HIGHWAY DESIGN MANUAL

CHAPTER 9 - SOILS, WALLS, AND FOUNDATIONS

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SOILS, WALLS, AND FOUNDATIONS

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9.1 INTRODUCTION

This chapter outlines the role of geotechnical engineering in the project development process. The designer should be aware that the entire highway system is supported by soil and rock, and that means for ensuring its adequate support must be considered in design. Soil and rock are the major construction materials used in the highway system, with about a fifth of construction costs spent on earthwork alone. Engineering properties of soils are more variable than any other material used in highway construction. Exploration programs thus are necessary to evaluate engineering properties of soil and rock that will be encountered, and to determine adequate and economical methods for using and dealing with them. The Geotechnical Engineering Bureau (in conjunction with the Office of Structures) designs foundations for all highway bridges. At the request of the Regional Geotechnical Engineer, the Geotechnical Engineering Bureau recommends cut slope and embankment foundation treatments, soil strength parameters, and assists in foundation design for other highway features, such as walls or culverts. Designers, in coordination with project managers, must give full consideration to soil exploration and testing requirements in determining resource and scheduling needs.

The Regional Geotechnical Engineer and Geotechnical Engineering Bureau are equipped to explore and analyze soil and rock conditions in the project area, and provide the designer with specific recommendations compatible with those conditions. Since the relationship between soil conditions and design of a highway project often are profound, coordination by the designer with the Regional Geotechnical Engineer throughout the project development process is essential. The designer's responsibility is to solicit those recommendations in a timely manner, and incorporate solutions into the contract documents.

Soil and rock problems occasionally require specialized explorations, laboratory testing, and design analysis for adequate solutions. The Geotechnical Engineering Bureau provides these services to the designer through the Regional Geotechnical Engineer. However, these services take time. It is essential that the Designer involve the Regional Geotechnical Engineer early in the design process to avoid any schedule conflicts. This chapter acquaints the designer with the soil exploration procedures used, soil conditions to consider in design, and some typical solutions.

On occasion, certain aspects of the geotechnical portion of the plan development are performed by Consultant Engineers. The designer must consult with the Regional Geotechnical Engineer prior to delegating this work to the consultant to determine if this work can be performed "in-house." If the geotechnical services of a consultant are required, the Regional Geotechnical Engineer will advise the designer of the specific responsibilities of the State and the consultant in performing this assignment. To help the designer decide what information is needed and who should provide it, a list of soils-related task assignments can be found in the following Geotechnical Engineering Bureau publication available on the IntraDOT or through the Regional Geotechnical Engineer: *Design Consultant Agreements: Soils-Related Task Assignments*[(4)].

9.2 GEOTECHNICAL INVESTIGATIONS

9.2.1 **Geotechnical Reports**

These describe the engineering significance of soil and foundation conditions in a project area, and relate those conditions to the project's design. As the project progresses through various phases of development and design, these reports become more refined and precise and must include recommendations to deal effectively with particular local soil conditions. The following sections correspond with the procedural phases of a project's development outlined in the Department's Project Development Manual. Sections 9.2.1.1 through 9.2.1.5 describe the progression of geotechnical reports, and the participation of both Regional Geotechnical Sections and the Geotechnical Engineering Bureau in their preparation. Section 9.2.1.6 outlines the geotechnical program during a project's construction.

9.2.1.1 Project Initiation Stage – Initial Project Proposal (IPP)

1. REGIONAL PLANNING AND PROGRAM MANAGEMENT GROUP
   Provides the Regional Geotechnical Engineer with draft of project management strategy and project objectives, and tentative identification of functional areas to be involved.
   Receives feedback from the Regional Geotechnical Engineer on activities, key products, final event, preliminary schedule, and resource requirements for project scoping.

2. REGIONAL GEOTECHNICAL SECTION
   Provides preliminary geotechnical information based on knowledge and experience with soil and rock conditions in the vicinity of the proposed project.
   Provides input to preliminary schedule for project scoping, including availability of additional resources (or possible need for them).
   Advises on preliminary estimates in the Initial Project Proposal (IPP).
   Provides generic costs [$/m^2 or $/km] and preliminary schedule.
   Indicates special technical activities that may be needed.
   Alerts IPP preparer of potential geotechnical problems.

3. GEOTECHNICAL ENGINEERING BUREAU
   Provides technical assistance, when requested.

9.2.1.2 Project Scoping Stage

1. REGIONAL DESIGN GROUP – Project Manager
   Provides the Regional Geotechnical Engineer with a corridor location plan, a contour plan and/or project description, project schedule data and a completed checklist for determining if subsurface explorations are required (See Appendix 9B).
   Receives Terrain Reconnaissance Report or Preliminary Geotechnical Report.

2. REGIONAL GEOTECHNICAL SECTION
   Performs a thorough review of the project site, reviews the checklist for determining if subsurface explorations are required (See Appendix 9B), establishes the need for subsurface explorations and schedules all necessary subsurface explorations.
   Prepares a Preliminary Geotechnical Report which includes identification of major soil and rock deposits on the project or in each corridor.
   Evaluates the engineering significance of major deposits that could affect highway performance, construction costs, and schedules. Occasionally, right-of-way considerations may be influenced by soil and rock deposits.
   Maintains close communication with the project developer as project details evolve.
   a. Field reconnaissance.
      - Evaluates performance of nearby facilities.
      - Evaluates existing pavement and shoulder performance, and determines need for drainage improvements to existing facilities.
   b. Review of previous subsurface explorations.
   c. Preliminary subsurface exploration if needed for major problem evaluation.
   d. Coordinates with Geotechnical Engineering Bureau for technical assistance:
      - Rock Slope Location and Identification Report.
      - Rock Outcrop Map.
   e. Provides input to project management process:
      - By participating in scoping meeting.
      - By providing comments on scoping document.
      - By providing input to Project Management Plan for design.

3. GEOTECHNICAL ENGINEERING BUREAU
   Provides technical assistance to the Region when requested.

   a. Terrain Reconnaissance Reports for major projects on new location when requested by Region.
   b. Assistance to Regional Geotechnical Section in preparing reports when requested.
   c. Engineering Geology Section reports when requested:
      - Rock Slope Location and Identification Report.
      - Rock Outcrop Map.
   d. Survey of existing geotechnical data:
      - Agricultural soils maps.
      - Geologic literature.
      - Analysis of air photos.

9.2.1.3 Project Design Stage - Preliminary Design (Design Phases I through IV)

1. REGIONAL DESIGN GROUP
   Provides Regional Geotechnical Engineer with plans, profiles, and typical sections of alternative designs.
   Receives Geotechnical Report for inclusion in Design Approval Document (or update of previously provided report, if necessary).

2. REGIONAL GEOTECHNICAL SECTION
   Prepares or updates Geotechnical Report for Design Approval Document.

   a. Identifies major soil and rock deposits for each alternative design and executes the subsurface exploration plan.
      - Field reconnaissance: Performance of existing highway(s) and nearby facilities.
      - Review of existing geotechnical data.
      - Subsurface explorations progressed for bridges, walls, fills, cuts, culverts, dams, major problem areas and evaluation of economic and environmental considerations.
   b. Determines engineering significance of major soil and rock deposits that could affect highway performance or construction costs, influence of soil and rock deposits on location, alignment, and right-of-way considerations, and appearance of completed project
   c. Evaluates existing pavement and shoulder structure and foundation conditions and provide input to the Regional Materials Engineer for projects requiring a pavement evaluation.
   d. Assists in determining the location and types of retaining walls and any specialized cut slope, embankment, or foundation treatment that may be required.
      - Soil parameters for retaining wall design, recharge basins, storm water management practices, and wetland creation sites.

3. GEOTECHNICAL ENGINEERING BUREAU
   Provides for subsurface exploration, special equipment, technical advisory service, and assistance in preparing drilling contracts.
   - Geologic surveys: seismic and resistivity explorations to determine depth to rock surface, water level, etc., for cut slope design and earthwork quantity estimates.
   - Data for recharge basin design, storm water management practices, and wetland creation sites.

Section 9.2.1.4 Project Design Stage - Advanced Detailed Plans (Design Phase V)

1. REGIONAL DESIGN GROUP
   Provides the Regional Geotechnical Engineer with plans, profiles, and generalized cross-sections (Advance Detail Plan or "ADP"), all plan and profile changes in development, and project schedule data.
   Receives Geotechnical Report for Advance Detail Plans (or update of previously provided report if necessary).

2. REGIONAL GEOTECHNICAL SECTION
   Provides the following geotechnical information for Advance Detail Plans, as appropriate:

   - Recommendations to provide designers necessary information to prepare plans and quantity and cost estimates for earthwork items.
   - Confirms location, alignment, and right-of-way treatments, based on soil and rock conditions.
   - Recommendations for treatment of embankment foundation problems and cut slopes, bank and channel protection.
   - A determination that sufficient subsurface explorations have been progressed.
   - Special earthwork specifications and typical sections, if required.
   - Accurate location of all subsurface explorations, and coordinate data for plotting on plans.

3. GEOTECHNICAL ENGINEERING BUREAU
   Provides technical services and assistance to Regions, when required or as requested:

   - Roadway foundation design: recommendations for treatment of embankment foundation soils in critical areas.
   - Earth cut slope design: recommendations in critical soil and groundwater seepage areas.
   - Retaining walls designed by Regions: recommendations for vertical and lateral earth pressure considerations.
   - Roadway design: general assistance to Regional Geotechnical Engineer where required.
   - Pipe design: unusual problems involving earth pressures and jacking procedures.
   - Channel slope protection: involving stone fill, riprap, gabions, stapods, except near structures.
   - Recommendations for Foundation Design Report.
   - Boring Location Plan and Subsurface Profile.
   - Special pavement design.

9.2.1.5 Project Design Stage - Final Plans, Specifications and Estimate (Design Phase VI)

1. REGIONAL DESIGN GROUP
   Receives Final Geotechnical Report (or update of previously provided report if necessary).

2. REGIONAL GEOTECHNICAL SECTION
   Compiles package of all geotechnical data for examination by bidders, including boring logs, test results, rock outcrop maps, Terrain Reconnaissance Report, and any other project information available from the Geotechnical Engineering Bureau.

3. GEOTECHNICAL ENGINEERING BUREAU
   Provides technical services and assistance to Regions, when required, or as requested, including lab test result summaries and special reports prepared by the Geotechnical Engineering Bureau.

9.2.1.6 Project Construction Stage

1. REGIONAL CONSTRUCTION GROUP
   Requests the Regional Geotechnical Engineer to attend or provide:
   a. Preconstruction geotechnical meeting.
   b. Preblasting meeting.
   c. Sampling and testing of granular material.
   d. Advice on quality of stone fill and riprap.
   e. Earthwork inspector training and equipment (state-inspected contracts).
   f. Geotechnical construction advice.
   Receives requested test data, training, and equipment.

2. REGIONAL GEOTECHNICAL SECTION
   Requests technical assistance from the Geotechnical Engineering Bureau, when required; Provides requested test data, services, training, equipment, and reports to the Construction Division.

3. GEOTECHNICAL ENGINEERING BUREAU
   Provides:
   - Test data (subbase, gravel, underdrain filter, compaction curves, topsoil).
   - Project testing equipment, training and supplies (for state-inspected contracts).
   - Technical assistance for constructing soils-related items.
   - Services:
     - Inspections, reports, pavement design.
     - Stone filling and riprap evaluation.
     - Presplitting and rock excavation inspection.
     - Water-well installation.
     - Instrumentation.
     - Vibration monitoring.
     - Recommendations on field construction problems.
     - Length analysis of piles for structures.
     - Analysis of pile-driving equipment.
     - Inspection of rock foundations.
     - Review of orders-on-contract.
     - Preconstruction, preblasting meetings.

9.2.2 **Subsurface Explorations**

The subsurface exploration program is an integral part of any comprehensive geotechnical engineering program. Unlike working with materials such as steel and concrete, whose properties can be specified and controlled, designers and contractors are forced to work with the subsurface conditions present at the site. Soil strength, rock location, ground water conditions, susceptibility to settlement, and soil behavior under various conditions cannot be specified. Due to the fact that these properties can be highly complex and variable, they need to be accurately characterized.

Subsurface explorations are progressed in order to determine the characteristics of the material that must be removed to complete the project or upon which the project will be supported. Prior to the practice of geotechnical engineering (Pre-1930's), civil engineering projects were constructed by trial and error, or over-designed so much that failure was impossible. With the advent of the science of soil mechanics, it became possible to design projects to be built in conditions that previously would not have permitted construction. However, it is imperative that the geotechnical engineer have an adequate knowledge of the subsurface conditions at the site to produce an appropriate design. Proposed structure locations, including retaining wall locations, require extensive exploration since the soil conditions have great influence on their foundation design. An initial investment into a subsurface exploration program will produce an appropriate, properly designed foundation saving costly failures or unnecessarily overly conservative designs.

The quality of the subsurface exploration information has a direct bearing on the quality of the product produced by the geotechnical designer. In addition to quality, schedule needs often require a quick response. Geotechnical engineers need to have an understanding of the reliability of the information obtained from subsurface explorations. The subsurface exploration program includes borings, manual methods, and geophysical methods.

Figure 9-1a Drilling in Roadway **Figure 9-1b Drilling in a Parking Lot**

Figure 9-1c Drilling on Steep Terrain

Figure 9-1d Drilling on Ice

Figure 9-1e Drilling with an All-Terrain Vehicle (ATV) Drill Rig

Figure 9-1f Drilling on Soft Soils

Figure 9-1g Drilling off a Barge

The value of proper subsurface explorations to the Department is to provide information to produce an economical, appropriate design that can be constructed quickly and safely while avoiding unanticipated subsurface conditions that would result in delays and orders-on-contract, or in extreme cases, premature distress or failure.

All Regions, except Region 11, have assigned drilling titles to operate drill rigs. Drill rigs are purchased and maintained by the Office of Fleet Administration and Support. Supplies are purchased and provided by the Geotechnical Engineering Bureau. The Office of Design controls and allocates drilling titles.

The subsurface exploration program is executed by Regional drill crews, and augmented by several different types of drilling contracts, including State-let, Consultant-let, Where-and-When, and Procurement. The knowledge, experience, and training of State drillers are essential to maintain quality assurance of contract drilling operations, as well as quality control of State drilling.

The Geotechnical Engineering Bureau and Regional Geotechnical Engineers are knowledgeable in the types of contracts available, and can assist the designer in assessing the need for one, determining the most appropriate type, and assembling it.

The Regional Geotechnical Engineer or consultant will prepare a log of the subsurface exploration. This log includes elevation and location data for this exploration. The Regional Geotechnical Groups utilize a Global Positioning System (GPS) receiver to obtain the location data for the log. In addition, the Regional Geotechnical Group will work closely with Regional survey forces to obtain information on bench marks in the area to establish the elevation for the subsurface exploration. If borings are done by contract, the consultant or contractor provides the elevation and location data to the Regional Geotechnical Engineer.

Exploration methods are varied, including disturbed-sample (or split-barrel) and undisturbed-sample drill holes, auger holes, probe holes, test pits, rock-core drilling, observation wells, geophysical exploration, the cone penetrometer, retractable-plug sampling, and falling weight deflectometer. Other methods of exploration, such as seismic survey, are also used in certain circumstances. Some methods of subsurface exploration, such as seismic survey, are performed by Geotechnical Engineering Bureau personnel. Other methods less commonly used will be obtained through contract. Obtaining subsurface explorations should be considered as early in the project development process as possible.

9.2.2.1 Split Spoon Sampling

This is the most common type of exploration. Drilling and sampling basically involve three operations: (1) driving a pipe (casing) to a desired depth, while recording the effort used (by blow counts), (2) removing loosened material from inside the casing, and (3) sampling material below the casing with one of several tube-like samplers. The casing keeps the hole open for the sampler. When occasionally a casing pipe is not used, the hole is kept open by filling it with a slurry called driller's mud.

The sampler is a cylindrical tube about 600 mm long. Once the soil sample is entrapped in it, the sampler is removed from the hole. It is fabricated so that the cylindrical tube can be opened (split) longitudinally, exposing the sample for examination and removal. This method is called disturbed or split-barrel sampling, providing samples used for identification and classification.

Additional information on the Standard Penetration Test (SPT) can be found in AASHTO T206 /ASTM D1586 Penetration Test and Split-Barrel Sampling of Soils(5e).

Figure 9-2a Split Spoon Sampler

Figure 9-2b Split Spoon Sampler Opened (Split)

The term "driving" implies dropping a specific weight from a certain height onto the end of the casing or sampler. The effort or "blow counts" required to drive the sampler, as well as the casing, are evaluated by the Regional Geotechnical Engineer as indicators of soil properties.

A variation of this process is the hollow-flight auger. The hollow-flight auger is a mechanically or hydraulically powered, helical instrument. Its core or stem is a hollow cylinder large enough in diameter to accept a disturbed or split-barrel sampler. The helix extends to the length of the hole and acts much like a casing. The sample is removed through the hollow stem. Unfortunately, although this method of progressing the exploration is often faster and easier than driving a casing, valuable casing blow data is not acquired.

Figure 9-3a Drilling with Augers

Figure 9-3b Hollow-Flight Auger with Split Spoon Sampler

9.2.2.2 Undisturbed-Sample Drill Holes

The purpose of this type of exploration is to acquire a sample in as near an undisturbed condition as possible, and it is used primarily in deposits of cohesive soil. By obtaining an undisturbed sample and mimicking the in-situ, or "in the original location", conditions, soil sample tests are used to enhance a subsurface profile, provide data on soil properties, and obtain parameters for empirical analysis and design applications. The method is similar to that for the drilling and sampling operation described in 9.2.2.1 Split Spoon Sampling, except for the sampling technique. The sample is taken by pressing a thin-walled tube sampler into the soil. The sample is then sealed into the tube, and tube and sample are sent to the Geotechnical Engineering Bureau laboratories. Samples are tested and analyzed to determine the soil's inherent strength and settlement characteristics. Additional information on undisturbed soil sampling can be found in AASHTO T207 /ASTM D1587 Thin-Walled Tube Sampling of Soils[5f].

Figure 9-4a Undisturbed Sampling (Setting Sampler)

Figure 9-4b Undisturbed Sampling (Breaking Down Sampler)

Figure 9-4c Undisturbed Sampling (Sealing Sample with Wax)

Figure 9-4d Undisturbed Sampling (Capping Sample and Securing for Delivery to Lab)

Figure 9-4e Extracting Undisturbed Soil Sample in Lab

Figure 9-4f Undisturbed Soil Sample Testing: Triaxial Test

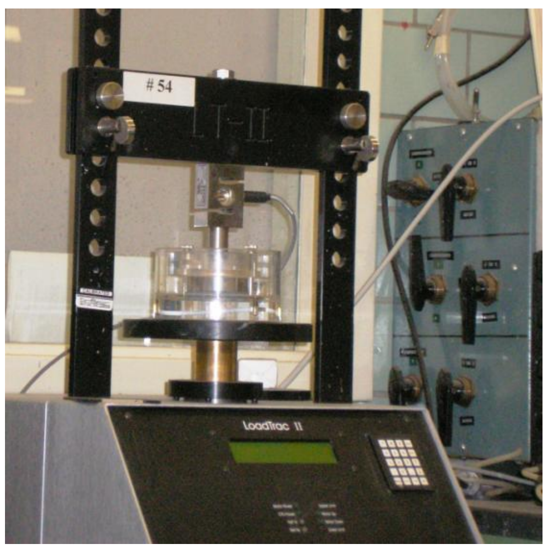


Figure 9-4g Undisturbed Soil Sample Testing: Consolidation Test

9.2.2.3 Auger Holes

This is another common type of exploration, and involves drilling an auger into the ground, either manually or mechanically, and extracting a sample for identification and classification. The manual auger is similar to a post-hole digger and usually varies in diameter from 50 mm to 150 mm. This type of exploration is limited by soil condition, groundwater level, and energy of the operator.

A power auger hole is made by a solid-stem, helical auger. Soil samples are taken from the flights. Those thus acquired are very disturbed, and it is uncertain at what depth they were encountered.

9.2.2.4 Probe Holes

This type of exploration consists of driving a rod into the ground until it is stopped by an obstruction, and is used primarily to determine if rock will be encountered during construction. However, samples are not acquired using this method, nor is it certain if refusal is actually caused by bedrock, a boulder, compact soil, or some other buried hard object.

Figure 9-5a Probe Hole (Progressed via Poyne Jar)

Figure 9-5a Probe Hole (Jack to Extract Rods)

9.2.2.5 Test Pits

These are simply manually or mechanically dug holes, often large enough for persons to work in, used to investigate subsurface strata, determine groundwater conditions, or sample granular material sources. They are particularly useful in examining characteristics of landfill deposits.

Small test pits may be used as percolation test pits or holes. Basically, a small 0.03 m^3 or 0.06 m^3 hole is dug and flooded with water. The percolation rate (or speed of the water level dropping in the hole) is related to the soil's permeability.

9.2.2.6 Rock Core Drilling

This is done in conjunction with a drill hole. A cylindrical sample of rock is taken in a sampler fitted with a diamond-toothed cutting bit. The resulting rock core is examined by an Engineering Geologist who determines the type of rock, its structural condition, the slope to which it may be safely cut, and its bearing capacity.

Figure 9-6a Extracting Rock Core Sample

Figure 9-6b Labeled & Boxed Rock Core

Figure 9-6c Point Load Test Set-Up

Figure 9-6d Point Load Test (Testing A Rock Core)

Figure 9-6e Unconfined Compression Test

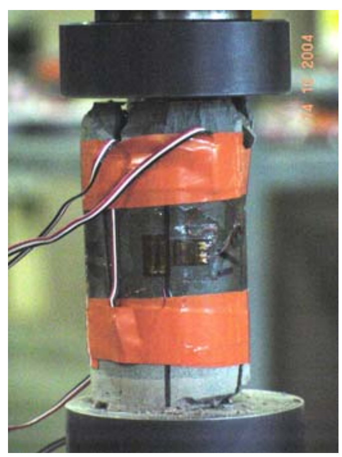


Figure 9-6f Unconfined Compression Test (Instrumented Rock Core)

9.2.2.7 Observation Wells and Piezometers

Occasionally holes are drilled to sample and/or measure depth to groundwater. Samples are sometimes tested for chemical or biological content. Both sampling and storage of samples must be carefully controlled. Since groundwater level fluctuates, the drill hole casing may be left in place, or a perforated tube may be left in the hole, for access in further monitoring of groundwater depth determinations on a predetermined schedule. This is commonly called a water observation well, but is more correctly referred to as an **open-well piezometer**.

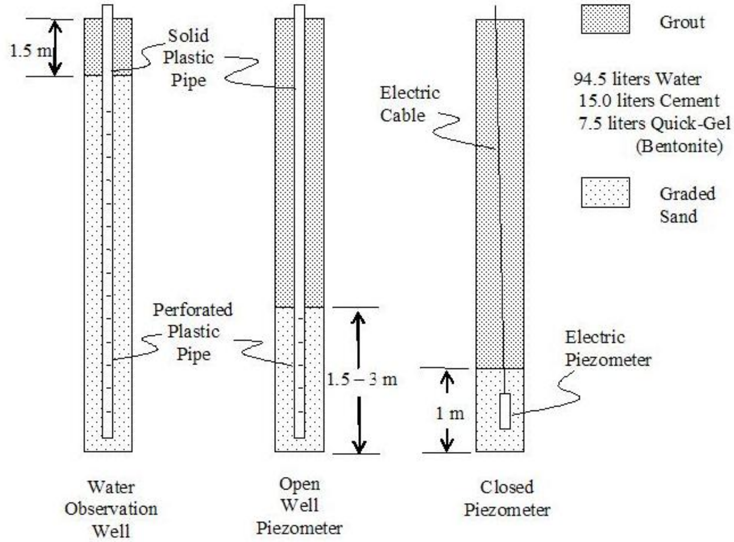
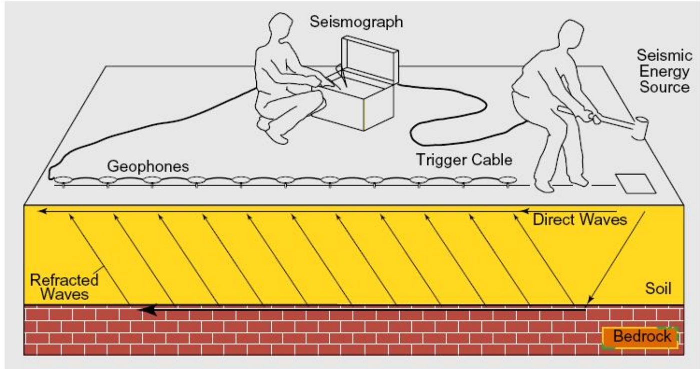


Figure 9-7 Typical Pore Water Pressure Monitoring Installation

The important distinction is that an open-well piezometer isolates the soil layer in question, and an observation well does not. Water observation wells are not recommended. More accurate measurements of pore water pressure are occasionally called for, requiring the installation of vibrating wire piezometers, which may be read remotely.

9.2.2.8 Geophysical Exploration

Depth to rock may be determined remotely by seismic refraction methods. This consists of introducing an impulse of energy into the soil, and computing the velocity of this seismic energy wave through various earth and rock strata. Results should be monitored or verified with drill holes.



from: Benson, 1983

Figure 9-8 Seismic Refraction

9.2.2.9 Cone Penetrometer

This involves a two-piece probe consisting of a cylindrical sleeve with a separate conical tip. As the probe is pressed into the ground, pressure meters measure resistance to penetration by the sleeve and conical tip. These measurements correlate to strength of the soil, but no samples are acquired, and resistance of the soil to penetration can severely limit the depth of exploration.

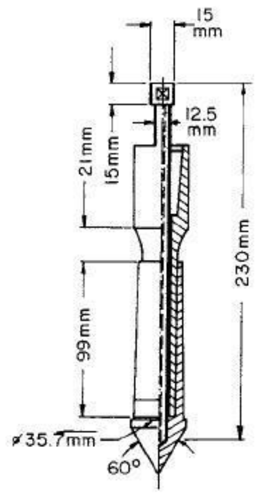


Figure 9-9a[5g] Collapsed Mechanical Cone Penetrometer Tip (Dutch Mantle Cone)

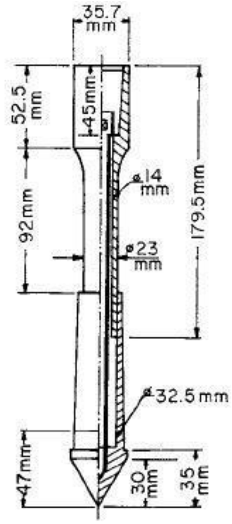
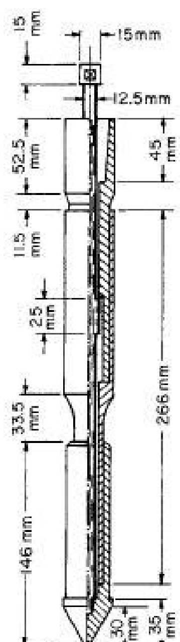


Figure 9-9b[5g] Extended Mechanical Cone Penetrometer Tip (Dutch Mantle Cone)



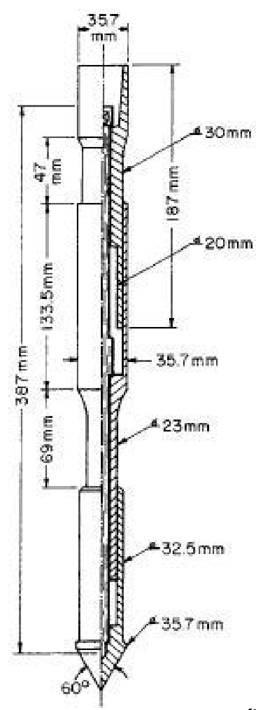


Figure 9-9c[5g] Collapsed Mechanical Friction-Cone Penetrometer Tip (Begemann Friction-Cone)

Figure 9-9d[5g] Extended Mechanical Friction-Cone Penetrometer Tip (Begemann Friction-Cone)

9.2.2.10 Retractable-Plug Sampling

This is a hand-driven tube fitted at its end with a pointed plug. When the tube (normally of 25 mm diameter) reaches the sampling depth, the plug is retracted and a thin-walled sampling tube is forced into the soil. The tube and entrapped sample are then removed. The device is highly portable, but severely limited and cannot be used in coarse soils or for deep explorations.

Figure 9-10 Retractable-Plug Sampling Equipment (Hammer, Drive Head, Tube, and Brass Sampling Tubes)

9.2.2.11 Falling-Weight Deflectometer

The falling-weight deflectometer is a trailer-mounted device that exerts a load on the pavement and records the response of the pavement structure to this loading. The recorded data is analyzed to determine the competency of the existing pavement structure and to determine parameters, such as resilient modulus, to assist in pavement design. The falling-weight deflectometer testing is performed by the Highway Data Services Bureau.

Figure 9-11 Falling-Weight Deflectometer (FWD)

9.3 SOIL AND FOUNDATION CONSIDERATIONS

Investigations described in Section 9.2 define and analyze soil and pavement conditions, so that adequate and economical solutions for any soil- or rock-related problems may be incorporated into the design. Those encountered unexpectedly during or after a project's construction are more expensive and usually more difficult to solve than those identified and solved during design. Nearly every project involving a significant amount of earthwork will encounter a soil-related problem. Most are relatively simple, requiring only a field evaluation and application of the proper solution. (Examples are underdrains, undercutting, or slope protection, as recommended by the Regional Geotechnical Engineer.) However, some are complex, requiring extensive exploration, testing, analysis, and construction control procedures. The Regional Geotechnical Engineer has direct access to the Geotechnical Engineering Bureau, which has testing facilities and engineers and geologists specially trained in soil and rock mechanics to solve these complex geotechnical problems. Obviously, critical soil-related situations should be avoided, but critical soil deposits are often hidden. Soil should be explored and analyzed as plans progress, to assure that all avoidable soil-related problems are exposed and avoided.

The following sections describe some of the more common soil and foundation conditions, and some treatments that must be considered by the designer and Regional Geotechnical Engineer. Since most solutions affect other significant project design considerations, such as right-of-way requirements, they must be identified and resolved early in plan development.

9.3.1 **Alignment**

A highway's profile has a major effect on project considerations. From a geotechnical perspective, the optimum section is generally a fill 2 m to 3 m high. A 2 m fill will usually bridge minor irregular soil deposits, by providing room to build a working platform on soft, unstable deposits. In addition, a 2 m profile is more likely to be free of subsurface drainage problems than a cut or profile that leaves the roadway near the level of the surrounding terrain. Except for extremely weak soil deposits, a 3 m fill will usually be stable, but this optimum profile may not be attainable because of other considerations, such as costs or effects on adjacent properties.

If cuts must be made, they should be of significant depth, as opposed to skim cuts in the upper, more-variable horizons of the soil deposit. However, location of the water table must be considered and if possible avoided. Profiles involving shallow (skim) cuts and fills, with numerous cut-fill interfaces thus will usually result in numerous subsurface drainage and foundation problems. These are difficult and costly to correct and should be avoided, if at all possible. As a rule of thumb, side-hill cuts are not desirable from a geotechnical viewpoint. If other alignment considerations permit, it is preferable to locate a cut so its alignment cuts directly into the contours of the land, as opposed to being at a skew to them.

9.3.2 Resurfacing, Restoration, and Rehabilitation (3R)

A highway requires 3R work when its surface no longer performs its function in an acceptable manner. The cause of this malfunction may be merely its age, but it often results from inadequate drainage or poor subbase or subgrade conditions, rendering it incapable of adequately supporting traffic loads. A satisfactory roadway is founded on a properly drained, uniform, firm subbase and subgrade. Poor subbase or subgrade conditions must be corrected before resurfacing.

Adequate pavement and shoulder drainage is necessary for the subbase and subgrade to function properly. Any deficiencies in such drainage should be corrected by cleaning, deepening, or constructing ditches (refer to Chapters 3 and 10 of this manual for desired details and location), or installing edge drains, or some combination of these treatments. Since these may constitute a significant portion of the total project cost, the project developer and/or designer should consult the Regional Geotechnical Engineer (RGE) early in development of the project. As appropriate, the RGE should also be included on the scope team. The Regional Geotechnical Engineer will work with the Regional Materials Engineer in preparing an evaluation of existing pavement and shoulder structures, and the foundation and drainage conditions necessary for the project's proper scoping and design.

9.3.3 **Widening**

Pavements are often widened in conjunction with resurfacing. To avoid post-construction differential settlement between the old pavement and added portion of roadway, the additional pavement must be as firm as the existing pavement structure. Thickness of the existing pavement, including the existing subbase, should be determined by reviewing record plans, coring, or other suitable means. Pavement and subbase of the widening should be no thinner than the total existing pavement and subbase.

The widening should not impede surface or subsurface drainage of the existing roadway. Cross-slopes providing internal drainage of the existing pavement and subbase must continue across the widened portion. The entire new width of pavement and shoulders must have proper drainage. Where right-of-way or other restrictions preclude adequate ditches, edge drains or stone-filled trenches should be considered.

To widen the pavement and/or raise the existing profile, the existing embankment often must be widened. Soil conditions in the area of this widening should be thoroughly explored in order to design measures avoiding potential problems of stability or settlement. Benching the existing embankment slope is also usually required, as shown on the standard sheet titled "Earthwork Transition and Benching Details."

Figure 9-12 Embankment Widening Utilizing Benching

9.3.4 **Settlement**

Most highway settlement problems result from consolidation of foundation soil due to a static load, such as an embankment. This settlement is caused by both rearrangement of soil particles and extrusion of water from pores or voids in the soil mass. Two aspects of settlement must be considered -- quantity and time. The solution must resolve both quantity and duration of settlement in a practical manner. This settlement may be substantial (on the order of a meter or more) or may seem insubstantial (millimeters), but the designer should recognize the differential effects of even a small settlement adjoining a rigid object, such as a pile-supported structure.

The most likely solution to a settlement problem is reducing the load by lowering the profile. If the profile cannot be adjusted, one method of reducing the amount of settlement is using lightweight fill. Available types and sources can be suggested by the Regional Geotechnical Engineer and the Geotechnical Engineering Bureau.

Figure 9-13a Lightweight Fill- Expanded Polystyrene Blocks (EPS)

Figure 9-13bLightweight Fill- Expanded Polystyrene Blocks (EPS)

Figure 9-13c Lightweight Foamed Concrete

Figure 9-13d Lightweight Foamed Concrete

**Figure 9-13e Lightweight Fill-
Tire Shreds**

**Figure 9-13f Lightweight Fill-
Tire Shreds**

**Figure 9-13g Lightweight Fill-
Pumice**

**Figure 9-13h Lightweight Fill-
Pumice**

**Figure 9-13i Lightweight Fill-
Expanded Shale**

**Figure 9-13j Lightweight Fill-
Expanded Shale**

Effects of consolidation on the finished project are sometimes reduced by a waiting period. This is a designed delay in construction sequence after building the embankment but before placing the pavement or structure. A surcharge is sometimes used in conjunction with the waiting period. By building the embankment higher than the final grade, a surcharge imposes a load on the underlying soil that exceeds the final embankment load. By selecting the appropriate surcharge load and duration of loading, most post-construction settlement may be eliminated.

