**, which provides the project description and basis of the design recommendations.
  
  The Design Analysis section is divided into subsections, including:
  
  - Site Information, which identifies the file and mapping information for the project.
  - Subsurface Information, which identifies the subsurface explorations progressed at the site and any additional subsurface information available (e.g. Natural Resources Conservation Services (NRCS) soil surveys).

- **General Subsurface Conditions**, which describes the subsurface exploration program performed for the project, the type of soil testing utilized in examination of the subsurface conditions and identifies the concluding subsurface conditions including subsurface strata and groundwater elevation.

- **Retaining Wall Analysis**, which may be divided into subsections, including:
  
  - Design Assumptions, which identifies the maximum retaining wall height, cross sectional geometry, anticipated loading, groundwater elevation, and soil parameters.
  - Design Results, which provides the results of the geotechnical analyses.
  - Design Alternatives, which provides the results of the geotechnical analyses of different wall types with corresponding cost estimates.

- **Installation Recommendations**, which addresses the requirements for the construction of the retaining wall. These requirements may identify actions such as:
  
  - Details for placement of wall elements in pre-excavated holes.
  - Construction procedures for placement of drainage material in specific locations.
  - Specific measures required prior to placing wall units.
  - Adhering to identified NYSDOT Standard Sheets.

- **Appendix**, which provides information supporting the design recommendations including subsurface explorations, boring location plan, general subsurface profile, results of any soil testing, specifications, etc.
26.3.2.4 Soil Forensic Report

Forensic Engineering is defined as the investigation of materials, products, structures or components that fail or do not operate or function as intended, causing personal injury or damage to property. Generally, the purpose of a forensic engineering investigation is to locate cause or causes of failure with a view to improve performance or life of a component. With respect to geotechnical engineering, forensic engineering is used in a variety of geotechnical engineering problems, including, but not limited to, the following:

- Slope failures due to instability of cut or fill slopes
- Retaining wall failures, including global and compound instability in either permanent systems or temporary shoring systems
- Shallow or deep foundation failures for structures located on slopes or over potentially unstable soils
- Mechanisms of failure for landslides

Forensic geotechnical analysis results are made to the Regional Geotechnical Engineer for distribution throughout the Region in a Soil Forensic Report. A Soil Forensic Report describes the subsurface conditions within the failure area and provides the results of the site observations and document and literature research in an investigative synthesis supporting the hypothesis of failure. The Soil Forensic Report is divided into sections, including:

- Purpose, which provides the project description, a summary of the salient aspects of the report, and an explanation of the overall purpose of the report.
- History, which provides the overview of the history of the area including:
  - Description of the failure with the chronological events and account of the investigation,
  - Construction – date of initiation and completion of construction.
  - Construction documents – a complete listing of the construction documents and other documents uncovered and used in the investigation.
- Description of Failure, which provides the project description with a detailed account of the failed construction.
- Observations, which provides a description of the site visits and detailed description of the site observations, field testing, sketches, photographs, and samples taken for laboratory testing.
- Testing, which provides an overview of the field and laboratory testing program with testing objectives and a summary of the results.
- Analysis, which provides an analysis of the failed construction and a result of the synthesis of the data resulting from the investigations.
  - A description of the methodology used for the review, computations, evaluation, and development of hypothesis.
- Conclusion, which provides the presentation of the most probable cause of the failure and discussion of the relevant factors contributing to the cause.
- Recommendations, which provides a statement of the recommended action based on the requirements of the investigation.
Remedial Action – if requested direction for repair of the failure, the report will recommend a remedial program.

Avoidance of Future Failure – statement of how this failure investigation and its findings may be used to prevent future failures in this or other similar facilities.

• Appendix, which provides detailed data utilized in the investigation report, including:
  o Subsurface explorations
  o Boring location plan
  o General subsurface profile
  o Test reports
  o Contract documents
  o Field sketches
  o Photographs
  o Relevant specifications

26.3.2.5 Foundation Design Report

Structure foundation recommendations are made to the Structure Designer in a Foundation Design Report (FDR). An FDR provides recommendations for the support of the proposed substructures along with recommendations for their construction. The FDR is typically a closure document which finalizes the agreed-upon foundation recommendations that were developed throughout the design phases of the project in an iterative process between the Structure Designer and Departmental Geotechnical Engineer.

An FDR is produced by the Geotechnical Engineering Bureau for all State or Consultant designed structure replacements or widenings. If the scope of services for a Consultant designed structure is such that the geotechnical design is task to the Consultant, the FDR will take on a different format.

The foundation design requirements outlined in an FDR are divided into sections, including:

• Design Analysis, which provides the project description, basis of the design recommendations, and overall site classification.

  The Design Analysis section is divided into subsections, including:
  o General
  o Abutments, Piers, Precast Concrete Arch, Box Culvert
  o Erosion Protection
  o Dewatering
  o Excavation and Backfill
  o Mechanically Stabilized Earth System

• Construction Requirements, which provides Special Notes to be included in the project documents.

  The Construction Requirements section is divided into subsections, including:
  o Monitoring
  o Spread Footings
  o Piles, Micropiles, Drilled Shafts
  o Dewatering, Cofferdams, Sheeting
In synchronization with the above outline, the following is an itemization of the FDR.

* The geotechnical designer should understand that these notes are written for the most common situations encountered when preparing an FDR. There is not a note written for every possible situation that a geotechnical designer will encounter. On many jobs, the geotechnical designer will have to write portions of the report due to unique features of the particular job. When drafting these portions, please keep the following things in mind:
  • The portion written by the geotechnical designer must blend with the rest of the FDR, so remember to use consistent terminology (ie, “toe treatment” instead of “pile tip”).
  • Write in the active voice (ie, “Perform building condition surveys...” instead of “The Contractor shall perform building condition surveys...”).
  • Read the final draft to be sure it all blends together and sounds like a coherent report.

There are no “standard notes” that are guaranteed to belong in every report. Consideration must be given to the appropriateness of every note used, as there are situations that make even the most commonly used note invalid.

The purpose of the footnotes is to provide commentary and uniformity in the preparation of the FDR. Commentary is not provided for every note, as many of the notes are self-explanatory.

26.3.2.5.1 General

The following are a series of tables outlining the results of the geotechnical design:

Note 26-1: The foundation design requirements are based on the following elevations:

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>(BOTTOM-OF-FOOTING)/ (BOTTOM-OF-STEM)/ (TOP-OF-FOOTING) ELEVATIONS (feet)</th>
<th>SCOUR ELEVATION$^{(1)}$ (feet)</th>
</tr>
</thead>
</table>

Notify the Foundation and Construction Unit if the above (footing)/(stem) elevations vary by more than 1 foot from the final design.

$^{(1)}$ The scour elevation at each substructure is provided by the Hydraulics Unit of the Office of Structures. If the scour predicted has a significant impact on the foundation design, contact the Office of Structures to set up a meeting with the Hydraulics Unit.
Note 26-2: The foundation design requirements are based on the following elevations:

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>CULVERT INVERT ELEVATIONS (feet)</th>
<th>WINGWALL (BOTTOM-OF-FOOTING)/(TOP-OF-FOOTING) ELEVATION (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inlet</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Outlet</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notify the Foundation and Construction Unit if the above elevations vary by more than 1 ft. from the final design.

Note 26-3(1): Based on the results of our analyses, the seismic soil classifications for the structure are:

<table>
<thead>
<tr>
<th>Operational Classification</th>
<th>Substructure</th>
<th>Seismic Soil Site Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(1) Based on current Load and Resistance Factor Design (LRFD) design code, Structural Designers will require seismic soil site classifications for all NY state bridges. Refer to Table 3.10.3.1-1 in the AASHTO Specifications for site class definitions. Bridges are also categorized into three Operational Classifications (normal, essential, or critical) which affects the level of seismic analysis. The Operational Classification should be identified in the Bridge Site Data Package.

26.3.2.5.1.1 Abutments, Piers, Precast Concrete Arch, Box Culvert

Note 26-4(1): Support the ___ on spread footings placed on (undisturbed soil(2))/(a 5 foot thick pad of compacted Select Structure Fill(3)).

The following LRFD soil parameters and resistance factors are provided for design of the structure:

**Foundation Soil Parameters**

<table>
<thead>
<tr>
<th>Substructure</th>
<th>Friction Angle (degrees)</th>
<th>Unit Weight (lbs/ft^3)</th>
<th>Max. Service Limit State Bearing Resistance (kips/ft^2)</th>
<th>Nominal Coefficient of Friction for Sliding(4)</th>
<th>Approximate Groundwater Elevation (feet)</th>
</tr>
</thead>
</table>
Strength Limit State Resistance Factors

<table>
<thead>
<tr>
<th>Condition</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing Resistance</td>
<td></td>
</tr>
<tr>
<td>Sliding</td>
<td></td>
</tr>
</tbody>
</table>

Place the following table on each substructure plan and elevation sheet in the Contract Plans. The designer shall fill in the bearing pressures to the nearest kip per square foot.

<table>
<thead>
<tr>
<th>Substructure</th>
<th>Strength Limit State Bearing Pressure (kips/ft²)</th>
<th>Service Limit State Bearing Pressure (5) (kips/ft²)</th>
</tr>
</thead>
</table>

(1) *Note:* This note is intended for the Office of Structures or a Consultant who will calculate the Strength Limit State bearing resistance based on soil parameters provided by the Geotechnical Engineering Bureau.

(2) *Undisturbed Soil (Cut Situations):* The nominal bearing resistance should be calculated using Naval Facilities Design Manual (NAVFAC) DM 7.2-131 and factored to determine the Strength Limit State bearing resistance. The Service Limit State bearing resistance is determined based on settlement calculations (typical Service Limit State bearing resistance values correspond to ASD allowable bearing capacities, i.e. 5 kips/ft² on fills, 6 kips/ft² on undisturbed soil).

(3) *Compacted Select Structural Fill (Fill Situations):* The Service Limit State bearing resistance is 5 kips/ft² in 1(V) on 2(H) slopes. The bearing resistance can vary, depending on settlement conditions or footing location (i.e. a footing on level ground will have a larger bearing capacity than a footing in a slope).

(4) *Nominal Coefficient of Friction:* The nominal coefficient of friction for sliding is tan φ. The values from NAVFAC DM 7.2-p. 63 are approximately equal to the factored coefficient of friction for sliding.

(5) *Mechanically Stabilized Earth System:* The maximum Service Limit State bearing resistance for a spread footing placed on a Mechanically Stabilized Earth System (M.S.E.S.) is 4 kips/ft². This value is what was previously used for the allowable bearing capacity in ASD design (based on 2002 AASHTO, Article 7.5.4, Abutments on Mechanically Stabilized Earth Walls).

**Note 26-5(1):** Support the ___ on spread footings placed on (undisturbed soil(2))/(a 5 foot thick pad of compacted Select Structure Fill(3)).
The following LRFD soil parameters and resistance factors are provided for design of the structure:

**Foundation Soil Parameters**

|--------------|--------------------------------------------------------|--------------------------------------------------------|--------------------------------------------------|

Place the following table on each substructure plan and elevation sheet in the Contract Plans. The designer shall fill in the bearing pressures to the nearest kip per square foot.

<table>
<thead>
<tr>
<th>Substructure</th>
<th>Strength Limit State Bearing Pressure (kips/ft²)</th>
<th>Service Limit State Bearing Pressure (kips/ft²)</th>
</tr>
</thead>
</table>

(1) *Note:* This note is for situations where the designer will not be calculating the Strength Limit State bearing resistance and requires the Geotechnical Engineering Bureau to provide one. An effective footing width needs to be assumed for preliminary design and then iterated upon for the final design. The FDR should document the final design effective footing width.

(2) *Undisturbed Soil (Cut Situations):* The nominal bearing resistance should be calculated using NAVFAC DM 7.2-131 and factored to determine the Strength Limit State bearing resistance. The Service Limit State bearing resistance is determined based on settlement calculations (typical Service Limit State bearing resistance values correspond to ASD allowable bearing capacities, i.e. 5 kips/ft² on fills, 6 kips/ft² on undisturbed soil).

(3) *Compacted Select Structural Fill (Fill Situations):* The Service Limit State bearing resistance is 5 kips/ft² in 1(V) on 2(H) slopes. The bearing resistance can vary, depending on settlement conditions or footing location (i.e. a footing on level ground will have a larger bearing capacity than a footing in a slope).

(4) *Strength Limit State:* The maximum Strength Limit State bearing resistance is based on an effective footing width (footing width – 2 x eccentricity) of ___ feet and a bearing resistance factor of ___. If the effective footing width varies by more than 1 foot, contact the Foundation and Construction Unit for a reanalysis.

(5) *Strength Limit State coefficient of Friction for Sliding:* The strength limit coefficient of friction for sliding includes a resistance factor of ___.

NYSDOT Geotechnical Design Manual
(6) *Mechanically Stabilized Earth System:* The maximum Service Limit State bearing resistance for a spread footing placed on a Mechanically Stabilized Earth System (M.S.E.S.) is 4 kips/ft$^2$. This value is what was previously used for the allowable bearing capacity in ASD design (based on 2002 AASHTO, Article 7.5.4, Abutments on Mechanically Stabilized Earth Walls).

**Note 26-6$^{(1)}$:** Support the _____ on spread footings placed on a pad of compacted Select Structure Fill$^{(2)}$. Use a 2 ft. thick pad under that portion of the footing located in a cut section and a 5 ft. thick pad under that portion of the footing located in a fill section.

The following LRFD soil parameters and resistance factors are provided for design of the structure:

**Foundation Soil Parameters**

<table>
<thead>
<tr>
<th>Substructure</th>
<th>Friction Angle (degrees)</th>
<th>Unit Weight (lbs/ft$^3$)</th>
<th>Max. Service Limit State Bearing Resistance (kips/ft$^2$)</th>
<th>Nominal Coefficient of Friction for Sliding$^{(3)}$</th>
<th>Approximate Groundwater Elevation (feet)</th>
</tr>
</thead>
</table>

**Strength Limit State Resistance Factors**

<table>
<thead>
<tr>
<th>Condition</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing Resistance</td>
<td></td>
</tr>
<tr>
<td>Sliding</td>
<td></td>
</tr>
</tbody>
</table>

Place the following table on each substructure plan and elevation sheet in the Contract Plans. The designer shall fill in the bearing pressures to the nearest kip per square foot.

<table>
<thead>
<tr>
<th>Substructure</th>
<th>Strength Limit State Bearing Pressure (kips/ft$^2$)</th>
<th>Service Limit State Bearing Pressure$^{(4)}$ (kips/ft$^2$)</th>
</tr>
</thead>
</table>

$^{(1)}$ *Note:* This note is used for situations where a spread footing is to be placed partially in a cut and partially in a fill (for example, on the side of an existing embankment). The pad of select material is extended under the footing in the cut portion to provide even bearing. A number other than 2 ft. may be used, for example, if 1 ft. were used instead, the difference between the two would match the dimensions used for benching the new fill into the existing embankment (refer to NYSDOT Standard Sheet 203-2).

This note is intended for the Office of Structures or a Consultant who will calculate the Strength Limit State bearing resistance based on soil parameters provided by the Geotechnical Engineering Bureau.
(2) **Compacted Select Structural Fill:** The Service Limit State bearing resistance is 5 kips/ft² in 1(V) on 2(H) slopes. The bearing resistance can vary, depending on settlement conditions or footing location (i.e. a footing on level ground will have a larger bearing capacity than a footing in a slope).

(3) **Nominal Coefficient of Friction:** The nominal coefficient of friction for sliding is \( \tan \varphi \). The values from NAVFAC DM 7.2–p. 63 are approximately equal to the factored coefficient of friction for sliding.

(4) **Mechanically Stabilized Earth System:** The maximum Service Limit State bearing resistance for a spread footing placed on a Mechanically Stabilized Earth System (M.S.E.S.) is 4 kips/ft². This value is what was previously used for the allowable bearing capacity in ASD design (based on 2002 AASHTO, Article 7.5.4, Abutments on Mechanically Stabilized Earth Walls).

**Note 26-7**(1): Support the blank on spread footings placed on a pad of compacted Select Structure Fill(2). Use a 2 ft. thick pad under that portion of the footing located in a cut section and a 5 ft. thick pad under that portion of the footing located in a fill section.

The following LRFD soil parameters and resistance factors are provided for design of the structure:

**Foundation Soil Parameters**

|--------------|--------------------------------------------------------|--------------------------------------------------------|-------------------------------------------------|

Place the following table on each substructure plan and elevation sheet in the Contract Plans. The designer shall fill in the bearing pressures to the nearest kip per square foot.

<table>
<thead>
<tr>
<th>Substructure</th>
<th>Strength Limit State Bearing Pressure (kips/ft²)</th>
<th>Service Limit State Bearing Pressure(5) (kips/ft²)</th>
</tr>
</thead>
</table>

(1) **Note:** This note is used for situations where a spread footing is to be placed partially in a cut and partially in a fill (for example, on the side of an existing embankment). The pad of select material is extended under the footing in the cut portion to provide even bearing. A number other than 2 ft. may be used, for example, if 1 ft. were used instead, the difference...
between the two would match the dimensions used for benching the new fill into the existing embankment (refer to NYSDOT Standard Sheet 203-2).

This note is for situations where the designer will not be calculating the Strength Limit State bearing resistance and requires the Geotechnical Engineering Bureau to provide one. An effective footing width needs to be assumed for preliminary design and then iterated upon for the final design. The FDR should document the final design effective footing width.

(2) **Compacted Select Structural Fill (Fill Situations):** The Service Limit State bearing resistance is 5 kips/ft² in 1(V) on 2(H) slopes. The bearing resistance can vary, depending on settlement conditions or footing location (i.e. a footing on level ground will have a larger bearing capacity than a footing in a slope).

(3) **Strength Limit State:** The maximum Strength Limit State bearing resistance is based on an effective footing width (footing width – 2 x eccentricity) of ___ feet and a bearing resistance factor of ___. If the effective footing width varies by more than 1 foot, contact the Foundation and Construction Unit for a reanalysis.

(4) **Strength Limit State coefficient of Friction for Sliding:** The strength limit coefficient of friction for sliding includes a resistance factor of ___.

(5) **Mechanically Stabilized Earth System:** The maximum Service Limit State bearing resistance for a spread footing placed on a Mechanically Stabilized Earth System (M.S.E.S.) is 4 kips/ft². This value is what was previously used for the allowable bearing capacity in ASD design (based on 2002 AASHTO, Article 7.5.4, Abutments on Mechanically Stabilized Earth Walls).

**Note 26-8:** Place the culvert on a 1 foot thick pad of compacted (Crushed Stone, Item 623.12)/(Crushed Gravel, Item 623.11)/(Crushed Gravel, Crushed Stone, or Crushed Slag, Item 05623.15) consisting of a mixture of stone size designation ___ and stone size designation ___.

(1) **Note:** This note applies to box culverts only and precast boxes in particular. The crushed stone or gravel serves as a leveling pad and a working platform. Consult the Geotechnical Engineering Bureau for the correct item number and size designations. There is a choice of material since in some Regions it is difficult to obtain Crushed Stone, therefore Crushed Gravel is preferred. Also, Region 5 has requested the use of Item 05623.15, with stone size designations of 1 and 2.

This note does not apply to precast concrete arch units without bottom slabs which rest on cast-in-place footings (i.e. Hyspan or Conspan). For these types of culverts, use the appropriate note depending on the foundation type chosen.

**Note 26-9:** Place the culvert on a 1 foot thick pad of compacted Crushed Stone - Modified, Item 08623.12.
Note 26-10(1): Support the cast-in-place wingwalls on spread footings placed on undisturbed soil. Design the footings for a maximum allowable bearing pressure of ___ kips per square foot. Use a coefficient of sliding friction of ____.

Note 26-11(1): The Contractor is responsible for the design of the precast wingwall footings. Their design is to be based upon the allowable bearing pressure, coefficient of sliding friction and soil parameters provided in the “Notes to be Included in the Contract Plans.”

Place the precast wingwall footings and cut-off walls on a 1 foot thick leveling pad of crushed material meeting the requirements of material designation 703-0201, 703-0202, or 703-0204 with a size designation of 2. Show the pad extending 1 foot beyond the footprint of these substructures. After placing and leveling the wingwall and cut-off wall units, the areas beneath and behind the units shall be grouted with flowable low strength material meeting the requirements of Section 204. The backfill around the wingwall footings and cut-off walls is with the appropriate item from the BD sheets.

Note 26-12(1): Support the ___ on spread footings placed on competent rock. Design the footings for a maximum Service Limit State bearing pressure of ___ kips per square foot and a maximum Strength Limit State bearing pressure of ___ kips per square foot. Use a coefficient of friction for sliding of ____.

Note 26-13(1): The rock at this site is (scourable)/(layered), therefore, key the footing a minimum of ___ feet into competent rock.

Note: Keying or doweling of footings into rock is sometimes necessary for one or more of the following reasons:
1. Additional sliding resistance may be needed. The Structural Designer determines if this is the case when designing the abutments.
2. The rock may be layered. The Engineering Geologist will identify this situation.
3. The rock is scourable. The Engineering Geologist will provide the scour potential of the rock as a numerical rating of 1 to 10, with 10 being the highest scour potential. The footing should be keyed into competent rock for ratings 4 or greater as indicated below:

<table>
<thead>
<tr>
<th>Geologist Scour Rating</th>
<th>Depth of Footing Key in Note 26-16</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2, 3</td>
<td>no key required, do not use note</td>
</tr>
<tr>
<td>4, 5, 6</td>
<td>0.5 ft</td>
</tr>
<tr>
<td>7, 8, 9</td>
<td>1.0 ft</td>
</tr>
<tr>
<td>10</td>
<td>2.0 ft</td>
</tr>
</tbody>
</table>

The Engineering Geologist will determine whether keying or doweling is appropriate for situations 1 and 2. Generally, doweling into shale is not possible and keying is preferred. The Structural Designer calculates the embedment into rock when additional sliding resistance is necessary. The Engineering Geologist provides the embedment for layered rock.

**Note 26-14**: If additional sliding resistance is necessary, (key the footing deeper into competent rock.)/(the footing may be doweled into competent rock.)

\[(1)\]  Keying or doweling of footings into rock is sometimes necessary for one or more of the following reasons:

1. Additional sliding resistance may be needed. The Structural Designer determines if this is the case when designing the abutments.
2. The rock may be layered. The Engineering Geologist will identify this situation.
3. The rock is scourable. The Engineering Geologist will provide the scour potential of the rock as a numerical rating of 1 to 10, with 10 being the highest scour potential. The footing should be keyed into competent rock for ratings 4 or greater as indicated below:

<table>
<thead>
<tr>
<th>Geologist Scour Rating</th>
<th>Depth of Footing Key</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2, 3</td>
<td>no key required, do not use note</td>
</tr>
<tr>
<td>4, 5, 6</td>
<td>0.5 ft</td>
</tr>
<tr>
<td>7, 8, 9</td>
<td>1.0 ft</td>
</tr>
<tr>
<td>10</td>
<td>2.0 ft</td>
</tr>
</tbody>
</table>

The Engineering Geologist will determine whether keying or doweling is appropriate for situations 1 and 2. Generally, doweling into shale is not possible and keying is preferred. The Structural Designer calculates the embedment into rock when additional sliding resistance is necessary. The Engineering Geologist provides the embedment for layered rock.
**Note 26-15(1):** Specify the depth of key, and the reason(s) the key is needed, on the Contract Plans. Refer to the "Notes to be Included in the Contract Plans" for the appropriate note to use.

(1) Note: Include this note, and Note 26-65, if a key is necessary for scour and/or keying would be appropriate for additional sliding resistance. The Structural Designer will have to complete Note 26-65, since we will not know if additional sliding resistance is needed.

**Note 26-16:** The estimated elevation(s) of competent rock, as determined by an Engineering Geologist, (is)/(are) ___ feet. (This)/(These) elevation(s) (is)/(are) based on evaluation of rock cores from drill hole(s) ____.

**Note 26-17(1):** Place the following table on each substructure plan and elevation sheet in the Contract Plans. Show the footing's actual maximum Service Limit State and Strength Limit State bearing pressures on the Contract Plans to the nearest kip per square foot. An Engineering Geologist uses this information during construction to determine whether the rock can support the designed bearing pressures.

<table>
<thead>
<tr>
<th>Substructure</th>
<th>Strength Limit State Bearing Pressure (kips/ft²)</th>
<th>Service Limit State Bearing Pressure (kips/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(1) Note: This note is to be used for all spread foundations on rock. Note 26-66 must also be included in the F.D.R.

**Note 26-18(4):** Support the ___ on ___ inch (O.D.) Cast-In-Place Concrete Piles, Item 551.11. Design the ___ so that these piles have a maximum Strength Limit State axial load(7) of ___ kips per pile; they will be driven to achieve a nominal resistance(4) of ___ kips per pile. Show a minimum wall thickness(2) of ___ inch and ____ as a toe treatment(3).

At a Service Limit State axial load(7) of ___ kips per pile, the piles will have a maximum vertical deflection(8) of ___ inch.

Vertical pile deflections at the Service Limit State(8) axial load will be less than 1 inch.

The piles have a maximum Strength Limit State lateral resistance(9) of ___ kips per pile.

The single pile maximum Service Limit State lateral resistance is ___ kips at a deflection of ___ inch. Use Table 10.7.2.4-1 to adjust this resistance according to the actual pile layout.

The calculated Service Limit State lateral deflection(10) at the ___ is less than ___ inch.

The estimated vertical pile lengths(6) for this structure are:
**CHAPTER 26**  
**Geotechnical Reporting and Documentation**

**LOCATION** | **ESTIMATED VERTICAL PILE LENGTH (feet)***
--- | ---

*Estimated vertical pile lengths include the pile projecting ___ feet above the bottom of (footing)/(stem).

Show the actual maximum pile Service Limit State axial load on the Contract Plans to the nearest kip.

Estimate the quantity of splices\(^{(5)}\) for piles as ½ the total number of piles used in the structure.

\(^{(1)}\) *Note:* This note is for Cast-In-Place Concrete Piles driven to a rated resistance. Rated resistance is defined as driving the piles to a specified blow count per foot of pile length. Rated resistance criteria is established during construction by wave equation analysis of pile foundations (WEAP) of the specific driving system submitted by the Contractor.

\(^{(2)}\) *Minimum Wall Thickness:* Minimum wall thickness is determined by running a WEAP during the design phase. The WEAP analysis should utilize a single acting diesel which generates a driving resistance between 20 blows/foot and 120 blows/foot. A single acting diesel is recommended because it will have a light ram with a long stroke. The stroke creates a high impact velocity which generates a larger force ($V_x EA/c$). This type of hammer will generally give a worst case scenario for pile overstress.

Short piles with high end bearing can create overstress problems. When the stress wave reaches the bottom of the pile and a relatively unyielding medium, the stress wave is reflected in compression. This compression wave travels back up to the unyielding hammer and compounds the force in the pile.

For integral abutments founded on cast-in-place (CIP) concrete piles, the minimum wall thickness required by the Structures Division is 0.25 inch.

\(^{(3)}\) *Toe Treatments:* Toe Treatments

- **Flat Plates** are the usual toe treatment and are included in the specification. The thickness of the plate is ¾ inch. The diameter of the plate is usually ½ inch, but not more than e inch, larger than the diameter of the pile. An exception to this is an oversize plate used in reducing drag loads.
- **Conical Shoes** are used when driving through bouldery material or when driving into a hard bearing layer.
- **Other Toe Treatments** available are Buffalo Star Points (two plates forming an "X" on the bottom of a flat plate) and an APF rock point (see point literature for pictures and descriptions). A Buffalo Star Point and a conical shoe could be used in the same conditions.
(4) **Practical refusal:** For situations where CIP’s are driven to practical refusal (i.e. for piles in corrosive soil environments or in soft rock), this note will have to be modified. Practical refusal is defined as 20 blows for an inch. This is difficult to achieve on any material other than rock.

(5) **Pile Splices:** Include the estimate of quantity of splices (last paragraph) when estimated pile lengths are 30 feet or greater.

(6) **Pile Lengths:** Include Estimated Vertical Pile Lengths given in the FDR must include the length of pile above the bottom of footing (embedment in the footing or stem). Use the following pile embedments when computing pile lengths and enter these embedments in the appropriate note beneath the table:

- Conventional foundations supported on:
  - Cast-In-Place Concrete Piles (tapered and non-tapered): 0.5 ft.
  - Steel Bearing Piles: 1.0 ft
  - Drilled Shafts: 1.0 ft
- Integral abutments with all superstructures:
  - All pile types: 2.0 ft

(7) **Axial Pile Loads:** Axial pile loads should be rounded to the nearest kip. The nominal resistance should reflect the effects of scour, liquefaction, and downdrag.

(8) **Pile Deflections:** One of the following notes regarding vertical pile deflections will be supplied for all pile foundations:

- At a Service Limit State axial load of ___ kips per pile, the piles will have a maximum vertical deflection of ___ inch.
- Vertical pile deflections at the Service Limit State axial load will be negligible.
- As the piles will be driven to rock, vertical pile deflections at the Service Limit State axial load will be negligible.

The Service Limit State vertical deflection can be calculated using either the Florida Pier or the GROUP computer programs. The Office of Structures has specified a maximum Service Limit State vertical deflection of 1 inch for all structure types. Note that most piles driven to a bearing layer and virtually all piles driven to rock will vertically deflect less than this value.

(9) **Strength Limit State Lateral Pile Resistance:** The following note regarding Strength Limit State lateral pile resistance may be necessary for pile foundations:

The piles have a maximum Strength Limit State lateral resistance of ___ kips per pile.

The maximum Strength Limit State lateral resistance of a pile is based on the structural resistance of a pile and takes into account the effect of combined axial and bending load.

(10) **Service Limit State Lateral Pile Resistance:** One of the following notes regarding Service Limit State lateral pile resistance will be supplied for all pile foundations:
The single pile maximum Service Limit State lateral resistance is ___ kips at a deflection of ___ inch. Use AASHTO Table 10.7.2.4-1 to adjust this resistance according to the actual pile layout.

The calculated Service Limit State lateral deflection at the ___ is less than ___ inch.

The Office of Structures has specified a maximum Service Limit State lateral deflection of 0.5 inch for all structure types. The following Table can be used to provide Service Limit State lateral resistances for typically used pile types:

<table>
<thead>
<tr>
<th>Pile</th>
<th>Service Limit State Lateral Resistance @ 0.5 Inches</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Deflection (kips)</td>
</tr>
<tr>
<td></td>
<td>$\varphi = 28^\circ$ (k = 20 lb/in$^3$)</td>
</tr>
<tr>
<td></td>
<td>$\varphi = 32^\circ$ (k = 60 lb/in$^3$)</td>
</tr>
<tr>
<td></td>
<td>$\varphi = 35^\circ$ (k = 125 lb/in$^3$)</td>
</tr>
<tr>
<td>HP 10 x 42</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>21</td>
</tr>
<tr>
<td></td>
<td>24</td>
</tr>
<tr>
<td>HP 12 x 53</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>26</td>
</tr>
<tr>
<td></td>
<td>30</td>
</tr>
<tr>
<td>CIP 12.75 in. x 0.25 in.</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>24</td>
</tr>
<tr>
<td>CIP 14.00 in. x 0.25 in.</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>27</td>
</tr>
</tbody>
</table>

The above values assume a fixed pile head connection, a minimum pile length of 15 feet, fully submerged SAND ($\gamma_t = 120$ lbs/ft$^3$, $\gamma_s = 57.6$ lbs/ft$^3$), and a non-linear (cracked EI) CIP pile (0.25 inch shell thickness) analysis. H-piles were loaded in the strong direction.

If the above values are unsatisfactory, a lateral pile resistance for the specific pile type and soil conditions can be calculated using the LPILE computer program.

Table 10.7.2.4-1, which is referenced in the above note, adjusts the pile’s lateral resistance due to pile spacing/shading effects.

If the Service Limit State lateral deflections were calculated for the entire pile foundation group using either the Florida Pier or GROUP computer programs, the 2nd lateral resistance note should be used.

**Note 26-19**(1): Support the ___ on ____ inch (O.D.), tapered Cast-In-Place Concrete Piles, Item 551.11. Design the ___ so that these piles have a maximum Strength Limit State axial load(5) of ___ kips per pile; they will be driven to achieve a nominal resistance of ___ kips per pile. Show the pile as ___ gauge(2) and tapered ___ inch per foot.

At a Service Limit State axial load(5) of ___ kips per pile, the piles will have a maximum vertical deflection(6) of ___ inch.

Vertical pile deflections at the Service Limit State(6) axial load will be less than 1 inch.
The piles have a maximum Strength Limit State lateral resistance\(^7\) of ___ kips per pile.

The single pile maximum Service Limit State lateral resistance\(^8\) is ___ kips at a deflection of ___ inch. Use Table 10.7.2.4-1 to adjust this resistance according to the actual pile layout.

The calculated Service Limit State lateral deflection at the ___ is less than ___ inch.

The estimated vertical pile lengths\(^4\) for this structure are:

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>ESTIMATED LENGTH OF TAPERED SECTION (feet)</th>
<th>ESTIMATED LENGTH OF EXTENSION (feet)</th>
<th>TOTAL ESTIMATED VERTICAL LENGTH (feet)*</th>
</tr>
</thead>
</table>

*Estimated vertical pile lengths include the pile projecting ___ feet above the bottom of (footing)/(stem).

Show the actual maximum pile Service Limit State axial load on the Contract Plans to the nearest kip.

Estimate the quantity of splices\(^3\) for piles as \(\frac{1}{2}\) the total number of piles used in the structure.

\(^1\) Note: This note is for Tapered Cast-In-Place Concrete Piles driven to a rated resistance.

\(^2\) Minimum Wall Thickness: Minimum wall thickness is determined by running a WEAP during design. The WEAP analysis should utilize a single acting diesel which generates a driving resistance between 20 blows/foot and 120 blows/foot. A single acting diesel is recommended because it will have a light ram with a long stroke. The stroke creates a high impact velocity which generates a larger force \((V, EA/c)\). This type of hammer will generally give a worst case scenario for pile overstress. Usually, a seven gauge shell is specified. A lower gauge number may be used if overstress is a problem. A higher gauge number would give a smaller wall thickness than the 3/16 inch allowed in the pile specification and, therefore, should not be used.

\(^3\) Pile Splices: Include the estimate of quantity of splices (last paragraph) when estimated pile lengths are 30 feet or greater.

\(^4\) Pile Lengths: Include Estimated Vertical Pile Lengths given in the FDR must include the length of pile above the bottom of footing (embedment in the footing or stem). Use the following pile embedments when computing pile lengths and enter these embedments in the appropriate note beneath the table:

- Conventional foundations supported on:
Cast-In-Place Concrete Piles (tapered and non-tapered): 0.5 ft.
Steel Bearing Piles: 1.0 ft
Drilled Shafts: 1.0 ft

- Integral abutments with all superstructures:
  - All pile types: 2.0 ft

**Axial Pile Loads:** Axial pile loads should be rounded to the nearest kip. The nominal resistance should reflect the effects of scour, liquefaction, and downdrag.

**Pile Deflections:** One of the following notes regarding vertical pile deflections will be supplied for all pile foundations:

- **At a Service Limit State axial load of ___ kips per pile, the piles will have a maximum vertical deflection of ___ inch.**
- Vertical pile deflections at the Service Limit State axial load will be negligible.
- As the piles will be driven to rock, vertical pile deflections at the Service Limit State axial load will be negligible.

The Service Limit State vertical deflection can be calculated using either the Florida Pier or the GROUP computer programs. The Office of Structures has specified a maximum Service Limit State vertical deflection of 1 inch for all structure types. Note that most piles driven to a bearing layer and virtually all piles driven to rock will vertically deflect less than this value.

**Strength Limit State Lateral Pile Resistance:** The following note regarding Strength Limit State lateral pile resistance may be necessary for pile foundations:

The piles have a maximum Strength Limit State lateral resistance of ___ kips per pile.

The maximum Strength Limit State lateral resistance of a pile is based on the structural resistance of a pile and takes into account the effect of combined axial and bending load.

**Service Limit State Lateral Pile Resistance:** One of the following notes regarding Service Limit State lateral pile resistance will be supplied for all pile foundations:

The single pile maximum Service Limit State lateral resistance is ___ kips at a deflection of ___ inch. Use AASHTO Table 10.7.2.4-1 to adjust this resistance according to the actual pile layout.

The calculated Service Limit State lateral deflection at the ___ is less than ___ inch.

The Office of Structures has specified a maximum Service Limit State lateral deflection of 0.5 inches for all structure types. The following Table can be used to provide Service Limit State lateral resistances for typically used pile types:
The above values assume a fixed pile head connection, a minimum pile length of 15 feet, fully submerged SAND ($\gamma_t = 120$ lbs/ft$^3$, $\gamma_s = 57.6$ lbs/ft$^3$), and a non-linear (cracked EI) CIP pile (0.25 inch shell thickness) analysis. H-piles were loaded in the strong direction.

If the above values are unsatisfactory, a lateral pile resistance for the specific pile type and soil conditions can be calculated using the LPILE computer program.

Table 10.7.2.4-1, which is referenced in the above note, adjusts the pile’s lateral resistance due to pile spacing/shading effects.

If the Service Limit State lateral deflections were calculated for the entire pile foundation group using either the Florida Pier or GROUP computer programs, the 2nd lateral resistance note should be used.

**Note 26-20**

Support the ___ on ___ Steel H-Piles, Item 551.01___. Design the ___ so that these piles have a maximum Strength Limit State axial load of ___ kips per pile; they will be driven to (practical refusal (2) (20 blows per inch) and a nominal resistance) (achieve a nominal a resistance) of ___ kips per pile. Show (a reinforced shoe) (APF HP77750 or equivalent) as a toe treatment.

At a Service Limit State axial load of ___ kips per pile, the piles will have a maximum vertical deflection of ___ inch.

Vertical pile deflections at the Service Limit State axial load will be less than 1 inch.

As the piles will be driven to rock, vertical pile deflections at the Service Limit State axial load will be negligible.

The piles have a maximum Strength Limit State lateral resistance of ___ kips per pile.

The single pile maximum Service Limit State lateral resistance is ___ kips at a deflection of ___ inch. Use Table 10.7.2.4-1 to adjust this resistance according to the actual pile layout.

The calculated Service Limit State lateral deflection at the ___ is less than ___ inch.
The estimated vertical pile lengths\(^{(5)}\) for this structure are:

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>ESTIMATED VERTICAL PILE LENGTH (feet)*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Estimated vertical pile lengths include the pile projecting ___ feet above the bottom of (footing)/(stem).

Show the actual maximum pile Service Limit State axial load on the Contract Plans to the nearest kip.

Estimate the quantity of splices for piles\(^{(4)}\) as \(\frac{1}{2}\) the total number of piles used in the structure.

\(^{(1)}\) **Note:** This note is for Steel H-Piles driven to either a rated resistance or practical refusal.

\(^{(2)}\) **Practical Refusal:** Practical refusal is defined as 20 blows for an inch. This is difficult to achieve on any material other than rock.

\(^{(3)}\) **Toe Treatments:** The toe treatment is usually a reinforced shoe. The S.F.S. designer can specify "APF HP77750 or equivalent" (i.e. a hard bite point) for sloping rock situations, or when piles are significantly battered on a rock surface.

\(^{(4)}\) **Pile Splices:** Include the estimate of quantity of splices (last paragraph) when estimated pile lengths are 30 feet or greater.

\(^{(5)}\) **Pile Lengths:** Include Estimated Vertical Pile Lengths given in the FDR must include the length of pile above the bottom of footing (embedment in the footing or stem). Use the following pile embedments when computing pile lengths and enter these embedments in the appropriate note beneath the table:

- Conventional foundations supported on:
  - Cast-In-Place Concrete Piles (tapered and non-tapered): 0.5 ft.
  - Steel Bearing Piles: 1.0 ft
  - Drilled Shafts: 1.0 ft
- Integral abutments with all superstructures:
  - All pile types: 2.0 ft

\(^{(6)}\) **Axial Pile Loads:** Axial pile loads should be rounded to the nearest kip. The nominal resistance should reflect the effects of scour, liquefaction, and downdrag.

\(^{(7)}\) **Pile Deflections:** Axial One of the following notes regarding vertical pile deflections will be supplied for all pile foundations:
• At a Service Limit State axial load of ___ kips per pile, the piles will have a maximum vertical deflection of ___ inch.
• Vertical pile deflections at the Service Limit State axial load will be negligible.
• As the piles will be driven to rock, vertical pile deflections at the Service Limit State axial load will be negligible.

The Service Limit State vertical deflection can be calculated using either the Florida Pier or the GROUP computer programs. The Office of Structures has specified a maximum Service Limit State vertical deflection of 1 inch for all structure types. Note that most piles driven to a bearing layer and virtually all piles driven to rock will vertically deflect less than this value.

(8) Strength Limit State Lateral Pile Resistance: The following note regarding Strength Limit State lateral pile resistance may be necessary for pile foundations:

The piles have a maximum Strength Limit State lateral resistance of ___ kips per pile.

The maximum Strength Limit State lateral resistance of a pile is based on the structural resistance of a pile and takes into account the effect of combined axial and bending load.

(9) Service Limit State Lateral Pile Resistance: One of the following notes regarding Service Limit State lateral pile resistance will be supplied for all pile foundations:

The single pile maximum Service Limit State lateral resistance is ___ kips at a deflection of ___ inch. Use AASHTO Table 10.7.2.4-1 to adjust this resistance according to the actual pile layout.

The calculated Service Limit State lateral deflection at the ___ is less than ___ inch.