Figure 9-14a Surcharge of Approach Embankment (Additional Fill)

Figure 9-14b Surcharge of Approach Embankment (Instrumentation of Fill)

As previously stated, some settlement is associated with extrusion of water from voids in the soil mass. Permeability of the soil and distance the water must travel determine the duration of settlement. The distance may be reduced by using sand or wick drains. Sand drains are vertical columns of porous sand constructed through the compressible layer at predetermined spacings. Wick drains are prefabricated continuous drainage elements pushed through the compressible layers to a specified elevation. Water collected by these drains is then allowed to flow away from the area by sand blankets and drainage ditches.

Figure 9-15a Wick Drain Installation (Layout of Drains)

Figure 9-15b Wick Drain Installation (Instrumentation of Some Drains)

These methods of solving settlement problems require detailed analysis and design, involving undisturbed sampling and laboratory testing. Settlement-monitoring devices, such as settlement stakes, surface and subsurface settlement gages, or piezometers are often required to determine if the treatment method is effective. Potential for settlement problems and their alleviation are specialties requiring the designer to consult the Regional Geotechnical Engineer during development of plans.

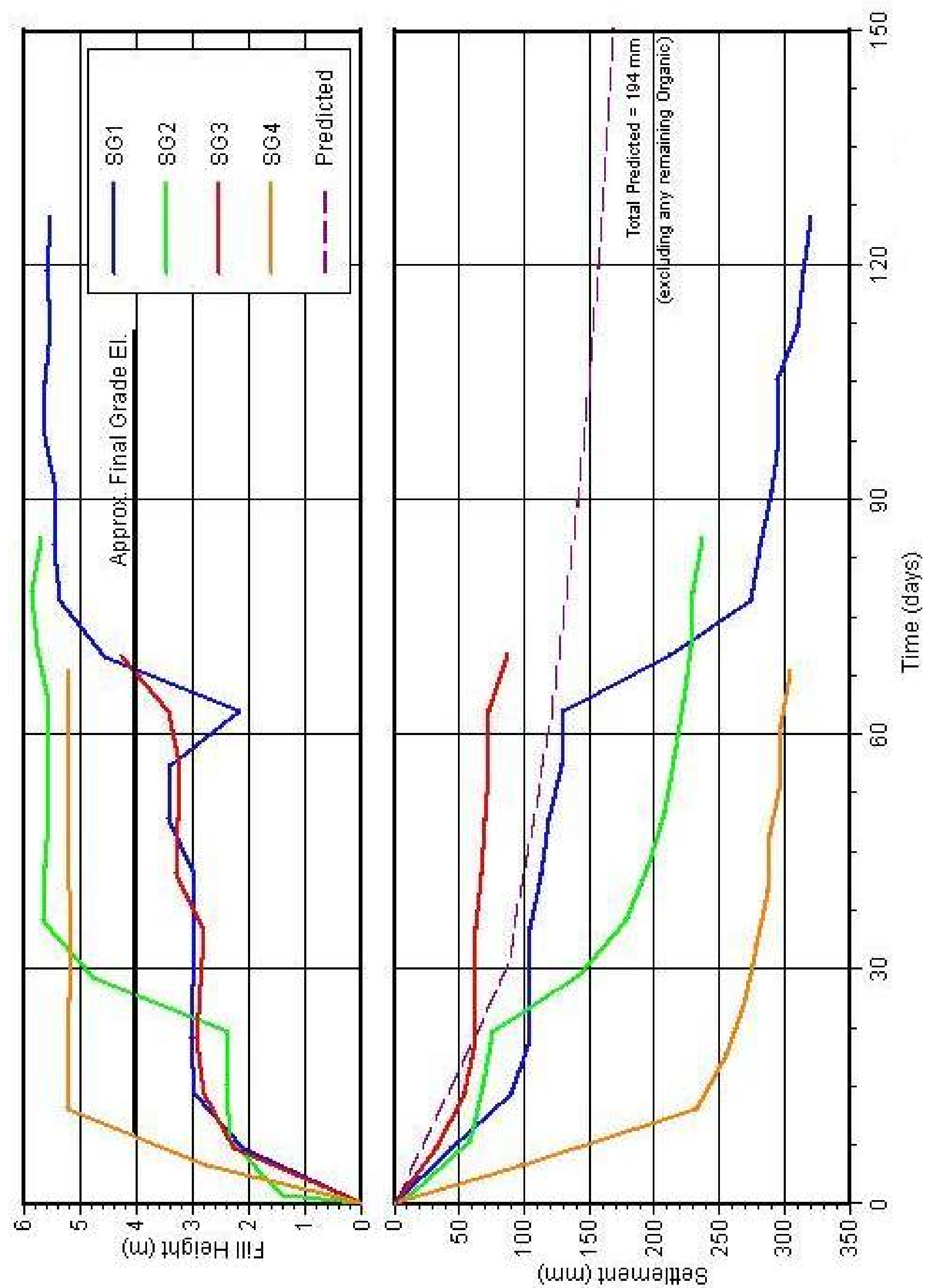


Figure 9-16 Example of Data Collection from Settlement-Monitoring Devices

9.3.5 **Stability**

When a load (such as is imposed by an embankment or structure) exceeds the soil's strength, the soil fails -- this is a stability problem.

Figure 9-17 Deep-Seated Slope Failure

Figure 9-18 shows a classic embankment stability failure. Typically, failure movement is rotational around an imaginary point located somewhere above the embankment slope. The embankment to the left of this imaginary point of rotation forms the driving force and the material to the right is the resisting force. When driving force exceeds resisting force and natural shear strength of the soil, the mass rotates around the point of rotation until equilibrium is attained.

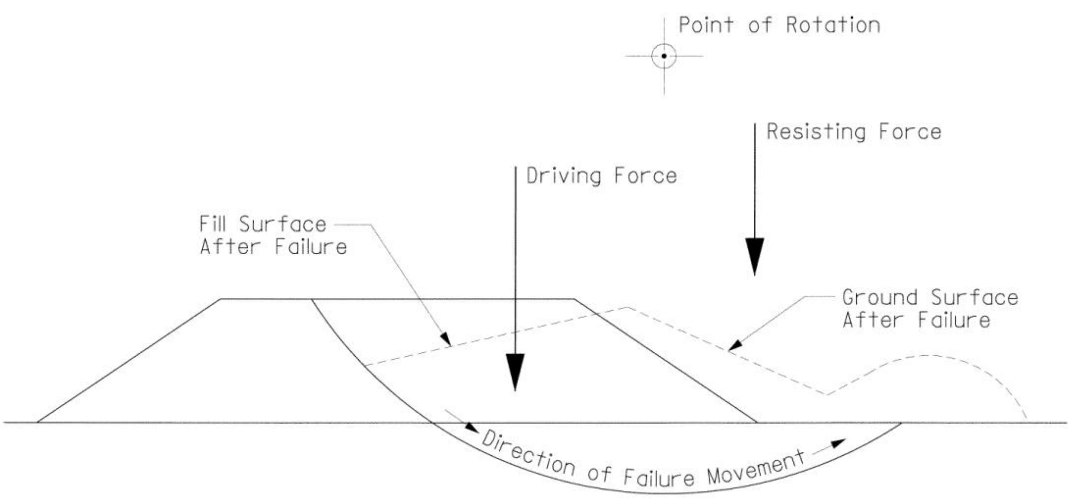


Figure 9-18 Stability Failure of an Embankment

One practical solution to a stability problem is reducing the driving force by lowering the profile. Since a change in profile usually has a significant effect on other elements of highway design, it is important that maximum safe embankment height be established early in project design.

Another solution is a stabilizing berm as shown in Figure 9-19. Resisting force is increased by the weight of the stabilizing berm. Normally, earth-fill slopes are no steeper than 1 vertical on 2 horizontal, and flattening the slope is a variation of this concept that sometimes suffices and may have traffic safety benefits.

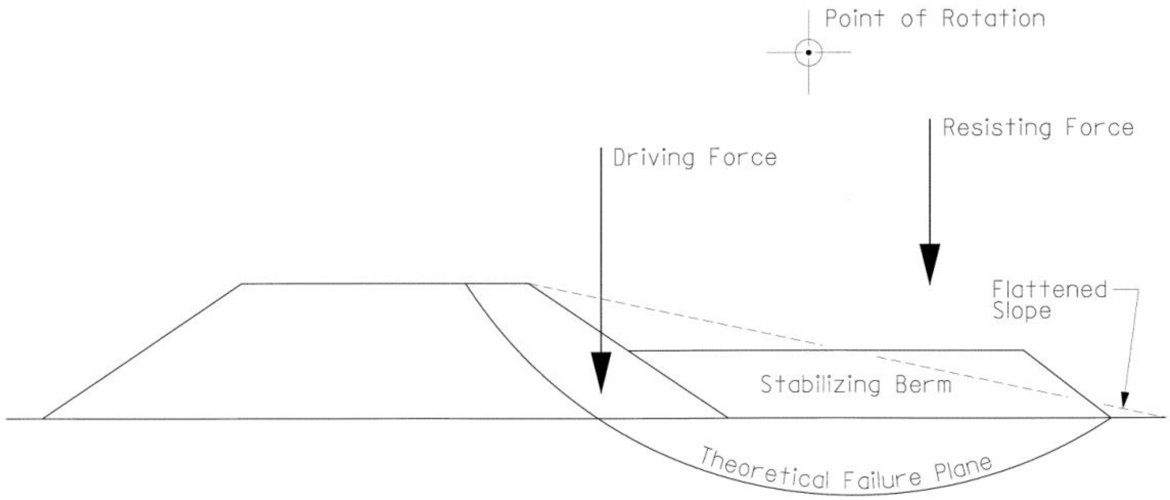


Figure 9-19 Embankment with Stabilizing Berm

Figure 9-20 Constructed Stabilization Berm

A third common solution is staged construction. The nature of most soil is that its strength will slowly increase under an imposed load, which occurs as water is squeezed out of the soil's pore spaces. Staged construction consists of raising the embankment to intermediate levels and delaying subsequent loading until the anticipated strength increase is realized. Sand drains or wick drains (described in Section 9.3.4 "Settlement") are sometimes used in conjunction with staged construction to accelerate the strength buildup and reduce the waiting time at the intermediate stages.

These three solutions are common, but cannot be applied indiscriminately. Once the potential for failure is recognized, a thorough investigation is required, including field studies, extensive sampling, testing, and analysis. As noted at the beginning of this section, a problem's solution may have marked effects on the entire project. An actual failure is usually highly undesirable, as well as expensive to correct.

Benching is steps cut into an existing sloping surface to discourage the imposed new embankment's tendency to slide down the slope. It may be required where embankments are placed against or on a sloping surface, such as a hillside or existing embankment slope. Cross-sections should be reviewed to determine the need for benching. They are constructed as shown on the standard sheet titled "Earthwork Transition and Benching Details." Contract documents and highway cross-sections should indicate this work, and sufficient quantities of earthwork estimated for this purpose as indicated on the standard sheet.

9.3.6 <u>**Embankment Foundation**</u>

9.3.6.1 Stripping (Topsoil Removal)

Topsoil and sod removal is normally required beneath low embankments (where the final pavement grade is 2 m or less above the existing ground surface), for some structure foundations, and on earth cut areas before general excavation when excavated material is to be used for embankment construction. Although topsoil and sod are considered unsuitable materials for embankment construction, they are valuable natural resources. Topsoil and sod excavated under an excavation item may be stockpiled for appropriate use on the project under Section 613 of the Standard Specifications. The Regional Geotechnical Engineer advises the designer and Regional Landscape Architect of the extent and estimated depths of topsoil removal (stripping) for each project. The quantities to be removed are shown on the highway cross-sections and included in the highway estimate. Where bridge structures are placed on fills less than 3 m high (measured from the original ground surface to the theoretical gradeline), topsoil and sod must be removed. Lateral limits of removal are defined by a rectangular or trapezoidal area bounded by lines extending 0.5 m horizontally from the outer footing limits, or to the toe of the fill, whichever is less. The Regional Geotechnical Engineer may require removal of topsoil under structures when the fills are higher than 3 m.

Under certain circumstances, it is desirable to leave topsoil and sod in place beneath embankments less than 2 m high, as when a thin layer of topsoil overlays fine-grained material in areas with a high water table. Removal of sod and topsoil could create an unstable condition in the underlying soil. The Regional Geotechnical Engineer identifies these areas, which the designer must clearly delineate on the plans.

9.3.6.2 Unsuitable Material

The term "unsuitable" as used here describes any material containing vegetable or organic matter (such as muck, peat, organic silt, topsoil, or sod) that is not satisfactory for use in embankment construction under §203-1.08 of the Standard Specifications. Certain man-made deposits of industrial waste, sludge, or landfill may also be found unsuitable, as well as soils contaminated with hazardous waste as determined by the Department of Environmental Conservation (NYSDEC).

As a general rule, when unsuitable material is encountered, it is removed and replaced with suitable material as shown on the standard sheet titled "Construction Details, Unsuitable Material, Excavation and Backfill." However, nature does not always adapt itself to a standard. The treatment method described by the standard sheet, although the most common, is not always the best for an unsuitable material deposit. Special treatment methods for organic deposits also include displacement and close-order excavation and backfill. The volume of unsuitable material to be removed and the required backfill must be determined during design. This information must be clearly presented with excavation and backfill payment lines in the contract documents. Reference should be made to the Standard Sheet entitled “Construction Details Unsuitable Material Excavation and Backfill.” Various methods are acceptable for presenting this information such as contours showing the bottom of removal, or a table indicating depth of removal at each section or at each exploration location.

Occasionally, construction has been necessary through or across landfills. By their nature, these deposits are variable and subject to extreme differential settlements. If this material is to be excavated, provisions should be made to ensure conformance with NYSDEC regulations regarding hazardous waste and to account for possible release of odorous gasses, spontaneous combustion, and pest control. Undercutting, dynamic compaction, and heavy rolling have been successful alternatives to complete removal. The deposit is undercut to provide a depth of at least 1.2 m between the final pavement surface and the landfill material. Heavy rolling of the landfill surface will usually accomplish some degree of uniformity by densifying its upper portion. Alternative technologies to heavy rolling exist, and should be evaluated by the Regional Geotechnical Engineer. This work is difficult, costly, and time-consuming, usually requiring services of specialty contractors. If possible, the best technique is to avoid areas known or suspected of containing such unsuitable material.

Any of these foundation treatments, including that outlined in the standard sheet just mentioned, require extensive exploration, analysis, and design by the Regional Geotechnical Engineer. The quantity of work must be clearly indicated on the plans.

9.3.6.3 Unstable Material

"Unstable" defines soils requiring special consideration because of their moisture content. They contain large portions of fine sand, silt, or clay, and are usually located at or below the groundwater level. These soils are suitable for embankment construction if their moisture content is reduced. An unstable foundation is most efficiently treated by raising the profile to avoid disturbing the wet, fine-grained material. If construction on unstable material cannot be avoided, special measures should be provided. Ditches may provide adequate drainage to stabilize the soil sufficiently. On advice of the Regional Geotechnical Engineer, temporary ditches may be required to expedite construction, and may or may not be incorporated into the final work.

On occasion, unstable soil conditions are encountered at the subgrade surface. In this situation the soil is usually fine-grained, inorganic, and saturated. It is incapable of supporting normal construction operations and would result in poor pavement performance if the pavement could be constructed. Along with the ground water level that results in the saturated soil condition, the instability may also be the result of liquefaction of the fine-grained soils by the action of vibrating compaction equipment.

Unstable material may have to be undercut to provide a stable foundation. This involves removing and replacing it with a more granular material less susceptible to the negative effects caused by water or excessive moisture content. In some cases, undercutting may be eliminated or reduced by using a geotextile or geogrid that allows water to pass through but retains the soil particles.

There are several methods for treating the unstable condition.

- Drain the area by means of ditches or underdrains to reduce the degree of saturation of the soil. Unfortunately, the terrain topography is frequently low and flat, precluding this option.

- Raise the profile to place the subgrade sufficiently above the unstable materials so that an adequate working platform can be constructed. Constraints such as design criteria and right of way may limit this option.

- Horizontal realignment to avoid the site of the unstable soil is even more subject to the constraints of design criteria and right of way, and is rarely viable.

- The most common treatment of an unstable subgrade is to undercut and replace some of the unstable material with granular material that will be stable in the wet environment. Undercuts of 0.6 m are common. Geotextiles are usually placed at the bottom of the undercut to act as a separator. This depth of removal and replacement (determined by trial and error) may be reduced substantially by the use of geogrids.

The Regional Geotechnical Engineer should be consulted for the appropriate method of treating an unstable material, with such measures incorporated into the plans.

9.3.7 **Geotechnical Design Guidance for Stormwater Management and Erosion and Sediment Control**

Soil erosion and sedimentation are natural results of surface water flow wearing away unprotected soil and then depositing it, usually at a lower elevation. See Chapter 8 for information the designer must consider in designing surface drainage facilities to minimize and control soil erosion and sedimentation both during construction, with the implementation of erosion and sediment control items, and on the completed project, with the construction of stormwater management practices. Refer to the Regional Landscape Environmental Manager for information relating to appropriate management practices, including landscape design and ground cover as erosion control measures.

To comply with the SPDES General Permit for Stormwater Discharges from Construction Activity GP-02-01, the Department will develop Stormwater Pollution Prevention Plans (SWPPPs) for all applicable projects in accordance with the New York State Stormwater Management Design Manual[(8d)]. All SWPPPs will contain Erosion and Sediment Control (E&SC) Plans in accordance with the New York State Standards and Specifications for Erosion and Sediment Control[(8c)].

9.3.7.1 Erosion and Sedimentation Control

Construction activities involving soil disturbances require erosion and sediment control measures. Section 209 of the Standard Specifications[(2)] provides specification provisions. See Chapter 4 (Section 4.3) of the Environmental Procedures Manual for information on procedures for determining the need for Erosion and Sediment Control Plans and coverage under stormwater general permits.

9.3.7.1.1 *Stormwater Diversion*

The designer should determine how runoff will drain from the site. Natural drainage channels should not be altered or relocated without proper approvals. The design of the E&SC plan shall emphasize erosion control as the main component and sediment control subsequently. Diversion of surface water away from exposed soils provides the most economic and effective erosion control possible since it is more advantageous to control erosion at the source than to design controls to trap suspended sediment. The reduction of soil loss decreases the cost and maintenance of sediment control practices, reduces the risk of degrading natural resources and improves the overall appearance of the construction site.

9.3.7.1.2 *Project Segmenting*

The Contractor is required to perform all construction operations using any and all methods at his/her disposal to minimize erosion and ensure sedimentation control on the project. Various E&SC practices are outlined in the New York State Standards and Specifications for Erosion and Sediment Control[8c]. As stated in the manual, the best way to protect the soil surface and limit erosion is to preserve the existing vegetative groundcover. For large-scale earthwork projects adjacent to sensitive waterbodies, an additional technique available is called *Project Segmenting*. Project segmenting breaks-up the extent of the projects exposed surface into "segments" thereby minimizing disturbance to the existing, extensive vegetative groundcover and limiting potential erosion to more controllable areas. Project segmenting is a restrictive process and, if chosen, should be analyzed in conjunction with the projects overall proposed schedule.
These segments, broken up into identified disturbed areas, are shown on the drawings and individually identified as one of the three following types:

- Balanced: In general, the earthwork needs of these segments are balanced within the segments themselves.
- Spoil Location Segment (Embankment Segment): These segments require additional embankment to be brought in from other locations.
- Borrow Source Segment (Excavation Segment): These segments provide excess material to be used for the construction of embankment locations. These areas are considered sources of borrow for embankment segments.

If this technique is chosen, the designer will develop an Earthwork Staging Diagram identifying each construction stage and a sequence of stabilized earthwork construction operations. The Contractor will be limited to the construction of only two (2) segments at any one time. The two segments may be a combination of two of the three types indicated above. At no time shall two of the same type of segments be allowed to be constructed at the same time. The Table of Earthwork Staging will provide a listing of the "build segments" within a construction stage. Build segments are a listing of segments (Balanced, Embankment or Excavation) by stationing, appropriate for the construction stage.

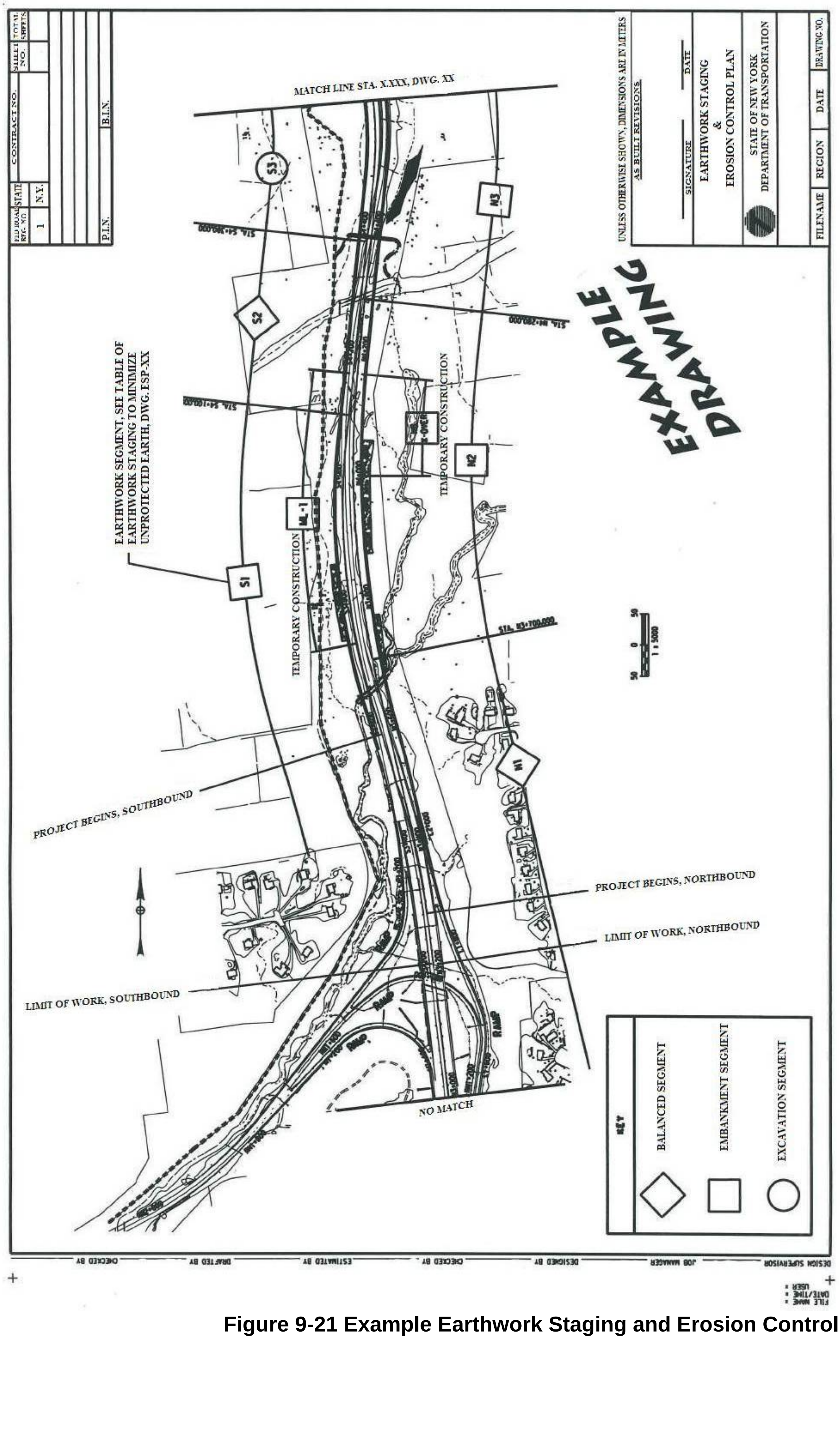


Figure 9-21 Example Earthwork Staging and Erosion Control Plan

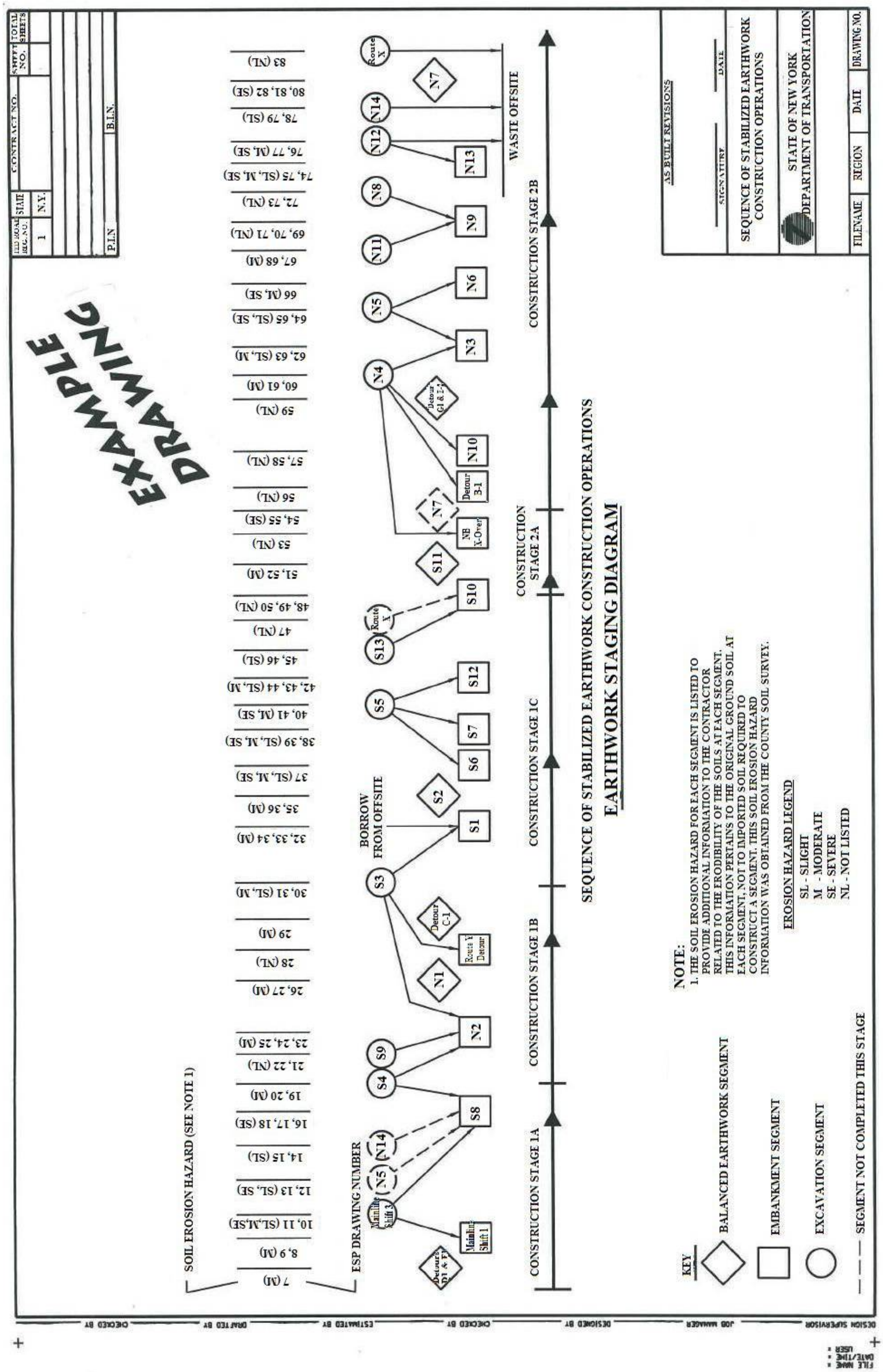


Figure 9-22 Example Sequence of Stabilized Earthwork Construction Operations

Table 9-1 Example Table of Earthwork Staging To Minimize Unprotected Earth

Table of Earthwork Staging To Minimize Unprotected Earth								
Construction Stage	**Build Segment**	**Disturbed Area (m^2)**	**Stations**	**Borrow or Waste**	**Borrow Source Segment**	**Borrow Source Disturbed Area (m^2)**	**Waste Location Segment**	**Waste Source Disturbed Area (m^2)**
1A	Detours D-1 & F-1	1500		Balanced				
	Mainline Shift 1	3650	S3+720 to S4+140	Borrow	Mainline Shift 3	6800		
	Mainline Shift 3	6800	N7+380 to N7+930	Waste			Mainline Shift 1	3650
	S8	25300	S5+880 to S6+280 and Ramp *A*	Borrow	N5	5750		
					N14	4950		
					S4	20000		
	S4	20000	S4+700 to S5+140	Waste			S8	25287
							N2	21000
1B	S9	5700	S6+280 to S6+600	Waste			N2	21000
	N2	21000	N3+700 to N4+280	Borrow	S4	20000		
					S9	5700		
					S3	14050		
	N1	6850	N3+280 to N3+700	Balanced				
	Route *X* Detour	1150		Borrow	S3	14050		
	Detour C-1	1150		Balanced				
1C	S3	14050	S4+380 to S4+700	Waste			N2	21000
							Route *X* Detour	1150
							S1	17650
	S1	17650	S3+580 to S4+100	Borrow	S3	14050		
					Offsite			
	S2	7950	S4+100 to S4+380	Balanced				
	S6	12000	S5+380 to S5+640	Borrow	S5	11750		
	S7	7250	S5+640 to S5+880	Borrow	S5	11750		
	S5	11750	S5+140 to S5+380	Waste			S6	12000
							S7	7250
							S12	13150
	S12	13150	N7+490 to N7+800 and Ramp *F*	Borrow	S5	11750		
	S13	3500	N7+800 to N8+200	Waste			S10	10700
	S10	10700	S6+600 to N7+340 and Ramps *C & D*	Borrow	S13	3500		
					Route *Y*	1700		
2A	S11	3800	N7+340 to N6+100	Balanced				
	Northbound Crossover	480		Borrow	N4	16850		
	N7	2900	N5+900 to N6+100	Balanced				
2B (continues)	Detour B-1	230		Borrow	N4	16850		
	N10	6450	N6+960 to N7+230	Borrow	N4	16850		
	Detours G-1 & E-2	4000		Balanced				

The Contractor will not be allowed to begin earthwork operations (including clearing and grubbing) of a new segment until the current working segments are completed.

A segment will be considered complete based on the following criteria:

a. Embankment Segments:
   i. Embankment constructed to subgrade.
   ii. All slopes are trimmed, topsoiled, seeded, mulched, and erosion control blankets (when required) are installed.
   iii. Subbase course installed. (Note: Drainage structures are to be installed to final roadway grade. Runoff across the subbase course is to sheet flow to slopes, or is to be collected in temporary swales and diverted to sediment control structures).

b. Excavation Segments:
   i. Excavated slopes are to be constructed to permanent payment lines from top of slope, down. (Note: It is not required that excavation segments be constructed to subgrade if the segment is not considered a "build segment" in the earthwork staging.)
   ii. All slopes are trimmed, topsoiled, seeded, mulched, and erosion control blankets (when required) are installed.
   iii. Top surface (non-slope) to be graded to drain to temporary swales and runoff diverted to sediment control structures.

c. Balanced Segments:
   i. All disturbed areas are trimmed, topsoiled, seeded, mulched, and erosion control blankets (when required) are installed.
   ii. Excavation and/or embankments within the balanced segment are completed as stated in a. Embankment Segment and/or b. Excavation Segment as appropriate.

9.3.7.1.3 *On-Site Material*

The erosion hazard for the soils within each project segment should be identified. An excellent resource available to categorize the erosion hazard is the Soil Survey produced by the Natural Resources Conservation Service (NRCS). The erosion hazard, as defined by the NRCS, is the probability that erosion can occur as a result of site preparation or cutting where the soil is exposed along roads. The erosion hazard ratings are based on the percent of the slope and are available from the descriptions of soil types as defined by the NRCS:

- *Slight:*
  A rating of slight indicates that no particular prevention measures are needed under ordinary conditions.
- *Moderate:*
  A rating of moderate indicates that erosion control measures are needed in certain activities.
- *Severe:*
  A rating of severe indicates that special precautions are needed to control erosion in most activities.

The erosion hazard and type of soil are significant elements in influencing the turbidity of construction runoff. The SPDES General Permit states that there shall be no increase in turbidity that will cause a substantial visible contrast to natural conditions; there shall be no increase in suspended, colloidal and settleable solids that will cause deposition or impair the waters for their best usages and; there shall be no residue from oil and floating substances, nor visible oil film, nor globules of grease. Turbidity, defined as the measure of clarity of an otherwise clear liquid by using colorimetric scales, is noted as the cloudy or hazy appearance of a waterway caused by a suspension of colloidal particles and/or fine soil/ solid particles. This limit on turbidity has been set because high turbidity may interfere with disinfectant efficiency, may stimulate growth of microorganisms and may also adversely affect fish. The result of sediment settling out into the streambed often suffocates eggs and leaves the streambed unsuitable for future egg incubation. In addition, turbidity reduces the food supply for fish and their ability to find food. The suspended particles can also clog or damage fish gills, directly killing or harming fish.

The rate at which eroded soil particles settle out of solution as sediment is largely determined by the size of the particle. Typical soil particle sizes are provided in the Soil Test Procedure (STP-2) manual published by the Geotechnical Engineering Bureau.

Table 9-2(8g) Settling Velocities Of Soil Particles In Still Water

Diameter of Particle (mm)	Order of Size	Settling Velocity (mm/sec)	Time Required to Settle One Meter (3.28 Ft)
10.0	Gravel	1.000	1.0 Seconds
1.0		100	9.8 Seconds
0.6	Coarse Sand	63	15.0 Seconds
0.3		32	30.0 seconds
0.15	Fine Sand	15	67.0 Seconds
0.015		0.35	47.6 Minutes
0.010	Silt	0.154	107.0 Minutes
0.003		0.0138	20.1 Hours
0.0015	Clay	0.0035	79.0 Hours
0.001		0.00154	180.0 Hours
0.0001		0.0000154	754.0 Days
0.00001	Colloidal Particles	0.000000154	207.0 Years
NOTE: Temperature 50°C; all particles assumed to have a specific gravity of 2.65.			

Table 9-2(8g) is a chart presenting settling rates for various particle sizes. Larger particles, which typically weigh more, will settle out faster than a smaller particle. Extremely small particles form colloidal suspensions. A colloidal suspension is defined as an intimate mixture of two substances, one of which, called the dispersed phase (colloid or in this case the eroded soil particle), is uniformly distributed in a finely divided state through the second substance, called the dispersion medium (in this case the waterway). As noted in the chart, colloidal particles will remain in suspension for years or centuries.

Discussion of the soil types and erosion hazard during the design phase may lead to revisions to the E&SC plan not only due to appropriateness but practicality. It is important for designers to identify the erosion hazard rating for a particular segment on the erosion and sediment control plans to provide field personnel with the appropriate information to promptly analyze any Contractors modification to the earthwork sequencing. By knowing the potential duration of the Contractors operation, impending weather conditions and the erosion hazard and type of soil in the segment in question, the Engineer will be able to direct the Contractor appropriately.

9.3.7.1.4 *Material Sequencing*

Concerning off-site activities, NYSDOT contractors routinely use sites outside of the project limits to conduct operations relative to the highway project. Surplus material (spoil) disposal areas, borrow areas, equipment staging areas, and temporary concrete batch plant operations established for the duration of a construction project and having no proposed permanent feature shall be considered temporary construction activities and will require only temporary controls during construction, provided they are stabilized at the end of construction and that they do not exacerbate fluctuations in water quantity leaving the site. The landowners of these sites will be the “Operators” as defined by the SPDES General Permit, and will be required to obtain permit coverage. NYSDOT will notify contractors of the need for permit coverage in such areas. NYSDOT will not include sites outside of the state right-of-way (except for easements, occupancies, and releases) in its permit coverage, but may include sites outside of the contract limits if the site is within the state right-of-way.

Although the overall project may be a spoil or borrow job, each construction stage, including individual segments, should be analyzed to determine how the material from each segment will be utilized and if there is a need to identify additional spoil or borrow areas into the contract. As an example, a project may have large cuts (rock cuts) and large fills and the job may balance out. However, there are limitations to some construction operations such as material top size in the subgrade area. If the Contractor is pulling from a rock cut at the time he/she is to finish an embankment section, will he/she will be directed to borrow appropriate sized material or crush the material on-site (an additional operation not expected in an overall balance job). If the segments are not analyzed, these decisions will have to be made in the field which will result in extra material costs, filing of additional permits, and an overall delay claim.

9.3.7.2 Post Construction Stormwater Management

Stormwater management practices are installed to minimize and control soil erosion and sedimentation on completed projects. See Chapter 8 of this manual for information regarding the design and implementation of acceptable permanent stormwater management practices and the development of Stormwater Pollution Prevention Plans (SWPPPs).

9.3.7.2.1 *Sizing of Stormwater Management Practices*

To compute water quantities to develop the size of a stormwater management practice, the New York State Stormwater Management Design Manual requires the use of the NRCS Technical Release TR55 “Urban Hydrology for Small Watersheds”.

The designer shall review the appropriate Soil Survey of the watershed area to determine the category(s), or soil description(s), for the terrain to identify the appropriate Hydrologic Soil Group (A-D). The Hydrologic Soil Group, as defined by the NRCS, refers to soils grouped according to their runoff-producing characteristics. The chief consideration is the inherent capacity of soil bare of vegetation to permit infiltration. The slope and kind of plant cover are not considered but are separate factors in predicting runoff. Soils are assigned to four groups:

- *Group A:*
  Group A soils have a low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sand or gravel and have a high rate of water transmission (greater than 0.30 in/hr).
- *Group B:*
  Group B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (0.15-0.30 in/hr).
- *Group C:*
  Group C soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (0.05-0.15 in/hr).
- *Group D:*
  Group D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soil with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0-0.05 in/hr).

If there is a question or concern expressed by the designer, the geotechnical engineer shall review the site. The geotechnical engineer’s site investigation, in addition to the above, should include an analysis of the surrounding soils to be compared with their research of the Soil Surveys. Their analysis may be based on visual observation of the soils within the watershed area or, if available, a review of any historical or recently progressed subsurface explorations. The geotechnical engineer will notify the designer of the appropriate Hydrologic Soil Group to be used in design of the features for the SWPPP.

9.3.7.2.2 *Design of Stormwater Management Practices*

An associated design criterion for stormwater management practices is the analysis of the subsurface conditions. The New York Standards and Specifications for Erosion and Sediment Control recommends that the design of a sediment basin shall be designed and constructed to conform to NRCS Standard And Specification No. 378 for Ponds in the National Handbook of Conservation Practices and the NYSDEC, "Guidelines for the Design of Dams".

The NRCS Standard And Specification No. 378 for Ponds states under Geological Investigations that "pits, trenches, borings, review of existing data or other suitable means of investigation shall be conducted to characterize materials within the embankment foundation, auxiliary spillway and borrow areas". The NYSDEC Guidelines for the Design of Dams states under Geological Investigations: Foundations, "subsurface explorations (drill holes, test pits and/or auger holes) should be located along the centerline of the dam, at the proposed service and auxiliary spillway locations, and in other critical areas". It continues to describe recommendations concerning the depth of the borings. In addition, "for even the smallest low hazard dams, at least three explorations should be made along the centerline of the dam, one in the deepest part of the depression across which the dam will be built and one on each side".

In addition to sediment basins and/or permanent pools, the New York Stormwater Management Design Manual recommends for the design of stormwater infiltration practices that "to be suitable for infiltration, underlying soils shall have an infiltration rate (f_c) of at least 0.5 inches per hour, as initially determined from NRCS soil textural classification, and subsequently confirmed by field geotechnical tests (outlined in an Appendix). The minimum geotechnical testing is one test hole per 5000 sf, with a minimum of two borings per facility (taken within the proposed limits of the facility). Soils shall also have a clay content of less than 20% and silt/clay content of less than 40%".

If infiltration practices are chosen, the designer will need an understanding of the groundwater elevation or the presence of a protected aquifer and the soil gradation below the bottom of the feature or the presence of bedrock at the proposed site. In addition to these, if there is knowledge of springs in the area, it is important that this information be addressed in design as it will increase the overall water treatment volume. The designer and geotechnical engineer shall discuss the proposed stormwater treatment practices and the appropriateness, location and quantity of subsurface explorations.

One essential element in the analysis of the subsurface conditions to be defined for these practices is permeability. Permeability is the capacity of a porous rock, soil, or sediment for transmitting a fluid without damage to the structure of the medium (also known as conductivity; perviousness). An additional term used to describe movement of water through the soil surface into the ground is infiltration.

The Pond Conservation Practice Standard states under Geological Investigations that "seepage control is to be included if (1) pervious layers are not intercepted by the cutoff, (2) seepage could create swamping downstream, (3) such control is needed to insure a stable embankment, or (4) special problem require drainage for a stable dam". The Guidelines for the Design of Dams states under Geological Investigations: Foundations, "whenever feasible, seepage under the dam should be controlled by means of a complete cutoff trench extending through all pervious foundation soils into a relatively impervious soil layer". It continues to describe recommendations concerning the cutoff or key trench.

Table 9-3[8h] Permeability and Drainage Characteristics of Soils

	Coefficient of Permeability k in cm per sec (log scale)											
	10^{2}	10^{1}	1.0	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}
Drainage				Good				Poor	Practically Impervious			
Soil types	Clean gravel		Clean sands, clean sand, and gravel mixtures			Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits, etc.				"Impervious" soils, e.g., homogeneous clays below zone of weathering		
					"Impervious" soils modified by effects of vegetation and weathering							
Direct determination of k	Direct testing of soil in its original position-pumping tests. Reliable if properly conducted. Considerable experience required											
	Constant-head permeameter. Little experience required											
Indirect determination of k			Falling-head permeameter. Reliable. Little experience required			Falling-head permeameter. Unreliable. Much experience required			Falling head permeameter. Fairly reliable. Considerable experience necessary			
	Computation from grain-size distribution, i.e., Hazen's formula. Applicable only to clean cohesionless sands and gravels									Computation based on results of consolidation tests. Reliable. Considerable experience required		

SOURCE: After Casagrande and Fadum (1940).

Table 9-4[8h] Coefficients of Permeability of Common Natural Soil Formations

Formation	k (cm/sec)
River deposits	
Rhone at Genissiat	up to 0.4
Small streams, eastern Alps	0.02–0.16
Missouri	0.02–0.20
Mississippi	0.02–0.12
Glacial deposits	
Outwash plains	0.05–2.00
Esker, Westfield, Mass.	0.01–0.13
Delta, Chicopee, Mass.	10^{-4}–0.015
Till	$< 10^{-4}$
Wind deposits	
Dune sand	0.1 –0.3
Silt (loess)	0.001 ±
Silt (loess loam)	10^{-4} ±
Lacustrine and marine offshore deposits	
Very fine uniform sand, $C_u = 5$ to 2	10^{-4}–0.0064
Silt (Bull's liver), Sixth Ave., N.Y.C., $C_u = 5$ to 2	10^{-4}–0.0050
Silt (Bull's liver), Brooklyn, $C_u = 5$	10^{-5}–10^{-4}
Clay	$< 10^{-7}$

SOURCE: Modified from Terzaghi and Peck (1967).

If subsurface explorations are progressed, the following additional test procedures are available to provide the designer with an understanding of the coefficient of permeability and/or infiltration rate at the site:

- Specific Surface Analysis: The Geotechnical Engineering Bureau has a test procedure to determine the specific surface of soil solids from grain size distribution data, GTP-5 Test Procedure for Specific Surface Analysis. The specific surface of solids, along with the porosity, may be used in determining the saturated permeability of a cohesionless granular material. The soil from the jar samples (of appropriate elevations) obtained from the subsurface exploration is tested. The test method involves (1) performing a grain size analysis of the soil specimen, (2) examining the shape characteristics of the grains contained in each sieve interval, and (3) an arithmetic or computer calculation of the specific surface.

  Limitations: There is a limitation on the test procedure. If the soil sample contains more than 5% passing the 75 µm sieve, the specific surface analysis is not performed. In addition, even though the value for the specimen may be correct, the results of the tests may differ from the actual field permeability for reasons explained in the appendix of GTP-5.

- Drill Hole Infiltration Test: Conduct a drill hole infiltration test in accordance with Appendix D of the NYS Stormwater Management Design Manual. Additional information on drill hole permeability tests are described in Soil Mechanics DM 7.1. The factors affecting the performance and applicability of the drill hole permeability test include; (1) position of the water level, (2) type of material (rock or soil), (3) depth of the test zone, (4) permeability of the test zone, and (5) heterogeneity and anisotropy of the test zone. To account for these factors, it is necessary to isolate the test zone.

  The determination of the test zone is critical to the design of the stormwater treatment feature. Typically, to identify the subsurface strata, two subsurface explorations would be progressed within 1.5 m of each other. The first subsurface exploration would identify the soil strata by progressing the hole and obtaining continuous samples. The hole should be progressed to a sufficient depth to locate and determine the extent and properties of all soil and rock strata that could affect the performance of the feature. Typically, the minimum depth of the exploration should be 3 m or equal to the proposed height of the features berm, whichever is greater. However, if the feature is proposed to be excavated, the depth of the exploration should be extended to identify the possibility of exposing pervious foundation layers. The initial subsurface exploration should then be compared to the proposed stormwater feature design and the test zone identified.

  The second subsurface exploration would be progressed to perform the drill hole permeability test. The hole would be progressed to a sufficient depth to isolate the test zone. Methods to isolate the test zone are also provided in the Soil Mechanics DM 7.1.

  Limitations: Attempt to conduct the drill hole permeability test in clement weather. If there is heavy rain, snow, ice or freezing rain, the test will be re-scheduled. The subsurface explorations will be progressed with a drill rig. The designer should contact the Regional Geotechnical Engineer to determine the scheduling limitations. As with all subsurface explorations, the locations are to be identified and cleared as per 16 NYCRR Part 753 Protection of Underground Facilities.

If subsurface explorations are not proposed and/or the following equipment is obtainable, a more common way of determining the coefficient of permeability at the site is a percolation test:

- Percolation Test: Percolation is defined as the gravity flow of groundwater through the pore spaces in rock or soil. A percolation test is typically used as a test to determine the suitability of a soil for the installation of a domestic sewage-disposal system, in which a hole is dug and filled with water and the rate of water-level decline is measured. The Department utilizes percolation data to determine infiltration rates. To provide national consistency, the NRCS Soil Surveys evaluated percolation rates of about 900 soils to define permeability classes (Uhland and O'Neal (1951)). As the surveys evolved, the NRCS transitioned from permeability to the term saturated hydraulic conductivity. The NRCS recommended that the term "saturated hydraulic conductivity be used for data expressed as a velocity and obtained by analysis using Darcy's Law on saturated cores".

  The percolation test involves, as described in Soil Mechanics DM 7.1, digging a hole (generally 0.6 m square), or drilled (100 mm min.) to a depth of the proposed absorption trench, cleaned of loose debris, filled with coarse sand or fine gravel over the bottom 50 mm, and saturated for a specified time. To eliminate any smearing during the excavation, the sidewalls should be scratched or scarified to provide open, natural soil which water may percolate. The percolation rate measurement is obtained by filling the hole to a prescribed level (usually 150 mm) and then measuring the drop over a set time limit (usually 30 minutes). In sandy soils, the time limit may be only 10 minutes.

  > *Limitations:* Attempt to conduct the percolation test in clement weather. If there is heavy rain, snow, ice or freezing rain, the test will be re-scheduled. Typically, depending on the depth, the absorption trenches are dug with a backhoe. The scheduling to obtain the equipment and hauling to the site should be a consideration. The depth of the absorption trench needs to be evaluated to determine it's safe and practical based on site constraints. Typically, a limit on the depth of an absorption trench is 1.5 m (1.2 m test pit with a 0.3 m infiltration test hole). As with all subsurface explorations, the locations are to be identified and cleared as per 16 NYCRR Part 753 Protection of Underground Facilities.

The designer's communication with the geotechnical engineer shall include discussion on the design approach for the stormwater management practices and the factors of safety in their design. Also, the geotechnical engineer shall point out to the designer the limited extent of the information obtained (breadth of visual review, size of a subsurface exploration or Soil Survey test pit vs. extent of the watershed) and the potential for encountering an undefined soil deposit or identifying a misrepresented extent of a soil deposit assumed in design within a stormwater management practice. Considering this and the particular design practice, the geotechnical engineer and designer should discuss the need to implement construction controls that can be interpreted so action can be taken to modify the design during construction if necessary.

9.3.8 **Underdrains and Edge Drains**

Underdrains and edge drains are perforated, slotted or porous pipes, installed in trenches and surrounded by underdrain filter material that is both pervious to water and capable of protecting the pipe from infiltration of the surrounding soil. *Underdrains* are used to lower the groundwater level, drain slopes, help stabilize cut slopes, and prevent water from entering the pavement section from below. *Edge drains* remove water from the pavement section.

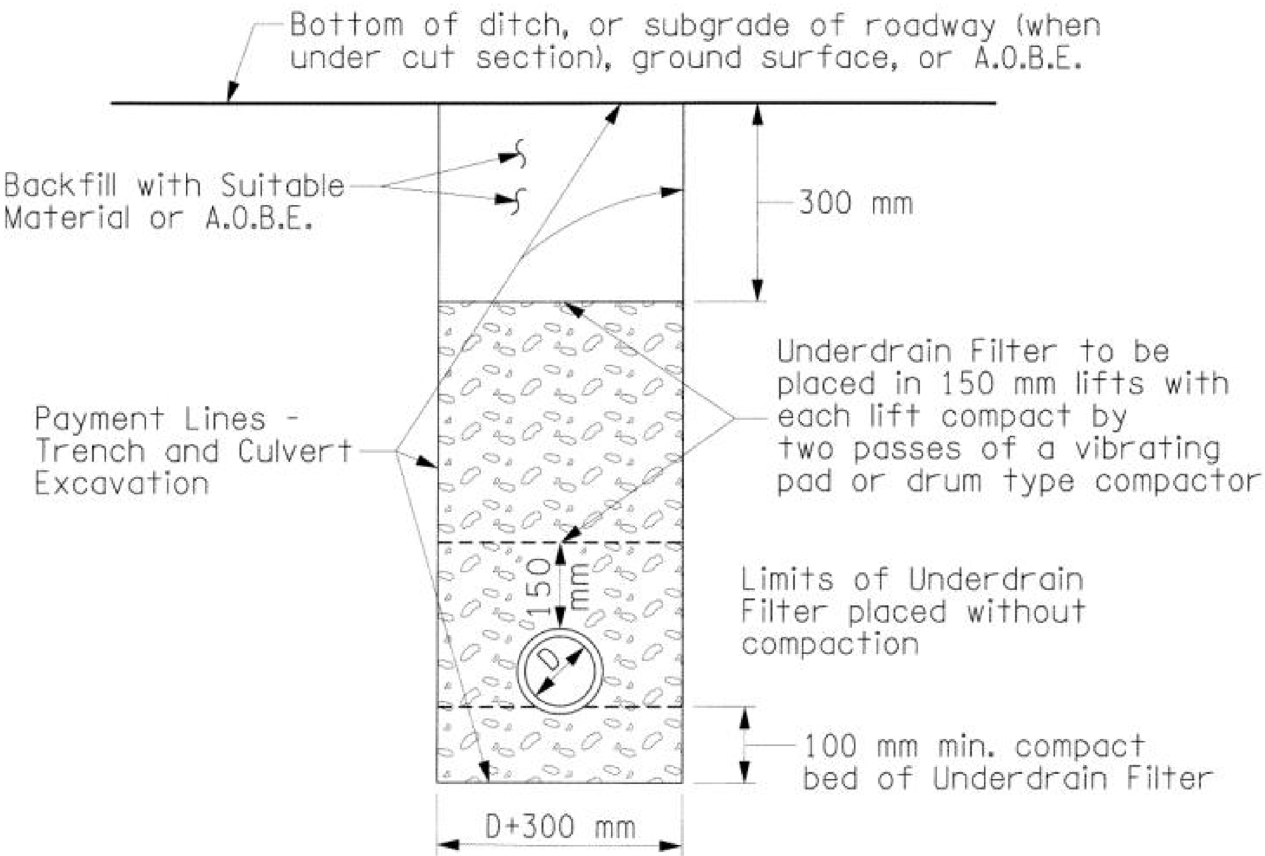
Underdrains and edgedrains remove water by means of gravity flow. To function, they thus must be continuously sloped to an outlet, such as a drainage channel or a closed drainage system, to ensure that no intercepted water is allowed to accumulate and that any transported sediments do not settle out and block low points.

Underdrains

Underdrain efficiency depends on the soil's permeability and the depth to which the groundwater level must be lowered. The Regional Geotechnical Engineer must be consulted if it is proposed to lower the groundwater level by means of underdrains.

Locations along the highway where future concentrations of water may require underdrains are sometimes difficult to predict during design, but some obvious locations for underdrains are areas of existing springs where a new highway is to be located, or where the pavement is located at the base of a side-hill cut, or on very long downhill grades where flow from infiltrated runoff and seepage zones tend to follow the direction of the pavement. For design purposes, it is recommended that quantities be estimated to provide for such underdrains. The Regional Geotechnical Engineer should be consulted to locate underdrains during design. Because underdrains are relatively expensive, good engineering requires discriminate application.

Ideally, the invert of the underdrain should be at least 1.2 m lower than the nearest edge of pavement. In areas where this can not be accomplished, considerations should be given to filling the ditch with stone which may then be surfaced with a geotextile and soil or topsoil. Installation details for underdrains are shown in HDM Figure 9-23 [formerly shown on the Standard Sheets titled “Installation Details for Circular and Elliptical Reinforced Concrete and Other Rigid Pipes” superseded by EB03-010, and “Installation Details for Corrugated Metal and Structural Plate Pipe and Pipe-Arches”].



Note: Consult with the Regional Geotechnical Engineer to determine subgrade soil type, effectiveness of underdrains and required depth to pipe invert.

Figure 9-23 Pipe Underdrain Bedding and Backfill Details for Trench Installations

Edge drains

Edge drain design for NYSDOT pavements extends 300mm into the subgrade, functioning as an underdrain too, so underdrain items are specified for edge drains.

Edge drain details depend on the highway geometry (e.g. superelevated, sag, etc.) as well as pavement section (e.g. curbing, permeable base, etc). For an edge drain to provide subsurface drainage of the pavement section, it must intercept the water-bearing layer as near as possible to the point of major concern. For instance, damage to conventional pavements because of subsurface water is usually most severe at the edge of traveled way. The edge drain thus should normally be placed at the pavement's vertical interface with the shoulder or curb (Figure 9-24).

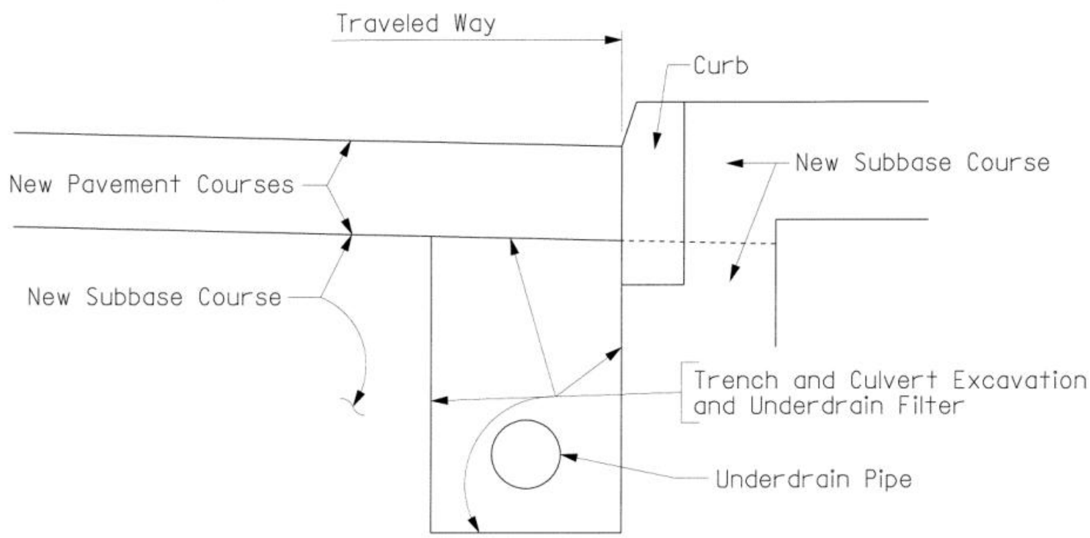


Figure 9-24 Edge-of-Pavement Drain: Curbed Section

In addition, the edge drain should intercept water from the highest water-bearing layer of the pavement section. In flexible pavements, this water is usually encountered in the asphalt base course. In rigid pavements, it is usually found between the concrete and subbase course. Typical edge drain installations for rehabilitation and restoration of conventional pavements are shown in the Comprehensive Pavement Design Manual (CPDM) Figures 7-6 & 7.

In pavement sections that include permeable base, the edge drain should be in contact with the permeable base at the low points of its cross-section. That would include the lower sides of superelevated sections and both sides of normal sections. CPDM Figures 7-1, 2 & 3 show edge drain details for full-depth PCC over permeable base. CPDM Figures 7-4 & 5 show edge drain details for full-depth HMA with permeable base.

Edge drains must be provided with lateral outlets to the roadway ditch or to appropriate structures in a closed storm-drain system. In practice, edge drains are normally placed in a trench dug after subbase construction. This method requires removing subbase and subgrade material, but it is used for ease of construction because it achieves uniform compaction of the roadway section, and adequately confines the underdrain filter material. For edge drain outlet installation details see CPDM Appendix 9A.

These figures are not standard and may be modified, but the criteria described here should be considered.

Preformed edge drains are also available, usually consisting of a ribbon of corrugated or dimpled plastic sheathed in an underdrain geotextile. The ribbons, referred to as Prefabricated Geocomposite Edge Drains (PGED) may be 0.6 to 1.0 m wide or wider. PGED are installed by a machine that excavates a slit-trench, places the edge drain, and backfills and compacts the trench in one pass. PGED are particularly advantageous where an open trench is not wanted, and have also been used to provide drainage behind walls. Figures 9-25a through 9-25d show installation details for PGED. Complete guidance for PGED installation and use is contained in CPDM subsection 9.4.2.

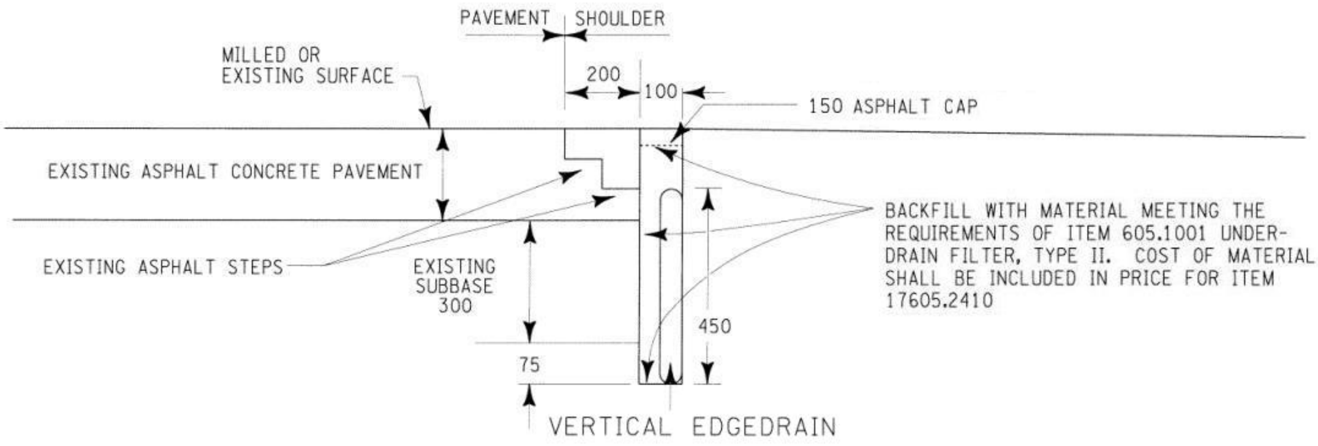


Figure 9-25a PGED with Stone Backfill Detail: Adjacent to Full Depth Asphalt Pavement

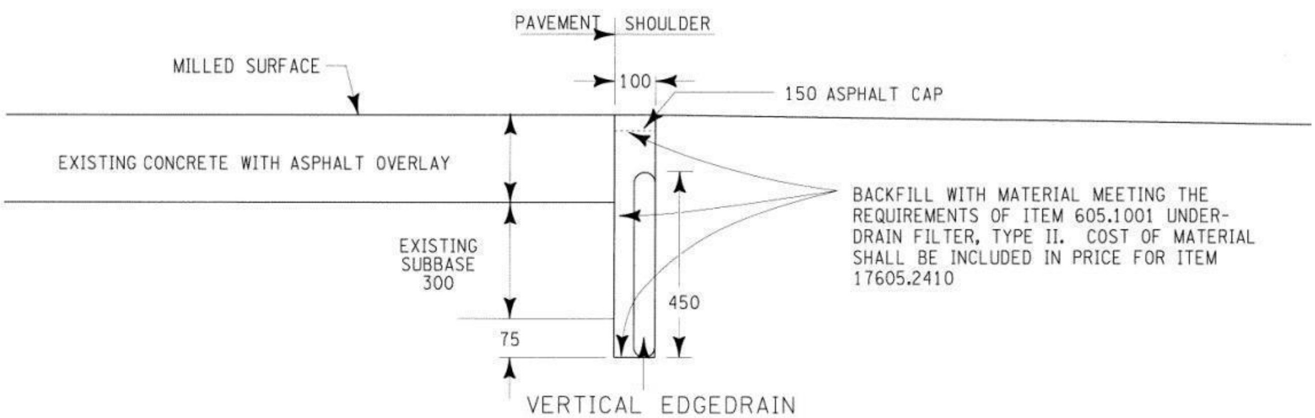


Figure 9-25b PGED with Stone Backfill Detail: Adjacent to Concrete Pavement

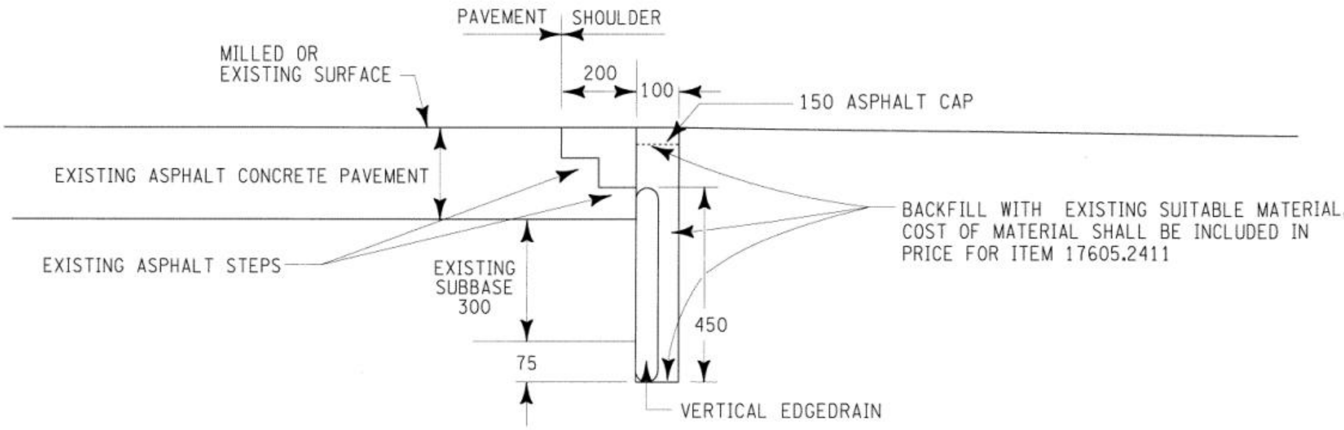


Figure 9-25c PGED with Existing Suitable Material Detail
Adjacent to Asphalt Pavement

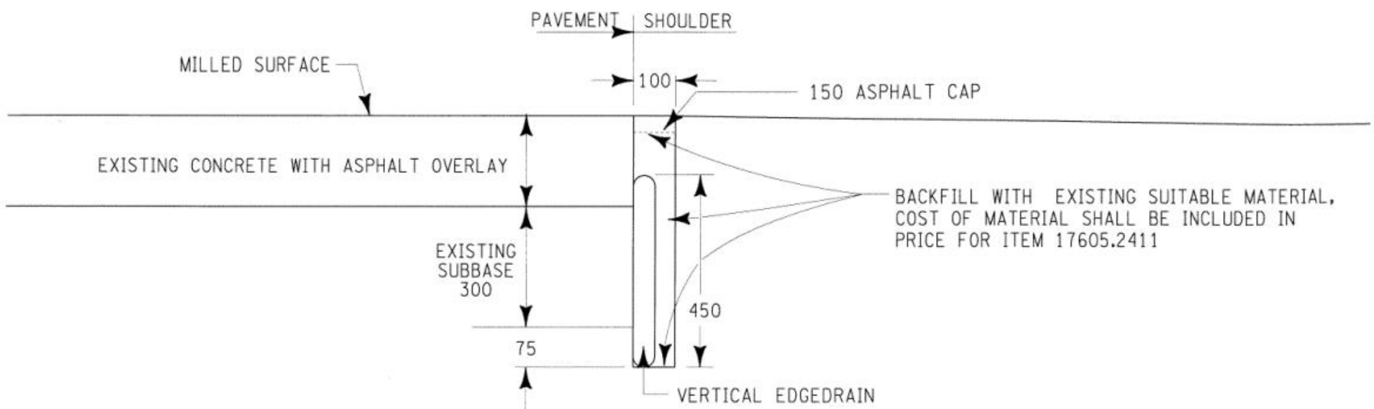


Figure 9-25d PGED with Existing Suitable Material Detail:
Adjacent to Concrete Pavement

Figure 9-25e PGED: Sample of Product

Figure 9-25f PGED: Drain Unrolled

Figure 9-25g PGED: Drain Installed Through Guides

Figure 9-25h PGED: Drain Installed (Outer side of Trench)

Figure 9-25i PGED: Granular Backfill Material (Pavement/Inner side of trench)

Figure 9-25j PGED: Capping Operation

As always, appropriate methods of pavement drainage should be discussed with the Regional Geotechnical Engineer.

9.3.9 **Cut Slopes**

Cut-slope problems are perhaps the most troublesome of major geotechnical difficulties encountered in highway construction, and the hardest to anticipate during design. There are two failure types: shallow-seated and deep-seated.

9.3.9.1 Soil Slopes

Shallow failures show up as sloughing or erosion of surface material on the cut slope.

Figure 9-26 Large Scale Surface Slough

Solutions to earth-cut problems may require no more than regrading and reseeding, or in extraordinary circumstances applying a slope-protection blanket of granular material or a slope drain (see the standard sheet titled "Installation Details for Granular Fill -- Slope Protection"). However, one difficulty is predicting location of the failure. Normal methods of subsurface exploration do not usually indicate a potential shallow cut-slope problem. Accuracy of prediction depends largely on the experience of the Regional Geotechnical Engineer. It is thus essential that the designer consult the Regional Geotechnical Engineer to determine if this problem may be a critical design factor. For cut slopes where need for slope protection is obvious during design, based on subsurface investigation or observation of existing conditions, excavation required for placement of the stone blanket can be performed during normal excavation operations. However, in most cases actual need for the slope-protection blanket does not become apparent until after finishing normal excavation. In that case, excavation for the blanket is more specialized.

Underdrains are also a means of stabilizing a cut slope. If a cut is made through a perched water table, an underdrain pipe may be installed to intercept perched groundwater before it causes damage to the cut slope. Design of a pipe-drain slope protection installation requires extensive subsurface exploration to determine location of the perched water table. Economics generally dictates that a slope-protection blanket be used instead of a pipe underdrain, where the perched water table is near the highway grade or the impervious stratum is discontinuous or poorly defined. Installation is described on the standard sheet titled "Installation Details for Granular Fill -- Slope Protection."

"Deep-seated" soil-cut failures often occur along circular arcs, similar to the embankment foundation shear failures previously described (Section 9.3.5 "Stability"). In effect, the cut reduces the natural resisting force, causing a weak stratum to fail in shear. A common solution for a deep failure is to make the slope flatter than a 1 vertical on 2 horizontal slope. Slope flattening has the effect of reducing the driving force of the material at the top of the slope. Another solution is to use a toe berm, which increases the resisting force, as shown in Figure 9-19. These or other solutions to cut-slope problems will probably have significant influence on the project, and it, thus, is essential that they be addressed early in the design process.