The Office of Structures has specified a maximum Service Limit State lateral deflection of 0.5 inch for all structure types. The following Table can be used to provide Service Limit State lateral resistances for typically used pile types:

<table>
<thead>
<tr>
<th>Pile</th>
<th>Service Limit State Lateral Resistance @ 0.5 Inch Deflection (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\varphi = 28^\circ$ (k = 20 lb/in$^3$)</td>
</tr>
<tr>
<td>HP 10 x 42</td>
<td>16</td>
</tr>
<tr>
<td>HP 12 x 53</td>
<td>20</td>
</tr>
<tr>
<td>CIP 12.75 in. x 0.25 in.</td>
<td>16</td>
</tr>
<tr>
<td>CIP 14.00 in. x 0.25 in.</td>
<td>18</td>
</tr>
</tbody>
</table>

The above values assume a fixed pile head connection, a minimum pile length of 15 feet, fully submerged SAND ($\gamma_l = 120$ lbs/ft$^3$, $\gamma_s = 57.6$ lbs/ft$^3$), and a non-linear (cracked EI) CIP pile (0.25 inch shell thickness) analysis. H-piles were loaded in the strong direction.
If the above values are unsatisfactory, a lateral pile resistance for the specific pile type and soil conditions can be calculated using the LPILE computer program.

Table 10.7.2.4-1, which is referenced in the above note, adjusts the pile’s lateral resistance due to pile spacing/shading effects.

If the Service Limit State lateral deflections were calculated for the entire pile foundation group using either the Florida Pier or GROUP computer programs, the 2nd lateral resistance note should be used.

**Note 26-21:** At the ____, the piles have a maximum Strength Limit State uplift resistance\(^{(1)}\) of ___ kips per pile.

\(^{(1)}\) *Strength Limit State Uplift Resistance:* Provide the Strength Limit State uplift resistance for piers supported on driven piles. Strength Limit State uplift pile resistance is the nominal calculated side friction on a pile times the appropriate resistance factor. When using Nordlund’s method, the resistance factor is 0.35. See AASHTO for the appropriate resistance factor when using other methods (i.e. Beta method, etc.).

The effects of vertical acceleration may be omitted for free standing abutments which may displace horizontally without significant restraint (e.g. superstructures supported by sliding bearings) as per AASHTO.

**Note 26-22:** ___ Dynamic Pile Test(s), Item 551.14, (has)/(have) to be performed at ___.

\(^{(1)}\) *DPLT:* For applicability of dynamic testing, refer to TRB paper entitled “Dynamic Pile Testing to Evaluate Quality and Verify Capacity of Driven Piles” by Steve Borg and Phil Walton.

**Note 26-23:** A restrike\(^{(1)}\) will be required on the(se) pile(s) approximately ___ hours after initial driving. This restrike (is)/(is not) considered a separate Dynamic Pile Test.

\(^{(1)}\) *Restricking:* If restriking occurs more than 28 hours after initial pile driving, the Contractor gets paid for a second Dynamic Pile Test.

**Note 26-24:** A detail showing the piles placed in holes\(^{(1)}\) excavated prior to driving (is)/(is not) required for this structure.

\(^{(1)}\) *Pre-excavated Holes:* Piles supporting integral abutments are placed in pre-excavated holes when the span is greater than 90 feet.

**Note 26-25\(^{(1)}\):** Support the ___ on Micropiles - ___. Item 17551.99__. Design the ___ so that the micropiles have a maximum Strength Limit State axial load of ___ kips per pile. Place the actual Service Limit State and Strength Limit State axial load (within 1 kip) and the required nominal geotechnical resistance (actual Strength Limit State axial load divided by a resistance
factor of ___), on the Contract Plans (refer to the "Notes to be Included on the Contract Plans" section of this report for the appropriate note to use).

At a Service Limit State axial load of ___ kips per pile, the micropiles will have an estimated maximum vertical deflection of ___ inch.

Estimated vertical micropile deflections(2) at the Service Limit State axial load will be (negligible)( less than 1 inch).

The micropiles have a estimated maximum Strength Limit State lateral resistance(3) of ___ kips per pile.

The estimated single micropile maximum Service Limit State lateral resistance(4) is ___ kips at a deflection of ___ inch. Use Table 10.7.2.4-1 to adjust this estimated resistance according to the actual pile layout.

The estimated Service Limit State lateral deflection(4) at the ___ is less than ___ inch.

(1) Note: This note is for micropile installations.

(2) Pile Deflections: One of the following notes regarding vertical pile deflections will be supplied for all pile foundations:
   • At a Service Limit State axial load of ___ kips per pile, the piles will have a maximum vertical deflection of ___ inch.
   • Vertical pile deflections at the Service Limit State axial load will be negligible.
   • As the piles will be driven to rock, vertical pile deflections at the Service Limit State axial load will be negligible.

   The Service Limit State vertical deflection can be calculated using either the Florida Pier or the GROUP computer programs. The Office of Structures has specified a maximum Service Limit State vertical deflection of 1 inch for all structure types. Note that most piles driven to a bearing layer and virtually all piles driven to rock will vertically deflect less than this value.

(3) Strength Limit State Lateral Pile Resistance: The following note regarding Strength Limit State lateral pile resistance may be necessary for pile foundations:

   The piles have a maximum Strength Limit State lateral resistance of ___ kips per pile.

   The maximum Strength Limit State lateral resistance of a pile is based on the structural resistance of a pile and takes into account the effect of combined axial and bending load.

(4) Service Limit State Lateral Pile Resistance: One of the following notes regarding Service Limit State lateral pile resistance will be supplied for all pile foundations:
The single pile maximum Service Limit State lateral resistance is ___ kips at a deflection of ___ inch. Use AASHTO Table 10.7.2.4-1 to adjust this resistance according to the actual pile layout.

The calculated Service Limit State lateral deflection at the ___ is less than ___ inch.

The Office of Structures has specified a maximum Service Limit State lateral deflection of 0.5 inch for all structure types. The following Table can be used to provide Service Limit State lateral resistances for typically used pile types:

<table>
<thead>
<tr>
<th>Pile</th>
<th>Service Limit State Lateral Resistance @ 0.5 Inch Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \varphi = 28^\circ ) ( (k = 20 \text{ lb/in}^3) )</td>
</tr>
<tr>
<td>HP 10 x 42</td>
<td>16</td>
</tr>
<tr>
<td>HP 12 x 53</td>
<td>20</td>
</tr>
<tr>
<td>CIP 12.75 in. x 0.25 in.</td>
<td>16</td>
</tr>
<tr>
<td>CIP 14.00 in. x 0.25 in.</td>
<td>18</td>
</tr>
</tbody>
</table>

The above values assume a fixed pile head connection, a minimum pile length of 15 feet, fully submerged SAND \( (\gamma_s = 120 \text{ lb/ft}^3, \varrho_s = 57.6 \text{ lbs/ft}^3) \), and a non-linear (cracked EI) CIP pile (0.25 inch shell thickness) analysis. H-piles were loaded in the strong direction.

If the above values are unsatisfactory, a lateral pile resistance for the specific pile type and soil conditions can be calculated using the LPILE computer program.

Table 10.7.2.4-1, which is referenced in the above note, adjusts the pile’s lateral resistance due to pile spacing/shading effects.

If the Service Limit State lateral deflections were calculated for the entire pile foundation group using either the Florida Pier or GROUP computer programs, the 2\textsuperscript{nd} lateral resistance note should be used.

**Note 26-26:** Show a ___ inch maximum diameter Permanent Casing\((\textsuperscript{1})\), Item 17551.___, for the micropiles at the ___. Show a Grade 2 permanent casing extending ___ feet below pile cut-off elevation and having a minimum wall thickness of ___ inch.

\((\textsuperscript{1})\) *Permanent Casing:* The purpose of the permanent casing is to allow the micropile to supply the same bending resistance per pile as a Cast-In-Place pile. The casing should extend at least 10 feet below the bottom-of-footing elevation. A typical wall thickness is 0.25 inch. For design purposes, when a permanent casing is used, a micropile can be assumed to behave similarly to a CIP pile. Therefore, when modeling micropiles in design, cast-in-place pile bending and lateral resistances (see commentary on notes I. B. 13-15.) can be assumed in the initial analysis. Our cast-in-place piles normally are ASTM A252 Grade 2 material with 35 ksi yield strength, and can be the starting point for the
design. When necessary, Grade 3 steel with 45 ksi yield strength may be specified.

**Note 26-27:** The mobilization of equipment is paid for under Furnishing Equipment for Installing Permanent Casing and Micropiles, Item 17551.4020.

**Note 26-28:** ___ Static Pile Load Test\(^{(1)}\), Item 17551.5022, (has)/(have) to be performed on a non-production pile installed in the vicinity of the ___. This test is to be performed prior to the installation of any production piles.

\(^{(1)}\) *SPLT:* Usually, a load test is required for situations where the micropiles are contractor designed or do not terminate in a rock socket.

**Note 26-29:** Support the ____ on Drilled Shafts, Item 17551.9949nn having an outside diameter of ___ feet and rock socket diameter\(^{(1)}\) of ___ feet. The drilled shaft design information is contained in the following table:

<table>
<thead>
<tr>
<th>Location</th>
<th>Maximum Service Limit State Axial Load (kips)</th>
<th>Maximum Strength Limit State Axial Load (kips)</th>
<th>Nominal Geotechnical Axial Resistance (kips)</th>
<th>Bottom-of Footing Elevation (feet)</th>
<th>Assumed Top-of Rock Elevation (feet)</th>
<th>Minimum Rock Socket Depth (feet)</th>
<th>Estimated ((^{(2)}) Vertical Shaft Length (feet))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\(^{(1)}\) *Rock Socket Diameter:* Usually As per AASHTO and ADSC recommendations, rock socket diameters of drilled shafts are typically 6 inches less than the overburden shaft diameters to facilitate the use of rock cutting tools inside a temporary or permanent casing. The casing is usually seated on the rock to seal out possible socket contaminates.

\(^{(2)}\) *Estimated Vertical Shaft Length:* Measured from bottom of footing (BOF) to bottom-of (shaft)/(rock socket), does not include projection into footing.

**Note 26-30:** Lateral load analyses were performed on the drilled shafts by the Geotechnical Engineering Bureau using loads provided by the (structural designer)/(design consultant). Results of these analyses were originally transmitted to your office in a memorandum dated ___.

**Note 26-31:** Equipment required to install the shafts shall be mobilized under Item 17551.60 Furnishing Equipment for Installing Drilled Shafts.

**Note 26-32:** All shafts will be tested to verify concrete integrity in accordance with Item 17551.96 - Crosshole Sonic Logging (CSL) of Drilled Shafts. Detail (_ – 2 inch I.D. steel access pipes\(^{(1)}\)) for CSL testing, attached to the (inside)/(outside) of the reinforcement cage with the bottom of the pipes located a maximum of 2 inch above the bottom of the cage. Show access
pipes at __° intervals or as close to __° as possible. Designate the pipes “A”, “B”, “_”, “_”, and “_” in the detail.

(1) **CSL Access Pipes:** CSL access pipes for rock socketed drilled shafts should be attached to the inside of the reinforcing cage to facilitate shaft construction. If the access pipes are placed outside the reinforcing cage, they tend to get displaced and/or damaged when the cage is lowered into the socket. Drilled shafts constructed solely in soil can have CSL access pipes either inside or outside the reinforcing cage at the designer’s discretion. Access pipes located outside of the cage allow for greater shaft cross-section coverage during the testing. Note that drilled shaft contractors always prefer CSL access pipes to be placed on the inside of the reinforcing cage.

**Note 26-33:** A trial shaft(1) (will)/(will not) be necessary for this project.

(1) **Trial Shaft:** On large projects or small projects with unusual or difficult subsurface conditions, a trial shaft to demonstrate the Contractor’s ability to install the shaft foundations as designed can be beneficial. Note that trial shafts are usually very expensive.

**Note 26-34:** The ___ (micropiles)/(drilled shafts) at the ___ have an estimated Strength Limit State uplift resistance(1) of ___ kips per (pile)/(shaft).

(1) **Strength Limit State Uplift Resistance:** Provide Strength Limit State uplift resistance for piers supported on micropiles or drilled shafts. Strength Limit State uplift resistance for shafts and micropiles is calculated using the appropriate factors contained in AASHTO based on the analysis method and soil condition. See the FHWA Drilled Shaft Design Manual for more information on this topic.

**Note 26-35(1):** Place the following table on each substructure's (pile)/(micropile)/(drilled shaft) layout plan sheet in the Contract Plans to document the installed lengths:

<table>
<thead>
<tr>
<th>ACTUAL PILE LENGTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>PILE NO.</td>
</tr>
<tr>
<td>---</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

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<table>
<thead>
<tr>
<th>ACTUAL (PILE)/(MICROPILE)/(DRILLED SHAFT) LENGTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>(PILE)/(SHAFT) NO.</td>
</tr>
</tbody>
</table>

\(^{(1)}\) Note: Include this note for all pile/micropile/drilled shaft supported structures. This table is meant to expand the record plans to include foundation lengths. This will allow future designers to evaluate foundation capacity. It also gives inspectors an estimation of scour protection.

**Note 26-36\(^{(1)}\):** The soil at this site cannot provide lateral resistance for the piles. Design the pile foundation so that all the lateral load is carried by the horizontal component of the actual axial load on the batter pile.

\(^{(1)}\) Note: This note should be used when one or more of the following conditions are present. Refer to NYSDOT GDM Chapter 11.

- The top 10 feet of the pile is in organic soils, soft clays, very loose silts, or loose fill.
- The approach embankments have a low factor of safety (F.S. < 1.5) because of buried soft soils.

The balance of the lateral load is then supported by batter piles. This note does not apply to integral abutments.

**Note 26-37:** The maximum unfactored lateral force\(^{(1)}\) on the integral abutment stems is ___ kips per linear foot of wall, based on an abutment height of ___ feet. This force acts ___ feet from the bottom of the stem.

\(^{(1)}\) Maximum Unfactored Lateral Load: The maximum unfactored lateral load is determined by calculating the full passive earth pressure with a triangular distribution. Unit weight and friction angles should be for select structure fill material \(\gamma = 130 \text{ lbs/ft}^3\) and \(\phi = 35^\circ\).

**Note 26-38\(^{(1)}\):** Include Item 17634.9901 - Building Condition Survey at ______.

\(^{(1)}\) Note: Include this item when pile installation or support system installation have potential to disturb nearby structures. This item can help the Department avoid paying for pre-existing structural damage. The pay unit for this item is lump sum. Also use Note 26-62 when this note is used.
26.3.2.5.1.2 Erosion Protection

Note 26-39\(^{(1)}\): Do not include a layer of bedding material beneath the Stone Filling (___), at this site.

\(^{(1)}\) *Note:* The Departmental Geotechnical Engineer should be consulted on the necessity of a bedding layer.

Note 26-40\(^{(1)}\): Include a layer of Geotextile Bedding, Item 207.20, meeting the material requirements of Material Code 737.0103 or Material Code 737.0106 and Bedding Class B, beneath the Stone Filling (___), Item 620.__, at this site.

\(^{(1)}\) *Note:* The Departmental Geotechnical Engineer should be consulted on the necessity of a bedding layer.

26.3.2.5.1.3 Dewatering

Note 26-41\(^{(1)}\): Include an appropriate Item from Section 553 for the ___ excavations.

\(^{(1)}\) *Note:* This note should be included when ordinary high water (O.H.W.) elevation is higher than the bottom-of-footing elevation. Cofferdams can be as simple as a pile of earth or an old tank car with the ends cut off to a designed sheeting cell with internal bracing. Refer to Section 4 of the N.Y.S.D.O.T. Bridge Manual for complete guidelines for cofferdams.

Note 26-42\(^{(1)}\): Due to the potential for seepage through the foundation soils, include a ___ feet thick tremie seal\(^{(2)}\) below the bottom of footing at the ___. All appropriate load and resistance factors have been applied.

\(^{(1)}\) *Note:* Although the design of cofferdams is done by the Contractor, the Department does have a responsibility to consider dewatering, as it may affect the constructability of our foundation. When the amount of dewatering is extensive (the difference between ordinary high water and bottom-of-footing elevation is great), a depth of cut-off wall analysis should be done (NAVFAC DM 7.1-268 to 270) to determine if it is possible (and economical) to drive sheeting deep enough to achieve a stable excavation bottom.

\(^{(2)}\) *Tremie Seal:* For situations where this is not practically possible, a tremie seal should be specified. The thickness of the tremie seal (X) is calculated as follows:
H = head of water, O.H.W. - B.O.F.
X = required tremie thickness
γₜ = unit weight of water = 62.4 lbs/ft³
γ_c = unit weight of unreinforced concrete = 145 lbs/ft³
γ_cs = unit weight of submerged concrete = 145 - 62.4 = 82.6 lbs/ft³

Balance column of water (Hγᵣ) against tremie seal (Xγ_cs)
Hγᵣ = Xγ_cs
X = Hγᵣ/γ_cs = H(62.4/82.6)
X = H(0.76)

When a tremie seal is needed for a pier supported on a pile foundation or spread footing on rock, the need for an additional 1.25 safety factor is unwarranted based upon the following built in factor of safety:

In the pile foundation cases, the uplift resistance of the piles are ignored creating a built in factor of safety. For the spread footings on rock case, the inability of the water to generate pressure through the rock to the tremie seal is not taken into account, creating a built in factor of safety.
26.3.2.5.1.4 Excavation and Backfill

Note 26-43\(^{(1)}\): Detail the excavation and backfill requirements of this structure in accordance with the appropriate BD sheets.

\(^{(1)}\) Note: This note applies to most FDR’s. One example of when this note does not apply is when lightweight fill is being used behind the abutments.

Note 26-44: Locate the payment lines for Select Structure Fill, Item 203.21, and Structure Excavation, Item 206.01, a distance\(^{(1)}\) from the back of the stem of one-half the height of the retaining structure.

\(^{(1)}\) Excavation and Backfill Limit: The excavation and backfill limit should be determined by the ability of the embankment material to act like an equivalent fluid with a unit weight of 30 lb/ft\(^3\) \((\gamma = 120 \text{ lbs/ft}^3 \text{ and } \phi = 37^\circ)\). This is the value that is used by the Structural Designer to design the abutments. If the embankment material cannot supply this, or the embankment material is unknown (i.e. new embankment construction), then an excavation and backfill limit of ½ the height of the retaining structure should be considered. Consult with the Regional Geotechnical Engineer about local embankment material.

The excavation and backfill limit is particularly critical in the case of in-line wingwalls, such as for some railroad structures. The least critical situation is for bridges with U-walls (walls parallel to the roadway). The case of flared wingwalls, of course, becomes more critical as the flared walls approach the in-line situation.

This note does not apply to integral abutments.

Note 26-44\(^{(1)}\): The proposed ___ consists of new fill placed on an existing embankment. Bench the existing embankment according to NYSDOT Standard Sheet 203-2 Earthwork Transition and Benching Details.

\(^{(1)}\) Note: This note will only apply to widenings or to raises in grade greater than 4 feet.

Note 26-45\(^{(1)}\): Design and detail a support system for excavations greater than 5 feet in height which cannot be laid back on a 1 vertical on 1 \(\frac{1}{2}\) horizontal slope or flatter.

For excavations greater than 20 feet in height, design and detail either a support system or a slope layback.

\(^{(1)}\) Note: Include second paragraph for projects where it is likely deep excavations will be necessary.

Some locations in the State, especially parts of Rockland and Orange Counties, have soils which allow steeper slopes (i.e. 1 vertical on 1 horizontal). Consult with the Departmental Geotechnical Engineer.

Refer to NYSDOT GDM Chapter 17 and NYSDOT Bridge Manual for complete guidelines for the use of all sheeting items.
Note 26-46\(^{(1)}\): Do not show a support system for excavations which can be laid back on a 1 vertical on 1 ½ horizontal slope or flatter.

\(^{(1)}\)Note: Some locations in the State, especially parts of Rockland and Orange Counties, have soils which allow steeper slopes (i.e. 1 vertical on 1 horizontal). Consult with the Departmental Geotechnical Engineer.

Refer to NYS DOT GDM Chapter 17 and NYS DOT Bridge Manual for complete guidelines for the use of all sheeting items.

Note 26-47\(^{(1)}\): Traffic is being maintained by an off-site detour. We anticipate the excavations for the proposed substructures can be laid back on a 1 vertical on 1 ½ horizontal slope, therefore, a support system is unnecessary.

\(^{(1)}\)Note: Some locations in the State, especially parts of Rockland and Orange Counties, have soils which allow steeper slopes (i.e. 1 vertical on 1 horizontal). Consult with the Departmental Geotechnical Engineer.

Refer to NYS DOT GDM Chapter 17 and NYS DOT Bridge Manual for complete guidelines for the use of all sheeting items.

Note 26-48\(^{(1)}\): The proximity of the on-site detour to the existing alignment may make it necessary for a support system to be installed in order to progress the excavations for the permanent structure's abutments and/or wingwalls. If a support system is necessary, the Contractor will be responsible for the design under the provisions of Temporary Structures and Approaches, Item 619.06. The Contractor will base the design on soil parameters and water elevations shown on the Contract Plans.

\(^{(1)}\)Note: Refer to NYS DOT GDM Chapter 17 and NYS DOT Bridge Manual for complete guidelines for the use of all sheeting items.

Note 26-49\(^{(1)}\): Where designed cantilevered sheeting is required at any of the proposed substructure locations, refer to Section 4 of the N.Y.S.D.O.T. Bridge Manual and BD-EE16E for appropriate sheeting information to include on the Contract Plans.

Determine the lateral forces acting on the sheeting using the soil parameters and water elevations given in the “Notes to be Included in the Contract Plans” section of this report. Notify the Foundation and Construction Unit if assistance with the design is required.

\(^{(1)}\)Note: Refer to NYS DOT GDM Chapter 17 and NYS DOT Bridge Manual for complete guidelines for the use of all sheeting items.

Use this Note when cantilevered sheeting can actually be used (if in doubt, run an analysis):

- Cobbles, boulders, obstructions, very compact material or rock, are not present to a significant depth below B.O.F. elevation.
- Exposed wall height is less than 15 feet.
Use Note 26-50 when cantilevered sheeting won't work and either braced, anchored or tied-back sheeting or a soldier pile wall may be necessary. Use Note 26-52 when sheeting cannot be used:

- Vibrations from driving sheeting will be a problem.
- Cobbles, boulders, obstructions or very compact material present throughout soil profile, particularly directly beneath the B.O.F. elevation.
- Rock at B.O.F. elevation (spread footing on rock).

In general, counting on cohesion in cohesive soils will result in a non-conservative design. Cohesion should be listed as zero and a drained friction angle used.

Omit the last sentence (“Notify the Foundation and Construction Unit...”) when the Structural Designer is a Consultant.

Note 26-50(1): Due to____________________________________________________, it will not be possible to support excavations with cantilevered sheeting. If sheeting is used, it must be braced, tied back or anchored.

Refer to Section 4 of the N.Y.S.D.O.T. Bridge Manual for the appropriate sheeting information to include on the Contract Plans. Show the following additional information for tied-back or anchored sheeting (refer to BD-EE10E and BD-EE12E):

- Sheet with steel ties to a deadman – Appropriate steel tie item, all information listed on the "Information Required on Contract Plans" attachment to the steel tie item, deadman location and details.
- Sheet with grouted anchor tiebacks - Appropriate anchor item, anchor position, anchor design load, minimum free length.

A soldier pile and lagging wall is also an option. If this type of wall is chosen, refer to the guidelines included in GDP-11 Rev. 3 Appendix D and BD-EE11E for the appropriate information to include on the Contract Plans.

Determine the lateral forces acting on the support system using the soil parameters and water elevations given in the "Notes to be Included in the Contract Plans" section of this report. Notify the Foundation and Construction Unit if assistance with the design is required.

(1) Note: Refer to NYSDOT GDM Chapter 17 and NYSDOT Bridge Manual for complete guidelines for the use of all sheeting items. Use Note 26-49 when cantilevered sheeting can actually be used (if in doubt, run an analysis):

- Cobbles, boulders, obstructions, very compact material or rock, are not present to a significant depth below B.O.F. elevation.
- Exposed wall height is less than 15 feet.

Use this Note when cantilevered sheeting won't work and either braced, anchored or tied-back sheeting or a soldier pile wall may be necessary.
Use Note 26-52 when sheeting cannot be used:
- Vibrations from driving sheeting will be a problem.
- Cobbles, boulders, obstructions or very compact material present throughout soil profile, particularly directly beneath the B.O.F. elevation.
- Rock at B.O.F. elevation (spread footing on rock).

In general, counting on cohesion in cohesive soils will result in a non-conservative design. Cohesion should be listed as zero and a drained friction angle used.

Omit the last sentence (“Notify the Foundation and Construction Unit...”) when the Structural Designer is a Consultant.

**Note 26-51**: If steel sheeting is used, indicate a minimum section modulus of ______ in³/ft and a reinforced tip to withstand the expected hard driving.

\(^{(1)}\) Note: Refer to NYSDOT GDM Chapter 17 and NYSDOT Bridge Manual for complete guidelines for the use of all sheeting items.

**Note 26-52**: Due to (vibrations, boulders and obstructions, rock at B.O.F., etc.), it will not be possible to support excavations with driven sheeting. A soldier pile and lagging wall is an option. If soldier pile and lagging wall is used, the soldier piles must be installed in pre-excavated holes and socketed into rock. Provide the following information to the Foundation and Construction Unit so that a rock socket depth can be determined: rock socket diameter and spacing and the maximum moment in the soldier pile at the top-of-rock surface. Refer to the guidelines included in GDP-11 Rev. 3 Appendix D and BD-EE11E for the appropriate information to include on the Contract Plans.

Determine the lateral forces acting on the support system using the soil parameters and water elevations given in the “Notes to be Included in the Contract Plans” section of this report. Notify the Foundation and Construction Unit if assistance with this design is required.

\(^{(1)}\) Note: Refer to NYSDOT GDM Chapter 17 and NYSDOT Bridge Manual for complete guidelines for the use of all sheeting items.

Use Note 26-49 when cantilevered sheeting can actually be used (if in doubt, run an analysis):
- Cobbles, boulders, obstructions, very compact material or rock, are not present to a significant depth below B.O.F. elevation.
- Exposed wall height is less than 15 feet.

Use Note 26-50 when cantilevered sheeting won’t work and either braced, anchored or tied-back sheeting or a soldier pile wall may be necessary.

Use this Note when sheeting cannot be used:
- Vibrations from driving sheeting will be a problem.
- Cobbles, boulders, obstructions or very compact material present throughout soil profile, particularly directly beneath the B.O.F. elevation.
- Rock at B.O.F. elevation (spread footing on rock).

In general, counting on cohesion in cohesive soils will result in a non-conservative design. Cohesion should be listed as zero and a drained friction angle used.
Omit the last sentence (“Notify the Foundation and Construction Unit...”) when the Structural Designer is a Consultant.

**Note 26-53**(1): Due to ______________, the interim steel sheeting item should be used. The cut-off elevation is given in the “Notes to be Included in the Contract Plans” section of this report.

(1) _Note:_ Refer to NYSDOT GDM Chapter 17 and NYSDOT Bridge Manual for complete guidelines for the use of all sheeting items.

This note is used when temporary sheeting must be left in place (for example, in a clay soil or a loose saturated soil where pulling sheeting could cause subsidence, or where extraction of sheeting could damage adjacent structures). The interim sheeting item was developed for these situations.

**Note 26-54**(1): Sheet ing is required to support the railroad tracks during construction of the ______. The excavation will be located within the theoretical railroad embankment, therefore, design the sheeting for earth support and a Cooper E80(2) loading.

(1) _Note:_ For excavations located near railroad tracks, refer to NYSDOT GDM Chapter 17.

(2) _Cooper E-80 Live Load:_ A Cooper E-80 live load is modeled as a strip load having a width equal to the width of a RR tie (8.5 feet) and a load of 1.882 kips/ft². The load is arrived at by dividing the axle weight of 80 kips by the axle spacing (5 feet) times the RR tie width. A copy of “Norfolk Southern Lateral Pressures from Copper E-80 Train Loads” is available through your supervisor.

**Note 26-55**(1): Sheet ing is required to support the railroad tracks during construction of the ______. The excavation will be located within the railroad live load influence line, therefore, use the interim steel sheeting item and design the sheeting for earth support and a Cooper E80(2) loading. The required sheeting cut-off elevations are given in the “Notes to be Included in the Contract Plans” section of this report.

(1) _Note:_ For excavations located near railroad tracks, refer to NYSDOT GDM Chapter 17.

(2) _Cooper E-80 Live Load:_ A Cooper E-80 live load is modeled as a strip load having a width equal to the width of a RR tie (8.5 feet) and a load of 1.882 kips/ft². The load is arrived at by dividing the axle weight of 80 kips by the axle spacing (5 feet) times the RR tie width. A copy of “Norfolk Southern Lateral Pressures from Copper E-80 Train Loads” is available through your supervisor.

**Note 26-56**(1): The excavation for the construction of the ___ is outside of the theoretical railroad embankment. Sheet ing is not required.

(1) _Note:_ For excavations located near railroad tracks, refer to NYSDOT GDM Chapter 17.
26.3.2.5.1.5 Mechanically Stabilized Earth System Notes

Note 26-57: For preliminary span length determination, locate the abutment so the minimum distance (1) between the centerline of bearings and the front of the MSE panel is 7.5 feet and the minimum distance between the footing and the back of the MSE panel is 2 feet.

(1) Minimum Distances: These distances are greater than those specified in AASHTO to allow access for inspectors.

Note 26-58: Show a precast or cast-in-place reinforced concrete coping along the top of the concrete panels that are part of the Mechanically Stabilized Earth System.

Note 26-59: The piles (1) must be installed before the construction of the Mechanically Stabilized Earth System.

(1) Pile Installation: In general, piles cannot be driven through a MSES due to interference with the reinforcement. If an abutment placed on a MSES must be pile supported, the piles must be either:

1. Driven prior to MSES construction, and the MSES is constructed around the piles, or
2. The MSES is constructed around casings, which the piles are later driven through.

If Option 1 is specified, use this Note. The portion of the piles which extends through the MSES may need to be treated with bituminous coating to prevent downdrag loads from developing.

If Option 2 is chosen, the casing requirements must be specified on the Contract Plans since there is no special specification for this situation. A detailed construction sequence may also be necessary.

When pile supported abutments on MSES are designed, the piles must carry vertical loads, only. No value for lateral resistance should be applied to the piles, since this load would be transmitted to the MSE fill. All lateral load on the abutment will be resisted by MSE reinforcement attached to the back of the abutment. Additional FDR notes are available through your supervisor for this situation, and for Option 2.

26.3.2.5.2 Notes to be Included in the Contract Documents

26.3.2.5.2.1 Monitoring

Note 26-60 (1): PERFORM (A) SURVEY(S) TO ESTABLISH (A) BASELINE PROFILE(S) OF THE RAILROAD TRACK(S) AS DESCRIBED BELOW. INCLUDE THE COST OF THIS SURVEY WORK IN THE BID PRICE FOR SURVEY AND STAKEOUT, ITEM 625.01.

A. PROVIDE TOP OF RAIL ELEVATIONS ON EACH RAIL AT 15 FOOT STATIONS FROM STATION ___ TO STATION ___. ESTABLISH HORIZONTAL CONTROL FOR THESE SAME POINTS.
B. PROVIDE TRACK MONITORING AS DIRECTED BY THE ENGINEER, IN CONSULTATION WITH THE RAILROAD'S FIELD REPRESENTATIVE, WHEN IT IS SUSPECTED THAT ___ OPERATIONS NEAR THE TRACK(S) IS CAUSING A SUBSIDENCE IN THE RAIL OR TRACK PROFILE.

C. IF DIRECTED BY THE ENGINEER, TAKE COMPARISON ELEVATIONS HOURLY DURING ___ OPERATIONS THROUGHOUT THE AFFECTED AREA.

D. IMMEDIATELY REPORT ALL DEVIATIONS IN EXCESS OF 0.25 INCH TO THE ENGINEER AND THE RAILROAD’S FIELD REPRESENTATIVE. HALT ALL ___ OPERATIONS UNTIL FURTHER DIRECTION IS GIVEN BY THE ENGINEER.

(1) Note: This note would be appropriate in a situation where there is a sheeted excavation close to railroad tracks. Pile driving can cause a loss of soil strength and sheeting deflections. This note would also be appropriate when MSES walls are constructed near the tracks, or pile driving operations are occurring near the tracks. The station limits should be 100 feet on either side of the operation.

Note 26-61(1): GOVERN METHODS OF OPERATIONS TO MINIMIZE VIBRATIONS SO THAT PEAK PARTICLE VELOCITIES MEASURED AT THE ___ RESULTING FROM THE ___ DO NOT EXCEED ___ INCHES PER SECOND. INCLUDE THE COST OF MONITORING IN ____.

(1) Note: The cost of the monitoring is usually included in the item of work that will potentially cause structural distress, such as a sheeting or a pile item. Contact the Office of Structures to confirm the appropriate item.

Note 26-62: PERFORM ITEM 17634.9901 - BUILDING CONDITION SURVEY AT______.

26.3.2.5.2.2 Spread Footings

Note 26-63(1): THE FOOTING FOR THE ___ IS DESIGNED TO EXERT A MAXIMUM BEARING PRESSURE OF ___ KIPS PER SQUARE FOOT.

(1) Note: This note is required for culverts with cast-in-place wingwalls which are not yet covered by the LRFD design code. The S.F.S. designer fills in the blank for the bearing pressure. This is the same bearing pressure given in Note I. B. 5.

Note 26-64(1): DESIGN THE PRECAST WINGWALL FOOTINGS FOR A MAXIMUM ALLOWABLE BEARING PRESSURE OF ___ KIPS PER SQUARE FOOT. USE A COEFFICIENT OF SLIDING OF ___. CALCULATE THE ACTIVE EARTH PRESSURE ON THE WINGWALLS USING A SOIL UNIT WEIGHT OF ___ POUNDS PER CUBIC FEET AND A FRICTION ANGLE OF ___ DEGREES.
(¹) Note: Use this note for culverts with precast wingwalls. The coefficient of sliding for precast units is lower than for a cast-in-place footing on the same soil. The active wedge extends behind the back of the wall a distance of about ½ the height of the wall. For a wingwall up to 7 feet in height, the select structure fill (3 feet behind the wall) contributes to the active pressure and parameters for select material can be specified (i.e., $\gamma = 130$ lbs/ft³ and $\phi = 37^\circ$).

**Note 26-65:** KEY THE _____ FOOTING ____ FEET INTO COMPETENT ROCK FOR
(SCOUR)/(SLIDING RESISTANCE)/(SCOUR AND SLIDING RESISTANCE).

**Note 26-66**(¹): AT EACH OF THE SUBSTRUCTURES SUPPORTED ON ROCK, AN
ENGINEERING GEOLOGIST FROM THE GEOTECHNICAL ENGINEERING BUREAU
WILL BE REQUIRED TO INSPECT THE ROCK TO DETERMINE IF IT IS COMPETENT TO
SUPPORT THE SERVICE LIMIT STATE BEARING PRESSURES SHOWN ON THE
CONTRACT PLANS.

(¹) Note: This note is required for all footings on rock.

**Note 26-67**(¹): ADHERE TO THE FOLLOWING PROCEDURES IF THE ROCK SURFACE
AT A SUBSTRUCTURE IS NOT FOUND AT THE ELEVATION SHOWN ON THE
CONTRACT PLANS:

A. ROCK SURFACE WITHIN 2 FEET OF THE PROPOSED BOTTOM-OF-FOOTING
   ELEVATION
   - IF THE ROCK SURFACE IS HIGHER, REMOVE THE ROCK SO THAT THE
     MINIMUM FOOTING THICKNESS CAN BE PLACED.
   - IF THE ROCK SURFACE IS LOWER, PLACE ADDITIONAL FOOTING
     CONCRETE SO THAT THE TOP-OF-FOOTING ELEVATION CAN BE
     ACHIEVED.

B. ROCK SURFACE MORE THAN 2 FEET FROM THE PROPOSED BOTTOM-OF-
   FOOTING ELEVATION
   - THE ENGINEER WILL NOTIFY THE DCES OF THIS CONDITION. THE
     DCES WILL DETERMINE IF: THE FOUNDATION FOR THE
     SUBSTRUCTURE HAS TO BE REDESIGNED, ADDITIONAL FOOTING
     CONCRETE HAS TO BE PLACED, OR ADDITIONAL ROCK HAS TO BE
     EXCAVATED.

(¹) Note: In general, with footings on rock, the top of footing elevation is set and a
minimum footing thickness of 2 feet is specified. The minimum thickness will determine
where the bottom reinforcing will be placed. For footings where rock is deeper, the depth
may be made up with concrete. However, if the depth becomes excessive, overturning
may become critical and some redesign may be necessary.
This note is usually included for footings on rock, however, it may not be appropriate for all situations. For example, setback requirements may necessitate a large rock cut for a proposed footing and the rock surface may be much higher than 2 feet above the proposed bottom-of-footing. There may also be situations where the note is appropriate, but with a number other than 2 feet.

**Note 26-68:** ADHERE TO THE FOLLOWING PROCEDURES IF THE ROCK SURFACE AT A SUBSTRUCTURE IS NOT FOUND TO BE AT THE ELEVATION AS SHOWN ON THE CONTRACT PLANS:

A. ROCK SURFACE WITHIN 2 FEET OF THE PROPOSED BOTTOM-OF-TREMIE SEAL ELEVATION
   - IF THE ROCK SURFACE IS HIGHER, RAISE THE TOP-OF-TREMIE SEAL AND FOOTING AND SHORTEN THE STEM SO THAT THE MINIMUM TREMIE SEAL AND FOOTING THICKNESS CAN BE PLACED.
   - IF THE ROCK SURFACE IS LOWER, PLACE ADDITIONAL CONCRETE FOR STRUCTURES, CLASS G (DEPOSITED UNDER WATER), ITEM 555.06, IN THE TREMIE SO THAT THE TOP-OF-TREMIE ELEVATION CAN BE ACHIEVED.

B. ROCK SURFACE MORE THAN 2 FEET FROM THE PROPOSED BOTTOM-OF-TREMIE SEAL ELEVATION
   - THE ENGINEER WILL NOTIFY THE DCES OF THIS CONDITION. THE DCES WILL DETERMINE IF: THE FOUNDATION FOR THE SUBSTRUCTURE HAS TO BE REDESIGNED, ADDITIONAL CONCRETE FOR STRUCTURES, CLASS G, HAS TO BE PLACED, OR ADDITIONAL ROCK HAS TO BE EXCAVATED.

**26.3.2.5.2.3 Piles, Micropiles, Drilled Shafts**

**Note 26-69**(1): DYNAMIC PILE TESTS WILL BE CONDUCTED BY REPRESENTATIVES OF NEW YORK STATE ON ___ PILES AT ___, OR AT OTHER LOCATIONS ORDERED BY THE ENGINEER. PERFORM THE WORK IN ACCORDANCE WITH DYNAMIC PILE TESTING, ITEM 551.14. NOTIFY THE DCETS THREE WORKING DAYS PRIOR TO DYNAMIC TESTING.

(1) *Note:* The first blank is the number of piles to be tested. The second blank is the substructure locations.

**Note 26-70**(1): THE DYNAMIC PILE TEST WILL CONSIST OF ONE TEST AT INITIAL DRIVE AND A RESTRIKE AFTER A ___-HOUR WAITING PERIOD. ADDITIONAL PILES MAY BE DRIVEN DURING THIS TIME, STARTING A MINIMUM DISTANCE OF 10 FEET FROM THE TEST PILE AND PROGRESSING AWAY FROM THE TEST PILE.

(1) *Note:* If restriking occurs more than 28 hours after initial pile driving, the Contractor
gets paid for a second Dynamic Pile Test.

**Note 26-71**

THE ___ PILES ARE DESIGNED TO SUPPORT A MAXIMUM STRENGTH LIMIT STATE AXIAL LOAD OF ___ KIPS PER PILE. DRIVE THESE PILES TO ACHIEVE A NOMINAL RESISTANCE OF ___ KIPS PER PILE.

THE MAXIMUM SERVICE LIMIT STATE AXIAL LOAD (FOR/APPLIED TO) THE PILES AT THE ___ IS ___ KIPS PER PILE.

(1) *Note:* For the service limit state note either use “FOR” and fill in the maximum service load, or use “APPLIED TO” and leave the service load blank to be filled in by the structural designer.

**Note 26-72**

THE ___ PILES ARE DESIGNED TO SUPPORT A MAXIMUM STRENGTH LIMIT STATE AXIAL LOAD OF ___ KIPS PER PILE. DRIVE THESE PILES TO PRACTICAL REFUSAL (20 BLOWS PER INCH), AND A NOMINAL RESISTANCE OF ___ KIPS PER PILE.

THE MAXIMUM SERVICE LIMIT STATE AXIAL LOAD (FOR/APPLIED TO) THE PILES AT THE ___ IS ___ KIPS PER PILE.

**Note 26-73**

DRIVE THE PILES AT THE ___ TO A MINIMUM LENGTH OF ___ FEET. THE ENGINEER WILL IMMEDIATELY CONTACT THE DCES IF THE MINIMUM LENGTH IS NOT ACHIEVED.

(1) *Note:* This note is usually used to ensure piles get their resistance below the scour elevation (when scour is excessive). Another use for this note is to prevent a pile from "hanging up" on an obstruction or bearing layer over a compressible layer.

**Note 26-74**

THE EXISTING ___ (IS)/(ARE) PILE SUPPORTED. THESE EXISTING PILES MAY INTERFERE WITH THE INSTALLATION OF THE PROPOSED PILES. IF THERE ARE PILE INTERFERENCES, THE DCES WILL DIRECT THE CONTRACTOR HOW TO PROCEED.