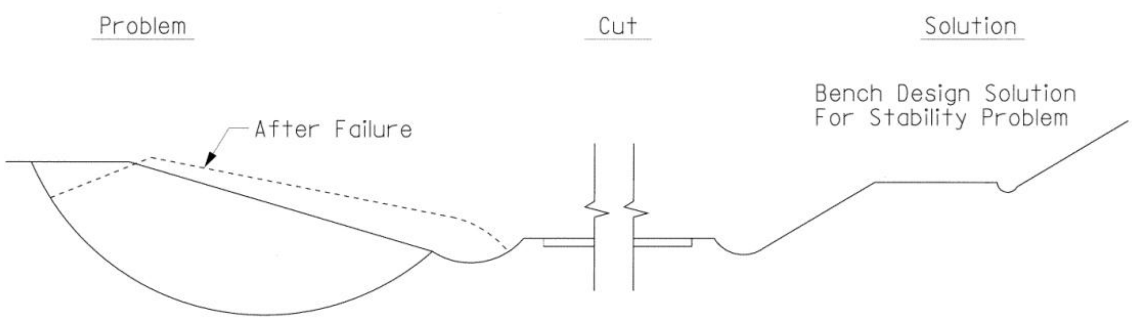


Figure 9-27 Cut-Slope Failure and Benching Solutions

9.3.9.2 Rock Slopes

Rock slopes are also subject to failures. The rock in New York State is highly variable. Some failures may be caused by weathering when rock breaks down into small pieces that accumulate as talus at the toe of the slope. This talus may clog ditches and cause flooding of the roadway. Rock-slope failures may also take the form of rock slides or rock falls, where many larger blocks, or individual blocks, detach from the rock mass and fall or roll into the highway.

Figure 9-28 Large Scale Rock Fall

9.3.9.3 Slope Protection for Soil Slopes

The purpose of applying a blanket of slope protection on a given slope is to prevent shallow surface failures or sloughing due to erosion. The cause of erosion can be either from the surface or from groundwater seepage from the cut slope itself.

Figure 9-29 Stone Slope Protection

The need for slope protection is difficult to predict unless surface sloughing has occurred in the past. On slopes to be recut, or in the case of a new cut slope, groundwater elevation needs to be accurately determined in the initial subsurface investigation.

The most troublesome slopes are those in which perched groundwater or seepage occurs along well defined zones or layers. The presence of perched groundwater is difficult to determine from conventional borings unless varying soil deposits are well defined or are thickly bedded. Thin, less-permeable soil deposits may be missed during standard 1.5 m sampling intervals.

Erosion of soil from seepage through the slope generally occurs in sand and silts where seepage forces actually carry soil particles from the slope face. Where the soils consist of silt or silty sand, the slope protection should be underlain by a geotextile (see Standard Specifications Section 207, Geotextile Slope Protection) to prevent migration of fine particles through the stone. In cases where saturated sands are present, a well-graded, slope-protection stone can be used which will create a natural filter with time allowing only water to pass through.

Cut slopes consisting of dense glacial tills with appreciable amounts of fines are also susceptible to surface sloughing when continually or periodically saturated. Saturation can occur either from perched groundwater flowing from a more pervious soil deposit above the till, natural moisture content, precipitation, or snow melt. Fine particles within the till face tend to retain moisture and after one or more freeze-thaw cycles the slope face becomes unstable and sloughs. Slope protection in this case provides both erosion and frost protection.

There are basically five (5) types of slopes which might be encountered:

1. Slopes that do not need slope protection at all.
2. Slopes which need only minimal slope protection during construction. This would be the case where fresh cut slopes will drain themselves more quickly. There is no recharge area behind the cut such as a crest of a hill.
3. Slopes that need protection for a period of time to allow the groundwater surface to recede into the slope. Generally this applies to granular uniform deposits with or without a recharge area behind the slope.

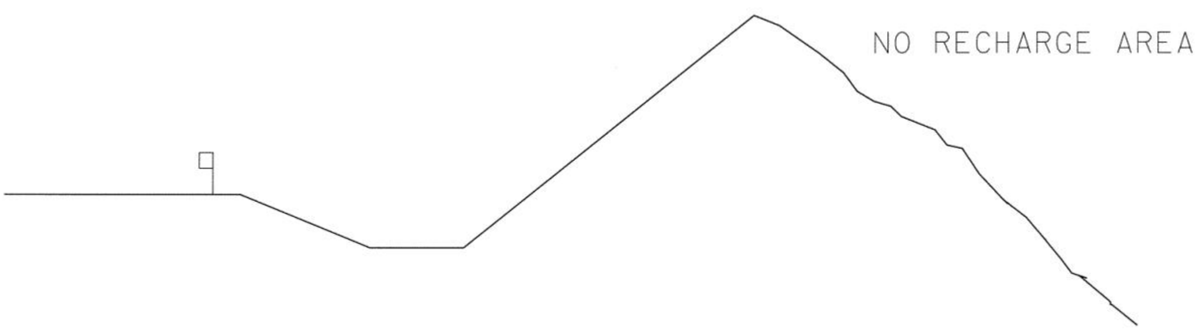


Figure 9-30 No Recharge Area behind Slope

4. Slopes which need permanent protection due to continual or even seasonal seepage. Deposits are usually layered with perched groundwater and have a large recharge area above or behind the slope.

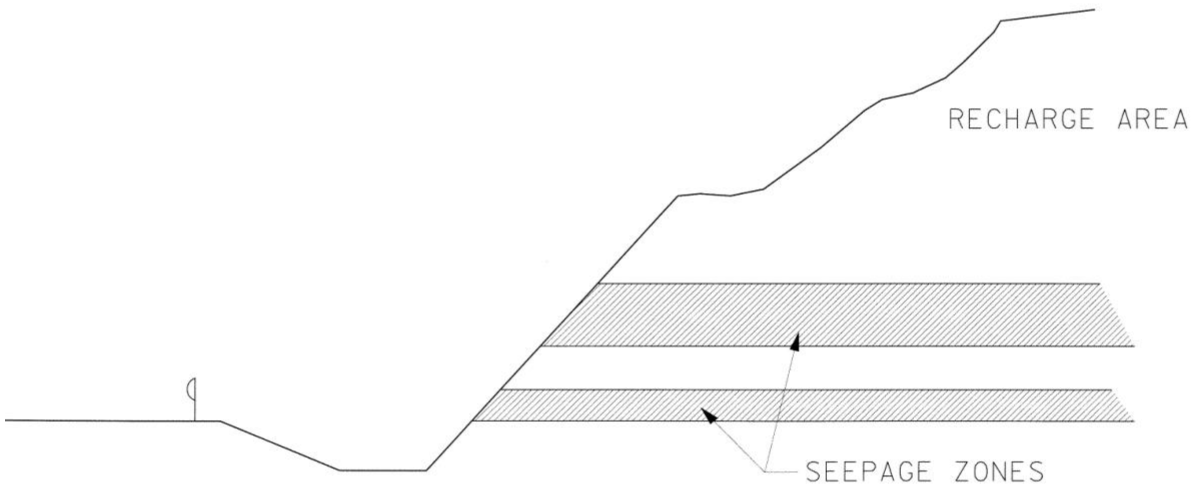


Figure 9-31 Recharge Area behind Slope

5. Slopes which may have potential for deep-seated failures at proposed slope inclinations. Slope protection will not stabilize a deep-seated slope failure.

The need for slope protection can be difficult to predict during the design phase of a project and in many cases, the need is discovered during construction when slopes are initially cut. For this reason there are two items for slope protection:

- Item 203.0801 SELECT GRANULAR FILL, SLOPE PROTECTION-TYPE A
- Item 203.0802 SELECT GRANULAR FILL, SLOPE PROTECTION-TYPE B

Type A is used at locations where we are confident that it is needed and is therefore shown on the contract plans. Type B is used when exact location and/or limits are not known. For this item we include an estimated quantity in the contract. It is to be placed at locations specified by the Engineer-In-Charge, typically with guidance from the Regional Geotechnical Engineer.

9.3.9.4 Slope Protection for Rock Slopes

The Geotechnical Engineering Bureau provides rock slope recommendations which consider major items in designing slopes for a proposed rock cut: stability and cost. Although rockfalls rarely cause injury and only occasionally cause property damage, when they occur they draw considerable public attention. Rock slope recommendations for creating a new slope or trimming an existing slope include cut angles and ditch catchment dimensions. Recommended cut angles are the steepest angles which will provide the steepest, low maintenance slope. Catchment ditches prevent falling rock from reaching the roadway. Other methods of mitigating an existing slope may include rock scaling, rock bolts, slope mesh, buttresses, and rock fall barriers.

The designer must consult a Department Engineering Geologist (through the Regional Geotechnical Engineer) to determine an appropriate rock-slope treatment consistent with other aspects of the design such as safety, right-of-way availability, utilities, etc. (refer to Geotechnical Design Procedure for Preparing Rock Slope Recommendations (GDP-13)).

Figure 9-32a Rock Scaling (Using Long Reach Excavator)

Figure 9-32b Rock Scaling (Using Long Reach Excavator)

Figure 9-32c Rock Scaling (Using Hoe Ram)

Figure 9-32d Rock Bolts

Figure 9-32e Rock Bolts

Figure 9-32f Wire Mesh Drape

Figure 9-32g Wire Mesh Drape

Figure 9-32h Wire Mesh Drape (Drape Connections)

Figure 9-32i Wire Mesh Drape (Anchor)

Figure 9-32j Rock Buttress

Figure 9-32k Rock Buttress

Figure 9-32l Thrie Beam Railing/
Corrugated Beam Rock Fence

Figure 9-32m Thrie Beam Railing/
Corrugated Beam Rock Fence

Figure 9-32n Wire Rope Catchment Fence
(Medium Impact)

Figure 9-32o Wire Rope Catchment Fence
(Medium Impact)

9.3.10 **Frost Heaves and Boulder Heaves**

Both these phenomena are caused by a combination of three conditions: frost-susceptible soils, a source of water, and freezing temperatures. The most susceptible soils for frost heaves are fine sands and silts. Frost heaves can also occur in areas where the top of bedrock is 1.2 m or less below the top of pavement. As the ground freezes, water forms ice lenses, and as low temperatures persist and more water freezes, the lenses continue forming and increasing in quantity and size. This expansion is reflected at the pavement surface as bumps and cracks. In spring, as ice lenses melt from the top, the upper subsurface strata become supersaturated and will not support traffic loads. This lack of support results in the deformations and potholes that are frequently experienced in late winter and spring.

Boulder heaves are caused by freeze-thaw of soil around boulders. As soil water freezes and expands, it elevates the boulder. As it thaws, loosened soil adjoining the void beneath the elevated boulder partially fills the void and supports the boulder in a raised position. With time, these solid objects may actually be forced to the pavement surface, causing very abrupt surface distortions. Eliminating the bump created by a boulder heave requires physically removing the boulder and replacing it with material similar to that formerly surrounding the boulder.

Frost and boulder heaves may be relieved by removing at least one of the three essential components: water, frost-susceptible soils, or freezing temperatures. The water component may sometimes be eliminated by proper drainage. The soil or bedrock component is minimized by undercutting the susceptible material and replacing it with granular material. The freezing-temperature component is inherent to our climate and cannot be eliminated from the soil.

Figure 9-33 Boulder Heave

9.3.11 **Rock Excavation**

This is considerably more difficult and usually more expensive than excavation of earth. Unlike earth excavation, rock must first be broken up before its removal, but the designer should be aware of certain advantages to rock excavation and should not avoid rock cuts without considering them. Rock-cut slopes are usually steeper than earth-cut slopes; right-of-way requirements thus may be reduced. Rock excavation and placement may be conducted through the winter, increasing available construction time. Good-quality rock may be used for production of granular material such as subbases, underdrain filters, stone fill-items, or underwater backfill.

Figure 9-34a Rock Excavation- Drilling

Figure 9-34b Rock Excavation- Drilling

Figure 9-34c Rock Excavation- Drilling

Figure 9-34d Rock Excavation- Explosive Loading

Figure 9-34e Rock Excavation- Explosive Loading

Figure 9-34f Rock Excavation- Matting over Explosives

Rock in New York State is of variable quality, and rock-cut slopes and/or their special treatment depend on composition and structure of the rock encountered. As indicated in Section 9.3.9.2, rock-cut slopes and any required special treatments will be recommended by the Geotechnical Engineering Bureau. As noted before, early coordination and decisions are important to permit timely determination of right-of-way requirements. Normal blasting and rock excavation techniques and requirements with which the designer should be familiar are included in Standard Specifications Sections 107-05 and 203-3 and Geotechnical Engineering Manual (GEM-22) Procedures for Blasting.

After the initial rock slope design has been provided by the Geotechnical Engineering Bureau's Engineering Geology Section, the designer should check the design to ensure that it meets the requirements of Context Sensitive Solutions (CSS). Designers, as the Department's representatives, need to be aware of the effects that compromises on rock slope design made to conform to CSS goals have on safety and liability. Designers should work closely with the Engineering Geologists to understand these effects and risks, thus enabling them to make an informed decision.

9.3.12 **Excavation Protection and Support**

Consideration for excavation protection and support shall be given when specifying excavation type work (e.g., unclassified excavation and disposal, trench and culvert excavation). Standard Specification[2] provisions are provided for unclassified excavation and disposal, trench and culvert excavation, and support and protective systems. Refer to Section 4 of the Bridge Manual[11b] for guidance regarding excavation protection and support (e.g., when a separate pay item is needed for excavation protection and support).

Figure 9-35 Trench Box

Figure 9-36 The ICON Group (Slide Rail System)

9.3.13 **Design of Spoil Sites**

Disposal of construction-and-demolition (C&D) waste and debris is regulated by NYSDEC under Title 6 of the Official Compilation of Codes, Rules and Regulations (6NYCRR) Part 360-7, or (for Nassau and Suffolk Counties) Part 360-8. Burning solid waste, which is defined as "disposal" by 6NYCRR Part 360, is prohibited except at a facility permitted by NYSDEC. Burning material of any kind, including that generated by clearing and grubbing, is prohibited on Department projects. Disposal of spoil from Department projects is exempt, under certain conditions, from Part 360 regulations if it occurs on property under Department jurisdiction -- that is, on the highway right-of-way. These regulations are identified to the contractor in §107-10 Managing Surplus Material and Waste, §107-11 Air Quality Protection, and §107-12 Water Quality Protection of the Standard Specifications.

For these reasons, disposal sites for material not marketable or recyclable should (where feasible) be designed into each project. Figure 9-37 indicates suggested sites and categorizes them in order of preference.

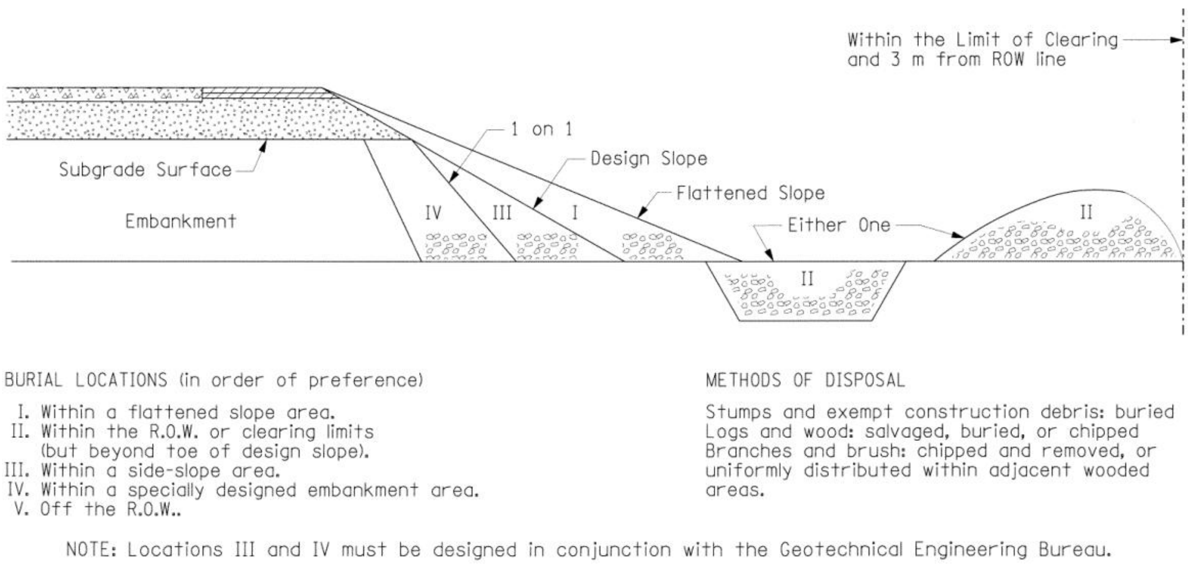


Figure 9-37 Suggested Spoil Sites

In selecting or designing disposal sites, special consideration must be given to environmental consequences and potential impact on the area's ecology. No excess or unsuitable material, wood, or other debris from clearing and grubbing, excess earth, or any other waste materials from the project may be disposed of in environmentally sensitive areas, either on or off the right-of-way, such as at the edges of lakes, ponds, rivers, streams, flood plains, marshes, or wetlands of any kind, or closer than 30 m to a state-designated wetland, without the necessary state permit. Such disposal must also be in conformance with Title 6 regulations concerning disposal of construction and demolition debris. As Figure 9-37 shows, the preferred location is on the highway right-of-way by flattening slopes, filling gore areas, etc. Where disposal of such materials is contemplated near a wetland and/or other body of water, all necessary precautions must be taken to control erosion and minimize their intrusion through proper location, grading, drainage, and cover. In addition, appropriate federal regulatory agencies must be contacted and their requirements satisfied.

Note that Locations III and IV in Figure 9-37 are within the embankment and require special design consideration. Disposal in these locations may be advantageous in areas of limited right-of-way or high fills. Since these locations provide a certain amount of support to the entire embankment, they must be evaluated individually. It is important for the designer to anticipate use of these disposal areas during design, and specify the desired method of disposal in the contract documents. These disposal methods are to be designed in cooperation with the Geotechnical Engineering Bureau. As noted previously, early coordination and decisions are important to permit timely determination of right-of-way requirements.

9.3.14 **Optional Borrow Areas**

Existing laws on Mined Land Reclamation and the Adirondack Park Agency, as well as other state and local laws and ordinances, may delay or sometimes even prohibit contractors from obtaining local borrow material. These regulations are identified to the contractor in §107-08 Protection and Restoration of Property and Landscape, B. Outside the Right of Way, of the Standard Specifications. Legal restrictions have had the effect of increasing NYSDOT earthwork costs.

The Department's policy is to make optional borrow areas within the highway right-of-way available to Contractors wherever practicable, and to show all such areas on the plans. When an optional borrow area is designated, the Contractor will be permitted to excavate material, subject to any state-imposed restrictions and conditions.

Designation of optional borrow areas should have the effect of reducing earthwork cost and minimizing hauling of materials over local roads outside the project limits. Projects requiring borrow should be carefully examined by the designer to determine if optional borrow areas can be designated without detriment to project objectives. Some ideas are illustrated in Figures 9-38:

- Removing mounded medians and gores.
- Excavating oversize ditches.
- Flattening or lateral shifting of cut slopes.
- Flattening rock-cut slopes, and daylighting hill cuts.
- Constructing drainage retention basins.

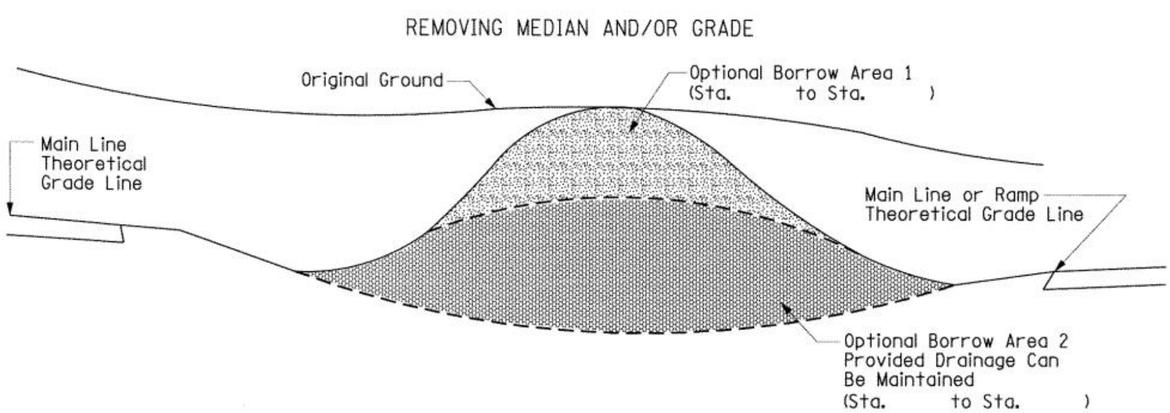


Figure 9-38a Optional Borrow Areas

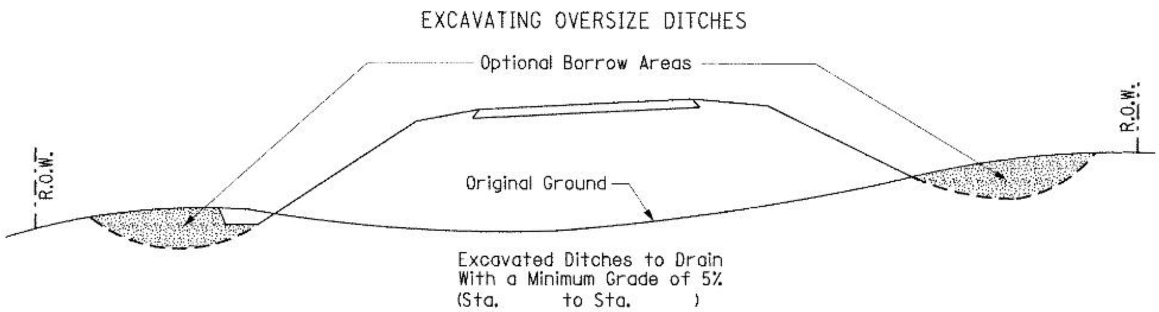


Figure 9-38b Optional Borrow Areas

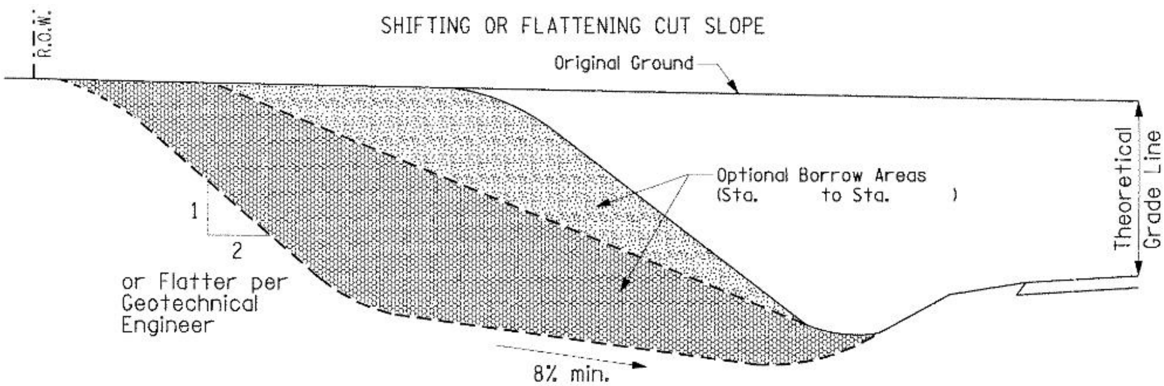


Figure 9-38c Optional Borrow Areas

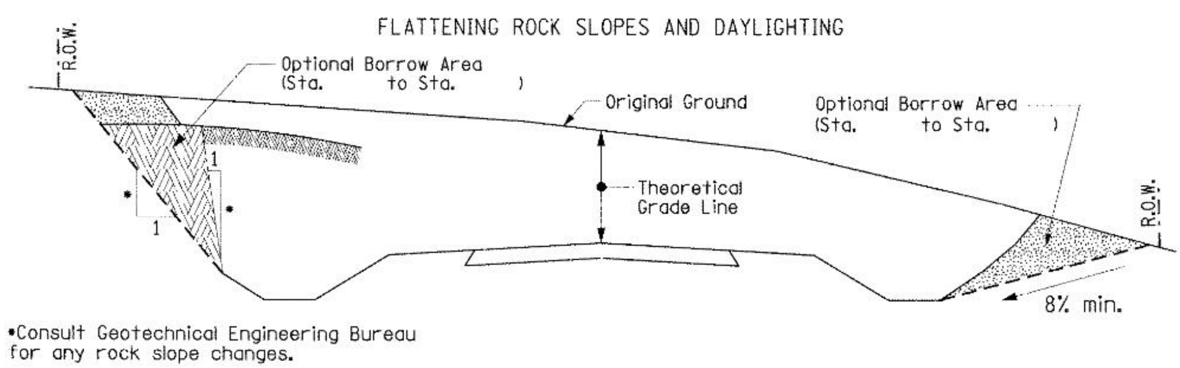


Figure 9-38d Optional Borrow Areas

If one or more of these concepts appears beneficial and does not adversely affect esthetic or environmental conditions, a designer should:

1. Consult the Regional Geotechnical Engineer or other appropriate functional unit representative.
2. Include sketches or other appropriate details in the plans. This information should be tailored to the particular project location. The objective should be to establish the limiting conditions under which an appropriate borrow source can be used.

3. To make these optional borrow areas compatible with the specifications, a note similar to the following must be included in the contract:

   "The Contractor's written request to use the optional borrow area(s) designated in the contract plans and/or proposal for this project will be approved by the Regional Geotechnical Engineer under the following conditions:

   1. The Contractor designates the locations and limits of the intended use.
   2. The Contractor agrees to conform to the requirements of '§107-08 Protection and Restoration of Property and Landscape' and '§107-12 Water Quality Protection' in Section 100 of the Standard Specifications in using these areas.
   3. No royalty will be required from the Contractor by the State for material obtained from areas designated in the contract documents.
   4. Payment will be made for such additional excavation beyond the design slope only when the extra material qualifies, and is used, as borrow for a project item that is paid for in its original position. No payment for excavation will be made when the material is used for an item paid for in its final compacted position.
   5. Cost of any additional grading, seeding, mulching, drainage appurtenances, guide railing, fencing, or other items of work related to the closing of the optional borrow areas to conditions satisfactory to the Engineer shall be borne by the Contractor."

As noted previously, early coordination and decisions are important to permit timely determination of right-of-way requirements.

9.3.15 **Geotextiles**

Geotextiles are sheets of porous fabric or plastic. In general, they are intended to allow water to pass through them while preventing the passage of solid particles. They must also be strong enough to withstand tearing and the pressure of the retained solids.

There are two basic types of fabrics manufactured: woven and nonwoven. The woven fabric manufacturing process results in a weave of distinct threads of plastic intersecting at roughly 90°. The threads of a nonwoven fabric intersect in an irregular and random pattern. The nonwoven fabrics are usually thicker than the woven.

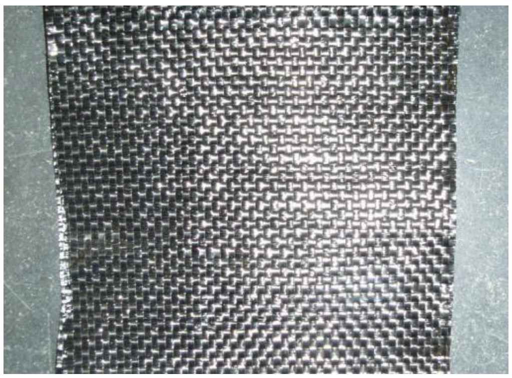


Figure 9-39a Slit Film Woven (Mirafi 500X)

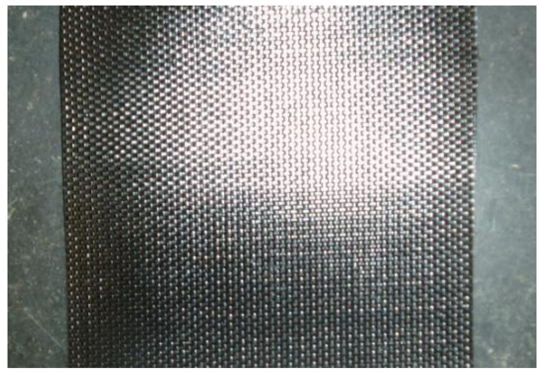


Figure 9-39b Monofilament Woven (SI GeoSolutions GEOTEX 104F)

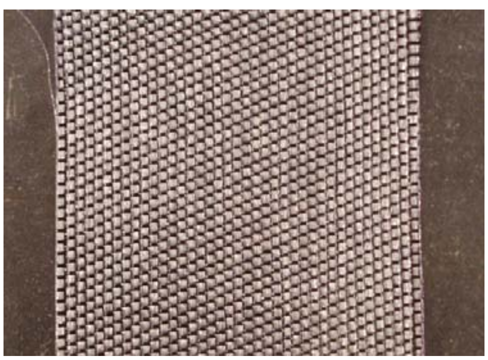


Figure 9-39c Combination Monofilament/ Fibrillated Yarn Woven (WEBTEC Terra Tex HPG 16)

Figure 9-39d Needle Punched NonWoven (SI GeoSolutions GEOTEX 1001)

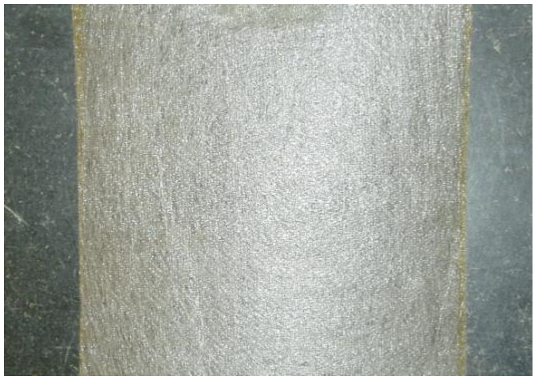


Figure 9-39e Heatbonded NonWoven (BBA FiberWeb Typar 3401G)

Standard Specifications Section 207 provides five geotextile payment items and uses: Geotextile Bedding, Geotextile Separation, Geotextile Drainage, Geotextile Slope Protection, and Geotextile Stabilization. Each of these have distinct permeability, filtration, and/or strength requirements. A brief discussion of each is provided below:

- Geotextile Bedding is used beneath stone fill where it must be strong enough to withstand the load of the stone placement, allow the passage of water and prevent the movement of fine particles from the underlying soil.

- Geotextile Separation provides permeability while preventing the passage of fine particles. It is usually used in conjunction with undercut operations in unstable subgrade situations.

- Geotextile Drainage is used as a filter to protect a draining medium from being clogged by the filtration of fines.

- Geotextile Slope Protection is used in areas where cut slope excavations have encountered a phreatic surface that is causing erosion of the slope surface. The Geotextile Slope protection is placed between the cut-slope surface and the Select Granular Fill Slope Protection to prevent migration of in-situ fine soil particles.

- Geotextile Stabilization, acts as a filter and is relatively strong. It is frequently used to reduce or replace the need for undercut in unstable subgrade conditions.

The strength, permeability and filtration characteristics of most of the manufactured geotextiles have been determined by extensive testing. Based upon the strength testing, the Department has divided the geotextiles into three strength classes: Class 1 being the strongest class, Class 2 of lesser strength and Class 3, the least strong.

Figure 9-40a Geotextile Testing (Grab Strength Test)

Figure 9-40b Geotextile Testing (Trapezoidal Tear Test)

Figure 9-40c Geotextile Testing (50 mm Puncture Test)

Figure 9-40d Geotextile Testing (Wide Width Strength Test)

The permeability and filtration characteristics have been grouped to be compatible with the soil material being drained. These are grouped according to apparent opening size (AOS) class as follows:

Class A is compatible with coarse material (0%-15% passing #200 sieve).
Class B, with average material (15%-50% passing #200 sieve).
Class C, with fine soil (greater than 50% passing #200 sieve).

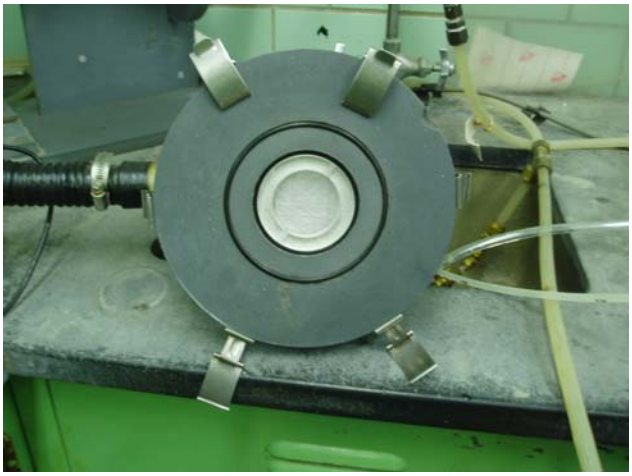


Figure 9-40e Geotextile Testing (Permittivity Test- Geotextile in Ring)

Figure 9-40f Geotextile Testing (Permittivity Test)

Figure 9-40g Geotextile Testing (Apparent Opening Size Test)

Designers must show the quantities, location/limits, geotextile type, strength class and AOS class for all geotextiles. The bidders use this information in conjunction with the Materials Bureau's Approved List to work up their bid prices. An example table (which should be provided in the Miscellaneous Tables section of the plans) identifying the type of information that should be for each geotextile type, is shown below:

Table 9-5 Example of Type of Geotextile Information to be Included in Miscellaneous Tables

Geotextile Item Number	Application	Location Limits (Sta. To Sta.)	Strength Class and Description	Apparent Opening Size (where appropriate)	Estimated Quantity (m^2)
207.10	Bedding	1+000 to 3+200	Class 2 Monofilament OR Class 1 Combination Monofilament Fibrillated Woven	B	887
207.11	Separation	3+210 to 7+125	2	___	1,200
207.12	Drainage	1+000 to 25+000	2	B	727
207.13	Slope Protection	4+025 to 5+025	1	B	1,010
207.14	Stabilization	6+070 to 8+125	1	___	517

The designer should contact the Regional Geotechnical Engineer for guidance on how to select the proper geotextile.

9.3.16 Controlled Low-Strength Material (CLSM)

CLSM consists of cement, water and, at the Contractor's option, fly ash, aggregate, or chemical admixtures in any proportions such that the final product meets the strength and flow consistency requirements included in the specification. The mix is proportioned to be self leveling and does not require compaction. Mix designs are approved based on a certified compressive strength. In addition, quality assurance samples for each batch are obtained for evaluation by the Geotechnical Engineering Bureau. It is much lower in strength than concrete, making future excavation possible. After set, CLSM has the characteristics of compact, undisturbed soil. Specification provisions for CLSM are provided in Standard Specification Section 204.

The flowability characteristic of the CLSM makes it an ideal material to fill voids in inaccessible areas, such as bedding under curb sections, and for cutoff walls (i.e., to separate water mains from storm sewers).

Due to the self-compacting properties of the material, CLSM is used as a replacement for compacted soil backfill in sites where obtaining compaction is difficult and labor intensive. In addition to backfill around pipes and drainage structures, CLSM is used as backfill behind abutments and around thrust blocks. When specifying CLSM as backfill material, the designer should always consider the equivalent fluid pressure exerted on structures by the CLSM before setup has occurred.

Because the CLSM is self compacting, construction personnel and equipment are not required in the trenches for compaction operations. Without workers in the trench, the width of excavation can be reduced, often eliminating laid back slopes and reducing the need for lane closures and flagging operations.

Specify a minimum soil cover to prevent CLSM from eroding. For short-term conditions (<3 months), use a minimum soil cover of 150 mm. For longer than 3 months, use a minimum soil cover of 300 mm.