(1) *Note:* Relocation of the new pile is the usual procedure. Although timber piles and H-piles can easily be extracted, doing so disturbs the soil that may be providing capacity for the proposed piles (friction piles). Concrete piles (both CIP and prestressed) are almost impossible to extract because they lack the necessary tensile strength.

**Note 26-75**

DIFFICULT DRIVING OF PILES MAY BE ENCOUNTERED AND IT MAY BE NECESSARY TO USE MECHANICAL EQUIPMENT TO REMOVE VERY COMPACT MATERIAL OR BOULDERS FROM THE LOCATION OF THE PILES. WHEN REQUIRED, SPUD OR EXCAVATE HOLES PRIOR TO DRIVING IN ACCORDANCE WITH SECTION 551.
Note: This note is used when there are boulders or obstructions that will make pile driving difficult. Consideration should be given to using a predrilling item instead of this note if the situation is such that a majority of the piles will experience hard driving.

Note 26-76(1): THE USE OF MECHANICAL PILE SPLICES FOR CAST-IN-PLACE CONCRETE PILES MAY BE ALLOWED ON THIS STRUCTURE IF THE FOLLOWING REQUIREMENTS ARE MET:

A. PLACE A SEAL WELD COMPLETELY AROUND THE TOP AND BOTTOM OF THE SPLICER SLEEVE.
B. DO NOT USE A SPLICER SLEEVE WITHIN 30 FEET OF THE PILE TOE.
C. AVOID THE USE OF MECHANICAL PILE SPLICES WITHIN 6 FEET OF THE PILE CUT-OFF ELEVATION.

Note: Use this note only for conventional abutments supported by non-tapered CIP piles.

Note 26-77(1): DO NOT USE MECHANICAL PILE SPLICES ON THIS STRUCTURE.

Note: Use this note for integral abutments supported by non-tapered CIP piles or for piles subjected to tension. Tapered CIP piles (monotubes) do not use mechanical splices, so this note does not apply. Descriptions of splicing procedures for different pile types are listed below:

- Splicing Non-Tapered CIP Piles
  Cast-In-Place piles are spliced by welding or using mechanical splicers. Welding is accomplished by using a backing or (chill) ring, which is a thin metal band with spacers attached. The bottom pile section is cut square and ground smooth. The ring is placed and then the top pile section, with a 45 degree bevel, is placed on the spacers. The welder then makes the necessary passes to complete the splice.
The mechanical splicer sleeve is a collar tapered top and bottom which narrows or crimps the pile shell to fit into the splicer sleeve. The majority of non-tapered CIP pile shells are spiral welded. This type of pipe looks like a toilet paper roll and is made by wrapping a steel sheet on a mandrel, placing a rod over the joint and welding the steel sheet to each side of the rod. When this type of pile shell is mechanically spliced, the rods press in farther than the rest of the pile and creates an opening to allow water leakage. This is the reason a seal weld is required around the top and bottom of the splicer sleeve.

• Splicing Tapered CIP Piles (Monotubes)

Monotube piles are spliced by welding, only. The top pile section is crimped by burning or cutting slits into the valleys of the shell and then using a crimping tool or a sledge hammer to squeeze the edges in. This top pile section is then slid into the bottom pile section and welded all around with a fillet weld. Mechanical splicer sleeves are not used.
• Splicing H-Piles

H-piles are spliced by welding using either a backing plate or air-carbon arc gouge method. In both methods the bottom pile section is cut square and ground smooth. The top pile section is prepared by beveling at 45 degrees.

For the backing plate method, the top pile section is lowered onto the spacers, then the area is filled with weldment.
In the Air Carbon Arc method, the top pile section is placed on the bottom pile section and the area filled with weldment. Then the welder changes to a rod that looks like a copper tube. Air is blown through the tube to gouge out the back of the weld. This area is then filled with weldment to complete the weld.

Currently, our pile specification does not allow the use of mechanical splices for steel bearing piles.
Note 26-78\(^{(1)}\): PLACE THE PILES FOR THE INTEGRAL ABUTMENTS IN ___ INCH DIAMETER HOLES THAT EXTEND 8 FEET BELOW THE BOTTOM OF EACH ABUTMENT STEM. KEEP EACH OF THESE HOLES OPEN DURING THE INSTALLATION OF PILES SO THAT CUSHION SAND (MEETING THE REQUIREMENTS OF SECTION 703-06) CAN BE PLACED LOOSELY AROUND EACH PILE FOR THE FULL DEPTH OF THE HOLE. INCLUDE THE COST OF EXCAVATING THE HOLES, CASING AND CUSHION SAND IN THE UNIT PRICE BID FOR THE PILE ITEM.

\(^{(1)}\) Note: This note is used on integral abutment jobs with a span length greater than 90 feet. The diameter of the hole is usually 20 inches and should be at least 6 inches larger than the pile diameter. These holes are typically backfilled with a loose sand so as to generate a compressible zone to allow the pile to flex.

Note 26-79: PROVIDE ___-GAUGE, TAPERED CAST-IN-PLACE CONCRETE PILES.

Note 26-80: PROVIDE CAST-IN-PLACE CONCRETE PILES WITH A MINIMUM WALL THICKNESS OF ___ INCH.

Note 26-81\(^{(1)}\): EQUIP ALL CAST-IN-PLACE CONCRETE PILES WITH 60-DEGREE CONICAL SHOES. ATTACH THE SHOE TO THE PILE WITH A ___ INCH FILLET WELD, WELD ALL AROUND.

\(^{(1)}\) Note: The fillet weld size is the shell thickness with a maximum of 5/16 (0.313) inches (e.g. for a 0.25 inch shell - use a 0.25 inch weld, for a 0.5 inch shell - use a 0.313 inch weld).

Note 26-82\(^{(1)}\): EQUIP ALL CAST-IN-PLACE CONCRETE PILES WITH ___ INCH THICK FLAT CLOSURE PLATES. SUPPLY A FLAT PLATE WITH A DIAMETER THAT DOES NOT EXCEED THE PILE DIAMETER BY MORE THAN 5/8 INCH. ATTACH THE PLATE TO THE PILE WITH A ___ INCH FILLET WELD, WELD ALL AROUND.

\(^{(1)}\) Note: The fillet weld size is the shell thickness with a maximum of 5/16 (0.313) inches (e.g. for a 0.25 inch shell - use a 0.25 inch weld, for a 0.5 inch shell - use a 0.313 inch weld).

Note 26-83\(^{(1)}\): PROVIDE CAST-IN-PLACE CONCRETE PILES MEETING THE REQUIREMENTS OF ASTM A252 GRADE ___ STEEL.

\(^{(1)}\) Note: The designer should specify the steel grade of the CIP shell assumed in design (35 or 45 ksi). As per the specification, the default steel grade for CIP shells is 35 ksi, if this note is not used.

Note 26-84\(^{(1)}\): PROVIDE STEEL H-PILES MEETING THE REQUIREMENTS OF ASTM A572 GRADE 50 STEEL.
Note: Grade 50 ksi H-piles are now more common than Grade 36 ksi piles, but are not yet required by our pile specifications. If Grade 50 ksi H-piles are intended in design, this note should be used.

Note 26-85: EQUIP ALL STEEL H-PILES WITH REINFORCED SHOES.

Note 26-86: EQUIP ALL STEEL H-PILES WITH APF HP77750 OR EQUIVALENT.

Note 26-87: THE MICROPILES AT THE ___ WILL SUPPORT A MAXIMUM STRENGTH LIMIT STATE AXIAL LOAD OF ___ KIPS PER PILE. INSTALL THESE PILES TO ACHIEVE A NOMINAL RESISTANCE OF ___ KIPS PER PILE.

THE MAXIMUM SERVICE LIMIT STATE AXIAL LOAD (FOR/APPLIED TO) THE PILES AT THE ___ IS ___ KIPS PER PILE.

(1) Note: The Structural Designer fills in the actual Service and Strength Limit State load and nominal resistance.

Note 26-88: PERFORM ___ STATIC PILE LOAD TEST, ITEM 17551.5022 ON A NON-PRODUCTION PILE INSTALLED IN THE VICINITY OF ___. PROVIDE A BOND BREAKER ABOVE ELEVATION ___. PERFORM THIS TEST PRIOR TO THE INSTALLATION OF ANY PRODUCTION PILES.

(1) Note: If a micropile with extended length is specified, a bond breaker is necessary so the load test represents the capacity below the specified elevation. If an extended length is not used, delete the sentence referring to the bond breaker.

Note 26-89: INCLUDE THE COST OF THE TEST PILE(S) IN THE STATIC PILE LOAD TEST ITEM.

Note 26-90: DESIGN THE MICROPILES AT THE ___ TO DEVELOP THEIR CAPACITY BELOW ELEVATION ___.

(1) Note: Use this note when a micropile with an extended length is used (i.e., scour, liquefaction, etc.).

Note 26-91: DESIGN THE MICROPILES AT THE ___ TO RESIST A STRENGTH LIMIT STATE AXIAL LOAD OF ___ KIPS PER PILE COMBINED WITH A STRENGTH LIMIT STATE BENDING MOMENT OF ___ KIP-FEET PER PILE.

Note 26-92: ITEM 17551.9945, PERMANENT CASING FOR MICROPILES MAY BE SUBSTITUTED WITH ANOTHER PERMANENT CASING HAVING THE FOLLOWING:

\[ S \times F_Y \]
WHERE: \( S = \text{PROPOSED CASING SECTION MODULUS} \)
\( F_Y = \text{PROPOSED CASING YIELD STRENGTH} \)

ANY CASING SUBSTITUTION REQUEST WILL BE SUBJECT TO APPROVAL BY THE D.C.E.T.S.

(1) Note: This note allows the Contractor to substitute a different permanent casing section than one that is specified in the Contract Plans. Gas line pipe seconds are a common alternative for micropile permanent casing. The blank is a maximum bending moment which is calculated by multiplying the section modulus of the permanent casing specified in note I. B. 21. by the yield stress of the permanent casing. If pile deflection was a major issue in design, casing substitutions (which would change the pile’s assumed EI) should not be entertained and this note should not be used.

**Note 26-93**(1): DESIGN THE MICROPILES SO THAT THE GROSS VERTICAL TOP MOVEMENT OF THE PILE DURING LOAD TESTING DOES NOT EXCEED ___ INCH AT THE SERVICE LIMIT STATE AXIAL LOAD.

(1) Note: The micropile specification requires the gross vertical top movement of the pile at Service Limit State load be shown on the Contract Plans if the pile is load tested. Contact the Office of Structures for this number. A typical value is 1 inch.

**Note 26-94:** THE DRILLED SHAFTS AT THE ___ WILL SUPPORT A MAXIMUM STRENGTH LIMIT STATE AXIAL LOAD OF ___ KIPS PER SHAFT. INSTALL THESE SHAFTS TO ACHIEVE A NOMINAL RESISTANCE OF ___ KIPS PER SHAFT.

THE MAXIMUM SERVICE LIMIT STATE AXIAL LOAD (FOR/APPLIED TO) THE SHAFTS AT THE ___ IS ___ KIPS PER SHAFT.

**Note 26-95:** THE MINIMUM LENGTH OF ROCK SOCKETS FOR THE ____ DRILLED SHAFTS IS ___ FEET BELOW TOP-OF-SOUND ROCK. ESTIMATED TOP-OF-SOUND ROCK ELEVATION IS ___ FEET.

**Note 26-96:** CONDUCT CROSSHOLE SONIC LOGGING (CSL) OF DRILLED SHAFTS - ITEM 17551.96 ON ALL DRILLED SHAFTS. PERFORM TESTING BETWEEN ACCESS PIPE PAIRS A-B, B-C, C-D, D-A, __, __, AND __.

CONDUCT CSL TESTING WITHIN 3 TO 45 DAYS AFTER CONCRETING EACH SHAFT.

**Note 26-97**(1): AFTER COMPLETION OF THE (PILE)/(MICROPILE)/(DRILLED SHAFT) INSTALLATION, THE ENGINEER WILL COMPLETE THE “ACTUAL (PILE)/(MICROPILE)/(DRILLED SHAFT) LENGTH” TABLE FOR INCLUSION IN THE AS-BUILT PLANS.

(1) Note: Include this note for all pile/micropile/drilled shaft supported structures.
26.3.2.5.2.4 Dewatering, Cofferdams, Sheeting

Note 26-98\(^{(1)}\):  THE WATER LEVELS NOTED ON THE BORING LOGS AND ON THE SUBSURFACE PROFILE DRAWING INCLUDED IN THE CONTRACT PLANS FOR THIS STRUCTURE MAY NOT BE INDICATIVE OF ACTUAL WATER CONDITIONS AT THE TIME OF CONSTRUCTION.

\(^{(1)}\) Note: This note is used when water levels observed in borings are not indicative of the groundwater, i.e. drilling water that is one third the way down the hole, or observed water level is well below ordinary water elevation at a stream crossing.

Note 26-99\(^{(1)}\):  PLACE THE COFFERDAMS FOR THE ___ SO THAT THEY WILL NOT INTERFERE WITH THE DRIVING OF BATTER PILES.

\(^{(1)}\) Note: This note applies to most cases where a cofferdam is required at a substructure where battered piles will be driven. Consider that piles are typically battered at 6 on 1 and have 1 foot of cover to the edge of the footing. Given 3 feet for construction, this makes 4 feet from face of pile to the cofferdam sheeting or 24 feet of pile depth before striking the cofferdam sheeting.

Note 26-100\(^{(1)}\):  DESIGN THE COFFERDAM AND TREMIE SYSTEM TO AUTOMATICALLY FLOOD BY NON-MECHANICAL MEANS WHEN THE WATER ELEVATION EXCEEDS ___ FEET.

\(^{(1)}\) Note: This note is used in conjunction with Note 26-99. The water elevation entered in this note should be the ordinary high water (O.H.W.) elevation used to design the tremie seal.

Note 26-101\(^{(1)}\):  CUT OFF THE INTERIM STEEL SHEETING, ITEM 552.15, (2 FEET BELOW SUBGRADE SURFACE) / (1 FOOT BELOW FINISHED GRADE).

\(^{(1)}\) Note: Sheeting within the roadbed limits is cut off 2 feet below subgrade surface. Sheeting outside the roadbed limits is cut off 1 foot below finished grade.

Note 26-102\(^{(1)}\):  CUT OFF THE INTERIM STEEL SHEETING, ITEM 552.15, AT THE TOP OF THE RAILROAD TIE DURING CONSTRUCTION. AFTER BACKFILLING THE ___, CUT OFF THE SHEETING 1.5 FEET BELOW EXISTING GROUNDLINE.

\(^{(1)}\) Note: Use this note in conjunction with Note 26-55.

Note 26-103\(^{(1)}\):  THE FOLLOWING INFORMATION WAS USED IN THE DESIGN OF THE (STEEL SHEETING)/(TIMBER SHEETING)/(SOLDIER PILE AND LAGGING WALL)/(EARTH SUPPORT SYSTEM):
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<table>
<thead>
<tr>
<th>LOCATION</th>
<th>ELEVATION (FEET)</th>
<th>UNIT WEIGHT (LBS/FT^3)</th>
<th>FRICTION ANGLE (DEGREES)</th>
<th>COHESION (LBS/FT^2)</th>
<th>WALL FRICTION (DEGREES)</th>
</tr>
</thead>
</table>

A. **DIVIDE THE PASSIVE EARTH PRESSURE COEFFICIENT (K_p)** BY (1.25)/(1.50).
B. **GROUNDWATER IS ASSUMED AT ELEVATION ____ FEET.**
C. **A SURCHARGE LOAD OF ____ POUNDS PER SQUARE FOOT IS ASSUMED AT THE TOP OF THE WALL.**
D. **SHEETING CANNOT BE DRIVEN BELOW ELEVATION ___, DUE TO (ROCK, BOULDERS, COMPACT MATERIAL, OBSTRUCTIONS, ARTESIAN WATER PRESSURE, ETC.).**

(1) *Note:* This note is used in conjunction with Note 26-49, 50 or 52 so that the soil parameters are included in the Contract Plans.

**Note 26-104(1):** THE CONTRACTOR’S PROPOSED DETOUR SCHEME MAY REQUIRE A SUPPORT SYSTEM AT THE EXCAVATIONS FOR THE PERMANENT STRUCTURE’S ABUTMENTS AND/OR WINGWALLS. IF A SUPPORT SYSTEM IS REQUIRED, DESIGN, SUBMIT FOR APPROVAL AND INSTALL THIS SYSTEM IN ACCORDANCE WITH TEMPORARY STRUCTURES AND APPROACHES, ITEM 619.06. INCLUDE THE COST OF THIS WORK IN THE LUMP SUM BID FOR ITEM 619.06 - TEMPORARY STRUCTURES AND APPROACHES. BASE THE DESIGN UPON THE FOLLOWING:

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>ELEVATION (FEET)</th>
<th>UNIT WEIGHT (LBS/FT^3)</th>
<th>FRICTION ANGLE (DEGREES)</th>
<th>COHESION (LBS/FT^2)</th>
<th>WALL FRICTION (DEGREES)</th>
</tr>
</thead>
</table>

A. **USE A GROUNDWATER ELEVATION OF ____ FEET.**
B. **IF STEEL SHEETING IS USED, DO NOT DESIGN THE SHEETING TO REQUIRE EMBEDMENT BELOW ELEVATION ____ FEET, DUE TO ___.**

(1) *Note:* Use this note in conjunction with Note 26-48.
26.3.2.5.2.5 Excavation and Backfill Details

Note 26-105\(^{(1)}\): CONSTRUCT THE APPROACH EMBANKMENT AT THE ___ ABUTMENT TO SUBGRADE ELEVATION (INCLUDING)/(EXCLUDING) THE AREA TO BE OCCUPIED BY THE ABUTMENT. AFTER CONSTRUCTING THE APPROACH EMBANKMENTS TO THIS ELEVATION, OBSERVE A ___ WAITING PERIOD PRIOR TO (EXCAVATING FOR)/(CONSTRUCTING) THE ABUTMENT. THIS WAITING PERIOD MAY BE REDUCED BY THE DEPUTY CHIEF ENGINEER (STRUCTURES), BASED UPON THE INTERPRETATION BY THE GEOTECHNICAL ENGINEERING BUREAU OF READINGS FROM ___ INSTALLED AT THE FOLLOWING LOCATIONS:

___

INSTALL AND MONITOR THE INSTRUMENTATION IN ACCORDANCE WITH THE APPROPRIATE GEOTECHNICAL ENGINEERING BUREAU GEOTECHNICAL CONTROL PROCEDURE.

\(^{(1)}\) Note: Instrumentation is usually economical only when waiting periods are three months or more in length. Consult with the Departmental Geotechnical Engineer when waiting periods are contemplated as they have the potential to seriously impact the construction schedule.

Note 26-106\(^{(1)}\): ESTABLISH ____ REFERENCE POINTS ON THE ABUTMENTS AT LOCATIONS DESIGNATED BY THE ENGINEER. TAKE ____ READINGS ON THESE POINTS AS DIRECTED BY THE ENGINEER. THE ENGINEER WILL INTERPRET THIS INFORMATION TO DETERMINE WHETHER MOVEMENT HAS OCCURRED. NO SEPARATE PAYMENT WILL BE MADE FOR THIS WORK.

\(^{(1)}\) Note: This is used with abutments on spread footing that require a waiting period prior to setting pedestals.

Note 26-107\(^{(1)}\): THE PIER FOOTINGS AND COLUMNS MAY BE CONSTRUCTED PRIOR TO GENERAL EMBANKMENT CONSTRUCTION, BUT THE PIER COLUMNS SHOULD NOT EXTEND MORE THAN 5 FEET ABOVE THE PROPOSED END-SLOPE GRADE AT THIS STAGE. COMPLETE THE EMBANKMENTS IN THE VICINITY OF THESE STRUCTURES TO SUBGRADE ELEVATION, (INCLUDING)/(EXCLUDING) THE IMMEDIATE ABUTMENT AREA, PRIOR TO COMMENCING CONSTRUCTION OF THE REMAINDER OF THESE SUBSTRUCTURES.

\(^{(1)}\) Note: This procedure is used when a pier on spread footings is located in the toe of an embankment, where a waiting period is required. In order to eliminate the need for future excavations into the toe, the pier footing and part of the column are constructed before embankment placement.
This is a form of staged construction and the pier will be subject to settlement and possibly tilting.

**Note 26-108**(1): UNLESS OTHERWISE SHOWN ON THE CONTRACT PLANS, REMOVE EXISTING SUBSTRUCTURES AS FOLLOWS:

1. COMPLETELY REMOVE THE PORTION OF THE EXISTING SUBSTRUCTURE WITHIN A LATERAL LIMIT OF 3 FEET OF THE NEW SUBSTRUCTURE.

2. REMOVE THE PORTION OF THE EXISTING SUBSTRUCTURE THAT IS OUTSIDE OF THIS LATERAL LIMIT AS FOLLOWS:

   A. EXISTING SUBSTRUCTURE LOCATED UNDER ROADWAY - REMOVE TO 2 FEET BELOW SUBGRADE SURFACE.

   B. EXISTING SUBSTRUCTURE LOCATED UNDER APPROACH EMBANKMENT END SLOPE - REMOVE TO ELEVATION WHERE IT INTERSECTS THE BOTTOM OF THE STONE FILLING.