The typically rapid setup time of the CLSM allows pipe crossings under roadways to be completed in very short time periods, usually between the morning and evening rush hours. Steel plates can span the narrower trenches, allowing almost immediate access across the excavation. Note that each additional foot of lift thickness can add 3-4 hours to the initial set time, unless accelerators are used. Wet soil conditions will increase the set time, while dry soil conditions will reduce the set time. The designer should consider the need to specify a set time in the plans.

If CLSM is used as backfill around lightweight pipes, flotation of the pipes may occur as the CLSM is placed. Appropriate notes should be included in the plans to inform the contractor of this possibility (See the appropriate Standard Sheet for placement of CLSM). Methods for weighting the pipes include anchors, straps and gravel bags, and partially filling watertight pipes with water. Caution is advised with the use of anchors, straps and gravel bags, as arching between the anchor points can occur.

The designer should consider the maximum lift thickness that can be placed to avoid flotation, as well as the maximum vertical deflection and lateral displacement that the pipe can tolerate.

Do not place CLSM in contact with aluminum pipe, including connections, fixtures, etc., unless the aluminum has been coated with an approved primer. Do not place CLSM containing fly ash in contact with cast iron pipes.

Refer to the Construction Inspectors Manual, Standard Sheet M204-1 for CLSM placement, and the Regional Geotechnical Engineer for additional guidance on the use of CLSM.

Figure 9-41a Placement of CLSM

Figure 9-41b Placement of CLSM

9.4 RETAINING WALLS AND REINFORCED SOIL SLOPES AND WALLS

Retaining walls are used in areas where normal earth slopes are undesirable, usually because of space restrictions. These walls have in the past normally been poured reinforced concrete (gravity or cantilever), timber, steel, or precast concrete cribbing, stone-filled wire-basket gabions, timber or steel sheeting, or steel soldier pile and lagging walls, all of which provide external support to the retained soil mass. Several innovations in types of retaining walls have become available, including using inherent characteristics of retained or reinforced soil as part of the support system.

There are three basic categories of support systems based on their intended function: permanent, temporary, and interim. A “permanent” system provides a structural support function for the life of the facility. A “temporary” system is designed to provide structural support during construction, and is removed when construction is complete. An “interim” system is identical to a temporary system in function, except it remains in place (although it no longer provides a structural function) after it is no longer needed.

A classification of retaining wall systems is provided in Table 9-6. The table provides a breakdown of available retaining wall systems, its associated method of construction, means of stability, design requirements and constraints (e.g. typical height range, maximum wall height).

Table 9-6 Classification of Retaining Wall Systems[14m]

Wall Category	Wall Type	Construction Type[1]	Wall Group	Design	Constraints
Externally Stabilized Wall Types	Anchored Walls (Sheeting or Soldier Pile & Lagging Walls)	Cut Wall	Deadman Anchors	Designed & detailed in contract.	Typical Height Range: 5 to 20m
			Grouted Tiebacks	Detailed in contract. Designed by Contractor's Design Consultant.	
			Braced Walls	Designed & detailed in contract.	
	Soldier Pile & Lagging Walls	Cut Wall		Designed & detailed in contract.	Typical Height Range: 2 to 5 m
	Sheeting Walls	Cut Wall		Designed & detailed in contract.	Typical Height Range: 2 to 5 m Maximum Wall Height= 4.5 m
Internally Stabilized Wall Types	Mechanically Stabilized Earth System (MSES)	Fill Wall		Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Typical Height Range: 3 to 20m
	Geosynthetic Reinforced Earth System (GRES)	Fill Wall		Designed & detailed in contract.	Typical Height Range: 2 to 15m
	Soil Nail Wall System	Cut Wall		Detailed in contract. Designed by Contractor's Design Consultant	Typical Height Range: 3 to 20m
	Mechanically Stabilized Segmental Block Retaining Wall System	Fill Wall		Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Typical Height Range: 2 to 11m
Gravity and Cantilever Wall System	Gravity Wall	Primarily Fill Wall. May be installed as a Cut wall.	Segmental Block Retaining Wall System	Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Typical Height Range: 1 to 2 m
			Sta-Wall	Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Maximum Wall Height = 7 m
			T-Wall	Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Typical Height Range: 2 to 15 m
			Evergreen Wall	Detailed in contract. Designed by Contractor's Designer/Supplier (Proprietary Wall)[2].	Maximum Wall Height = 7 m
			Gabion	Designed & detailed in contract.	Typical Height Range: 2 to 6 m Maximum Wall Height = 6 m
			CIP Mass Gravity	Designed & detailed in contract.	Typical Height Range: 1 to 3 m Maximum Wall Height = 7 m
	Cantilever Wall	Primarily Fill Wall. May be installed as a Cut wall.	Precast Cantilever Wall	Designed & detailed in contract.	Typical Height Range: 2 to 9 m
			CIP Cantilevered Wall	Designed & detailed in contract.	Typical Height Range: 2 to 9 m Maximum Wall Height = 7 m

[1] Cut wall construction is refers to a wall system in which the wall is constructed from the top of the wall to the base (i.e., “top-down” construction). Fill wall construction is refers to a wall system in which the wall is constructed from the base of the wall to the top (i.e., “bottom-up” construction).

[2] Use of proprietary systems must be justified in writing, in accordance with 23 CFR 635.411.

9.4.1 **Proprietary Wall Systems - Description and Evaluation Process**

Proprietary retaining wall systems are vendor designed retaining wall systems that are protected by specific patents that make them unique from other retaining wall systems. The term "proprietary" is really a misnomer today because many of the vendor-designed retaining wall systems that are used no longer have patents attached to them. In essence, many retaining walls could be designed using component parts from various systems, utilizing proper design procedures. For all but the simplest structures, however, this is impractical, and now, any vendor-designed and supported retaining wall system is considered a proprietary retaining wall system.

The proprietary retaining wall systems now used by the Department fall within two categories; mechanically stabilized earth systems (MSES) and precast modular gravity walls.

9.4.1.1 Mechanically Stabilized Earth Systems (MSES)

MSES are systems comprised of natural select granular backfill (reinforced backfill), dry-cast, modular block or segmental, precast concrete panels (facing), and high-strength, metallic or polymeric inclusions (reinforcement) to create a reinforced soil mass. The reinforcement is placed in horizontal layers between successive layers of granular soil backfill. Each layer of backfill consists of one or more compacted lifts. A free-draining, nonplastic backfill is required to ensure adequate performance of the retaining wall system. Each reinforcement is connected to the facing either with a mechanical or friction connection, depending on the system. Load is transferred from the backfill soil to the metallic or polymeric inclusion by shear along the interface and/or through the passive resistance on the transverse members of the inclusion. Stability of these systems is achieved by the weight of the reinforced soil mass resisting the overturning and sliding forces generated by the lateral stresses from the retained soil behind the reinforced mass.

Standard Specifications Section 554 provides specification provisions for mechanically stabilized earth systems. Currently Section 554 of the Standard Specification covers only those retaining wall systems that utilize segmental precast concrete panels.

9.4.1.2 Precast Modular Gravity Walls

Precast modular gravity walls are concrete or metal bin, gravity retaining structures built of adjoining closed-face or open-face bins. Each unit of a metal bin retaining wall is comprised of lightweight steel members that are bolted together on the construction site. Each unit of a precast concrete bin wall is comprised of interlocking prefabricated reinforced concrete modules that are placed together like building blocks. Each bin unit is filled with granular, free-draining soil which is compacted inside each unit. Stability of these systems is achieved by the weight of the soil-filled bin units resisting the overturning and sliding forces generated by the lateral stresses from the retained soil behind the bin units.

Standard Specifications Section 632 provides specification provisions for precast modular gravity walls. In addition to the Standard Specifications, the Geotechnical Engineering Bureau has created special specifications for the precast concrete wall unit type systems identified in Table 9-6.

Figure 9-42a Sta-Wall: Plain Concrete Finish (Leveling Beam Installation)

Figure 9-42b Sta-Wall: Plain Concrete Finish (Wall Construction)

Figure 9-42c Sta-Wall: Exposed Aggregate Finish (Elevation View)

Figure 9-42d Sta-Wall: Exposed Aggregate Finish (Stair Construction)

Figure 9-42e T-Wall: Plain Concrete (Wall Construction)

Figure 9-42f T-Wall: Plain Concrete Finish Finish (Wall Construction)

Figure 9-42g T-Wall: Exposed Aggregate Finish (Wall Construction)

Figure 9-42h T-Wall: Exposed Aggregate Finish (Wall Construction)

Figure 9-42i Evergreen Wall (Constructed Wall)

Figure 9-42j Evergreen Wall (Plant Growth)

Wall selection should follow the FHWA procedure referenced in Section 9.4.3. Once a wall type is chosen, Regional designers will prepare a line diagram layout for the retaining wall for inclusion in the plans. This can be done in this fashion because there is usually more than one retaining wall system that can be bid under the specification for the retaining wall type chosen for the project. This allows for competition among wall producers and hopefully a better price for the retaining wall. There are instances where true proprietary retaining walls (those protected by patents) are desired for a project site. In these situations, the Regions must justify and document their reasons for using a proprietary system because they are in essence sole sourcing the retaining wall for the project and eliminating competition. Once the justification is done, Regional designers can then contact the proprietary retaining wall company directly and have a design prepared for inclusion in the contract plans. Regional Geotechnical Engineers are very familiar with the above process and work very closely with the Regional design and construction personnel to ensure that it is followed closely.

9.4.2 **Wall Types**

9.4.2.1 Externally Stabilized Wall Types

The mechanism for stability of an externally stabilized wall system is obtained by installing a structural wall of sufficient strength to resist the overturning and sliding forces generated by the lateral stresses from the retained soil behind it. Externally stabilized wall types include anchored walls, soldier pile and lagging walls, and sheeting walls.

A. Anchored Walls

Anchors have been used where it is difficult to attain sufficient embedment to provide cantilever support for a retaining wall. The wall is anchored to stable earth or bedrock by wire strand or steel bar tendons. Anchors receive their resistance either by being attached to a deadman, or by being grouted in soil or rock. Deadman anchors are attached to an object that is constructed or already exists, located outside the zone of influence. A deadman may be an existing abutment or wall, a length of driven sheeting, a section of cast concrete, or any other member that is buried in the soil. For a deadman to provide maximum resistance, the passive failure wedge for the deadman has to be outside the active failure wedge of the wall to be supported.

A more common type of treatment is similar to deadman anchors, but is more versatile which allows their use under more diverse site and loading conditions. This commonly used category of anchor is called a *grouted tieback*. These anchors may be used to support temporary, permanent, or interim walls. They are most often used to augment soldier pile and lagging walls and sheeting walls.

A tieback consists of a steel tension element called a *tendon* (may be multistrands or a bar) that transfers tensile forces from the ground to a structural element.

Typical uses for tiebacks include supporting retaining walls (either temporary or permanent), stabilizing abutments, and increasing down force on dams.

Because the tendons extend behind the wall, it is necessary to make sure the Department has the legal right to the property directly above the tiebacks. It may be necessary to obtain temporary or permanent easements or ROW.

To design tiebacks, the Geotechnical Engineering Bureau designer determines the loads on the wall for each maximum depth of excavation. The designer then shows the loading on the tiebacks, tieback spacing, angle of declination, minimum free length, soil parameters, and other design assumptions on the contract plans. It is the Specialty Contractor's responsibility to design the tendon size, tendon type, and bond length, and use appropriate installation procedures and equipment to properly install the tiebacks.

It is also a good idea for the designer to come up with a table or graph that relates excavation height to tieback design load for various spacings. This is not very time consuming to complete at this stage, when the designer is familiar with the demands of the project, and it may save a lot of time later. Quite often, the Subcontractor adjusts the tieback spacing and recalculates the loads to suit his/her own operations and expertise.

Figure 9-43a Sheeting Wall (Braced Excavation)

Figure 9-43b Anchored Sheeting Wall

Figure 9-43c Anchored Soldier Pile & Lagging Wall

Figure 9-43d Anchor (Sheeting Deadman)

Figure 9-43e Anchor (Concrete Deadman)

Figure 9-43f Anchor (Soldier Pile Deadman)

B. Soldier Pile and Lagging Walls

Soldier piles used as part of a shoring system are vertical structural units, or members, which are spaced at set intervals. Soldier piles in a soldier pile and lagging wall system are typically spaced at 1.8 m to 3.0 m intervals. A lagging material is placed between the soldier piles to complete the shoring system. In New York State, the majority of the soldier piles used are made of steel, with concrete or timber used less often.

A soldier pile and lagging wall derives its resistance from the embedded portion of the wall. The lagging material selected is usually dependent upon the design life of the wall. A temporary wall will usually incorporate untreated timber lagging, with steel sheeting used less often. A permanent wall will usually incorporate treated timber lagging or concrete lagging with an architectural finish.

Because of the higher available section moduli, greater excavation depths can be supported by soldier piles and lagging as compared to those supported by sheeting. Cantilevered soldier piles are usually practical for excavations up to approximately 5 m in height.

Figure 9-44a Soldier Pile & Lagging Wall (Permanent Timber Lagging)

Figure 9-44b Soldier Pile & Lagging Wall (Permanent Precast Concrete Lagging)

Figure 9-44c Soldier Pile & Lagging Wall (Cast-In-Place Finish)

C. Sheeting

Sheeting members of a shoring system are structural units which, when connected one to another, will form a continuous wall. The wall continuity is usually obtained by interlocking devices formed as part of the manufactured product. In New York State, the majority of the sheeting used is made of steel, with timber and concrete used less often.

Sheeting is driven to a depth sufficient for the passive pressure exerted on the embedded portion to resist the lateral active earth pressures acting on the cantilevered section. To achieve the required passive earth pressure resistance, embedment depths can often be quite high. Therefore, due to limitations on the availability of certain section moduli and the associated costs, cantilevered sheeting walls are usually restricted to a maximum height of 4.5 m. When the height of excavation exceeds 4.5 m, or if the embedment depth is limited (e.g., the presence of boulders or bedrock), it becomes necessary to investigate the use of additional support for the wall system. Additional support may be provided by grouted tieback anchors, anchors to a deadman or struts, braces or rakers.

Figure 9-45a Sheeting Wall Installation

Figure 9-45b Sheeting Wall (Permanent Timber Facing)

Figure 9-45c Sheeting Wall (Permanent Timber Facing)

9.4.2.2 Internally Stabilized Wall Systems

The mechanism for stability of an internally stabilized wall system is obtained by improving the strength of a soil by placing tensile reinforcing elements (inclusions) in the soil to create a reinforced mass. The weight of the reinforced soil mass resists the overturning and sliding forces generated by the lateral stresses from the retained soil behind it.

It should be noted that, since internally stabilized wall systems rely on a reinforced mass of soil for stability, these wall systems may not be appropriate where it may be necessary to gain future access to underground utilities.

Internally stabilized wall types include Mechanically Stabilized Earth Systems, Geosynthetic Reinforced Earth Systems, Soil Nail Wall, and Mechanically Stabilized Segmental Block Retaining Wall System.

A. Mechanically Stabilized Earth Systems (MSES)

A MSES wall consists of straps, mesh, or membrane buried in the retained embankment as it is constructed. These members, along with precast concrete facing panels, provide sufficient tensile strength to retain the earth slope on a vertical or nearly vertical plane. Additional information on MSES walls is discussed in Section 9.4.1.1 Mechanically Stabilized Earth Systems (MSES).

MSES walls are most applicable when constructed in new fills, such as embankment widenings. An important consideration in determining the use of these systems is the space required for embedment of the reinforcing. A guide in determining the preliminary embedment lengths is to use 70% of the proposed MSES wall height. To install the reinforcing to the required embedment lengths will sometimes involve cutting and benching adjacent fills. In addition, MSES walls should not be used in areas where utilities other than highway drainage must be constructed within the reinforced zone where future access for repair would require the reinforcement layer to be cut.

Final selection of this wall type depends on the corrosiveness of the soil mass to be retained, and its effect on buried members, such as steel straps.

A list of approved MSES systems is maintained on the Materials Bureau's Approved List.

Inspection guidance for the construction of MSES walls is summarized in a manual published by the Geotechnical Engineering Bureau[(17a)].

Figure 9-46a MSES Wall (Reinforcement Layout)

Figure 9-46b MSES Wall (Form Liner Finish)

Figure 9-46c Terraced MSES Wall

Figure 9-46d MSES Wall (Temporary Stage Line Wall)

Figure 9-46e MSES Wall (Cast-In-Place Facing)

Figure 9-46f MSES Wall (Cast-In-Place Facing)

B. Geosynthetic Reinforced Earth Systems (GRES) - Temporary and Permanent

It is very common to use geosynthetics to reinforce a soil mass, to allow the construction of embankments with very steep and even vertical slopes. These systems are called Geosynthetic Reinforced Earth Systems (GRES). Simply put, a GRES is a non-proprietary version of an MSES. Temporary GRES systems are very commonly used for staged construction, as they are inexpensive and easy to construct. GRES slopes or walls can also be constructed with an engineered facing, enabling these systems to serve a permanent support function. These systems are typically designed in-house by the Geotechnical Engineering Bureau, who has created a special specification and common details for the installation of GRES walls and slopes.

As a more economical alternative to the typical retaining wall, the Geotechnical Engineering Bureau has developed a specification and details for a GRES. These systems can be used to construct permanent and temporary over-steepened slopes and permanent and temporary vertical embankment fills. They are most applicable when constructed in new fills, such as a raise in grade between stages, embankment widenings, or when constructed within the backfill where the proposed abutment is located in front of the existing abutment.

An important consideration in designing these systems is the space required for embedment of the reinforcing. A guide in determining the preliminary embedment lengths is to use 70% of the proposed GRES wall height. To install the reinforcing to the required embedment lengths will sometimes involve cutting and benching adjacent fills. This should also be accounted for when determining available space for GRES construction. GRES should not be used in the top 2 m of a fill if significant future utility work is expected. Since its development, many applications for GRES have evolved. These are briefly described below:

B.1 Staged Construction

One of the major considerations during design of a structure replacement is the maintenance of traffic during construction. When detours are not feasible, the new structure is typically constructed in stages using part of the existing structure for stage one traffic. This often requires design and construction of temporary retaining walls such as steel sheeting or soldier pile and lagging walls to support the roadway between stages. The GRES wall was originally developed as a more economical wall system for this application where new backfills will be placed in front of the existing abutment in stages.

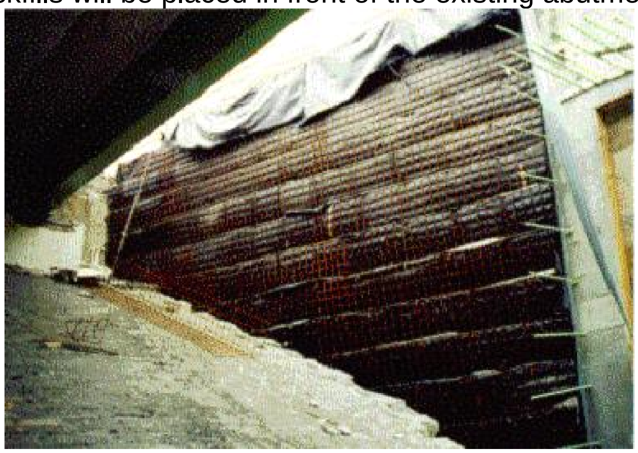


Figure 9-47a GRES (Temporary Stage Wall)

The construction procedure for a vertical GRES wall is simple. A 450 mm by 450 mm L-shaped welded wire mesh is placed on the leveled foundation soil to act as a form for the backfill. (This method to form the face is not required, but is common in most Regions. Regardless of the method used, the face should be constructed with a relatively uniform slope.) Sheets of geosynthetic reinforcing of the required embedment lengths are then placed perpendicular to the face of the wall. A geotextile wrap is laid along the bottom of the welded wire form and up around the face to retain the backfill. The select granular backfill is then placed and compacted. Once the backfill reaches the top of the form, the geotextile wrap is pulled back over the top of the backfill. The next welded wire form is placed and the process is repeated until the final design height is reached.

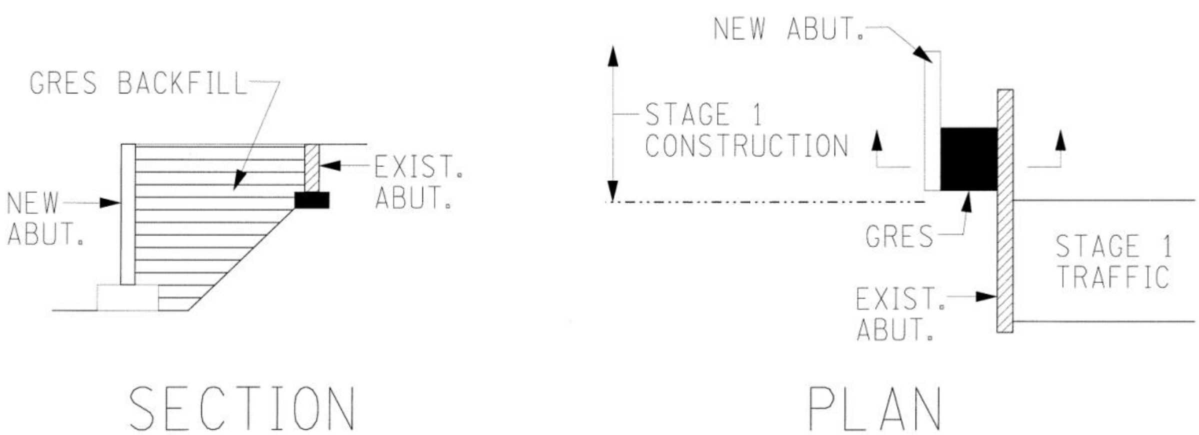


Figure 9-47b GRES (Temporary Stage Wall)

B.2 GRES Slopes (Permanent and Temporary)

Another typical application for GRES is to reinforce over-steepened slopes. In many cases, roadway improvements require widening embankments. These widenings are often restricted by R.O.W. limits, environmental constraints such as wetland areas or at culvert locations. Retaining walls or culvert extensions were typically used to overcome these restrictions. These widenings can be accommodated by constructing steeper embankment side slopes with GRES while maintaining the same toe of slope. The GRES slopes can be constructed without forms and seeded at slopes of 1V on 1H or flatter. The requirements for the backfill material for a GRES slope are less strict as compared to those for a GRES wall. The requirements for both situations are provided in the specification. Backfill (Type B) identified in the specification is used for slopes.

Figure 9-48a GRES (Permanent Oversteepened Slope)

Figure 9-48b GRES (Permanent Oversteepened Slope)

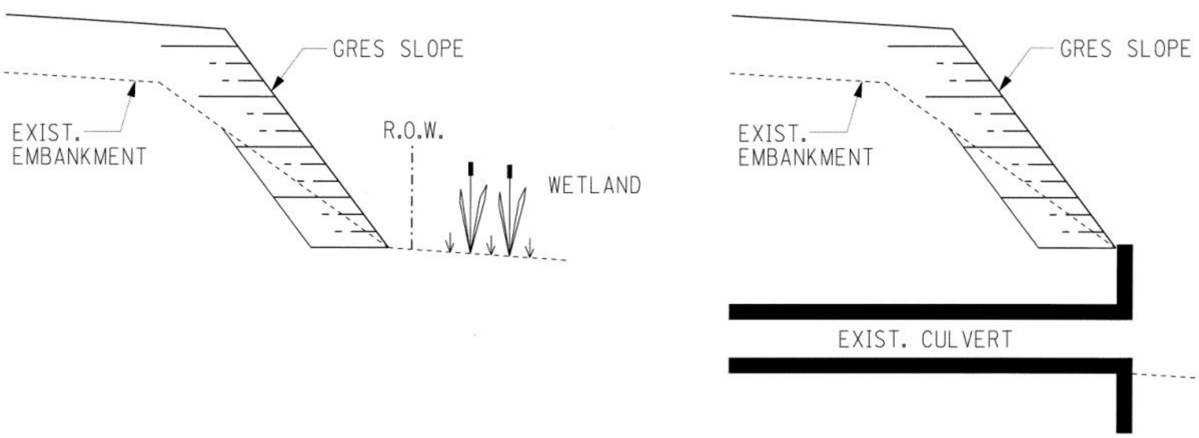


Figure 9-48c GRES for Widening/Grade Change

B.3 Vegetated Face Vertical GRES (Permanent)

For realignment or new embankment construction, retaining walls are sometimes required to minimize impact on wetlands or R.O.W. The vegetated-face GRES wall is an economical alternate, especially in environmentally sensitive areas. The construction of the vegetated-face GRES is similar to the temporary GRES for staged construction. However, this "green" wall incorporates a geocell facing. A geocell is a three-dimensional, High Density Polyethylene (HDPE) cellular confinement unit which is infilled. The outer cell is typically filled with topsoil to provide for plant or turf establishment.

Figure 9-49a Geocell Faced GRES

Figure 9-49b Geocell Faced GRES

B.4 Timber Faced Vertical GRES (Permanent)

Another option for realignment or new embankment construction is a timber faced GRES wall. Construction methods are similar to the temporary GRES for staged construction except that the timbers act as a form and the welded wire mesh forms are not required. Other facings, such as precast concrete block units or gabions can be incorporated into GRES walls.

Figure 9-50a Timber Faced GRES Wall

Figure 9-50b Timber Faced GRES Wall

C. Soil Nail Wall

Soil nailing is an operation in which the soil is reinforced by steel to increase its tensile strength, thus forming a gravity retaining mass. This treatment requires highly specialized construction techniques.

Soil nails are steel bars or tendons installed to reinforce or strengthen the existing ground. They are used to support the existing soil for a cut situation. Soil nails are installed into a slope or excavation as construction proceeds from the existing ground surface to the proposed bottom of excavation. The soil nailing process creates a reinforced section that is itself stable and able to retain the ground behind it.

Soil nails are similar in construction but different in function from a grouted tieback. Specifically, soil nails are passive reinforcements which develop their reinforcing action through nail-ground interactions as the ground deforms during and following construction. Soil nailing provides some advantages over grouted tiebacks as they improve construction flexibility where overhead access is limited, reduce right-of-way requirements by being typically shorter than tiebacks and, by eliminating soldier pile/sheeting installation, they reduce construction time and improve construction flexibility in heterogeneous soils with cobbles and boulders. They should not be used where the water table elevation is above the bottom of the excavation for the soil nail wall.

The Geotechnical Engineering Bureau has created a special specification for a soil nail wall, along with design and construction guidelines for the use of the special specification for a soil nail wall system[19c].

Figure 9-51a Soil Nail Wall (Temporary Shotcrete Facing)

Figure 9-51b Soil Nail Wall (Permanent Shotcrete Facing)

Figure 9-51c Soil Nail Wall (Cast-In-Place Facing)

Figure 9-51d Soil Nail Wall (Stone Masonry Facing)

D. Mechanically Stabilized Segmental Block Retaining Walls

Because of the small size and weight of the blocks used in the segmental block retaining wall systems, wall heights are extremely limited. By using internal reinforcing, wall heights can be increased to meet nearly any highway fill application. Systems like these are similar in function and construction to a permanent GRES system, which utilizes the segmental blocks as a permanent facing.

Allowable segmental block types used to construct these systems can be found on the Materials Bureau's Approved List.

Figure 9-52a Mechanically Stabilized Segmental Block Wall System (Unilock Siena Stone 500 & Strata grid 200)

Figure 9-52b Mechanically Stabilized Segmental Block Wall System (Back Face) (Unilock Siena Stone 500 & Strata grid 200)

9.4.2.3 Gravity and Cantilever Wall Systems

The mechanism for stability of a gravity or cantilever wall system is obtained by gravity providing the righting moment and base friction to resist overturning forces. There are several wall types that fall under the general heading of gravity wall systems, including:

- Cast-in-place gravity walls.
- Precast gravity walls.
- Segmental Block Retaining Wall Systems (SBRWS) (i.e., Mesa). Segmental block retaining walls are usually proprietary systems that consist of interlocking units that stack vertically or at a nominal batter. Their heights are usually quite limited, unless they are internally reinforced. Allowable segmental block wall types can be found on the Materials Bureau's Approved List.
- Precast retaining wall systems (i.e., Precast cantilever walls, Sta-Wall, T-Wall).
- Gabions.

Gravity walls are best constructed in fill situations.

Figure 9-53a Precast Gravity Wall (Foundation Construction)

Figure 9-53b Precast Gravity Wall (Stone Facing)

Figure 9-53c Segmental Block Retaining Wall System (Unilock Siena Stone 500)

Figure 9-53d Segmental Block Retaining Wall System: Back Face (Unilock Siena Stone 500)

Figure 9-53e Precast Concrete Cantilever Wall

Figure 9-53f Gabion Wall

Figure 9-53g Gabion Wall

9.4.2.3.1 General

All retaining walls must be considered to be structures. Figures 9-54a through 54h (at the end of this subsection) are intended to divide those structures into two categories:

- Standard design structures which can be detailed completely from these sheets and;
- Special design structures which require unique designs.

The purpose of establishing two separate categories of structures is to identify the more expensive and the more structurally critical structures for careful review. Special design structures are those which must be designed independently (not by these Figures) and then reviewed by both the Geotechnical Engineering Bureau and the Office of Structures. Standard structures are defined as those which may be designed and detailed entirely from Figures 9-54a through 54h and do not require review beyond the region level.

Figure 9-54a gives details for cantilever walls. Figures 9-54b through 54h gives details for both gravity and cantilever walls. These standard designs were developed with three (3) specific goals in mind. First, the standards are developed in order to provide the regions with the capability to generate contract plans from the standards without the need to refer to any outside organization. Second, criteria are to be provided for making quick preliminary cost estimates, and for converting the theoretical individual designs into practical overall wall details. These details are to emphasize simplicity and standardization. Third, costs are minimized by the use of low pressure wall designs, made possible by a limited amount of soil preparation beneath the footing.

To achieve the capability of performing complete designs at the Regional office, it is necessary to involve the skills of the Regional Geotechnical Engineer, as well as those of the structure designer. Standard designs shall not be used unless both the designer and the geotechnical engineer agree that they are applicable to the individual case. The tabulated designs include soil pressures as low as 95.76 kPa and sliding friction coefficients down to 0.35. These values enable the use of a spread footing design for almost any site. The function of the geotechnical engineer is to determine the design parameters for the soil at all footing locations along with recommended bottom of footing elevations. In the event that undercutting of unstable material beneath the proposed spread footing is required to provide a stable foundation, this undercut shall be limited to 1.8 m maximum for cantilever walls (Figure 9-54b) and 1.2 m maximum for gravity walls (Figure 9-54g). Greater depths of soil removal require careful analysis and review by the Office of Structures and the Geotechnical Engineering Bureau to determine the feasibility of using standard design as compared to special pile-supported (or other) designs.

Standard designs are restricted to the use of spread footings. They are available up to a maximum stem design height of 7.3 m. Separate tables for level surcharge and sloping surcharge are provided to enable the designer to compensate for varying earth pressures due to backfill conditions. Whenever soil conditions dictate the use of a pile foundation, the design stem height exceeds 7.3 m, or a railroad live load surcharge may be applied to the retaining wall, a special design is required.

Standard cantilever wall designs may be used at any wall location. Walls required for the retention of highway fills (not connected with a major structure) may be either of cantilever or gravity design. When highway fills are high enough to require the use of retaining walls which exceed 3.7 m in height, other wall types should be considered.

Fencing to protect pedestrians should be provided where appropriate. This would usually be required when a wall is retaining a cut slope. In built up areas, a 1.8 m chain link fence should be used. In rural areas where few pedestrians are anticipated and fencing is not provided at the right of way line, woven wire fencing would be satisfactory. When a wall is retaining a fill and it is located at the outside edge of a shoulder, bridge rail should be used to protect vehicles.

9.4.2.3.2 Design – Cantilever Walls

9.4.2.3.2.1 BASIS OF DESIGN

These walls have been designed by means of computer program No. B5000 as maintained by the Structures Design Systems Unit of the Structures Subdivision. Some of the assumptions and input data for these designs are listed below:

1. Unit weight of earth: 19.0 kN/m^3
2. Unit weight of concrete: 24.0 kN/m^3
3. Active earth pressure of soil behind wall = 1.4 kPa for level surcharge only.
4. Calculated by program for a 1 on 2 sloping surcharge = 2.2 kPa maximum.
5. Earth fill behind wall was assumed with the top of finished ground 230 mm below the wall coping.
6. A 0.3 m depth of surcharge was applied above the toe of the footing. No passive earth pressure was applied.
7. A 0.6 m depth of surcharge, simulating the effect of live load, was applied above the heel when level surcharge was used. No provision for Railroad live load surcharge was included.
8. No loads, vertical or horizontal, other than that caused by the earth backfill and surcharge were imposed on the wall.
9. The designs were developed with the absence of hydrostatic pressure behind the wall. The material placed behind the retaining wall is a select, free-draining granular material and the walls are to be detailed with weep holes as identified in the notes.
10. The placement of backfill was assumed to extend full depth behind the wall. No sheeting or intervening rock which would limit the effect of the backfill upon the wall was considered to exist.
11. All the cantilever walls are supported by 0.6 m thick footings except for certain walls with a stem design height of 6.1 m or greater which have a sloping surcharge imposed on them. All walls have stems which measure 0.45 m at the coping. The exposed face of wall is vertical and the face against which earth is placed is battered at 24 vertical on 1 horizontal starting at the coping and extending to the footing.
12. Reinforcement is designed based on a 76 mm concrete cover over footing steel and a 50 mm cover over the stem steel.
13. Allowable concrete stress: 8.3 MPa.
14. Allowable reinforcing bars stress: 165.5 MPa.
15. Ratio of elastic moduli (n): 10.
16. Allowable shear in concrete: 620.5 kPa.
17. Allowable bond stress for reinforcement: 2.1 MPa.
18. Factor of safety, sliding: 1.5.
19. Factor of safety, overturning: 2.0.

The following input values were varied over a range to produce the family of designs from which the tables were extracted:

1. Stem height: 1.8 m to 7.3 m.
2. Soil pressure: 100.0 kPa to 500.0 kPa
3. Sliding friction coefficient: 0.35, 0.50, 0.65.

9.4.2.3.2.2 METHOD OF DESIGN

The program checks the wall for both permanent loadings and temporary loadings during construction except that wind forces during the construction are not analyzed. Since no restrictions were placed on either toe projection or heel projection, the design is accomplished by incrementing both toe and heel until a width of footing is attained which will meet the design criteria. In cases where the projection of the footing must be limited, a special design or review should be requested from the Office of Structures if the toe or heel projection given by Table 9-3 exceeds the allowable. The program calculates the required areas of steel for stem steel and footing steel and the required perimeter for bond. The reinforcement shown in the table was chosen in accordance with a collated summary of these values. Calculated values of soil pressure and safety factors are also listed in the output. These values were the basis for the selection of the individual designs to be included in the table. Separate designs for footings with shear keys were automatically computed but were discarded as being less desirable than other designs which were offered. Table 9-7 is the result of editing over 1,500 designs to provide a complete coverage of soil conditions. The 350.0 kPa thru 500.0 kPa pressure designs were eliminated because the extra soil capacity was not required for reasonable economical design.

The 0.35 sliding friction coefficient will satisfactorily represent designs in the 0.30 to 0.44 range, the 0.50 sliding friction coefficient designs are accurate within the 0.45 to 0.59 range, and the 0.65 sliding friction coefficient designs pertain to the 0.60 to 0.70 range. It remains then for the Regional Geotechnical Engineer to classify soils as either being in the “LOW” sliding resistance range, the “NORMAL” sliding resistance range, or in the “HIGH” sliding resistance range in accordance with these ranges of values.

The concrete unit quantity column was added by means of an independent computation. The sole purpose of this value is to provide a means for making comparative estimates when preparing preliminary plans.

9.4.2.3.2.3 DESIGN AND DETAILING SEQUENCE

1. Locate retaining wall in plan by offsets from adjacent center line of improvement or base line.
2. Plot boring holes in plan.
3. Regional Geotechnical Engineer shall be consulted to ascertain from the borings the required depth of excavation, bearing value of the soil, and sliding friction coefficient of the soil.
4. The bottom of footing elevations should be tentatively set at various points along the wall. A 1.2 m minimum earth cover below final grade should be maintained and an effort made to place the proposed footing subgrade in a consistent band of subsoil.

5. Wall should be subdivided into separate footing segments (pours) between footing expansion joints and into the individual wall panels between stem contraction joints. Stem panel lengths should not exceed 9.1 m. Footing segments should not exceed 27.45 m.
6. At appropriate intervals along the wall (not exceeding 30.5 m even stations), the height of wall required to retain the earth fill should be calculated from profile and topographic data.
7. From the raw data now available, the bottom of footing elevations shall be finally set and the top of wall profile established forming a smooth, aesthetic profile rather than a series of straight line connections to computed theoretical points. Footing segments (pours) on rock or hardpan should be stepped as necessary in 0.6 m minimum step increments. Footing segments in earth cut or on fill should not be stepped, but the footing segment lengths should be limited such that the height of fill over the toe of the footing does not vary by more than 2.4 m. The coping shall protrude above the earth fill a minimum of 230 mm.
8. Compute the height of wall at the joints between each stem panel. If the wall height within the panel varies by 1.8 m or less, select the height of the higher third point as the design height for the panel. For panels of more than 1.8 m variation in height, the greatest height must be used for one design height and a second design height may be selected from the height of the lower third point if the panel is long enough to warrant more accurate design.
9. Enter the Table of Proportions with the appropriate stem height, allowable soil pressure and sliding friction coefficient, and list the design values for all design sections.
10. If either toe or heel projection is limited by clearance requirements, the footing dimensions which have been listed should be checked for interference. A special design will be prepared by the Office of Structures in cases of unavoidable interference.
11. Design values of toe and heel projections should be plotted about the respective points along the layout control line at which the design sections were taken.
12. Actual footing dimensions, extrapolated from the required theoretical dimensions, are next determined for each footing segment. Variations in either toe or heel projections of less than 0.45 m may be neglected in order to maintain a constant toe or heel dimension. In such cases, the designer should use the larger design projection.
13. Once the concrete dimensions are fixed, the footing reinforcement must be decided upon. If the footing segment is of constant cross-section, the maximum required heel and toe reinforcement must be carried throughout the length of the footing segment. If the footing is trapezoidal in shape, heel and toe reinforcement should be varied in accordance with the respective design section values.
14. Relatively level panels, which have been designed on the basis of a higher third point design height, shall carry the design dowel reinforcement for the entire length of the stem panel. Those steeply sloped panels, which are designed on the basis of the greatest height in the panel, should use two dowel sizes in the panel. More than two dowel sizes will not be economical except in walls with the most extreme (1 vertical on 2 horizontal) slopes. The variation in the dowel size may be extracted from the table in accordance with the stem height.
15. Main stem reinforcement shall be detailed as follows: Use #22 bars at 450 mm centers for all stem heights above 6.1 m. Use #16 bars at 450 mm centers for all portions of wall where the stem height is 6.1 m or less.
16. In using the concrete volume figures for steeply sloping walls, a sufficiently accurate preliminary estimate may be arrived at by assigning a corresponding length of wall to each different volume figure.

17. The designer should develop scoring details and patterns appropriate to enhance the aesthetic quality of the retaining walls especially if wall heights exceed 2.4 m of exposed stem and the walls can be readily viewed by the public.

9.4.2.3.3 Design – Gravity Walls

Gravity walls are designed by manual calculations. They are intended to provide a low cost alternative to the cantilever designs where design heights do not exceed 3.7 m. Provided the appearance of the gravity wall is suitable to the overall site plan, a gravity design may be used to retain highway fills more economically than the corresponding cantilever wall.

Gravity walls will be most appropriate to support shallow cuts, driveways, local streets, etc. They may be used adjacent to mainline traffic provided the select fill behind the wall can be placed beyond the edge of highway pavement.

9.4.2.3.4 Cantilever and Gravity Wall Figures and Table of Proportions

9.4.2.3.4.1 GENERAL NOTES:

1. Walls which cannot be designed within the range of designs listed must be referred to the Office of Structures for review. The applicability of the wall designs to a given situation shall be determined within the Region by the concurrence of both the structures designer and the geotechnical engineer. In cases where doubt exists as to the relative economy of alternate designs, approval to vary from the standard wall design shall be requested from the Deputy Chief Engineer (Structures).
2. Two weep holes shall be provide in every panel, located at the quarter points.
3. The invert of the weep hole at the front of the wall shall be located 150 mm above the finished grade or 150 mm above low water for stream bridges, whichever is higher.
4. Waterstops:
   In accordance with BD-MS3 R1, use Type “E” or Type “D” Waterstops behind joints thru the stem, if staining would be objectionable.
5. Construction Joints:
   Detail in accordance with BD-AB6 R1.

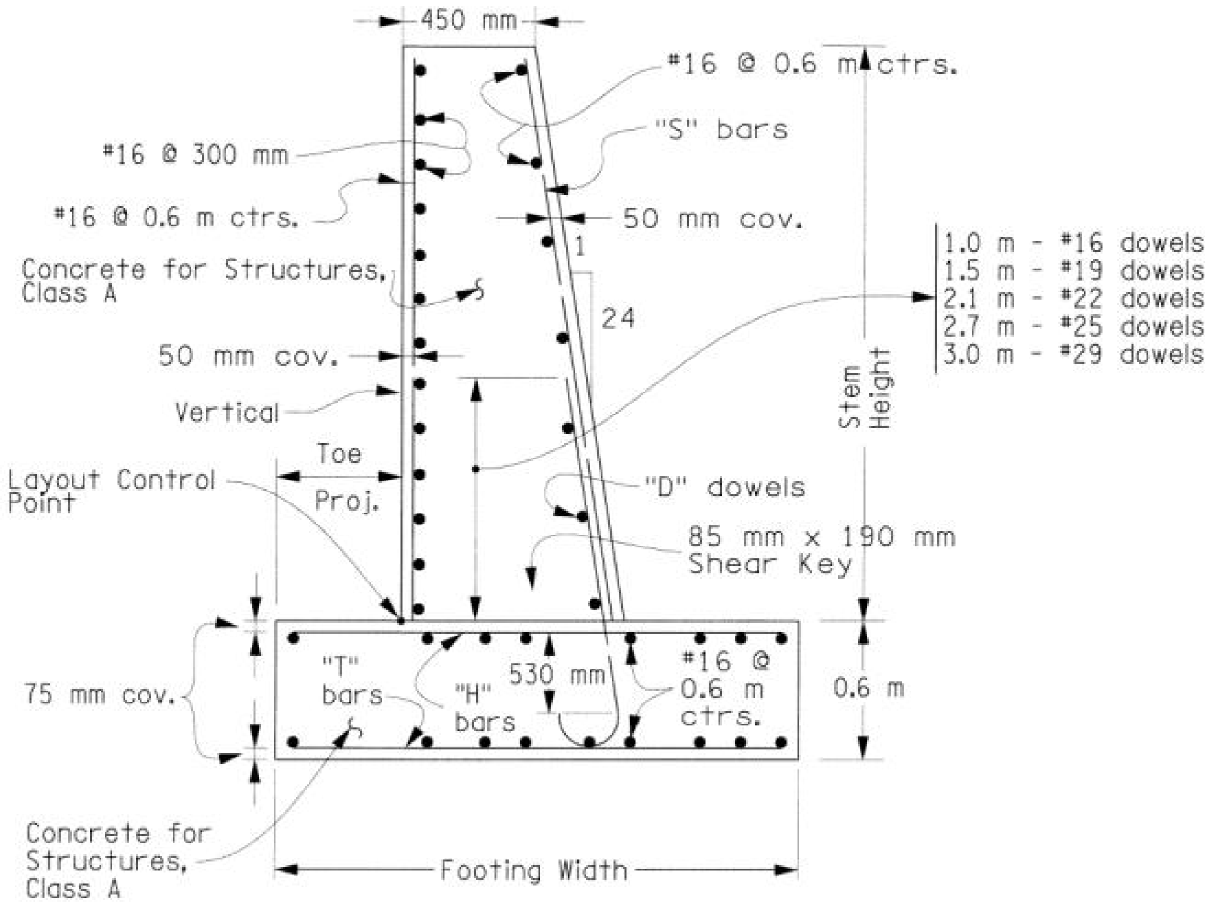
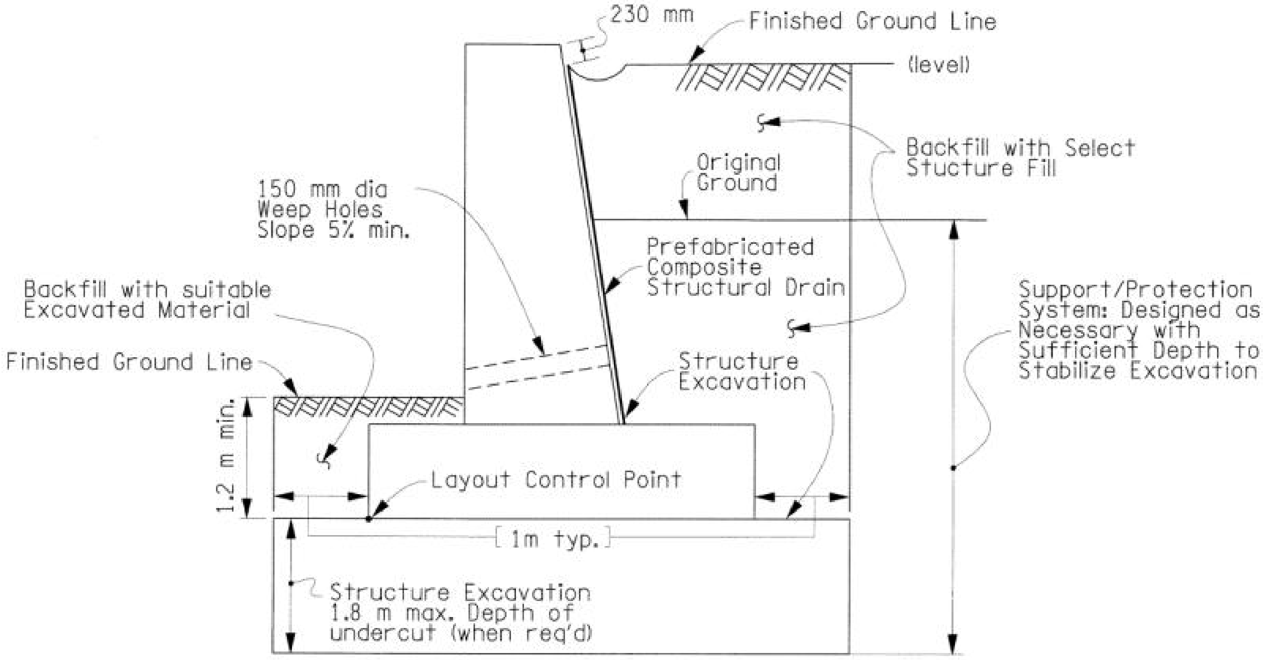


Figure 9-54a Section Through Wall

Cantilever Wall Notes:
See 9.4.2.3.2.1 BASIS OF DESIGN



Notes:
1. When required, footing excavation shall be undercut and backfilled with Select Structure Fill.
2. For excavation, drainage and backfill details, see BD-EE1 R1 & 3 R1

Figure 9-54b Section Foundation & Backfill Preparation

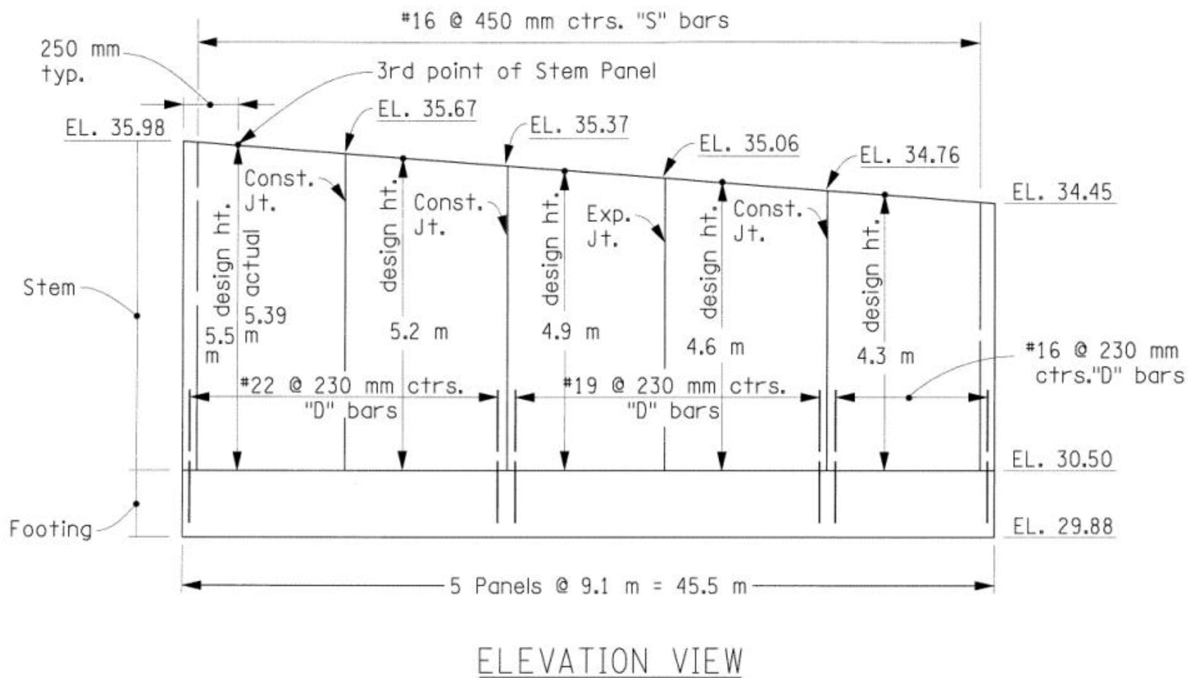


Figure 9-54c Illustrative Example (Slightly Sloping Wall – Elevation View)

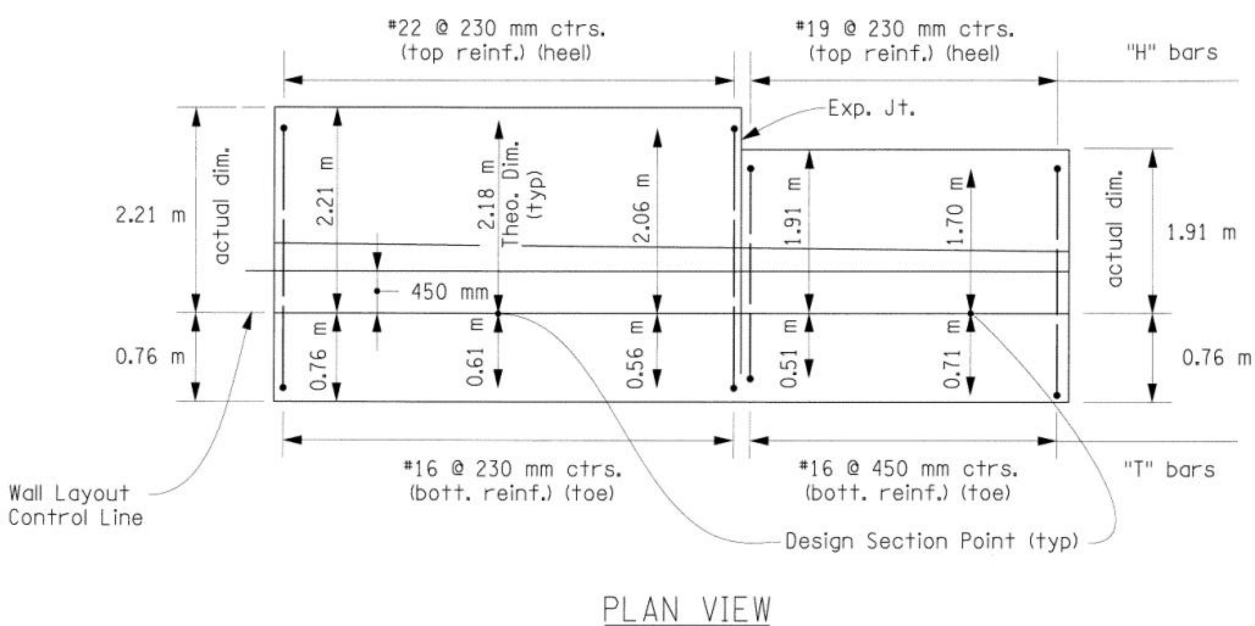


Figure 9-54d Illustrative Example (Slightly Sloping Wall - Plan View)

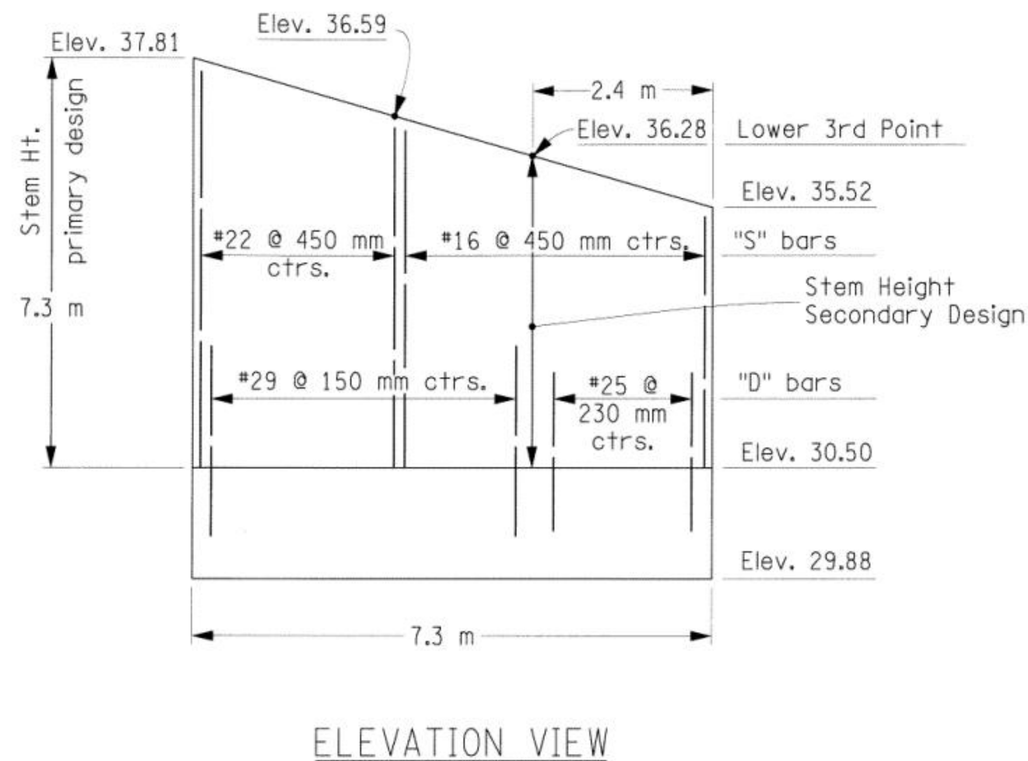


Figure 9-54e Illustrative Example (Steeply Sloping Wall – Elevation View)

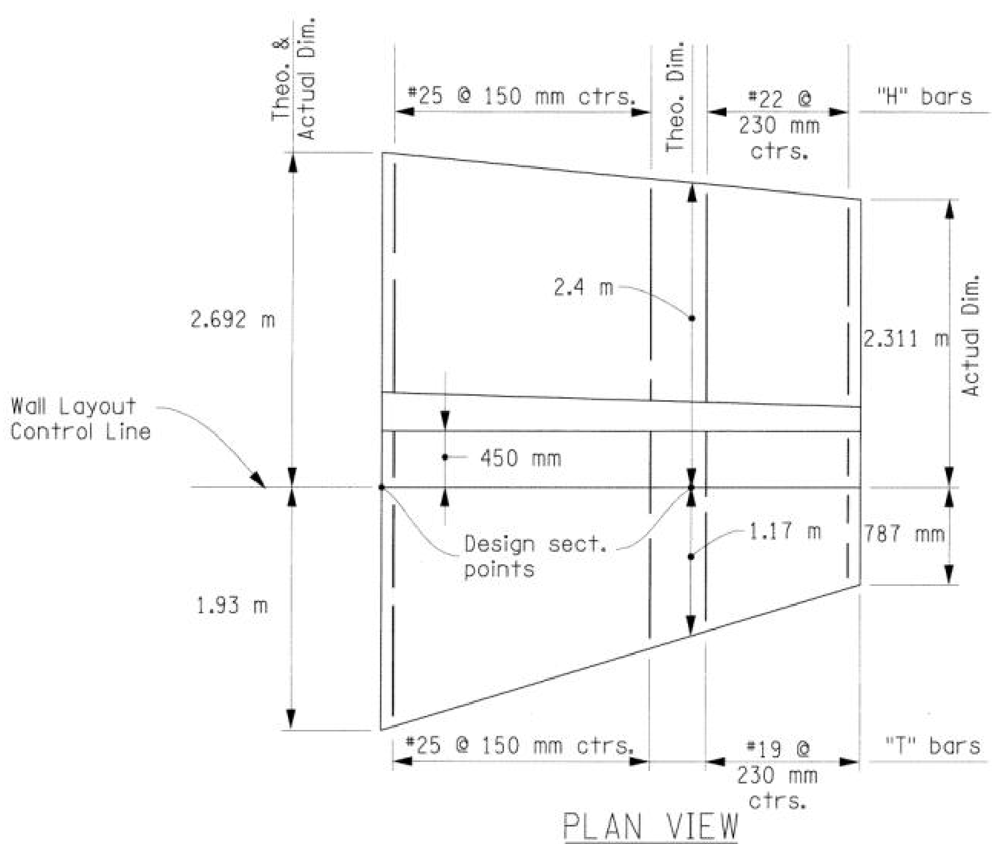


Figure 9-54f Illustrative Example (Steeply Sloping Wall – Plan View)

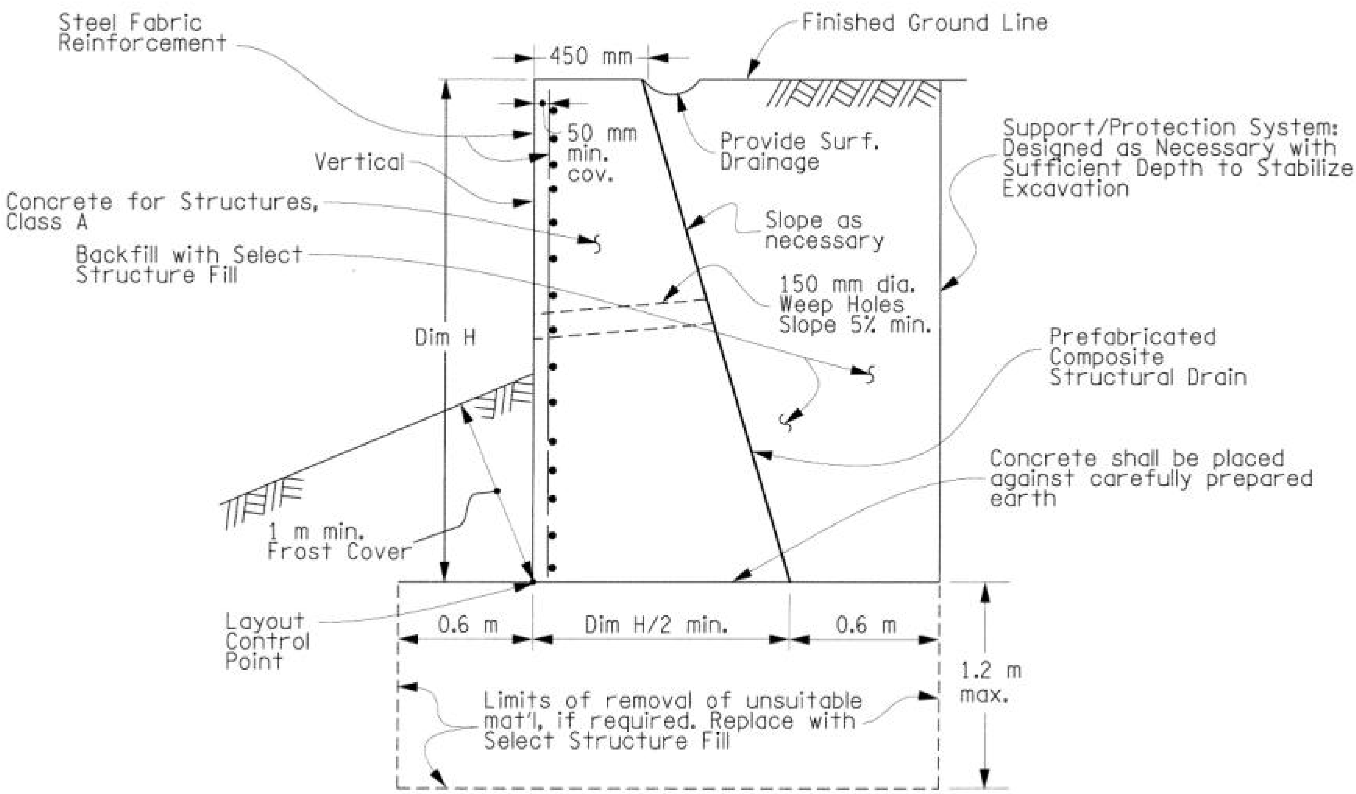


Figure 9-54g Section Gravity Wall

Gravity Wall Notes:

1. The gravity wall section may be used as an alternate to the cantilever section where it will not be compatible with adjacent work and the wall height (dimension "H") is a minimum of 1.2 m and a maximum of 3.66 m.
2. Required Soil Capacity:
   Allowable Pressure: 150.0 kPa.
   Friction Coefficient: 0.35.
3. Steel fabric reinforcement shall be placed as shown behind the full area of the vertical form.
4. Maximum Panel Length Between Construction Joints: 6.1 m.
5. Maximum Length Between Expansion Joints: 30.5 m.
6. All joints shall be formed with an adequate shear key.

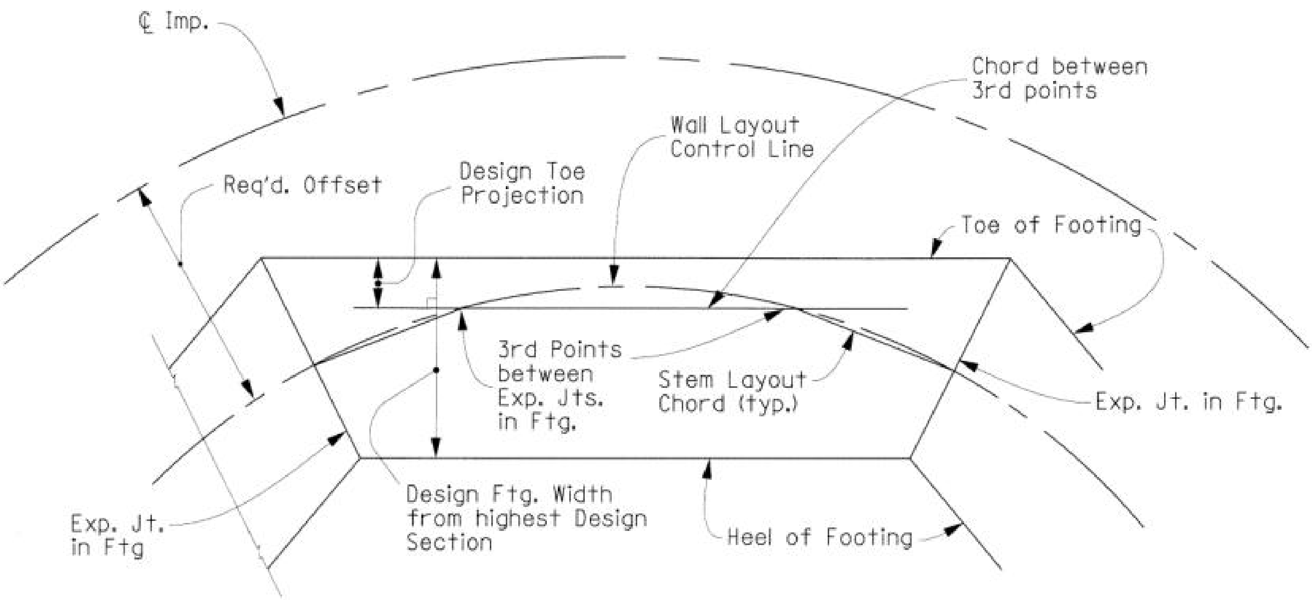


Figure 9-54h Layout Control

<u>Layout Control Notes:</u>

Walls situated adjacent to a curved centerline of improvement should be detailed as a series of straight segmental walls. Where it is necessary that the wall layout control line be curved itself, one of the following layout methods shall be used:

1. Curved walls (Radius grater than 138 m): Layout on chords, unless horizontal clearance is critical. Locate footing as shown above by perpendicular offsets to a chord between the third points of the footing pour between expansion joints. Lay out stem panels on chords between the contraction joints.
2. Curved walls (Radius 138 m or less): Lay out both footing and stem on curve. If the distance between footing expansion joints is reduced so that it does not exceed one-fifth of the radius, lay out on chords may be used on radii shorter than 138 m.