   C. EXISTING SUBSTRUCTURE AT ALL OTHER LOCATIONS - REMOVE TO 1 FOOT BELOW FINISHED GRADE.

(1) *Note:* The designer must carefully consider the individual project circumstances when using this note. Although it looks like a "standard" note, there are many situations where it needs revision. Below are two examples:

1. When replacing a long, multi-span structure with approach embankments and a smaller structure, if settlement of the approach embankments is an issue, the designer may wish to specify that existing piers be cut off at the original ground surface, rather than 2 feet below subgrade.

2. For a situation with an integral abutment with cantilever wingwalls, the removal limits are 1 foot below proposed bottom-of-stem elevation within a lateral limit of 3 feet of the proposed wingwalls.

**Note 26-109**(1): POUR THE PEDESTALS FOR THE ABUTMENTS _____ (WEEKS/MONTHS) AFTER THE ABUTMENTS HAVE BEEN BACKFILLED TO SUBGRADE SURFACE.

(1) *Note:* This note is used in order to get the settlement out before the pedestals are set. If the settlement will happen immediately, eliminate reference to a waiting period.

**Note 26-110**(1): ____ MAY BE ENCOUNTERED AT THE PROPOSED FOOTING ELEVATION OF THE ___. IF THIS MATERIAL IS ENCOUNTERED, REMOVE IT AND REPLACE WITH ____, ITEM ___, TO THE DEPTH AND EXTENT DIRECTED BY THE REGIONAL GEOTECHNICAL ENGINEER.
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(1) Note: This note is used when unsuitable soils are anticipated near the footing elevation. This note is also used when boulders are apt to be present at footing elevation, as it prevents uneven bearing or hard spots due to the boulders. Consult with the Departmental Geotechnical Engineer for the appropriate backfill item.

26.3.2.5.2.6 Mechanically Stabilized Earth System Notes

Note 26-111(1): INSTALL ALL PILES FOR THE ___ BEFORE CONSTRUCTING THE MECHANICALLY STABILIZED EARTH SYSTEM.

(1) Note: In general, piles cannot be driven through a MSES due to interference with the reinforcement. If an abutment placed on a MSES must be pile supported, the piles must be either:

1. Driven prior to MSES construction, and the MSES is constructed around the piles, or
2. The MSES is constructed around casings, which the piles are later driven through.

If Option 1 is specified, use Note 26-59. The portion of the piles which extends through the MSES may need to be treated with bituminous coating to prevent downdrag loads from developing.

If Option 2 is chosen, the casing requirements must be specified on the Contract Plans since there is no special specification for this situation. A detailed construction sequence may also be necessary.

When pile supported abutments on MSES are designed, the piles must carry vertical loads, only. No value for lateral resistance should be applied to the piles, since this load would be transmitted to the MSE fill. All lateral load on the abutment will be resisted by MSE reinforcement attached to the back of the abutment. Additional FDR notes are available through your supervisor for this situation, and for Option 2.

Note 26-112: PLACE THE COPING FOR THE MECHANICALLY STABILIZED EARTH SYSTEM AFTER ALL WAITING PERIODS FOR THE EMBANKMENT HAVE BEEN COMPLETED.

Note 26-113: INCLUDE THE COST FOR THE COPING IN THE BID PRICE FOR MECHANICALLY STABILIZED EARTH SYSTEM, ITEM 554.01

26.3.2.5.3 Appendix

1. Soil liquefaction due to a seismic event (is)/(is not) a design concern at this site.

2. Minimum pile lengths are specified because ___.

3. The strength limit state axial pile load is less than the nominal axial pile resistance times the resistance factor of ___ because of ___.

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4. The foundation design requirements have been discussed with ___ of Main Office Structures, Foundations and Construction Unit.

26.3.2.6 Geotechnical Summary Report

For large-scale projects involving numerous geotechnical design elements, one or more geotechnical reports may be prepared in evaluating the available data to address a broad range of design issues and communicate design recommendations for the Project Manager’s consideration. These geotechnical reports are also used to evaluate various design alternatives, assess the impact of construction on adjacent structures and facilities, focus on individual elements of the project, and discuss construction issues.

A compilation of geotechnical recommendations are made to the Project Manager in a Geotechnical Summary Report. A Geotechnical Summary Report is comprised of documents that have been previously issued and does not contain new recommendations. The summary document is provided to the Project Manager developing the design for the proposed facilities as a comprehensive geotechnical resource used to support and explain an element(s) incorporation.

While the Geotechnical Summary Report content and format will vary by project size and extent of geotechnical elements, Geotechnical Summary Reports should comprise all geotechnical reports developed such as:

- Terrain Reconnaissance Report (TR),
- Embankment Foundation Report,
- Retaining Wall Analysis Report,
- Soil Forensic Report,
- Foundation Design Report (FDR)
- Building Foundation Report
- Appendix, which provides information supporting the design recommendations including subsurface explorations, boring location plan, general subsurface profile, results of any soil testing, specifications, etc.

26.3.3 Special Reporting Requirements for LRFD Foundation and Wall Designs

The Departmental Geotechnical Engineer should provide the following information to the Structural Designer for Load and Resistance Factor Design (LRFD):

26.3.3.1 Footings

To evaluate bearing resistance, the Departmental Geotechnical Engineer provides $q_{un}$, the unfactored nominal (ultimate) bearing resistance available for the strength and extreme event limit states, and $q_{serv}$, the settlement limited nominal bearing resistance for the specified settlement (typically 1 inch) for various effective footing widths likely to be used for the service limit state, and resistance factors for each limit state. The amount of settlement on which $q_{serv}$ is based shall be stated. The calculations should assume that $q_{un}$ and $q_{serv}$ resist uniform loads applied over effective footing dimensions $B'$ and $L'$ (i.e., effective footing width and length ((B or L) - 2e) as
determined using the Meyerhof method, at least for soil). For footings on rock, the calculations should assume that $q_n$ and $q_{serv}$ resist the peak load in the footing stress distribution and that the stress distribution is triangular or trapezoidal rather than uniform. The Departmental Geotechnical Engineer also provides embedment depth requirements or footing elevations to obtain the recommended bearing resistance.

To evaluate sliding stability and eccentricity, the Departmental Geotechnical Engineer provides resistance factors for both the strength and extreme event limit states for calculating the shear and passive resistance in sliding, as well as the soil parameters $\phi$, $K_p$, $\gamma$ and depth of soil in front of footing to ignore in calculating the passive resistance, and $\phi$, $K_a$, $\gamma$, $K_{ae}$, and the earth pressure distributions to use for the strength and extreme event (seismic) limit states for calculating active force behind the footing (abutments only – see NYSDOT GDM Chapter 17 on walls).

To evaluate soil response and development of forces in foundations for the extreme event limit state, the Departmental Geotechnical Engineer provides the foundation soil/rock shear modulus values and Poissons ratio ($G$ and $\nu$). These values should typically be determined for shear strain levels of 0.02 to 0.2%, which span the strain levels for typical large magnitude earthquakes.

The Departmental Geotechnical Engineer evaluates overall stability and provides the maximum (unfactored) footing load which can be applied to the design slope and still maintain an acceptable safety factor (typically 1.5 for the strength and 1.1 for the extreme event limit states, which is the inverse of the resistance factor). A uniform bearing stress as calculated by the Meyerhof method should be assumed for this analysis. An example presentation of the LRFD footing design recommendations to be provided by the Departmental Geotechnical Engineer is as shown in Tables 26-2 and 26-3, and Figure 26-2. See NYSDOT GDM Chapter 17 for examples of the additional information submitted for abutment wall design.
### Table 26-2 Example Presentation of Soil Design Parameters for Sliding and Eccentricity Calculations

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Abutment Piers</th>
<th>Interior Piers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Unit Weight, $\gamma$ (soil above footing base level)</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Soil Friction Angle, $\varphi$ (soil above footing base level)</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Active Earth Pressure Coefficient, $K_a$</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Passive Earth Pressure Coefficient, $K_p$</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Seismic Earth Pressure Coefficient, $K_{ae}$</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Coefficient of Sliding, $\tan \delta$</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

### Table 26-3 Example Presentation of resistance Factors for Footing Design

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Resistance Factor, $\varphi$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bearing</td>
</tr>
<tr>
<td>Strength</td>
<td>X</td>
</tr>
<tr>
<td>Service</td>
<td>X</td>
</tr>
<tr>
<td>Extreme</td>
<td>X</td>
</tr>
</tbody>
</table>

### Figure 26-2 Example Presentation of Bearing Resistance Recommendations

Unfactored strength and extreme event limit states

Service limit state at ___ in. of settlement

Effective Footing Width, $B'$
26.3.3.2 Drilled Shafts

To evaluate bearing resistance, the Departmental Geotechnical Engineer provides as a function of depth and at various shaft diameters the unfactored nominal (ultimate) bearing resistance for end bearing, $R_p$, and side friction, $R_s$, used to calculate $R_n$, for strength and extreme event limit state calculations (see example figures below). For the service limit state, the unfactored bearing resistance at a specified settlement, typically 0.5 to 1.0 inch (mobilized end bearing and mobilized side friction) should be provided as a function of depth and shaft diameter. See Figure 26-3 for an example of shaft bearing resistance information that would be provided. Resistance factors for bearing resistance for all limit states will also be provided, as illustrated in Table 26-4.

If downdrag is an issue, the ultimate downdrag load, DD, as a function of shaft diameter will be provided, as well as the depth zone of the shaft that is affected by downdrag, the downdrag load factor, and the cause of the downdrag (settlement due to vertical stress increase, liquefaction, etc.). If liquefaction occurs, the lost side friction resistance, $R_{sd}$, due to downdrag will be provided (see NYSDOT GDM Chapter 11).

If scour is an issue, the magnitude and depth of the skin friction lost due to scour, $R_{scour}$, will also be provided (see NYSDOT GDM Chapter 11).

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Resistance Factor, $\phi$</th>
<th>Skin Friction</th>
<th>End Bearing</th>
<th>Uplift</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Service</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Extreme Event</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td></td>
</tr>
</tbody>
</table>

Table 26-4 Example Presentation of Resistance Factors for Shaft Design
If lateral loads imposed by special soil loading conditions such as landslide forces are present, the nominal (ultimate) lateral soil force or stress distribution, and the load factors to be applied to that force or stress, will be provided.

For evaluating uplift, the geotechnical designer provides, as a function of depth, the nominal (ultimate) uplift resistance, \( R_u \). The skin friction lost due to scour or liquefaction to be applied to the uplift resistance curves should be provided (separately, in tabular form). Resistance factors should also be provided.

The Departmental Geotechnical Engineer also provides group reduction factors for bearing resistance and uplift if necessary, as well as the associated resistance factors. The Departmental Geotechnical Engineer also provides soil/rock input data for P-Y curve generation or as input for conducting strain wedge analyses (e.g., the computer program S-Shaft) as a function of depth. Resistance factors for lateral load analysis generally do not need to be provided, as the lateral load resistance factors will typically be 1.0.

### 26.3.3.3 Piles

To evaluate pile resistance, the Departmental Geotechnical Engineer provides information regarding pile resistance using one of the following two approaches:

1. A plot of the unfactored nominal (ultimate) bearing resistance (\( R_u \)) as a function of depth for various pile types and sizes for strength and extreme event limit state calculations are provided. This design data would be used to determine the feasible ultimate pile resistance.
and the estimated depth for pile quantity determination. See Figure 26-4 for example of pile data presentation.

2. Only $R_n$ and the estimated depth at which it could be obtained are provided for one or more selected pile types and sizes.

Resistance factors for bearing resistance for all limit states will also be provided (see Table 26-5 for an example).

If downdrag is an issue, the ultimate downdrag load, $DD$, as a function of pile diameter should be provided, as well as the depth zone of the pile that is affected by downdrag, the downdrag load factor, and the cause of the downdrag (settlement due to vertical stress increase, liquefaction, etc.). If liquefaction occurs, the lost side friction resistance, $R_{sd}$, due to downdrag should be provided (see NYSDOT GDM Chapter 11).

If scour is an issue, the magnitude and depth of the skin friction lost due to scour, $R_{scour}$, should also be provided (see NYSDOT GDM Chapter 11).

If lateral loads imposed by special soil loading conditions such as landslide forces are present, the ultimate lateral soil force or stress distribution, and the load factors to be applied to that force or stress, should be provided.

For evaluating uplift, the Departmental Geotechnical Engineer should provide, as a function of depth, the nominal (unfactored) uplift resistance, $R_u$. This should be provided as a function of depth, or as a single value for a given minimum tip elevation, depending on the project needs. The skin friction lost due to scour or liquefaction to be applied to the uplift resistance curves should also be provided (separately, in tabular form). Resistance factors should also be provided for strength and extreme event limit states.

The Departmental Geotechnical Engineer should also provide group reduction factors for bearing resistance and uplift if necessary, as well as the associated resistance factors, but these will be rarely needed.

The Departmental Geotechnical Engineer should provide P-Y curve data as a function of depth. Resistance factors for lateral load analysis do not need to be provided, as the lateral load resistance factors will typically be 1.0.

Minimum tip elevations for the pile foundations should be provided as appropriate. Minimum tip elevations should be based on pile foundation settlement, and, if uplift loads are available, the depth required to provide adequate uplift resistance (see NYSDOT GDM Chapter 11). Minimum pile tip elevations provided in the geotechnical report may need to be adjusted depending on the results of the lateral load and uplift load evaluation performed by the Structural Designer. If adjustment in the minimum tip elevations is necessary, or if the pile diameter needed is different than what was assumed by the Departmental Geotechnical Engineer for pile resistance design, the Departmental Geotechnical Engineer should be informed so that pile drivability, as discussed below, can be re-evaluated.
Pile drivability should be evaluated at least conceptually for each project, and if appropriate, a wave equation analysis performed and the results of the analysis provided in terms of special requirements for hammer size and pile wall thickness, etc. The maximum driving resistance required to reach the minimum tip elevation should also be provided.

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Resistance Factor, $\phi$</th>
<th>Resistance Factor, $\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bearing Resistance</td>
<td>Uplift</td>
</tr>
<tr>
<td>Strength</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Service</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Extreme Event</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

**Table 26-5 Example Presentation of Resistance Factors for Pile Design**

**Figure 26-4 Example Presentation of Pile Bearing Resistance and Uplift**

**26.3.3.4 Retaining Walls**

To evaluate bearing resistance for footing supported gravity walls, the Departmental Geotechnical Engineer provides $q_n$, the unfactored nominal (ultimate) bearing resistance available, and $q_{serv}$, the settlement limited bearing resistance for the specified settlement for various effective footing widths (i.e., reinforcement length plus facing width for MSE walls) likely to be used, and resistance factors for each limit state. The amount of settlement on which $q_{serv}$ is based shall be stated. The calculations should assume that $q_n$ and $q_{serv}$ will resist uniform loads applied over effective footing dimension $B'$ (i.e., effective footing width ($B - 2e$)) as determined using the Meyerhof method, at least for soil. For footings on rock, the calculations should assume that $q_n$ and $q_{serv}$ will resist peak loads and that the stress distribution is triangular or trapezoidal rather
than uniform. The Departmental Geotechnical Engineer also provides wall base embedment depth requirements or footing elevations to obtain the recommended bearing resistance.

To evaluate sliding stability, bearing, and eccentricity of gravity walls, the Departmental Geotechnical Engineer provides resistance factors for both the strength and extreme event limit states for calculating the shear and passive resistance in sliding. In addition, the Departmental Geotechnical Engineer provides the soil parameters $\phi$, $K_p$, and $\gamma$, the depth of soil in front of the footing to ignore when calculating passive resistance, the soil parameters $\phi$, $K_a$, and $\gamma$ used to calculate active force behind the wall, the seismic earth pressure coefficient $K_{ae}$ (see NYSDOT GDM Chapter 17), the peak ground acceleration (PGA) used to calculate seismic earth pressures, and separate earth pressure diagrams for strength and extreme event (seismic) limit state calculations that include all applicable earth pressures, with the exception of traffic barrier impact loads (traffic barrier impact loads are developed by the Structural Designer).

The Departmental Geotechnical Engineer should also indicate in the report whether or not the wall was assumed to be free to move during seismic loading (i.e., was 0.5xPGA or 1.5xPGA used to determine $K_{ae}$). The Departmental Geotechnical Engineer should evaluate overall stability and provide the minimum footing or reinforcement length required to maintain an acceptable safety factor (typically 1.5 for the strength and 1.1 for the extreme event limit states, which is the inverse of the resistance factor, i.e., 0.65 and 0.9, respectively), if overall stability controls the wall width required. A uniform bearing stress as calculated by the Meyerhof method should be assumed for this analysis. An example presentation of the LRFD wall design recommendations to be provided by the Departmental Geotechnical Engineer is as shown in Tables 26-6 and 26-7, and Figures 26-5 and 26-6.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Unit Weight, $\gamma$ (soil above wall footing base level)</td>
<td>X</td>
</tr>
<tr>
<td>Soil Friction Angle, $\phi$ (soil above wall footing base level)</td>
<td>X</td>
</tr>
<tr>
<td>Active Earth Pressure Coefficient, $K_a$</td>
<td>X</td>
</tr>
<tr>
<td>Passive Earth Pressure Coefficient, $K_p$</td>
<td>X</td>
</tr>
<tr>
<td>Seismic Earth Pressure Coefficient, $K_{ae}$</td>
<td>X</td>
</tr>
<tr>
<td>Coefficient of Sliding, $\tan \delta$</td>
<td>X</td>
</tr>
</tbody>
</table>

**Table 26-6 Example Presentation of Soil Design Parameters for Sliding and Eccentricity Calculations for Gravity Walls**
### Table 26-7 Example Presentation of Resistance Factors for Wall Design

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Bearing</th>
<th>Shear Resistance to Sliding</th>
<th>Passive Pressure Resistance to Sliding</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Service</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Extreme Event</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

### Figure 26-5 Example Presentation of Bearing Resistance Recommendations for Gravity Walls

- Unfactored strength and extreme event limit states
- Service limit state at __ in. of settlement
- Effective Footing Width, B'

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For non-proprietary MSE walls, the spacing, strength, and length of soil reinforcement should also be provided, as well as the applicable resistance factors.

For non-gravity cantilever walls and anchored walls, ultimate bearing resistance of the soldier piles or drilled shafts as a function of depth (see NYSDOT GDM Section 26.3.3.2 Drilled Shafts, and Figure 26-3), the lateral earth pressure distribution (active and passive), the minimum embedment depth required for overall stability, and the no load zone dimensions, ultimate anchor resistance for anchored walls, and the associated resistance factors should be provided. Table 26-8 and Figure 26-7 provide an example presentation of earth pressure diagrams for nongravity cantilever and anchored walls to be provided by the Departmental Geotechnical Engineer.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Unit Weight, $\gamma$ (all applicable strata)</td>
<td>X</td>
</tr>
<tr>
<td>Soil Friction Angle, $\phi$ (all applicable strata)</td>
<td>X</td>
</tr>
<tr>
<td>Active Earth Pressure Coefficient, $K_a$</td>
<td>X</td>
</tr>
<tr>
<td>Passive Earth Pressure Coefficient, $K_p$</td>
<td>X</td>
</tr>
<tr>
<td>Seismic Earth Pressure Coefficient, $K_{ae}$</td>
<td>X</td>
</tr>
<tr>
<td>Averaged $\gamma$ used to determine $K_{ae}$</td>
<td>X</td>
</tr>
<tr>
<td>Averaged $\phi$ used to determine $K_{ae}$</td>
<td>X</td>
</tr>
</tbody>
</table>

**Table 26-8 Example Presentation of Soil Design Parameters for Design of Non-Gravity Cantilever Walls and Anchored Walls**
26.3.4 Specialized Reports for Procured Services

26.3.4.1 Building Foundation Report

The Office of General Services (OGS) Capital Planning Unit manages their Capital Construction program by providing guidance and recommendations for the expenditure of capital funding for building and infrastructure improvements. The process includes identifying short term Preventative Maintenance/Rehabilitation & Improvement (PM/R&I) and long term (major designed and bid Capital) projects. The OGS Design & Construction Group (D&C) provides a full range of high quality architectural, engineering and construction management services to state agencies. The D&C procures the services of the Geotechnical Engineering Bureau to provide foundation recommendations for OGS proposed building sites.

A Building Foundation Report is produced by the Geotechnical Engineering Bureau for OGS in an effort of continuing efficiencies in government operations by capitalizing on state resources.
By taking advantage of shared services, both agencies are successful in obtaining a well designed, economic product while maximizing the efforts of the states workforce.