Table 9-7 Table of Proportions

Design Parameters						Reinforcement							
Stem Height (m)	Req'd Min. Soil Press. (kPa)	Design Coeff. of Friction	Conc. Unit Quant. (m^3 per m of wall)	Ftg. Width (m)	Toe Proj. (m)	"T" Bars		"H" Bars		"D" Bars		"S" Bars	
						Size	Spac (mm)	Size	Spac (mm)	Size	Spac (mm)	Size	Spac (mm)
1.83	100.0	0.50	1.67	1.25	0.203	#16	450	#16	450	#16	450	#16	450
1.83	100.0	0.35	1.85	1.55	0.254	#16	450	#16	450	#16	450	#16	450
2.13	100.0	0.50	1.89	1.35	0.406	#16	450	#16	450	#16	450	#16	450
2.13	100.0	0.35	2.09	1.68	0.254	#16	450	#16	450	#16	450	#16	450
2.44	150.0	0.50	2.07	1.37	0.203	#16	450	#16	450	#16	450	#16	450
2.44	100.0	0.50	2.20	1.57	0.508	#16	450	#16	450	#16	450	#16	450
2.44	100.0	0.35	2.42	1.93	0.305	#16	450	#16	450	#16	450	#16	450
2.74	150.0	0.50	2.37	1.57	0.254	#16	450	#16	450	#16	450	#16	450
2.74	100.0	0.50	2.49	1.78	0.610	#16	450	#16	450	#16	450	#16	450
2.74	100.0	0.35	2.73	2.16	0.356	#16	450	#16	450	#16	450	#16	450
3.05	150.0	0.50	2.64	1.73	0.457	#16	450	#16	450	#16	450	#16	450
3.05	100.0	0.50	2.81	2.01	0.711	#16	450	#16	450	#16	450	#16	450
3.05	100.0	0.35	3.06	2.41	0.457	#16	450	#16	450	#16	450	#16	450
3.35	150.0	0.50	2.96	1.96	0.457	#16	450	#16	450	#16	450	#16	450
3.35	100.0	0.50	3.11	2.21	0.813	#16	450	#16	450	#16	450	#16	450
3.35	100.0	0.35	3.42	2.72	0.610	#16	450	#19	450	#16	450	#16	450
3.66	150.0	0.50	3.25	2.13	0.508	#16	450	#19	450	#16	230	#16	450
3.66	100.0	0.50	3.44	2.44	0.914	#19	450	#16	450	#16	230	#16	450
3.66	150.0	0.35	3.58	2.67	0.457	#16	450	#16	230	#16	230	#16	450
3.66	100.0	0.35	3.73	2.92	0.660	#16	450	#19	450	#16	230	#16	450
3.96	150.0	0.50	3.52	2.26	0.660	#16	450	#19	450	#16	230	#16	450
3.96	100.0	0.50	3.75	2.64	1.106	#19	450	#16	450	#16	230	#16	450
3.96	150.0	0.35	3.91	2.90	0.457	#16	450	#19	230	#16	230	#16	450
3.96	100.0	0.35	4.09	3.20	0.864	#16	450	#16	230	#16	230	#16	450
4.27	150.0	0.50	3.80	2.41	0.711	#19	450	#16	230	#16	230	#16	450
4.27	100.0	0.50	4.08	2.87	1.118	#16	230	#19	450	#16	230	#16	450
4.27	150.0	0.35	4.25	3.15	0.559	#16	450	#19	230	#16	230	#16	450
4.27	100.0	0.35	4.39	3.38	0.914	#19	450	#16	230	#16	230	#16	450
4.57	200.0	0.50	3.99	2.41	0.508	#16	450	#19	230	#19	230	#16	450
4.57	150.0	0.50	4.15	2.67	0.813	#19	450	#16	230	#19	230	#16	450
4.57	100.0	0.50	4.40	3.07	1.219	#19	230	#19	450	#19	230	#16	450
4.57	150.0	0.35	4.59	3.38	0.610	#16	450	#22	230	#19	230	#16	450
4.57	100.0	0.35	4.77	3.68	1.016	#16	230	#19	230	#19	230	#16	450
4.88	200.0	0.50	4.32	2.62	0.559	#16	450	#22	230	#19	230	#16	450
4.88	150.0	0.50	4.53	2.95	0.914	#16	230	#19	230	#19	230	#16	450
4.88	100.0	0.50	4.74	3.30	1.321	#19	230	#16	230	#19	230	#16	450
4.88	150.0	0.35	4.94	3.63	0.660	#16	450	#22	230	#19	230	#16	450
4.88	100.0	0.35	5.17	4.01	1.118	#16	230	#19	230	#19	230	#16	450

Table 9-7 (cont.) Table of Proportions

Design Parameters						Reinforcement							
						"T" Bars		"H" Bars		"D" Bars		"S" Bars	
Stem Height (m)	Req'd Min. Soil Press. (kPa)	Design Coeff. of Friction	Conc. Unit Quant. (m^3 per m of wall)	Ftg. Width (m)	Toe Proj. (m)	Size	Spac (mm)	Size	Spac (mm)	Size	Spac (mm)	Size	Spac (mm)
5.18	200.0	0.50	4.63	2.79	0.610	#19	450	#22	230	#22	230	#16	450
5.18	150.0	0.50	4.80	3.07	0.965	#16	230	#19	230	#22	230	#16	450
5.18	100.0	0.50	5.13	3.61	1.473	#22	230	#16	230	#22	230	#16	450
5.18	150.0	0.35	5.28	3.86	0.711	#19	450	#25	230	#22	230	#16	450
5.18	100.0	0.35	5.51	4.24	1.372	#19	230	#19	230	#22	230	#16	450
5.49	200.0	0.50	4.95	2.97	0.762	#16	230	#22	230	#22	230	#16	450
5.49	150.0	0.50	5.18	3.35	1.067	#19	230	#22	230	#22	230	#16	450
5.49	100.0	0.50	5.48	3.84	1.575	#22	230	#19	230	#22	230	#16	450
5.49	150.0	0.35	5.64	4.11	0.762	#19	450	#25	230	#22	230	#16	450
5.79	250.0	0.50	5.13	2.92	0.559	#19	450	#29	230	#25	230	#16	450
5.79	200.0	0.50	5.25	3.12	0.813	#16	230	#25	230	#25	230	#16	450
5.79	150.0	0.50	5.55	3.61	1.168	#19	230	#22	230	#25	230	#16	450
5.79	100.0	0.50	5.81	4.04	1.676	#25	230	#19	230	#25	230	#16	450
5.79	150.0	0.35	5.99	4.34	0.914	#16	230	#29	230	#25	230	#16	450
6.10	250.0	0.50	5.48	3.15	0.610	#19	450	#29	230	#25	230	#16	450
6.10	200.0	0.50	5.67	3.45	0.914	#19	230	#25	230	#25	230	#16	450
6.10	150.0	0.50	5.86	3.76	1.219	#22	230	#25	230	#25	230	#16	450
6.10	100.0	0.50	6.17	4.27	1.778	#25	230	#22	230	#25	230	#16	450
6.10	150.0	0.35	6.35	4.57	0.965	#19	230	#29	230	#25	230	#16	450
6.40	250.0	0.50	5.82	3.35	0.660	#16	230	#32	230	#29	230	#22	450
6.40	200.0	0.50	5.98	3.61	0.965	#19	230	#29	230	#29	230	#22	450
6.40	150.0	0.50	6.22	4.01	1.321	#22	230	#25	230	#29	230	#22	450
6.40	150.0	0.35	6.69	4.78	1.016	#19	230	#32	230	#29	230	#22	450
6.71	250.0	0.50	6.19	3.58	0.711	#16	230	#29	150	#25	150	#22	450
6.71	200.0	0.50	6.31	3.78	1.219	#25	230	#22	150	#25	150	#22	450
6.71	150.0	0.50	6.62	4.29	1.778	#29	230	#19	150	#25	150	#22	450
6.71	150.0	0.35	7.05	5.00	1.219	#22	230	#25	150	#25	150	#22	450
7.01	300.0	0.50	6.35	3.48	0.610	#16	230	#29	150	#25	150	#22	450
7.01	250.0	0.50	6.52	3.76	0.864	#19	230	#29	150	#25	150	#22	450
7.01	200.0	0.50	6.61	3.91	1.270	#25	230	#25	150	#25	150	#22	450
7.01	150.0	0.50	6.97	4.50	1.880	#25	150	#22	150	#25	150	#22	450
7.01	150.0	0.35	7.39	5.18	1.270	#22	230	#29	150	#25	150	#22	450
7.32	300.0	0.50	6.66	3.61	0.711	#19	230	#32	150	#29	150	#22	450
7.32	250.0	0.50	6.88	3.96	1.067	#22	230	#29	150	#29	150	#22	450
7.32	200.0	0.50	7.02	4.19	1.372	#29	230	#25	150	#29	150	#22	450
7.32	150.0	0.50	7.28	4.62	1.930	#25	150	#22	150	#29	150	#22	450
7.32	200.0	0.35	7.58	5.11	0.965	#19	230	#32	150	#29	150	#22	450
7.32	150.0	0.35	7.93	5.69	1.372	#25	230	#29	150	#29	150	#22	450

9.4.3 Wall Selection Process

Personnel from the Geotechnical Engineering Bureau, together with the Regional Geotechnical Engineer, are responsible for visiting proposed project sites and then discussing and providing Regional designers and construction personnel with the appropriate selection and design for the various wall system(s) chosen for the project. Selection of a retaining wall system for a particular site is based on the criteria established in **FHWA publication "Geotechnical Circular No. 2, Earth Retaining Structures, FHWA-SA-96-038**."[14m] In general, selection is based primarily on whether a wall is going to be placed in a cut or fill section, whether a wall should or must be externally stabilized, internally stabilized, or a gravity type, and whether the wall will be permanent, temporary, or interim. Other factors such as aesthetics, economy, etc., come into play to make the final decision.

All these treatments have particular advantages and disadvantages. The designer should consult the Regional Geotechnical Engineer when wall systems are under consideration.

9.5 DEEP FOUNDATIONS

9.5.1 Drilled Shafts

A drilled shaft is a foundation which is constructed by placing fresh concrete in a drilled hole. Reinforcing steel can be installed in the excavation, if desired, prior to placing the concrete. Drilled shafts may also be referred to as caissons, drilled piers, or bored piles (although in NYS, bored piles, or bored-in piles, are arbitrarily defined as being less than 300 mm in diameter). Guidelines for the design and construction of drilled shafts are summarized in a manual published by the Geotechnical Engineering Bureau[20e].

The advantages[20a] of drilled shafts over conventional driven piles are:

- Excavated material can be examined for comparison to the design assumptions.
- Applicable to a wide variety of soil conditions (e.g. can be progressed through obstructions that would prevent a pile from being driven).
- Very large loads can be carried by a single shaft so that a pile cap is often not needed.
- Allows a non-destructive evaluation of installed shaft for quality control and quality assurance purposes.

The disadvantages[20a] are:

- The successful installation depends on the skills and equipment of a specialty Contractor. The average prime Contractor cannot install these shafts.
- Drilled shafts are much more expensive than conventional driven piles.
- Thorough inspection is essential to catch potential problems early.
- A single drilled shaft is frequently designed to replace a number of driven piles, eliminating the redundancy present in a group of driven piles, which again requires diligence and expertise in construction and inspection.

The design of a drilled shaft is based on a static analysis of the shaft element installed in a soil profile developed through interpretation of the subsurface explorations obtained at the site. During foundation design, if the Geotechnical Engineering Bureau determines that the soil cannot support the anticipated loads of a shallow foundation, the foundation can be designed to transfer the loads to a more suitable material at a greater depth by means of drilled shafts.

Drilled shafts were originally designed as end-bearing shafts that act as columns and derive their resistance primarily from the material on which the tips rest. Design methods have been developed to predict the skin friction resistance (shaft surface to the surrounding soil acting along the length of the shaft).

A drilled shaft design is an iterative process between the structure designer and geotechnical designer. The structural designer is required to provide the geotechnical designer with the vertical load, longitudinal shear, transverse shear, longitudinal moment and transverse moment for the AASHTO loading groups on the foundation cap. The structural designer should provide a sketch of the proposed drilled shaft cross-section showing sizes, number, and spacing of rebars, including any changes in section with depth. The structure designer should indicate the design load and ultimate load of the member and supply a sketch of the actual shaft group layout, including all shaft and row spacing. The geotechnical designer will analyze the subsurface profile and determine the required length of the drilled shafts and also perform a lateral load analysis[20c]. The resulting deflection from the lateral load analysis will be provided to the structural designer to determine if it is acceptable. If it is not acceptable, the iterative design process continues with the structure designer tweaking the cross sectional elements of the drilled shaft and re-submitting to the geotechnical designer for evaluation.

The Foundation Design Report will provide the designer with the maximum allowable axial load per shaft developed during the design of the foundation along with the estimated shaft depths and notes to the Contractor stating what capacity is to be achieved from the shaft installation process, estimated shaft length, estimated rock socket length (if appropriate), and explanation and outline of testing (e.g., crosshole sonic logging) [20k].

Figure 9-55a Drilled Shaft Installation
(Barge Set-Up)

Figure 9-55b Drilled Shaft Installation (Drilling & Excavation)

Figure 9-55c Drilled Shaft Reinforcement (Rebar Cage & CSL Access Tubes)

Figure 9-55d Drilled Shaft Installation (Concrete Pour)

Figure 9-55e Drilled Shaft Layout

Figure 9-55f Crosshole Sonic Log Testing (Ultrasonic Probe Installation)

Figure 9-55g Crosshole Sonic Log Testing (Mobile Test Lab)

9.5.2 **Driven Piles**

Driven piles are used to support walls, abutments, and other foundation elements when spread footings are not appropriate. The Structures Design and Construction Division and Geotechnical Engineering Bureau should be consulted.

The design of a pile is based on a static analysis of the pile element installed in a soil profile developed through interpretation of the subsurface explorations obtained at the site. During foundation design, if the Geotechnical Engineering Bureau determines that the soil cannot support the anticipated loads of a shallow foundation, the foundation can be designed to transfer the loads to a more suitable material at a greater depth by means of driven piles.

Driven piles may be categorized based on load transfer method into two groups: friction piles and end-bearing piles. Friction piles are piles that derive its resistance primarily from skin friction (pile surface to the surrounding soil acting along the pile shaft). End-bearing piles are piles that act as columns and derive its resistance primarily from the material on which the tip rests.

A driven pile foundation has to be installed to meet the design requirements for compressive, lateral and uplift capacity[21a]. The Foundation Design Report will provide the designer with the maximum allowable load per pile for the design of the foundation along with the estimated pile lengths and a note to the Contractor stating what capacity is to be achieved from the pile driving operations. A pile may also be designed to a predetermined length. The Foundation Design Report will provide the designer with an estimated vertical pile length and a note to the Contractor stating the minimum length to be obtained from the pile driving operations.

Equally important to installing a driven pile foundation to meet its design requirements is to avoid pile damage or foundation cost overruns by excessive driving. These objectives can all be satisfactorily achieved by use of wave equation analysis, dynamic monitoring of pile driving and static load testing[21a]. Wave Equation Analysis of Pile Foundations (WEAP) is a program which simulates a foundation pile under the action of an impact pile driving hammer. The Geotechnical Engineering Bureau utilizes the program during the initial foundation design to assure the recommended pile may be installed without sustaining damage. In addition, the Geotechnical Engineering Bureau utilizes the program during construction to evaluate the Contractors proposed pile driving operation.

Dynamic Pile Load Test[21b](DPLT) of a pile may be recommended by the Geotechnical Engineering Bureau to estimate the static axial compressive pile capacity from dynamic measurements of pile strain and acceleration. The Foundation Design Report will provide the designer with the number of, and appropriate item number for, DPLT's to be performed on the proposed foundations and a note to the Contractor stating that DPLT's will be conducted (typically by representatives of the Geotechnical Engineering Bureau) on the foundation piles to alert the Contractor of the required monitoring installation.

Static Pile Load Test[21b](SPLT) of a pile may be recommended by the Geotechnical Engineering Bureau to measure the response of a pile under applied load. The SPLT provides the best means of determining pile capacity. The Foundation Design Report will provide the designer with the number of, and appropriate item number for, SPLT's to be performed on the proposed foundations and a note to the Contractor requiring the performance of SPLT's on the foundation piles.

Figure 9-56a Pile Driving Operation

Figure 9-56b Dynamic Pile Load Test Set-Up

Figure 9-56c DPLT: Strain Transducer and Accelerometer Bolted To CIP Pipe Pile

Figure 9-56d DPLT: Pile Driving Analyzer (Pile Dynamics, Inc.)

9.5.3 **Micropiles**

Micropiles are nondisplacement piles formed by drilling a hole in the ground, and filling it with steel reinforcing and grout under pressure (pressure grouting). Typical diameters range from 190 mm to 230 mm. Typical design loads range from 440 kN in a clayey silt soil, to 1330 kN in a dense granular soil, with 710 kN to 900 kN being typical design loads for most competent soils.

The advantages of these types of piles over conventional driven piles are:

- Vibrations from installation are minimized.
- Can be installed in low-overhead conditions.
- Can be used to underpin existing structures.
- Can be progressed through obstructions that would prevent a pile from being driven.

The disadvantages are:

- Micropiles are much more expensive than conventional driven piles.
- The successful installation depends on the skills and equipment of a specialty Contractor. The average prime Contractor cannot install these piles.
- Thorough inspection is essential to catch potential problems early.
- Actual load capacity can only be verified through a Static Pile Load Test (SPLT). These can be very expensive, and take several days to set up and run. A driven pile is a tested pile.
- Failed load tests cause problems. How do you differentiate the good piles from the bad ones? Expertise and persistence is required to review inspection records, soil conditions, etc.

Micropiles are usually selected when a pile-supported foundation is necessary, but there are vibration concerns or restrictions, or obstructions, or there are low-overhead conditions. Typically, though not always, this occurs in the more urban areas of the state.

Micropiles get their capacity through side friction. Literature states, and pile load tests have verified, that very little of a micropiles capacity is derived through end bearing. Because the pile is not driven, the soil at the tip is not compacted. The pile would have to move downward over 25 mm to develop full, end-bearing resistance, at which point the majority of the side resistance would have been lost, resulting in a failed pile.

A 440 kN design load is relatively easy to achieve, even if the contractor uses less than optimal procedures. However, achieving design loads above 440 kN requires the use of proper equipment and procedures, even in a competent soil.

Micropile capacities can be increased by increasing the length or the diameter or by post grouting. Post grouting is performed by attaching a small diameter PVC tube along the reinforcement of a micropile. After the grout for the pile has set for a day or so, water is pumped through the tube at high pressure to crack the grout. Then, grout is pumped through the tube. This process can be repeated many times to continue increasing the side resistance. Be aware that post-grouting provides a more dramatic increase in side resistance in clayey soils than in granular soils. Also, excessive post-grouting can cause damage behind and above the piles, such as damaging utilities or causing walls to deflect or slabs to lift.

There are two micropile specifications: one is set up as a performance-based, Contractor designed foundation element and the other is a method-based, State designed foundation element. The Contractor designed micropile specification is the most desirable approach as it allows for specialty contractors to introduce innovative, cost-competitive solutions, making use of the latest available construction techniques. With this specification, design loads for the micropiles are specified in the contract documents and the Contractor hires a professional engineer, licensed to practice in New York State, to design the piles. Therefore, this allows the Contractor to use his/her expertise to select the pile diameter, length, and any other procedure to achieve the capacities shown in the contract documents.

Although this performance-based approach is the most desirable, the Geotechnical Engineering Bureau has seen a need for an additional specification for a micropile to apply only to certain subsurface profiles which could minimize delays during construction. With this specification, the micropile will be designed and detailed in the contract documents, eliminating any design disagreements in the submittal process. For these instances, the Contractor is only required to submit the proposed method of installation procedures.

Sometimes, to ensure that the pile's capacity is provided by a certain soil layer (i.e., to get below a liquefiable layer, or a compressible layer), it is necessary to specify a minimum pile length, or state that the capacity needs to be developed below a certain elevation. This can become problematic when performing a SPLT to verify capacity. Be aware that the pile will pick up side resistance during testing in the undesirable layer, which will affect the load-settlement curve. To subtract the effect that this layer would have, it is necessary to require the use of **telltales** in the pile. More information on telltales can be found in the SPLT manual in the Structure Foundation Section.

Figure 9-57a Micropile Installation

Figure 9-57b Micropile Installation (Down-The-Hole Hammer)

Figure 9-57c Micropile Installation (Centralizer with Telltales)

Figure 9-57d Micropile Installation (Footing Connection)

Figure 9-57e Static Pile Load Test of Micropile

Figure 9-57f Static Pile Load Test of Micropile

9.6 BUILDING CONDITION SURVEY AND VIBRATION MONITORING (NON BLASTING)

9.6.1 General

The revised special specification for building condition survey(s) and vibration monitoring (non blasting) is not intended to be used as a pay item for vibration monitoring during blasting operations. Monitoring vibrations generated via blasting operations are controlled by the Standard Specifications §203-3.05 Rock Excavation C. Explosive Loading Limits and are described in the Geotechnical Engineering Manual (GEM-22) Procedures for Blasting[10a].

Non blasting construction operations (e.g., excavation, pavement removal, backfill and compaction, demolition, driving of piles and sheeting, etc.) may damage or distress adjacent sensitive buildings, structures, or utilities. The special specification for building condition survey and vibration monitoring is intended to assess the condition of the building, structure or utility prior to these adjacent construction operations to develop a condition baseline. A companion special specification includes requirements for monitoring vibrations to record the intensity of the adjacent construction operation(s). This information may be used to resolve disputes.

Figure 9-58a Vibration Monitoring Set-Up

Figure 9-58b Instatel's Blastmate III (Includes full keyboard and thermal printer)

Figure 9-58c Vibration Monitoring (Microphone)

Figure 9-58d Vibration Monitoring (Triaxial Geophone)

9.6.2 **Design Considerations**

The use of the special specification with respect to "adjacent sensitive buildings, structures, or utilities" should be discussed with the Geotechnical Engineering Bureau Engineering Geology Section. Ground vibrations from construction activities very rarely reach the levels that can damage structures, but can be within the range of human perception in buildings very close to the site. A possible exception is the case of old, fragile buildings of historical significance where special care must be taken to avoid damage. The construction activities that typically generate the most severe vibrations are blasting and impact pile driving[23b]. If, due to project constraints, construction activities are scheduled adjacent to such sensitive buildings, the following is recommended:

9.6.2.1 **Adjacent Sensitive Structure**

The assessment of the potential for damage is two-fold. Vibrations propagate from a piece of construction equipment through the ground to a distant vibration-sensitive receiver predominately by means of Rayleigh (surface) waves and secondarily by body (shear and compressional) waves. The amplitude of these waves diminishes with distance from the source. This attenuation is due to two factors: expansion of the wave front (geometrical attenuation) and dissipation of energy within the soil itself (material damping) [23a]. Material damping in soil is a function of many parameters, including soil type, moisture content, and temperature. The designer will assess the potential for damage by determining the assumed geometrical attenuation or distance from the source to the receiver.

The most common generic model of construction vibrations as a function of distance was developed by Wiss (1981) [23a], as shown in Figure 9-59:

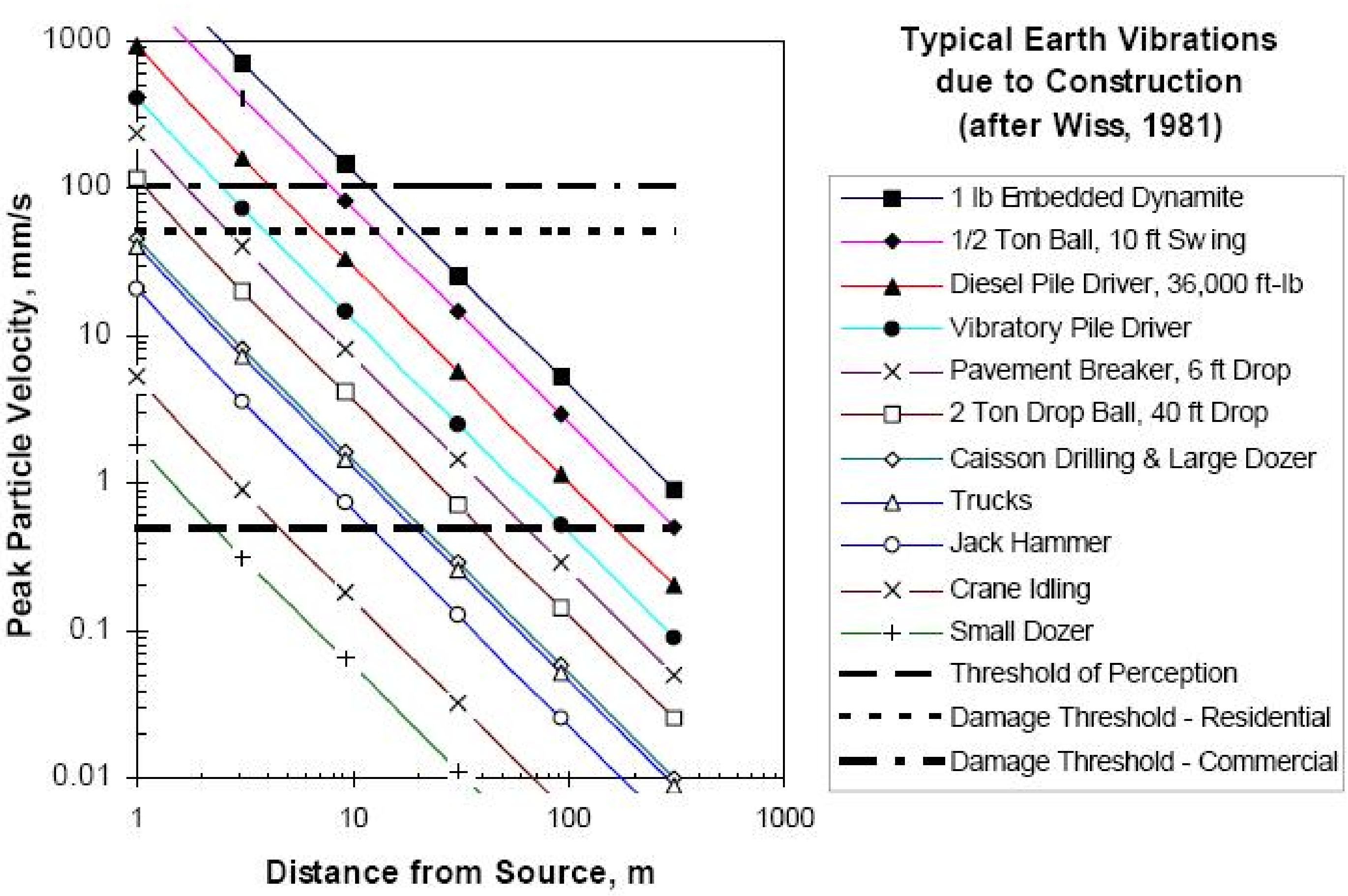


Figure 9-59[23a] – Construction Vibrations as a Function of Distance, after Wiss (1981)

Various types of construction equipment have been measured under a wide variety of construction activities with an average of source levels reported in terms of velocity levels[2]. It should be noted that there is a considerable variation in reported ground vibration levels from construction activities. However, reasonable estimates may be made for a wide range of soil conditions. For example, the upper range of an impact pile driver at a distance of 7.6 m from measured data is 38.6 mm/sec (1.518 in/sec) PPV[23b]. To compare these results with Figure 9-59, using the distance from the source as 7.6 m and the construction activity of a diesel pile driver, the resulting peak particle velocity is approximately 40 mm/sec.

The 7.6 m distance from the source to the receiver and the resulting measured PPV of 38.6 mm/sec is a good starting limit for the designer to use. As a comparison, the designer should note that for blasting operations, the Standard Specifications §203-3.05 Rock Excavation C. Explosive Loading Limits states “in the absence of more stringent requirements, the maximum quantity of explosives allowed per delay period shall be based on a maximum particle velocity of 50 mm/sec at the nearest structure to be protected”. Considering the impact pile driver ranks high on the list of construction activities producing ground vibrations yet yields a PPV of 38.6 mm/sec at a distance of 7.6 m, this buffer is a conservative distance.

If the project requires a construction activity adjacent to a sensitive structure closer than the 7.6 m limit described above or the designer has a concern with the existing condition and/or there is an historic significance, consult with the Geotechnical Engineering Bureau’s Engineering Geology Section.

9.6.2.2 **Consultation**

If the designer determines the adjacent distance may pose a potential for damage, consultation with the Geotechnical Engineering Bureau's Engineering Geology Section is recommended. Site specific information will be reviewed, including material damping (soil type, moisture content and temperature) and appropriate maximum allowable peak particle velocity (PPV) to be assigned to the structure.

Figure 9-60 Vibration Monitoring of Adjacent "Suspect" Building

9.6.2.3 Action

If the Designer and Engineering Geologist determine the need for the special specification, the following Special Note entitled Vibration Criteria shall also be included in the contract documents:

VIBRATION CRITERIA

The Contractor's attention is directed to the close proximity of the existing (buildings, structures, utilities) located ______________________. Excavation, pavement removal, backfill and compaction, demolition, driving of piles and sheeting, and any other construction operations shall be conducted in a manner which will not damage or distress any of the above, including but not limited to, adjacent buildings and structures, historic structures, utilities or tunnels. Any damage caused by or related to the Contractor's operations shall be repaired by the Contractor at no additional cost to the State.

The Contractor is required to engage the services of a New York State licensed Professional Engineer to conduct a condition survey of the existing (buildings, structures, utilities) indicated in Table 1 of this Special Note and an experienced vibration monitoring Consultant to measure peak particle velocities prior to, and during construction operations. The Contractor must perform all work in a manner that will limit construction vibration at the specified locations to within the limits set forth within this Note or the limits determined by his Professional Engineer, whichever is less.

Table 1

Structure & Location	Vibration Measurement Locations	Maximum Allowable Peak Particle Velocity (PPV)
	(Closest point on the dwelling to construction operation) (At a distance from a given pile to model the distance from closest pile to the building: monitoring impact at that distance to make adjustments to pile driving operation as work proceeds toward building) (Etc.)	(5.08 cm/sec (2.0 in/sec)) (2.54 cm/sec (1.0 in/sec)) (Etc.)

9.7 CONTRACT INFORMATION

Contract documents represent many months of analysis and design. They must clearly, concisely, and completely tell the bidder what work will be required.

9.7.1 **Earthwork Summary Sheets**

Earthwork Summary Sheets shall be provided in the contract plans or proposal, depending on the quantity of items 203.02 and/or 203.03 in the contract and the project length. Earthwork Summary Sheets are an important component of the contract documents because they provide the contractor with a summary of:

- Earthwork (i.e., a pay item quantity summary of item 203.02 Unclassified Excavation and Disposal, and item 203.03 Embankment In Place).
- Trench and culvert excavation (i.e., a pay item quantity summary of item 206.02 Trench and Culvert Excavation, and item 206.04 Trench and Culvert Excavation - O.G.).

In addition, they provide the:

- Contractor with a breakdown of the rock and nonrock quantity associated with the overall item quantity.
- Basis for any item adjustment under Standard Specification Section 104-04 Significant Changes in the Character of Work, D. Composite Items.

Earthwork Summary Sheets ES-1 and ES-2 shall be provided in the plans for all projects over 150 m in length which have over 4,000 m^3 of the sum of items 203.02 (Unclassified Excavation and Disposal) and 203.03 (Embankment In Place). When these criteria are not met, Earthwork Summary ES-1A and ES-2A (letter size paper) shall be prepared and provided for inclusion in the proposal. If items 203.02 or 202.03 are not part of the project, ES-2A shall be prepared when items 206.02 and/or 206.04 are in the estimate.

Guidance regarding completing these sheets is as follows:

- All five columns of the Summary of Trench and Culvert Excavation Table (on ES-1/ ES-2A as applicable) shall be completed when items 206.02 and/or 206.04 are in the engineer's estimate.
- All six columns of the Summary of Earthwork Table (on ES-1/ ES-2A as applicable) and all columns of ES-2 shall be completed when items 203.02 and/or 203.03 are in the engineer's estimate.
- Do not leave any blanks. Fill in blanks with zero's where appropriate.

Earthwork Summary Sheets are illustrated in Chapter 21, Appendix E, of this manual.

9.7.2 **Specifications**

The Standard Specifications and various Special Specifications include many references to information to be shown or indicated on the contract plans and/or in the proposal. These references most often refer to an illustration or listing of details and requirements, payment lines, or estimates of quantities for various segments of the work. This is done, particularly with soil- or foundation-related specifications, to allow for a generic specification to be used despite the vastly changing subsurface conditions throughout the State dictating site-specific geotechnical designs.

In the interest of clarity and completeness, the designer should carefully review the specifications for each contract item, to ensure that necessary and appropriate information is included in the plans and/or proposal.

9.7.3 **Proposal**

The proposal contains, in addition to other items, all Special Specifications, special notes, and the list of payment items and quantities.

9.7.3.1 Special Specifications

Special soil items, including modifications to existing Standard Specifications, should be prepared in consultation with the Regional Geotechnical Engineer to ensure that they adequately describe the work required.

9.7.3.2 Laboratory Building

The contract should include a payment item for a laboratory building when one is needed. It should be included on any project that will require significant field testing or processing samples for testing. On projects requiring minimal field testing or processing of samples, the Engineer's field office will serve this purpose. The Regional Geotechnical Engineer should be consulted regarding need for a laboratory building.

9.7.3.3 Special Notes

The special notes should describe any unusual soil conditions that are anticipated. This information may be extracted from geotechnical reports prepared by the Regional Geotechnical Engineer during progress of the design.

9.7.3.4 Supplemental Information Available to Bidders

Chapter 21 of this manual indicates the material to be made available to bidders. Rock cores should be made available for examination by prospective bidders at the Regional office. Logs of all subsurface information should also be made available to all bidders, and may be reviewed or copied by the bidder or copies purchased through the Region.

9.7.4 **Plans**

The following bullets present various sections of the plans organized consistent with Chapter 21 of this manual. Each bullet contains guidance regarding soils-, wall-, and/or foundations-related considerations.

- Typical Sections
  Earthwork payment lines and items must be shown or indicated on all typical sections, which should be reviewed by the Regional Geotechnical Engineer.

- List of Abbreviations - Subsurface Explorations
  The following abbreviations are included on the Abbreviations plan sheet and should be used to identify subsurface explorations:

Abbreviation	Identification
DA	60 mm Cased drill hole
DN	100 mm Cased drill hole
FH	Hollow flight auger
DM	Drilling mud
PA	Power auger
AH	Hand auger
RP	25 mm sampler (retractable plug)
PH	Probe hole
TP	Test pit
PT	Percolation test hole
SP	Seismic point
CP	Cone penetrometer

Subsurface explorations in the DA, DM, DN, and FH categories are further identified as follows:

Abbreviation	Identification
B	Bridge
C	Cut
D	Dam
F	Fill
K	Culvert
W	Wall
X	Misc.: Used if one of these six designations cannot be used at the time of exploration.

Each type of exploration is numbered sequentially; for example, DA-B-1 is a 60 mm cased-drill-hole exploration for a bridge. (Sequence numbers are not repeated for any other exploration on a project.)

- Symbology. The symbol on plans for a point of subsurface exploration is provided in resource files as discussed in Chapter 4 of the *CADD Standards and Procedure Manual*. This symbol is illustrated on the “Legend.”

Plans should also define locations of rock outcrops, as determined by a Department Engineering Geologist.

- Special Plans

  - Unsuitable Material. It is important that the extent of removal of such material, its disposal, and the required backfill be determined during design. This information must be clearly presented with excavation and backfill payment lines in the contract documents. Various methods are acceptable for presenting this information, such as contours showing the bottom of removal, or a table indicating depth of removal at each section or at each exploration location.

  - Special Earthwork Details. On some projects, soil conditions may dictate special construction procedures. Some are described in Section 9.3, such as vertical sand or wick drains, areas requiring undercutting, slope-protection measures, or construction-monitoring devices. Details and limits of these procedures must be clearly shown on the plans.

- Sign and Signal Structures

  Foundation designs for traffic signal poles are included in the Series 680 Standard Sheets. Overhead sign structure foundation designs are found in the OS Series Bridge Detail Sheets. The designer should contact the Regional Geotechnical Engineer if any special or unusual foundation conditions or loadings are encountered and the appropriate information included on the plans.

- Retaining Walls[14, 15, 16, 17, 18, 19]

  The Regional Geotechnical Engineer should be consulted regarding foundation parameters to be used in designing retaining walls or similar structures, including location of borings, setting footing elevations, maximum design soil pressure, and sliding-friction coefficient. This information should be included on the contract plans.

- Bridges

  Requirements for either pile or spread-footing foundations are included in individual bridge contract plan sheets, as well as a general soil profile. This geotechnical information is provided by the Geotechnical Engineering Bureau to the Structures Design and Construction Division.

- Standard Sheets[1]. Following is a list of soil- or foundation-related sheets:

  <u>203 Series</u>
  - Construction Details: Unsuitable Material Excavation and Backfill.
  - Earthwork Transition and Benching Details.
  - Installation Details for Granular Fill-Slope Protection.
  - Installation Details for Reinforced Concrete or “other” Rigid Pipes.
  - Installation Details for Corrugated and Structural Plate Pipe and Pipe Arches.

  <u>204 Series</u>
  - Controlled Low Strength Material (CLSM) Installation Details for Circular and Elliptical Corrugated Metal Arches, Structural Plate Pipe and Pipe Arches, and Reinforced Concrete and “other” Rigid Pipes.

<u>209 Series</u>

- Soil Erosion and Sediment Control, Linear Measures.
- Soil Erosion and Sediment Control, Check Dams.
- Soil Erosion and Sediment Control, Drainage Structure Inlet Protection.
- Soil Erosion and Sediment Control, Pipe Inlet/Outlet Protection, Pipe Slope Drain.
- Soil Erosion and Sediment Control, Construction Entrances.
- Soil Erosion and Sediment Control, Sediment Traps.

<u>605 Series</u>

- Porous Concrete Pipe Underdrain.

<u>632 Series</u>

- Concrete Cribbing.
- Installation Details for Concrete Cribbing and Metal Bin-Type Retaining Walls
- Installation Details for Precast Concrete Wall Units
- Structural Details for Precast Concrete Wall Units

These standard sheets should be listed on the plans title sheet when they are applicable. Their use eliminates duplication of typical sections and details for standard treatments. In the event actual conditions do not lend themselves to these standard treatments, a typical section or detail must be recommended by the Regional Geotechnical Engineer and included in the contract plans.

9.8 **REFERENCES**

1. New York State Standard Sheets, Design Quality Assurance Bureau, New York State Department of Transportation, 50 Wolf Road, Albany, New York 12232.

2. *Standard Specifications, Construction and Materials*, Design Quality Assurance Bureau, New York State Department of Transportation, 50 Wolf Road, Albany, New York 12232.

3. *Official Compilation of Codes, Rules and Regulations, 6NYCRR Part 360, Solid Waste Management Facilities, Title 6*, December 31, 1988. New York State Department of Environmental Conservation, 625 Broadway, Albany, New York 12233.

4. *Geotechnical Engineering Bureau Manual GDP-12 Design Consultant Agreements: Soils-Related Task Assignments*.