The foundation design requirements outlined in a Building Foundation Report are divided into sections, including:

- **Introduction**, which provides the project description, site layout, and basis of the design recommendations.
- **Subsurface Conditions**, which describes the subsurface exploration program performed for the building foundation design, the type of soil testing performed for the project and identifies the concluding subsurface conditions including subsurface strata and groundwater elevation.
- **Site Contract Recommendations**, which provides recommendations for site development. The Site Contract Recommendations section may be divided into subsections, including:
  - **Site Grading**, which describes the cut and fill areas accentuating particular areas which are expected to cause the Contractor difficulties (e.g. compactness of material making excavations difficult or pockets of unsuitable material causing instability).
  - **Temporary Excavations**, which describes the design recommendations for any temporary slopes or shoring of excavations for the foundation excavation.
  - **Permanent Walls**, which describes the design recommendations for any permanent walls required for the site development to allow for the construction of the building.
- **Building Contract Recommendations**, which provides recommendations for the building foundation. The Building Contract Recommendations section may be divided into subsections, including:
  - **Excavations**, which describes the subsurface conditions encountered in the building excavation and provides recommendations to address the groundwater table.
  - **Footing Design**, which provides recommendations for the foundation of the proposed building.
  - **Basement Walls**, which provides recommendations for the basement wall design including recommendations addressing the wall backfill material and foundation drains.
  - **Pavement Design**, which provides recommendations for the proposed pavement section for any local roads or parking lots that are included in the contract.
  - **Seismic Design**, which provides the site soil classification as defined in the International Building Code and addresses soil liquefaction.
- **Appendix**, which provides information supporting the design recommendations, including subsurface explorations, boring location plan, general subsurface profile, results of any soil testing, specifications, etc.

### 26.3.4.2 Canal Impounding Embankment Inspection Report

The New York State Canal System is a navigable 524-mile inland waterway that spans upstate New York. The waterway connects the Hudson River with Lake Champlain, Lake Ontario,
Cayuga Lake, Seneca Lake, and Lake Erie via the Niagara River. The Canal System includes four Canals: the Erie, Champlain, Oswego and Cayuga-Seneca; canalized natural waterways; five lakes: Oneida, Onondaga, Cross, Cayuga and Seneca; short Canal sections at Ithaca and Watkins Glen; feeder reservoirs, canals and rivers not accessible by boat from the Canal; and Canal terminals on Lake Champlain.

The New York State Canal Corporation is a subsidiary of the New York State Thruway Authority, formed through State legislation transferring the Canal System from the New York State Department of Transportation to the Authority on November 5, 1992. Through the use of a Shared Services Agreement, the Thruway Authority procures the services of the Geotechnical Engineering Bureau to inspect the Canal’s embankments.

A Canal Impounding Embankment Inspection Report is produced by the Geotechnical Engineering Bureau for Thruway Authority/Canal Corporation in an effort of continuing efficiencies in government operations by capitalizing on state resources. By taking advantage of shared services, both agencies are successful in obtaining a thoroughly inspected structural resource while maximizing the efforts of the State’s workforce.

The Canal Impounding Embankment Inspection Report is divided into sections, including:

- **Summary Sheets**, which provides a synopsis of the Field Data Sheets identified in the Embankment Inspection Results section.
- **County Index Maps**, which provides location identifiers for individual segments.
- **Embankment Inspection Results**. The Embankment Inspection Results are obtained from Field Data Sheets, which are divided into particular areas for detection concentrating on specific features of the canal composition, including:
  - General Information, which identifies the location along the waterway.
  - Embankment Geometry, which identifies the dimension of the cross section of the embankment.
  - Seepage Indicators, which identify a problem and its location.
  - Outboard Slope, which identifies the vegetative cover, presence of animal burrows, drainage structures through or under the slope, retaining wall at the toe of slope or a stream at the toe of slope.
  - Inboard Slope, which identifies an indication of the stability of the slope, the presence of a retaining wall, slope protection, vegetation or animal burrows along the slope.
  - Adjacent Terrain, which identifies the type of terrain adjacent to the embankment.
  - Risk Assessment System, which utilizes the above mentioned information to obtain a Rank. The Rank is then used to Rate the slope. The rating system is based on the following:
    1. Immediate emergency, contract recommended.
    2. Begin monitoring system and schedule repairs soon.
    3. Should be improved prior to next 5 year inspection.
    4. Should be improved as part of normal scheduling.
    5. Satisfactory but shows signs of aging.
7. No deterioration.

26.3.4.3 Canal Dredge Sampling Report

As stated in the Canal Impounding Embankment Inspection Report section, the New York State Canal Corporation is a subsidiary of the New York State Thruway Authority. Through the use of a Shared Services Agreement, the Thruway Authority procures the services of the Geotechnical Engineering Bureau to sample the Canal’s dredge sites.

Each year the Canal Corporation conducts maintenance dredging in order to maintain minimum water depths for navigation. Four floating plants are located on the Canal in Waterford, Utica, Syracuse and Albion. Each floating plant has the capability to dredge by hydraulic and mechanical methods. The average yearly volume of sediment dredged is approximately 415,000 cubic yards.

Hydraulic dredging involves the suction and pumping of sediments from the water into upland sites where solids are settled out before the water is returned to the Canal. Turbidity (a measure of the “cloudiness” of water) in the water being returned to the canal is monitored to ensure the waterway is not adversely impacted by the dredging operations.

A Canal Dredge Sampling Report is produced by the Geotechnical Engineering Bureau for the Thruway Authority/Canal Corporation as part of their environmental stewardship practices to ensure protection of the natural environment. The Geotechnical Engineering Bureau samples the effluent that flows from the spoil sites back into the Canal. A field turbidity test is performed and the samples are then sent to the Materials Bureau for testing. The information outlined in a Canal Dredge Sampling Report is divided into sections, including:

- Location, which identifies the dredge site.
- Initial Reading, which provides the initial Nephelometric Turbidity Units (NTU) at the following locations:
  - 500 ft. upstream of dredge site.
  - Near dredge site.
  - 500 ft. downstream of dredge site.
- Monitoring Sample Results, which provides information on the monitoring site including:
  - Nephelometric turbidity units (NTU).
  - Suspended solids.
  - Total volatile solids.

26.3.4.4 Monitoring Well Sampling and Testing Report

Under state law, all petroleum and most hazardous material spills must be reported to DEC Hotline (1-800-457-7362), within New York State, and to 1-518-457-7362 from outside New York State. Article 12 of the Navigation Law, the legislation which applies to Oil Spill Prevention, Control, and Compensation, defines a discharge as:

“any intentional or unintentional action or omission resulting in the releasing, spilling, leaking, pumping, pouring, emitting, emptying or dumping of petroleum into the waters of the state or onto lands from which it might flow or drain into said waters, or into waters
All petroleum spills that occur within New York State must be reported to the NYS Spill Hotline within 2 hours of discovery, except spills which meet all of the following criteria:

1. The quantity is known to be less than 5 gallons; and
2. The spill is contained and under the control of the spiller; and
3. The spill has not and will not reach the State's water or any land; and
4. The spill is cleaned up within 2 hours of discovery.

The NYSDOT has 60 Transportation Maintenance Residences and numerous subresidencies in New York State. Spills and storage tank leaks at NYSDOT maintenance facilities are not uncommon (there are approximately 30 facilities currently being monitored). The Environmental Sciences Bureau is the NYSDOT’s liaison with the NYSDEC. The Environmental Sciences Bureau procures the services of the Geotechnical Engineering Bureau to install/decommission wells, perform geoprobe investigations, and complete quarterly groundwater sampling at the facilities with open spills. Samples are obtained and sent to an environmental laboratory for testing. The results of the testing, groundwater levels, site history, and analysis are assembled in a Monitoring Well Sampling and Testing Report, which is provided to the Environmental Sciences Bureau for their transactions with NYSDEC.

A Monitoring Well Sampling and Testing Report is produced by the Geotechnical Engineering Bureau for the Environmental Services Bureau in accordance with state law and as part of the Departments environmental stewardship practices to ensure protection of the natural environment. The information outlined in a Monitoring Well Sampling and Testing Report is divided into sections, including:

- **Plan**, which provides an aerial view of the subject site including:
  - Monitoring well locations.
  - Contours.
  - Subsurface flow direction.

- **Monitoring Well Worksheet**, which identifies each monitoring well formation including:
  - Inside diameter.
  - Ground elevation and elevation at top of PVC.
  - Elevation of groundwater.
  - Depth of well.
  - Calculated purge volume.
  - Volume purged and characteristics (e.g. color, sheen, odor).

- Identification of contaminants from laboratory analysis in accordance with the NYSDEC Spill Technology and Remediation Series (STARS), which may include parameters such as volatile organic compound (VOC), semi-volatile organic compound (SVOC), polynuclear aromatic hydrocarbon (PAH), polychlorinated biphenyl (PCB), Target Analyte List (TAL) metals, etc., which may be presented in various formats including:
  - Table of the identified contaminant(s), which provides the results of the laboratory testing.
26.3.4.5 Earth Dam Inspection Report

As part of the NYS Environmental Conservation Law, Part 673: Dam Safety Regulations, §673.6 Inspection, Operation and Maintenance states that the owner of a dam that:

1. is equal to or greater than 15 feet in height or has a maximum impoundment capacity equal to or greater than three million gallons, unless the dam has a height equal to or less than six feet, regardless of impoundment capacity, or an impoundment capacity less than or equal to one million gallons, regardless of height, or
2. has been assigned a Hazard Classification of Class "B" or "C" pursuant to section §673.5, or
3. impounds waters which pose, in the event of a failure of such dam, a threat of personal injury, substantial property damage or substantial natural resource damage as determined by the department,

shall develop and fully implement a written Inspection and Maintenance Plan for the dam.

As stated in the Canal Impounding Embankment Inspection Report section, the New York State Canal Corporation is a subsidiary of the New York State Thruway Authority. The Thruway Authority/Canal Corporation owns many earth impounding structures installed to create reservoirs to supply water to the canal system. As it is the responsibility of an owner of a dam(s) for maintaining and operating the dam(s) in a safe condition at all times so it does not constitute a hazard to life, health, or property, the Thruway Authority/Canal Corporation (through the use of a Shared Services Agreement) procures the services of the Geotechnical Engineering Bureau to inspect the dams.

An Earth Dam Inspection Report is produced by the Geotechnical Engineering Bureau for the Thruway Authority/Canal Corporation in accordance with state law and as part of the Departments mission to ensure transportation users have a safe, efficient, balanced and environmentally sound transportation system. The information outlined in an Earth Dam Inspection Report is divided into sections, including:

- Summary; which provides an executive summary of the findings of the investigation.
- Hazard Classification; which provides the hazard classification categories including:
Class A; dam failure will damage nothing more than isolated farm buildings, undeveloped lands or township or county roads.

Class B; dam failure can damage homes, main highways, minor railroads, or interrupt use or service of relatively important public utilities.

Class C; dam failure can cause loss of life, serious damage to homes, industrial or commercial building, important public utilities, main highways, and railroads.

- Inspection and Rating Criteria; which provides the inspection and rating categories including:
  - Dams; an earth or rock fill barrier built across a river, stream or canal to impound water. If the top of dam also serves as a roadway, the ability to function as a roadway should be considered.
    - Rate no higher than “5” for a few isolated minor locations of rutting, roadway width reductions due to erosion, or disruptive vegetative growth.
    - Rate no higher than “3” for recurring locations of moderate rutting, roadway width reductions due to erosion, or disruptive vegetative growth.
    - Rate “1” for conditions which do not allow for vehicular passage.
  - Upstream Face; the sloped surface of an earth dam that faces upstream.
    - If a rating of “4” or lower is issued, or if there is any reason to suspect possible serious deterioration, loss of function, or unstable condition of the underwater portion of the upstream face, a recommendation for further investigation (diver inspection) should be made.
  - Downstream Face; the surface of an earth dam that faces downstream.
    - Rate no higher than “5” if a few isolated locations of minor leakage or seepage exist, but cause only minor erosion.
    - Rate no higher than “3” for minor leakage or seepage over a large area or for any areas of moderate to significant leakage, and causing significant erosion.
    - Rate “1” for heavy leakage or seepage and causing severe erosion.

- Condition Rating System; which provides the condition ratings including:
  - Numerical Ratings;
    7. Excellent. Like new condition.
    6. Used to shade between a rating of “5” and “7”.
    5. Good. Exhibits minor deterioration and is functioning as originally designed.
    4. Fair. Use dot shade between a rating of “3” and “5”. Moderate to significant deterioration or deficiency but judged to be capable of performing its design function until the next expected or scheduled inspection.
    3. Poor. Serious deterioration or not functioning as originally designed. For structure, significant increase in stresses with potential for excess deformation or even failure under full design loads.
    2. Use to shade between a rating of “1” and “3”.
    1. Serious. Completely failed or in a state of failure essentially lacking in any appreciable design function; or posing a clear and present structural or safety hazard.
N. Not applicable. No such element on this structure.
X. Unknown. Element is submerged or otherwise inaccessible and cannot be assigned a numerical rating.

- Rating Multi-Element Items
  - Worst-Of Ratings; where the item to be rated covers multiple elements, rate and enter the lowest rating.
  - System Ratings; where the item to be rated is a system of interconnected components, rate the effects of damage or deteriorate on the whole system and its ability to function as designed.

- Application of Ratings; for each category of structure, there is a set of inspection guidelines. All of the various inspection guidelines are related to the Condition Report Forms for the respective structure by means of the numbering systems for the various elements.
- Temporary Supports or Repairs; temporary supports or repairs are not to be considered in assigning numerical ratings.
- Use of Diving Inspection; items normally submerged may receive a diving inspection performed at another time. Items not normally submerged should be deferred unto a time of low water.
- Summary Recommendation; provided for each group of elements for a particular structure.
- General Condition Recommendation; provided for the entire structure.
  - Work Urgency Index; prepared for each repair as appropriate according to the following:
    - 7. No repairs needed, or, no immediate need for repair.
    - 6. Work should occur in the near future – add to scheduled work.
    - 5. Place in current years work schedule or – work to occur at first reasonable opportunity.
    - 4. Priority – work to occur during current season – adjustment in priority of scheduled work plan may be necessary.
    - 3. High priority – work to occur as soon as can be scheduled.
    - 2. Highest priority – discontinue other work if required – work to be treated on an emergency basis and emergency subsidiary actions taken if necessary.
    - 1. Emergency actions required – close canal and/or drain level; close to public access.

- Appendix, which provides information supporting the reports presentation including control data/summary sheet, condition report inspection forms, “notes for next inspector” forms, inspection forms, and photographs.

26.3.4.5.1 Dam Safety Report

The NYSDEC recognizes the NYSDOT as the owner of a few small dams. The Geotechnical Engineering Bureau is working with NYSDEC to develop a Dam Safety Report for these few sites. The Dam Safety Report will be in accordance with state law and consistent with the Departments mission to ensure transportation users have a safe, efficient, balanced and
environmentally sound transportation system.

The Dam Safety Report is a work plan for the creation and implementation of a State-wide program to inventory, inspect, and monitor all dams and impoundment structures owned by the NYSDOT. The following will be incorporated into the program:

- Development of Emergency Action Plans.
  - In accordance with Emergency Action Plans for Dams, NYS Department of Environmental Conservation, NYSDEC Program Policy.
- Engineering assessment reports, including maintenance and repair recommendations where warranted by dam conditions.
  - In accordance with Guidance for Dam Engineering Assessment, NYS Department of Environmental Conservation, NYSDEC Program Policy.

### 26.3.4.6 Water Well Report

As stated in the Building Foundation Report section, the Office of General Services (OGS) Capital Planning Unit manages their Capital Construction program by providing guidance and recommendations for the expenditure of capital funding for building and infrastructure improvements. The OGS Design & Construction Group procures the services of the Geotechnical Engineering Bureau to provide water well recommendations for OGS facilities.

Water wells are a thoroughly engineered and constructed method of delivering groundwater for drinking, irrigation, and other purposes. A drilled well consists of a hole bored into the ground, with the upper part being lined with casing. The casing prevents the collapse of the borehole walls and with a drive shoe or grout seal, prevents surface or subsurface contaminants from entering the water supply. The casing also provides a housing for a pumping mechanism and for the pipe that moves water from the pump to the surface. Below the casing, the lower portion of the boreholes is the intake through which water enters the well. The intake may be an open hole in solid bedrock or it may be screened and gravel-packed, depending on the geologic conditions. Once the well is completed, it is bailed or pumped to develop the well and determine the yield. Many areas need further work after drilling to remove fine material remaining from the drilling process so that the water can more readily enter the well. Possible development methods include compressed air, bailing, jetting, surging, or pumping. The quantity of water (yield test) is usually measured during development.

A Water Well Report is produced by the Geotechnical Engineering Bureau for OGS in an effort of continuing efficiencies in government operations by capitalizing on state resources. By taking advantage of shared services, both agencies are successful in obtaining a well designed, economic product while maximizing the efforts of the states workforce.

The information contained in a Water Well Report is divided into sections, including:

- Introduction, which provides the project description, requesting agency, and site layout.
- Subsurface Conditions, which describes the subsurface conditions anticipated along with the proposed minimum yield for the well(s).
- Drilling Observations, which provides an account of the drilling operations including:
26.3.5 Specialized Reports Documenting Projects and/or Geotechnical Elements

26.3.5.1 Case History

The Geotechnical Engineering Bureau documents particular projects as a resource to future individuals investigating similar geotechnical problems. A Case History is a summary document of experience gained on a particular problem. A Case History provides enough detail for a reader to recognize the situation and comprehend the conclusions resulting from the experience.

A Case History is divided into sections, including:

- Introduction, which states the type of problem and provides the purpose for the case history.
- Description, which provides the
  - Project description
  - Location of project area (with map)
  - Physiographic province
  - Agricultural mapping unit
  - Depositional unit
- Soil Conditions, Geologic Conditions, and Foundation Conditions, which provides a description of the applicable condition above.
  - Soil profile
  - Cross section
  - Test data
  - Other information which would describe the physical conditions in which the problem area exists.
CHAPTER 26
Geotechnical Reporting and Documentation

- Design, which provides a description of the design.
  o Reference to before construction photographs
  o Typical sections
  o Drawings
- Construction, which describes the conditions encountered during construction and the problems that were encountered. Liberal use of photography should be made to illustrate problems encountered during construction.
- Evaluation, which provides an analysis and evaluation of the proposed design in light of the construction problems encountered and suggest changes in design methods, construction procedures and specifications.
- Conclusions, which provides a tabulation (for easy reference) of the salient conclusions which can be made as a result of the experiences gained on the project.
- Appendix, which provides detailed data utilized in the case history report, including:
  o Subsurface explorations
  o Boring location plan
  o Photographs of the conditions before, during and after construction
  o Test reports
  o Soil and rock profile or sections at the site
  o Contract documents
  o Typical sections, profiles of the treatment
  o Relevant specifications or special notes applicable to the job

26.3.5.2 Technical Report

The Geotechnical Engineering Bureau documents studies to aid management in evaluating potential applications of an investigational item. A Technical Report is a summary document of the available design, construction and performance data. A Technical Report provides observations, related principles, practical applications, hypotheses, conclusions/recommendations, future investigations, etc. for a reader to analyze the performance and comprehend the conclusions resulting from the study.

A Technical Report is divided into sections, including:
- Introduction, which describes the geotechnical element, material, or investigational item.
- Description, which provides background information on the use of the geotechnical element, including
  o Project description
  o Location of project area (with map)
  o Physiographic province
  o Agricultural mapping unit
  o Depositional unit
- Soil Conditions, Geologic Conditions, and Foundation Conditions, which provides a description of the applicable condition above.
  o Soil profile
  o Cross section
  o Test data
26.3.5.3 Lessons Learned

The field of geotechnical engineering is complex and ever-changing. In order to rapidly disseminate knowledge gained from geotechnical design and construction activities, as well as to help preserve institutional knowledge, the Geotechnical Engineering Bureau issues a Lessons Learned report when directed or as needed.

The purpose of a Lessons Learned report is to document:

- new technology used
- solutions to unanticipated construction problems
- unique solutions to common problems
- interesting or unusual projects

A Lessons Learned report will typically be written in lieu of a case history. It may be as short as one page memorandum or a several page report. However, it should be brief, clear, and concise. A Lessons Learned report is divided into sections, including:

- Executive Summary, which provides a synopsis of the problem, solution and the lesson learned.
- Body, which provides the sequence of events in adequate detail
  - Problem description
  - Chronology
  - Relevant causal affects
    - Soil conditions
    - Specification requirements
    - Communication channels
    - Interpretations
- Conclusion, which provides a conclusion of how the problem could have been avoided, how well the solution worked, etc.
26.4 INFORMATION TO BE PROVIDED IN THE GEOTECHNICAL DESIGN FILE

Documentation that provides details of the basis of recommendations made in the geotechnical report or memorandum is critical not only for review by senior staff, but also for addressing future questions that may come up regarding the basis of the design, to address changes that may occur after the geotechnical design is completed, to address questions regarding the design during construction to address problems or claims, and for background for developing future projects in the same location, such as bridge or fill widenings. Since the Engineer who does the original design may not necessarily be the one who deals with any of these future activities, the documentation must be clear and concise, and easy and logical to follow. Anyone who must look at the calculations and related documentation should not have to go to the original Designer to understand what was done.

The project documentation should be consistent with FHWA guidelines, as mentioned at the beginning of this chapter. Details regarding what this project documentation should contain are provided in the sections that follow.

26.4.1 Documentation for Conceptual Level Geotechnical Design

Document sources of information (including the date) used for the conceptual evaluation. Typical sources include final records, as-built bridge or other structure layouts, existing subsurface exploration logs, geologic maps, previous or current geologic reconnaissance results, etc.

If a geologic reconnaissance was or is conducted, the details of that review, including any photos taken, should be included in this documentation. For structures, provide a description of the foundation support used for existing structure, including design bearing capacity, if known, and any foundation capacity records such as pile driving logs, load test results, etc. From the final contract records, summarize any known construction problems encountered when building the existing structure. Examples include undercut depth and extent, and why it was needed, seepage observed in cuts and excavations, dewatering problems, difficult digging, including obstructions encountered during excavation, obstructions encountered during foundation installation (e.g., for piles or shafts), slope instability during construction, changed conditions or change orders involving the geotechnical features of the project, and anything else that would affect the geotechnical aspects of the project.

For any geotechnical recommendations made, summarize the logic and justification for those recommendations. If the recommendations are based on geotechnical engineering experience and judgment, describe what specific information led to the recommendation(s) made.
26.4.2 Documentation for Final Geotechnical Design

In addition to the information described in NYSDOT GDM Section 26.4.1, the following information should be documented in the project geotechnical file:

1. List or describe all given information and assumptions used, as well as the source of that information. For all calculations, an idealized design cross section that shows the design element (e.g., wall, footing, pile foundation, buttress, etc.) located in context to the existing and proposed ground lines, and the foundation soil/rock must be provided. This idealized cross-section should show the soil/rock properties used for design, the soil/rock layer descriptions and thicknesses, the water table location, the existing and proposed ground line, and any other pertinent information. An example design cross-section for a deep foundation is shown in NYSDOT GDM Appendix 26-C. For slope stability, the soil/rock properties used for the design should be shown (handwritten, if necessary) on the computer generated output cross-section.

2. Additional information and/or a narrative should also be provided which describes the basis for the design soil/rock properties used. If the properties are from laboratory tests, state where the test results, and the analysis of those test results, can be found. If using correlations to SPT or cone data, state which correlations were used and any corrections to the data made.

3. Identify what is to be determined from these calculations (i.e., what is the objective?). For example, objectives could include foundation bearing resistance, foundation or fill settlement (differential and total), time rate of settlement, the cut or fill slope required, the size of the stabilizing berm required, etc.

4. The design method(s) used must also be clearly identified for each set of calculations, including any assumptions used to simplify the calculations, if that was done, or to determine input values for variables in the design equation. Write down equation(s) used and meaning of terms used in equation(s), or reference where equation(s) used and/or meaning of terms were obtained. Attach a copy of all curves or tables used in making the calculations and their source, or appropriately reference those tables or figures. Write down or summarize all steps needed to solve the equations and to obtain the desired solution.

5. If using computer spreadsheets, provide detailed calculations for one example to demonstrate the basis of the spreadsheet and that the spreadsheet is providing accurate results. Hand calculations are not required for well proven, well documented, and stable programs such as XSTABL or the wave equation. Detailed example calculations that illustrate the basis of the spreadsheet are important for engineering review purposes and for future reference if someone needs to get into the calculations at some time in the future. A computer spreadsheet in itself is not a substitute for that information.

6. Highlight the solutions that form the basis of the engineering recommendations to be found in the project geotechnical report so that they are easy to find. Be sure to write down which locations or piers where the calculations and their results are applicable.

7. Provide a results summary, including a sketch of the final design, if appropriate.
Each set of calculations must be signed and dated, and the reviewer must also sign and date the calculations. The name of the designer and reviewer should also be printed below the signature, to clearly identify these individuals.

A copy of the appropriate portion of the FHWA checklist for geotechnical reports (i.e., appropriate to the project) should be included with the calculations and filled out as appropriate. This checklist will aid the reviewer regarding what was considered in the design and to help demonstrate consistency with the FHWA guidelines.

### 26.4.3 Geotechnical File Contents

The geotechnical project file(s) should contain the information necessary for future users of the file to understand the historical geotechnical data available, the scope of the project, the dimensions and locations of the project features understood at the time the geotechnical design was completed, the geotechnical investigation plan and the logic used to develop that plan, the relationship of that plan to what was requested by the Region, Office of Structures, or other offices, the geotechnical design conducted, what was recommended, and when and to whom it was recommended. Two types of project files should be maintained: the geotechnical design file(s), and the construction support file(s).

The geotechnical design file should contain the following information:

- Historical project geotechnical and as-built data (see NYSDOT GDM Section 26.4.1 Documentation for Conceptual Level Geotechnical Design)
- Geotechnical investigation plan development documents
- Geologic reconnaissance results
- Critical end area plots, cross-sections, structure layouts, etc. that demonstrate the scope of the project and project feature geometry as understood at the time of the final design, if such data is not contained in the geotechnical report
- Information that illustrates design constraints, such as right-of-way location, location of critical utilities, location and type of adjacent facilities that could be affected by the design, etc.
- Boring logs
- Lab transmittals
- Lab data, including rock core photos and records
- Field instrumentation measurements
- Final calculations only, unless preliminary calculations are needed to show design development
- Final wave equation runs for pile foundation constructability evaluation
- Key photos (must be identified as to the subject and locations), including CD with photo files
- Key correspondence (including e-mail) that tracks the development of the project – this does not include correspondence that is focused on coordination activities

The geotechnical construction file should contain the following information:
• Change order correspondence and calculations
• Claim correspondence and data
• Construction submittal reviews
• Photos (must be identified as to the subject and locations), including CD with photo files
• CAPWAP reports
• Final wave equation runs and pile driving criteria development
• CSL reports

26.5 CONSULTANT GEOTECHNICAL REPORTS AND DOCUMENTATION PRODUCED ON BEHALF OF NYSDOT

Geotechnical reports and documentation produced by geotechnical Consultants shall be subject to the same reporting and documentation requirements as those produced by NYSDOT staff, as described in NYSDOT GDM Sections 26.3 Geotechnical Engineering Bureau Report Content Requirements and 26.4 Information to be Provided in the Geotechnical Design File. The detailed analyses and/or calculations produced by the consultant in support of the geotechnical report development shall be provided to the State.
26.6 REFERENCES


# Appendix 26-A  PS&E Review Checklist

## A. General

<table>
<thead>
<tr>
<th>No.</th>
<th>Question</th>
<th>Yes</th>
<th>No&lt;sup&gt;(1)&lt;/sup&gt;</th>
<th>N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Has the appropriate Departmental Geotechnical Engineer reviewed the PS&amp;E to ensure that the design and construction recommendations have been incorporated as intended and that the subsurface information has been presented correctly? This is an absolute necessity.</td>
<td></td>
<td></td>
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<tr>
<td>2</td>
<td>Are the finished profile exploration logs and locations included in the plans?</td>
<td></td>
<td></td>
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<tr>
<td>3</td>
<td>Have geotechnical designs prepared by Regional Offices or Consultants been reviewed and approved by the Geotechnical Engineering Bureau?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Do the contract documents contain the Special Notes as provided in the project geotechnical report?</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>5</td>
<td>Have the following common claim pitfalls been avoided:</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>a. Has an adequate site investigation been conducted?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>b. Has the use of “subjective” subsurface terminology (such as relatively soft rock or gravel with occasional boulders) been avoided?</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>c. If alignment has been shifted, have additional subsurface explorations been conducted along new alignment?</td>
<td></td>
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<tr>
<td></td>
<td>d. Has a note been included in the contract indicating all subsurface information is available to bidders? (CONR9)</td>
<td></td>
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<tr>
<td></td>
<td>e. Do you think the wording of the geotechnical Special Notes are clear, specific, and unambiguous?</td>
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<td></td>
</tr>
</tbody>
</table>

## B. Centerline Cuts and Embankments

<table>
<thead>
<tr>
<th>No.</th>
<th>Question</th>
<th>Yes</th>
<th>No&lt;sup&gt;(1)&lt;/sup&gt;</th>
<th>N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Where excavation is required, are excavation limits and description of unsuitable organic soils shown in the contract documents?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Are plan details and Special Notes provided for special drainage details – such as lined surface ditches, drainage blanket under sidehill fill, interceptor trench drains, etc.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Are Special Notes included for fill materials requiring special treatment, such as non-durable shales, lightweight fill, etc.?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Are Special Notes provided for any special rock slope excavation and stabilization measure called for in the contract documents, such as controlled blasting, wire mesh slope protection, rock bolts, shotcrete, etc.?</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
C. Embankments over Soft Ground

<table>
<thead>
<tr>
<th>No.</th>
<th>Question</th>
<th>Yes</th>
<th>No(1)</th>
<th>N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Where excavation is required, are excavation limits and description of unsuitable soils clearly shown in the contract documents?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Where settlement waiting period will be required, has estimated settlement time been stated in the Special Notes to allow bidders to fairly bid the project?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>If instrumentation will be used to control the rate of fill placement, do Special Notes clearly spell out how this will be done and how the readings will be used to control the Contractors operations?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Do Special Notes clearly state that any instrumentation damaged by Contractor personnel will be repaired at the Contractors expense?</td>
<td></td>
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</tr>
</tbody>
</table>

D. Landslide Corrections

<table>
<thead>
<tr>
<th>No.</th>
<th>Question</th>
<th>Yes</th>
<th>No(1)</th>
<th>N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Are plan details and Special Notes provided for special drainage details, such as lined surface ditches, drainage blankets, horizontal drains, etc.?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Where excavation is to be made into the toe of an active slide (such as for buttress or shear key construction) – and stage construction is required, do the Special Notes clearly spell out the stage construction sequence to be followed?</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>3</td>
<td>Where a toe buttress is to be constructed, do the Special Notes clearly state gradation and compaction requirements for the buttress materials?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>If the geotechnical report recommended that slide repair work not be allowed during the wet time of the year, is the proposed construction schedule in accord with the recommendation?</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
E. Retaining Walls

<table>
<thead>
<tr>
<th>No.</th>
<th>Question</th>
<th>Yes</th>
<th>No(^{(1)})</th>
<th>N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Are select materials specified for wall backfill with gradation and compaction requirements covered in the specifications?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Are limits of required select backfill zones clearly detailed in the contract documents?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Are excavation requirements specified, i.e. safe slopes for excavations, need for sheeting?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Where alternate wall types will be allowed, are fully detailed plans included for all alternates?</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>5</td>
<td>Is the right-of-way limit shown in the contract documents and mentioned in the specification where tiebacks are to be installed?</td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

F. Spread Footings

<table>
<thead>
<tr>
<th>No.</th>
<th>Question</th>
<th>Yes</th>
<th>No(^{(1)})</th>
<th>N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Where spread footings are to be placed in natural soil, is the specific bearing strata in which the footing is to be founded clearly described (i.e. place on Br. Sandy GRAVEL deposit, etc.)?</td>
<td>Yes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Where spread footings are to be placed in the bridge end fill, are gradation and compaction requirements – for the select fill and backfill drainage material – covered in the Special Notes, standard specifications, or standard structure sheets?</td>
<td>No(^{(1)})</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
G. Pile Foundations

<table>
<thead>
<tr>
<th>No.</th>
<th>Question</th>
<th>Yes</th>
<th>No(1)</th>
<th>N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Do plan details adequately cover pile splices, tip reinforcement, driving shoes, etc.?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Where friction piles are to be driven in silty or clayey soils – significant set-up or soil freeze affecting long-term capacity may occur – do specifications require re-tapping the piles after 24 or 48 hour waiting period when required bearing is not obtained at estimated length at end of initial driving?</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>3</td>
<td>Where friction piles are to be load tested, has a reaction load for four times design load been specified to allow load testing the pile to plunging failure so that the ultimate soil capacity can be determined?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Where end bearing steel piles are to be load tested, has load test been designed to determine if higher than 9 ksi allowable steel stress can be used (e.g. 12-15 ksi)?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Where cofferdam construction will be required, have soil gradation results been included in the contract documents or been made available to bidders to assist them in determining dewatering procedures?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>If a wave equation analysis will be used to approve the Contractors pile driving hammer – has a minimum hammer energy or estimated soil resistance (tons) to be overcome to drive the pile to the estimated length – been given in the Special Notes?</td>
<td></td>
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</tr>
</tbody>
</table>

H. Drilled Shaft Foundations

<table>
<thead>
<tr>
<th>No.</th>
<th>Question</th>
<th>Yes</th>
<th>No(1)</th>
<th>N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Where drilled shafts are to be placed in soil, is the specific bearing strata in which the drilled shaft is to be found clearly described (i.e., placed on Br. Sandy GRAVEL deposit, etc.)?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Where end bearing drilled shafts are to be founded on rock, has the rock elevation at the shaft pier locations been determined from borings at the pier location?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Where drilled shafts are to be socketed some depth into rock – have rock cores been extracted at depths to 10 ft. below proposed socket at location within 10 ft. of the shaft?</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## I. Material Sites

<table>
<thead>
<tr>
<th>No.</th>
<th>Question</th>
<th>Yes</th>
<th>No&lt;sup&gt;(1)&lt;/sup&gt;</th>
<th>N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Is a material site sketch included in the contract documents?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Has the material site investigation established a proven quantity of material sufficient to satisfy the project estimated quantity needs?</td>
<td></td>
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</tr>
<tr>
<td>3</td>
<td>Where specification material cannot be obtained directly from the natural deposit, do the Special Notes clearly spell out that processing will be required?</td>
<td></td>
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</tr>
<tr>
<td>4</td>
<td>Are Contractor special permit requirements covered in the Special Notes?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Are pit reclamation requirements clearly spelled out in the contract documents?</td>
<td></td>
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</tr>
</tbody>
</table>

<sup>(1)</sup> A response other than “Yes” or “N/A” for any of these checklist questions is cause to contact the appropriate Departmental Geotechnical Engineer for a clarification and/or to discuss the project.
Appendix 26-B  Special Notes

Contractors have complained that customary specification requirements are sometimes changed by Special Notes and, during the rush of preparing bids, they might overlook the note. Such oversight causes overbidding when they miss an economical change and underbidding when they miss a change that increases their cost. Moreover, the indiscriminate use of Special Notes can lead to a hard-to-follow set of contract documents and confusion when conflicts between plan, proposal and specifications are created.

The policy regarding the Designers use of Special Notes in the contract documents is as follows:

1. The Designer should make every effort not to use notes to modify customary and, therefore, expected specification provisions. Modifications to specifications should be done by a special specification.
2. The Designer should not include notes that simply restate parts of the General Provisions (Section 100) or other already stated specification provisions. This redundancy is time consuming at best and can lead to confusion and misunderstanding if the provision or specification is incorrectly paraphrased or if a typographical error occurs.
3. Notes should be used to convey information to the bidder regarding the conditions of the contract under which the work generally is to be performed, such as any time or site restrictions which may exist or any special permit requirements.
4. Notes may be used by the Designer when the specification itself alerts the bidder to look for a note and is specific as to where to look for it.

A. Dynamic Pile Load Testing of Existing Piles

Past experience has shown that the following notes should be included in the contract documents on projects where a dynamic pile load test is required on an existing CIP pile. These guidelines are necessary because past tests have resulted in incomplete test data due to:

- insufficient energy transferred to the piles because of an inefficient pile-hammer connection, and
- a question regarding the mode of transfer of the hammer impact to the concrete or steel of the pile.

To avoid these problems, the following outline should be placed in the Foundation Design Report for inclusion into the contract documents:

**Special Note for the Dynamic Load Test of Existing CIP Piles**

In addition to normal dynamic load test procedures, the following steps for dynamically testing existing CIP piles shall be taken:

1. The Contractor shall cut out two 6 in. x 6 in. windows in the steel pile shell opposite each other at a distance of 2 times the diameter of the pile down from the top of the pile, or as directed by the Engineer. Any concrete ripples due to
corrugation in the steel shell shall be removed, to provide a flat surface for attaching test instruments.

2. The State will drill holes into the concrete, place anchor plugs and attach the test instruments.

3. The Contractor shall use a female pile helmet suited to the pile size. The pile rebar shall be removed down to the top of pile, and a plywood pile cushion placed on the pile. A metal plate shall then be placed on the pile cushion, and the hammer placed over this. The metal plate shall fit inside the female helmet.

4. Before testing the pile, the hammer shall be warmed up by operating on the ground or on another pile.

B. Dewatering in Difficult Conditions

The Dewatering and Sheeting Task Force, composed of representatives from the Office of Construction, Office of Structures, Region 1, Claims Unit, and the Geotechnical Engineering Bureau has agreed on a Special Note to be used on projects with critical soil and water conditions. The Special Note, for trenches or excavations where it is essential that the competence of the foundation soils not be reduced by groundwater inflow, is as follows:

Special Note for Dewatering in Difficult Conditions
The Contractor shall install a dewatering system that will keep groundwater a minimum of 2 ft. below the bottom of the excavation at all times for the beginning of excavation until the facility in the excavation has been constructed and the excavation backfilled to 2 ft. above original groundwater level. Standby pumping equipment shall be readily available to assure continuous pumping. The Contractor shall install observation wells to permit monitoring of ht groundwater level during construction. Observation wells in trenches shall be spaced so that there is at least one observation well in every 50 ft. long section of trench. In excavation other than trenches, the minimum number of required observation wells shall be calculated by dividing the area of the excavation bottom by 120 yd² and rounding off to the next higher whole number. The Engineer may order additional observation wells to be installed at no extra cost to the State if groundwater problems are observed during excavation. Prior to beginning work, the Contractor shall submit to the Deputy Chief Engineer (Technical Services) for review a detailed plan of the proposed dewatering system, showing the arrangement and location of wells and well points, methods of installation, location of headers and discharge lines, points of discharge disposal, location and depth of observation wells and other features of the system. Review by the Deputy Chief Engineer (Technical Services) shall not relieve the Contractor of responsibility of the adequacy of the dewatering system to achieve the specified result. The cost of all labor, equipment and materials necessary to supply, install, operate and remove the dewatering system, including stand-by equipment and observation wells, shall be included in the price bid for the applicable excavation item.

It may be necessary to modify the Special Note under specific project and site conditions.
C. Unsuitable Excavation and Backfill

The following is an example of a Special Note used for this type of construction technique:

Special Note for Unsuitable Excavation and Backfill
Excavation of unsuitable material and backfill with Item 203.06 from approximate Station 235+00 as shown on the earthwork section shall be progressed in a close sequence operation.

The close sequence of excavation/backfill shall be such that the distance measured parallel with the roadway centerline, from the toe of excavation to the toe of backfill at all locations, shall not be greater than 15 ft. at all times during each day’s operations and less than 3 ft. at the conclusion of a day’s operations. These restrictions are essential in order to preserve the stability of the exiting embankment and pavement. This operation shall be among the first operations under this contract in order to allow the new embankment to obtain maximum stability prior to paving. Excavation shall be to the limit shown on the sections. These limits may only be extended by the Engineer in consultation with the Regional Geotechnical Engineer. Direction of mucking and backfilling shall be from west to east.

The excavation shall be backfilled with granular fill Item 203.06 to 2 ft. above the water surface and a steep (1V on 1.5H) backslope into the excavation shall be maintained at all times. The Contractors attention is drawn to Note A contained on Plan Sheet 34 limiting the top size of the backfill Item 203.06 in the culvert extension areas. The minimum water level within the excavation shall be maintained at elevation 138± to reduce or eliminate sloughing of the excavated side slope. No additional payment will be made for excavating material which sloughs into the excavation.

The Contractor shall use extreme caution when operating equipment on the existing embankment in the area immediately adjacent to the open excavation. Any operator of equipment in this area shall be responsible for the safety of the traveling public at all times. If existing highway should start to fail, the Contractor shall immediately take such steps necessary to repair such failure as ordered by the Engineer. Any additional material needed to accomplish this will be paid for under their respective items.

The Contractor may suggest changes by submitting alternate methods to the above identified construction procedures. Such changes must be submitted for approval by the Regional Construction Engineer and the Regional Geotechnical Engineer.

Any select material contaminated with unsuitable material as a result of the Contractors operations shall be removed and replaced in kind at no cost to the State.

It is necessary to modify the Special Note for specific project and site conditions.
Appendix 26-C  Typical Design Cross-Section for a Deep Foundation

The following figure is an example of a design soil cross-section for a deep foundation. This figure illustrates the types of information that should be included in an idealized cross-section to introduce a foundation design calculation. Depending on the nature of the calculation and type of geotechnical feature, other types of information may be needed to clearly convey to the reviewer what data was used and what was assumed for the design.