5. The following references provide additional guidance regarding subsurface explorations:

   - [5a]Geotechnical Engineering Drilling Manual, Geotechnical Engineering Bureau, July, 1999.
   - [5b]Soil Mechanics by the Department of Navy, Naval Facilities Engineering Command, Design Manual 7.1: May, 1982.
   - [5c]Geotechnical Engineering Manual (GEM-19) Guidelines for Preparing Drilling Contracts, Geotechnical Engineering Bureau, July, 2001.
   - [5d]Soils Engineering Manual (SEM-8) Techniques to Improve Undisturbed Sampling, Soil Mechanics Bureau, April, 1975.
   - [5e]AASHTO Designation T206-03/ ASTM Designation D1586-99 Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.
   - [5f]AASHTO Designation T207-03/ ASTM Designation D1587-00 Standard Test Method for Thin-Walled Tube Sampling of Soils.
   - [5g]ASTM Designation D3441-05 Standard Test Method for Mechanical Cone Penetration Tests of Soil.

6. The following references provide additional guidance regarding settlement and stability:

   - [6a]Soil Mechanics by the Department of Navy, Naval Facilities Engineering Command, Design Manual 7.1: May, 1982.

7. The following references provide additional guidance regarding embankment foundations:

   - [7a]Soil Mechanics by the Department of Navy, Naval Facilities Engineering Command, Design Manual 7.1: May, 1982.
   - [7b]Foundations and Earth Structures by the Department of Navy, Naval Facilities Engineering Command, Design Manual 7.2: May, 1982.
   - [7c]Soils Test Procedure (STP-2) An Engineering Description of Soils Visual-Manual Procedure, Geotechnical Engineering Bureau, December, 1994.
   - [7d]Soils Engineering Manual (SEM-12) Guidelines for Embankment Construction, Soil Mechanics Bureau, December, 1987.
   - [7e]Soil Test Method (STM-6) Test Method for Rapid Earthwork Compaction Control, Soil Mechanics Bureau, March, 1974.

- [7f]Soil Test Method (STM-9) Test Method for Earthwork Compaction Control by Sand Cone or Volumeter Apparatus, Soil Mechanics Bureau, July, 1980.
- [7g]Soil Test Method (STM-10) Test Method for Earthwork Compaction Control by Nuclear Gauge, Soil Mechanics Bureau, November, 1981

8. The following references provide additional guidance regarding the geotechnical design guidance for stormwater management and erosion and sediment control:

- [8a]NYSDOT Highway Design Manual, Chapter 8 Highway Drainage.
- [8b]NYSDOT Design Requirements and Guidance for State Pollutant Discharge Elimination System (SPDES) General Permit GP-02-01.
- [8c]Memorandum of Understanding (MOU) regarding the SPDES General Permit for Stormwater Discharges from Construction Activity, GP-02-01.
- [8d]SPDES General Permit for Stormwater Discharges from Construction Activity, GP-02-01, and SPDES General Permit for Stormwater Discharges from Municipal Separate Stormwater Sewer Systems (MS4s), GP-02-02.
- [8e]NYS Standards and Specifications for Erosion and Sediment Control, NYS Soil and Water Conservation Committee, August, 2005.
- [8f]NYS Stormwater Management Design Manual, Center for Watershed Protection, August, 2003.
- [(8g]Construction Site Erosion and Sediment Control Certification Course manual, Washington State Department of Transportation.
- [8h]Soil Mechanics Principles and Applications, William Perloff & William Baron, 1976 by John Wiley & Sons (Modified from Terzaghi and Peck (1967).

9. The following references provide additional guidance regarding underdrains and edgedrains:

- [9a]NYSDOT Comprehensive Pavement Design Manual, Chapter 9 Subsurface Pavement Drainage.
- [9b]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-SA-92-008: Drainable Pavement Systems, Participants Notebook, Demo. 87, March, 1992.
- [9c]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-NHI-99-030: Pavement Subsurface Drainage Design, Participant Workbook, ERES Consultants, April, 1999.
- [9d]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-NHI-99-028: Pavement Subsurface Drainage Design, Reference Manual, ERES Consultants, April, 1999.
- [9e]National Cooperative Highway Research Program, Pavement Subsurface Drainage Systems, Synthesis of Highway Practice 239, 1997.

10. The following references provide additional guidance regarding rock excavation:

- [10a]Geotechnical Engineering Manual (GEM-22) Procedures for Blasting, January, 2005.
- [10b]Geotechnical Design Procedure (GDP-13) for Preparing Rock Slope Recommendations, June 2003.
- [10c]Geotechnical Engineering Manual (GEM-15) Rock Slope Rating Procedure, June, 1996.

- [10d]Rockfall Catchment Area Design Guide, Final Report, SPR-3(032), Oregon Department of Transportation, FHWA-OR-RD-02-04, January, 2002.
- [10e]The Nature of Rockfall as the Basis for a New Fallout Area Design Criteria for 0.25:1 Slopes, Engineering Geology Group Oregon Department of Transportation, FHWA-OR-GT-92-05, September, 1994.
- [10f]Evaluation of Rockfall and Its Control by Arthur M. Ritchie, Washington State Highway Commission, Highway Research Record, Vol. 17: 1963.
- [10g]Rock Slopes: Design, Excavation, Stabilization, FHWA-TS-89-045, September, 1989.

11. The following references provide additional guidance regarding excavation protection and support:

- [11a]Safety and Health Regulations for Construction, Title 29 Code of Federal Regulations, Part 1926, Subpart P Excavations, US Department of Labor, Occupational Safety & Health Administration (OSHA).
- [11b]NYSDOT Bridge Manual, Section 4: Excavation, Sheeting, and Cofferdams.

12. The following references provide additional guidance regarding geotextiles:

- [12a]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-HI-95-038: Geosynthetic Design and Construction Guidelines, NHI Course No.13213 Participant Notebook, May, 1995.
- [12b]Geosynthetic Acceptance and Quality Assurance Procedures, Geotechnical Engineering Bureau Directive 124.1-4-2R8, May, 2001.
- [12c]Engineering Principles of Ground Modification, Manfred Hausmann, University of Technology, Sydney, McGraw-Hill Publishing Co., 1990.
- [12d]NYSDOT Materials Approved List, Geosynthetics for Highway Construction.
- [12e]National Transportation Product Evaluation Program (NTPEP), American Associations of State Highway and Transportation Officials (AASHTO): M-288 Specification.
- [12f]Guide to NYSDOT Usage of Geotextiles, Soil Mechanics Bureau, January, 1988.

13. The following references provide additional guidance regarding controlled low strength material:

- [13a]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-SA-94-081: Fly Ash Facts for Highway Engineers, American Coal Ash Association, August, 1995.
- [13b]Controlled Low Strength Material (CLSM) Technical Reports, A Collection of Five Research Reports on Controlled Low Strength Material for The Cincinnati Gas & Electric Co., Brewer & Associates, January, 1992.

14. The following references provide additional guidance regarding anchored walls:

- [14a]Geotechnical Design Procedure (GDP-11) for Flexible Wall Systems Rev. #2, Geotechnical Engineering Bureau, September, 2003.
- [14b]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-DP-68-1R: Permanent Ground Anchors, November 1984.

- [14c]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-RD-82/047: Tiebacks, Compliments of the Schnabel Foundation Co., July, 1982.
- [14d]Post Tensioning Institute (PTI), Recommendations for Prestressed Rock and Soil Anchors, 2004.
- [14e]Section 5: Retaining Walls in the Standard Specifications for Highway Bridges, adopted by the American Associations of State Highway and Transportation Officials (AASHTO).
- [14f]Foundations and Earth Structures by the Department of Navy, Naval Facilities Engineering Command, Design Manual 7.2: May, 1982.
- [14g]Permanent Ground Anchors - Nicholson Design Criteria, FHWA RD-81-151.
- [14h]Permanent Ground Anchors - Soletanche Design Criteria, FHWA RD-81-150.
- [14i]Permanent Ground Anchors - Stump Design Criteria, FHWA RD-81-152.
- [14j]Tiebacks, Executive summary, FHWA RD-82-046.
- [14k]Permanent Ground Anchors, Volume 1, Final Report, FHWA DP-90-068.
- [14l]Permanent Ground Anchors, Volume 2, Field Demonstration Project Summaries, FHWA DP.
- [14m]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-SA-96-038: Geotechnical Engineering Circular No. 2 Earth Retaining Walls, February, 1997.

15. The following references provide additional guidance regarding soldier pile and lagging walls:

    - [15a]Geotechnical Design Procedure (GDP-11) for Flexible Wall Systems Rev. #2, September, 2003.
    - [15b]Foundations and Earth Structures by the Department of the Navy, Naval Facilities Engineering Command, Design Manual 7.2: May, 1982.
    - [15c]Section 5: Retaining Walls in the Standard Specifications for Highway Bridges, adopted by the American Associations of State Highway and Transportation Officials (AASHTO).
    - [15d]Chapter 8: Excavations and Retaining Structures, Geotechnical Engineering Bureau, Design Manual (Working Draft), April, 1996. (Available upon request through the Geotechnical Engineering Bureau.)

16. The following references provide additional guidance regarding sheeting:

    - [16a]Geotechnical Design Procedure (GDP-11) for Flexible Wall Systems Rev. #2, September, 2003.
    - [16b]USS Steel Sheet Piling Manual, Updated and reprinted by US Department of Transportation/FHWA with permission: July, 1984.
    - [16c]Foundations and Earth Structures by the Department of the Navy, Naval Facilities Engineering Command, Design Manual 7.2: May, 1982.
    - [16d]Section 5: Retaining Walls in the Standard Specifications for Highway Bridges, Adopted by the American Associations of State Highway and Transportation Officials (AASHTO).
    - [16e]NYSDOT Bridge Manual, Section 4: Excavation, Sheeting, and Cofferdams.

- [16f]Chapter 8: Excavations and Retaining Structures, Geotechnical Engineering Bureau, Design Manual (Working Draft), April, 1996. (Available upon request through the Geotechnical Engineer Bureau.)

17. The following references provide additional guidance regarding mechanically stabilized earth system (MSES):

- [17a]Geotechnical Engineering Manual (GEM-16) Mechanically Stabilized Earth System Inspection Manual (Metric), December, 1996.
- [17b]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-NHI-00-043: Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design & Construction Guidelines, NHI course No. 132042, March, 2001.
- [17c]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-SA-96-072: Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, FHWA Demo 82, August, 1997.
- [17d]Section 5: Retaining Walls in the Standard Specifications for Highway Bridges, Adopted by the American Associations of State Highway and Transportation Officials (AASHTO).

18. The following references provide additional guidance regarding geosynthetic reinforced earth systems (GRES):

- [18a]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-SA-96-038: Geotechnical Engineering Circular No. 2, Earth Retaining Systems, February, 1997.
- [18b]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-HI-95-038: Geosynthetic Design and Construction Guidelines, NHI Course No. 13213 Participant Notebook, May, 1995.
- [18c]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-RD-89-043: Reinforced Soil Structures Volume I. Design and Construction Guidelines, November, 1990.
- [18d]Engineering Principles of Ground Modification, Manfred Hausmann, University of Technology, Sydney, McGraw-Hill Publishing Co., 1990.

19. The following references provide additional guidance regarding soil nails:

- [19a]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-IF-03-017: Geotechnical Engineering Circular No. 7 Soil Nail Walls, March, 2003.
- [19b]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-SA-93-068: Soil Nailing Field Inspectors Manual, Soil Nail Walls-Demonstration Project 103, April, 1994.
- [19c]Geotechnical Engineering Manual (GEM-21): Design and Construction Guidelines for the use of the Special Specification for a Soil Nail Wall System, January, 2005.
- [19d]Chapter 8: Excavations and Retaining Structures, Geotechnical Engineering Bureau, Design Manual (Working Draft), April, 1996.

20. The following references provide additional guidance regarding drilled shafts:

- [20a]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-IF-99-025: Drilled Shafts: Construction Procedures and Design Methods, Michael W. O'Neil and Lymon C. Reese, 1999.
- [20b]Specification Item 551.9949nn17 and Guidelines: General Information and Guidelines for the Designer of Drilled Shafts and Trial Shafts, September, 1997.
- [20c]Handbook on Design of Piles and Drilled Shafts Under Lateral Load, FHWA IP-84-11.
- [20d]Geotechnical Control Procedure (GCP-18): Static Pile Load Test Manual, July, 1999.
- [20e]Geotechnical Engineering Manual (GEM-18) Drilled Shaft Inspector's Guidelines, September, 2000.
- [20f]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-SA-91-042: Static Testing of Deep Foundations, February, 1992.
- [20g]Drilled Shaft Inspector's Manual, The Joint Caisson-Drilled Shaft Committee of the International Association of Foundation Drilling (ADSC) and Deep Foundation Institute (DFI), 1989.
- [20h]Section 4: Foundations, adopted by the American Associations of State Highway and Transportation Officials (AASHTO).
- [20i]NYSDOT Bridge Manual, Section 11: Substructures.
- [20j]Drilled Shafts, NHI Course 13214 Student Workbook, Publication No. FHWA HI-88-042, July, 1988.
- [20k]Manual for Non Destructive Testing and Evaluation of Drilled Shafts, Deep Foundation Institute (DFI) in cooperation with the International Association of Foundation Drilling (ADSC), 2004.
- [20l]Transportation Research Board Paper No. 98-0595, NDT Diagnosis of Drilled Shaft Foundations, Larry Olson, Marwan Aouad and Dennis Sack, January, 1998.

21. The following references provide additional guidance regarding driven piles:

- [21a]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-HI-97-013: Design and Construction of Driven Pile Foundations, Workshop Manual Volume I, NHI course Nos. 13221 & 13222, November, 1998.
- [21b]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-HI-97-014: Design and Construction of Driven Pile Foundations, Workshop Manual Volume II, NHI course Nos. 13221 & 13222, November, 1998
- [21c]Soils Engineering Manual (SEM-11), Charts to Facilitate Computation of Skin Friction on Driven Non-Tapered Piles in Cohesionless Soil, November, 1985.
- [21d]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-SA-91-042: Static Testing of Deep Foundations, February, 1992.
- [21e]US Department of Transportation, Federal Highway Administration, Publication No. TS-78-210: Negative Friction "Downdrag on a Pile", 1978.
- [21f]A Pile Inspector's Guide to Hammers, Equipment Applications Committee of the Deep Foundation Institute (DFI), 1995.
- [21g]Section 4: Foundations, adopted by the American Associations of State Highway and Transportation Officials (AASHTO).
- [21h]NYSDOT Bridge Manual, Section 11: Substructures.
- [21i]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-DP-66-1: Manual on Design and Construction of Driven Pile Foundations, April, 1986.
- [21j]Inspector's Manual for Pile Foundations, Inspection and Testing Committee of the Deep Foundation Institute (DFI), 1979.

22. The following references provide additional guidance regarding micropiles:

- [22a]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-SA-97-070: Micropile Design and Construction Guidelines, Implementation Manual, June, 2000.
- [22b]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-RD-96-016: Drilled and Grouted Micropiles: State-of-Practice Review, Volume I Background, Classification, Cost, July, 1997.
- [22c]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-RD-96-017: Drilled and Grouted Micropiles: State-of-Practice Review, Volume II Design, July, 1997.
- [22d]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-RD-96-018: Drilled and Grouted Micropiles: State-of-Practice Review, Volume III Design, July, 1997.
- [22e]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-RD-96-019: Drilled and Grouted Micropiles: State-of-Practice Review, Volume IV Design, July, 1997.
- [22f]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-SA-91-042: Static Testing of Deep Foundations, February, 1992.
- [22g]US Department of Transportation, Federal Highway Administration, Publication No. FHWA-TS-86-206: Bored Piles, 1986.

- It is also helpful to study references for grouted tiebacks, as the technologies are identical.

23. The following references provide additional guidance regarding building condition surveys and vibration monitoring (non blasting):

    - [23a]Construction Vibrations and Their Impact on Vibration-Sensitive Facilities, Hal Amick and Michael Gendreau, Colin Gordon & Associates, Presented at ASCE Construction Congress 6, February 22, 2000.
    - [23b]Transit Noise and Vibration Impact Assessment, Federal Transit Administration, Harris Miller Miller & Hanson Inc., Final Report April, 1995.
    - [23c]Standard Recommended Practice for Evaluation of Transportation-Related Earthborne Vibrations AASHTO Designation: R 8-96 (2004).

APPENDICES

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APPENDIX 9A
PROPRIETARY WALL SYSTEM EVALUATION PROCESS

This appendix presents the Department's process for evaluating new proprietary wall systems.

9A.1 PROCEDURE OVERVIEW

The Technical Services Division, through the Materials Bureau, maintains Materials Bureau Approved Lists of Materials and Equipment (Approved Lists) for use on New York State Department of Transportation projects. The Department currently uses several categories of prefabricated and field-assembled retaining wall systems, including some that are proprietary. Because of their complexity and the variability involved in their design and construction, the procedure described below has been developed as the standard preapproval process for proprietary retaining wall systems. Suppliers/Manufacturers wishing to have their retaining wall systems eligible for use on the State Highway system should seek preapproval of their systems in conformity with the procedures described in this appendix.

9A.2 DEFINITIONS AND ROLES

Approval - With respect to proprietary wall systems, "approval" signifies the Department's agreement that a given system (1) is technically satisfactory to be included on the Approved Lists and (2) meets the requirements of the Department's Specifications for its particular general classification.

Conditional Approval - A probationary status, prior to "approval" (as defined above). The Department's "conditional approval" signifies that, subject to prebid review and within case-specific "conditional" limitations, a system may be proposed for and used on Department projects where the wall's particular general classification has been specified. The Department may either approve or reject a system at the end of the "conditional approval" phase.

Construction Engineer - The individual responsible for reviewing the application package on behalf of the Construction Division. Areas of responsibility include review of construction specifications, constructability per Department specifications, special equipment requirements, testing recommendations, schedule implications, and field construction manuals.

Design Engineer - The individual responsible for reviewing the application package on behalf of the Design Services Bureau.

Design Quality Assurance Engineer - The individual responsible for reviewing the application package on behalf of the Design Quality Assurance Bureau.

Geotechnical Engineer - The individual responsible for coordinating the review of the retaining wall system and reviewing the application package on behalf of the Geotechnical Engineering Bureau. The Geotechnical Engineer's administrative responsibilities include keeping records of the application and the review process, preparing and transmitting the response(s) to the Supplier/Manufacturer, and recommending the addition of accepted systems to the Approved Lists. Technical responsibilities include review of (1) foundation requirements, (2) retaining wall stability requirements, (3) excavation and backfill requirements, (4) special geotechnical equipment and testing requirements, (5) durability and/or corrosion requirements of system components, and (6) the construction specifications. The Geotechnical Engineer will also be responsible for seeing that any necessary changes to the Department specifications are made.

Maintenance Engineer - The individual responsible for reviewing the application package on behalf of the Transportation Maintenance Division. Responsibilities include evaluation of durability issues, field inspection problems, routine maintenance requirements, and ease of repair/replacement.

Materials Engineer - The individual responsible for reviewing the application package on behalf of the Materials Bureau. Technical responsibilities include the review and evaluation of proposed system components, comparing proposed materials specifications with the Department's existing materials specifications, reviewing fabrication shop drawings for content and completeness, and maintenance of the Department's Approved Lists. Evaluation may include visits to the manufacturing facility(ies), review of the manufacturer's quality control and the sampling and laboratory testing of the individual components.

Reviewers - Collectively, the individuals responsible for reviewing a properly submitted application.

Structures Engineer - The individual responsible for reviewing the application package on behalf of the Structures Design and Construction Division. Technical responsibilities include strength of structural elements, reinforcing of foundation, connection details, and settlement tolerance of the rigid elements.

Supplier/Manufacturer - Agent wishing to obtain Department's acceptance of a retaining wall system for use on the State Highway system and inclusion on the official Approved Lists. Responsibilities include preparation and submittal of application package, submittal of requests for modifications to approved systems and prompt notification to the Department of any system problems discovered with an approved system.

9A.3 PROCEDURAL GUIDELINES

Supplier/Manufacturers submitting a preapproval application package should divide the information necessary for review into three sections as detailed in Attachment 1 to this procedure.

Section 1 - Selection Information, should contain general information about the system. After a system is conditionally approved, this information will be made available to designers and, upon request, to s to support the process of selecting suitable candidate systems for a given project need. The information should include the limits of applicability, the location of the plant and its proven production capacity, Supplier/Manufacturer contact information, and specific cost information.

Section 2 - Implementation Information should contain the detailed design procedures and construction specifications for the system. This information will form the basis of the guidance that will be provided to designers.

Section 3 - Evaluation Information, should contain the information that will be used in the product evaluation process by the reviewers. This should include information on the development of the system, its theoretical background, test results, case histories, and when available, names, addresses, and phone numbers of users who may be contacted for their experience with the system.

The Department's review of the application package will usually take between three and six months. New systems or modifications to approved systems may be (1) approved, (2) conditionally approved, or (3) rejected. If substantial improvements are made, the Department may elect to reconsider a rejected system that is resubmitted no earlier than six months from its date of rejection. Systems will not be reconsidered if significant technical deficiencies are not corrected. Systems rejected solely for a lack of proven performance history may be resubmitted after a period of one year.

Conditional approval implies that the Department is willing to use the new or modified system in a number of trial installations before making a final decision to either add the system to the Approved Lists or reject the system (or its modification). When research is still being conducted on a new system to support or provide a performance history, the Department may grant conditional approval.

The Department will determine whether a proposed installation on a project is suitable as a test site and will work with the Supplier/Manufacturer to develop a satisfactory testing and monitoring program if the Department's reviewers deem one necessary. For conditional approval, the Department will determine what limitations will be imposed on the conditions under which the system may be used.
When modifications are to be made to an approved system, the responsible Supplier/Manufacturer should submit a written request for the revision of the approved system to the Geotechnical Engineering Bureau for review. The request package need only address the specific modifications requested and document the justification for the revision. Reviews of minor modifications will normally require two months. Review of major modifications will be treated as if they were new systems.

The Supplier/Manufacturer has a responsibility to promptly notify the Geotechnical Engineering Bureau of any changes made to the approved system or of any problems with an approved system that are discovered. This includes any need to further limit the breadth of applicability of a system, revisions to the design process, details or construction procedures, or points of manufacture. Failure to notify the Department of changes or problems (including long-term performance problems) may result in disapproval of the system, or in the event of serious failures to notify, suspension or disapproval of all systems supplied by that Supplier/Manufacturer.

9A.4 STEPS IN THE PREAPPROVAL PROCESS

SUPPLIER/MANUFACTURER

1. Obtains a copy of this procedure from the Geotechnical Engineering Bureau. Telephones or meets formally with the Geotechnical Engineering Bureau after reviewing procedure and prior to preparing application package. This is strongly advised but not mandatory and is intended to clarify any questions on or concerns over preparing the application package. Prepares the package in general conformance with the guidance in this procedure's Attachment 1. Submits the completed package to the Director of the Geotechnical Engineering Bureau with a written request for an evaluation of their system.

GEOTECHNICAL ENGINEER

2. Documents receipt of the application package. Inspects the package for basic adequacy. Notifies the Supplier/Manufacturer and requests a re-submittal if the package is found to be inadequate. If found adequate, distributes copies to the Directors of, and to the designated reviewers in the Geotechnical Engineering Bureau, Construction Division, Design Services Bureau, Design Quality Assurance Bureau, Materials Bureau, Structures Design and Construction Division, and Transportation Maintenance Division. Provide the Structures Design and Construction Division reviewers with the necessary geotechnical information for their review.

REVIEWERS

3. Review the application package, paying particular attention to individual areas of responsibility. Contact the Supplier/Manufacturer for clarifications of material in the application package related to their review. Upon completion of the review, provide written comments to the Geotechnical Engineer, indicating preference for (1) Approval, (2) Conditional Approval, or (3) Rejection, including the reasons for the rejection and any recommended limitations on use. Areas of concern that need to be resolved during the test installation phase should be identified.

GEOTECHNICAL ENGINEER

4. Prepares a draft response to the Supplier/Manufacturer’s request for evaluation, indicating any areas of concern and distributes to the reviewers.

REVIEWERS

5. Review the draft response and notify the Geotechnical Engineer of concurrence or identify required corrections.

GEOTECHNICAL ENGINEER

6. Transmits the response to the Supplier/Manufacturer. If the system is conditionally approved, the letter will (1) be copied to Regional Design and Geotechnical Engineers, (2) be written to indicate (particularly to potential s that the Supplier/Manufacturer may later contact) that the system is conditionally approved, (3) list the specific limitations for use of the system, and (4) list the desired conditions for sites that will be considered for test installations. If the system is approved, go to Step 11 below. If the system is rejected, the letter will (1) state the reasons for the rejection and the areas within the application package that were affected, and (2) the time frame allowed for resubmission.

SUPPLIER/MANUFACTURER

7. Markets the conditionally approved system to Contractors. Notifies the Geotechnical Engineer of potential application on specific Department projects.

GEOTECHNICAL ENGINEER

8. Notifies the reviewers of the proposed use of the conditionally approved system. Arranges a meeting of reviewers to confirm site suitability and, if appropriate, to plan details of site specific testing and monitoring program. Acts as the single-point, formal contact with the Supplier/Manufacturer. Notifies the Supplier/Manufacturer of the site suitability and any site-specific monitoring/testing requirements.

REVIEWERS

9. Analyze the results from any site-specific monitoring/testing, as necessary, to evaluate systems that progress to installation. Provide a written summary evaluation to the Geotechnical Engineer, including the need for subsequent monitoring/testing, modifications to the limitations for use, and results relative to new or former areas of concern.

GEOTECHNICAL ENGINEER

10. Drafts a letter summarizing the monitoring/test results and, if the system is retained in conditional approval status, identifies any changes in limitations on the use or subsequent monitoring/testing requirements. If necessary, announces the system rejection and reasons, as previously outlined. When the letter is approved by reviewers, transmits it to the Supplier/Manufacturer, the Main Office Design Services Bureau, and the Regional Design and Geotechnical Engineers.

11. If reviewers agree that the system should be approved, documents the approval and requests that the Materials Bureau add the system to the Approved Lists. Files a record copy of the application package, review comments, and letter of response approving the system. Adds any design procedure information to the Geotechnical Engineering Bureau's Retaining Wall Design Guidelines and provides copies to the Main Office Design Services Bureau and to all Regional Design Engineers (RDEs) and Regional Geotechnical Engineers. Works with the Structures Design and Construction Division and Design Quality Assurance Bureau to add the description and any appropriate selection guidance to the *Highway Design Manual*. Works with the Construction Division to develop field inspection guidelines. Issues an Engineering Bulletin to designers, consultants, localities, and s on the EB mailing lists to announce the addition of the system to the Approved Lists. The EB should include brief descriptions and contact information and identify the RDEs as holders of the design procedure information.

MATERIALS ENGINEER

12. Adds information for the approved system to the Department's *Approved Lists*.

REVIEWERS

13. If required, the Reviewers having responsibility for specified items will transmit construction specifications to the Design Quality Assurance Bureau's Specifications and Standards Section for inclusion on the list of Special Specifications or as revisions to the Standard Specifications.

9A.5 RELATED OFFICIAL ISSUANCES

Title	Source
Highway Design Manual	Design Quality Assurance Bureau
Retaining Wall Design Guidelines	Geotechnical Engineering Bureau
APPROVED LIST - Materials and Equipment For Use on New York State Department of Transportation Projects	Materials Bureau

ATTACHMENT 1

INSTRUCTIONS FOR PREPARATION OF THE APPLICATION PACKAGE FOR PRODUCT EVALUATION OF PROPRIETARY RETAINING WALL SYSTEMS

Proprietary retaining wall Suppliers/Manufactures are required to provide the following information with their formal request for product evaluation. Each page of this supplemental material should bear the name of the system. All items should be marked with the same date of preparation to avoid problems with subsequent revisions or re-submittals. Seven (7) copies of the material should be submitted to aid the review process.

The application package should be in three separate parts. While each of the three parts will be used during the review process, the parts will be used in different ways, if and when the system is approved. If the system is approved, Section 1, Selection Information, will be used as a reference to aid designers and s in the selection of appropriate wall types. Section 2, Implementation Information, will contain the information necessary to properly design and construct the system. Section 3, Evaluation Information, would be used during the initial product evaluation phase by the Department's reviewers to evaluate whether the system should be approved, conditionally approved, or rejected. A detailed list of the recommended content for each part is listed below. Note that use of the metric system is preferred and, for some information, required.

<u>A. SELECTION INFORMATION PACKAGE</u> (Section 1)

1. Provide the trade name(s) under which the product is being marketed.

2. Provide the name, address, and telephone number of the company and/or manufacturer(s), of the product and, if different, the plant location(s).

3. If different from above, provide the name, address, and telephone number of the representative or Supplier/Manufacturer, and/or the designer.

4. Describe the system.
    a. List the advantages and best application(s) of the system.
    b. List any cautions that apply to use of the system.
    c. Brochures or reproducible illustrations are encouraged.
    d. Describe, and preferably illustrate, aesthetic treatment options.

5. List the limitations on the use of the system. This should be a thorough listing. Consider such parameters as:

   a. Wall height.
   b. Allowable settlement.
   c. Soil chemistry.
   d. Backfill type and compaction.
   e. Surcharge.
   f. Minimum wall radius.
   g. Expected design life.
   h. Warranty period.
   I. Maintenance requirements.
   j. Necessary impact protection.

6. Provide cost information. Where possible, actual case history data should be provided in sufficient detail to permit estimating the cost on dissimilar projects.

7. List the design activities that are required when the system is used. Describe these in the sequence by which design would normally proceed. This information should be sufficient for designers and/or s to quickly determine the level of design effort needed. Consider such factors as:

   a. Subsurface investigation.
   b. Excavation stability analysis.
   c. Foundation design.
   d. Stability analysis.
   e. Tieback design, if applicable.
   f. Drainage and backfill design.
   g. Face design, etc.

   Be specific about what design work will have to be done independently and what design work should follow the design charts, procedures and guidance provided with the system documentation. If design services are available through the Supplier/Manufacturer, so state. Note throughout that the design must be by a professional engineer licensed to practice in New York State.

8. List the typical construction equipment requirements, highlighting any special equipment needs. Indicate typical durations and progress rates. Describe any likely impacts on the construction effort. Describe the type and rate of any testing that should be done during construction. (The Department may require more testing than the amount specified by the Supplier/Manufacturer.)

B. IMPLEMENTATION INFORMATION (Section 2)

1. Outline the design process. Provide and explain the use of any charts or equations that should be used. Provide references to any recognized design standards/guides that are utilized. Provide recommended values or procedures for design processes not directly covered in the guidance referenced in Part A, step 7. Include representative sample design problems (at least two) with hand solutions for typical conditions. Show solutions for sloped backfill and for level surcharged backfill, at a minimum.

2. Provide all appropriate details or standard drawings required for fabrication and proper use of the system. Standard drawings used for fabrication of precast concrete units shall be prepared as follows:

   a. The size of the drawing shall be 559 mm x 864 mm with an image area of 529 mm x 799 mm. Top, bottom, and right-edge margins will be 15 mm; left margin 50 mm.

   b. Each drawing shall have a title block in the lower right hand corner, a minimum of 64 mm x 127 mm in size, with the following information:
      (1) Company name and location.
      (2) Title of drawing.
      (3) Unique drawing number for each drawing.
      (4) Date drawing was prepared.
      (5) Name or initials of the person preparing drawing.

   c. The drawings shall contain with the following information:
      (1) The minimum 28-day compressive strength of the concrete.
      (2) The yield strength or grade of reinforcement.
      (3) The minimum concrete cover over the reinforcing steel.
      (4) Type and capacity of lifting devices.
      (5) Complete and accurate views and sections of each precast unit with all dimensions and details necessary for fabrication.
      (6) Complete and accurate reinforcing details showing type, size spacing and location of all reinforcing steel.
      (7) A reinforcing steel table showing bar mark numbers, size, center-to-center spacing, length, details for all bends and quantity per unit.
      (8) Type and location of all lifting devices.

3. Provide information on all materials used to fabricate and install the wall system. Include the following:

    a. Material type and grade, referenced to applicable national standards.

    b. Detailed drawings for all component parts showing dimensions with working tolerances.

    c. Strength and durability test information.

    d. Material properties, tested for quality control purposes, with acceptance limits.

4. Provide all required specifications, preferably in general conformance with the Department's format for Standard Specifications. Where possible, any specifications referenced should be those of the Department.

5. Provide the Field Construction Manual for the system. (If the system is approved, the Supplier/Manufacturer will be expected to provide two copies of this manual for any project on which the system is used.) The equipment and testing requirements discussed above (in item 8 of the previous Section 1) shall be included in or appended to the Manual.

6. Provide a maintenance manual or list of maintenance considerations, if appropriate.

<u>C. EVALUATION INFORMATION</u> (Section 3)

1. Describe the development history of the system. When was it conceptualized? Patented? Was it previously known by other names? When was the first prototype developed?

2. If anything has been written or published to describe the theoretical basis of the system, this should be provided.

3. Provide any relevant information on testing of the system and the test results and interpretation.

4. Explain the derivation of charts and any unique equations that are provided for the design process.

5. Detail case histories and current follow-up of up to five previous projects. Photographs are considered helpful. When possible, contact information for user references should be provided.

6. Explain the rationale for any special details or procedures that are specified in the above section on implementation information.

7. Provide samples, if appropriate. The Materials Bureau reserves the right to require the Supplier/Manufacturer to supply samples of specific components for testing when deemed necessary during the course of the evaluation.

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APPENDIX 9B
PROJECT CHECKLIST FOR DETERMINING IF SUBSURFACE EXPLORATIONS ARE REQUIRED

Send a completed copy of this Checklist to the Regional Geotechnical Engineer.

PROJECT: ____

PIN: ________________

DATE: ________________

DESIGNER/CONTACT PERSON: ______________________________

☐ New Roadway Alignment
- ☐ Large Cuts
- ☐ Large Fills
- ☐ Multiple Alternates Being Considered

☐ Roadway Widening
- ☐ Permanent Retaining Walls
- ☐ Large Cuts
- ☐ Large Fills

☐ Bridge and/or Culvert Reconstruction
- ☐ Replacement
- ☐ Widening/ Extension

☐ Maintenance & Protection of Traffic (On-Site Detour)
- ☐ Temporary Retaining Walls
- ☐ Temporary Structure
- ☐ Temporary Approach Embankments

☐ Pavement Reconstruction
- ☐ Pavement Rubblization
- ☐ Pavement Full Depth Reclamation

☐ Shoulder Reconstruction

☐ Signals and/or Large Sign Structure Installations

☐ Deep and/or Extensive Drainage/ Utility Installations
- ☐ Deep and/or Extensive Drainage Pipe Installations
- ☐ Pipe Jacking, Boring, or Tunneling
- ☐ Excavations Where Rock Outcrops Are Present or Rock Is Expected At Shallow Depths

☐ Environmental Considerations
- ☐ Stormwater Management Practices
- ☐ Wetland Mitigation

This list may not be all-inclusive. Contact the Regional Geotechnical Engineer if you have questions regarding whether a project activity not listed above might require soil borings

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APPENDIX 9C
SUBJECT INDEX

APPENDIX 9C
SUBJECT INDEX