CHAPTER 12

EMBANKMENTS
(Intentionally left blank)
## Table of Contents

12.1 OVERVIEW AND DATA NEEDED................................................................. 12-6  
12.1.1 Site Reconnaissance........................................................................... 12-6  
12.1.2 Field Exploration and Laboratory Testing Requirements............... 12-7  
12.1.3 Soil Sampling and Stratigraphy......................................................... 12-8  
12.1.4 Groundwater ..................................................................................... 12-8  

12.2 DESIGN CONSIDERATIONS ...................................................................... 12-9  
12.2.1 Embankment Design in New York State........................................... 12-9  
12.2.2 Typical Embankment Materials and Compaction............................. 12-10  
  12.2.2.1 Earth Embankments and Bridge Approach Embankments.......... 12-12  
    12.2.2.1.1 Material Requirements...................................................... 12-12  
    12.2.2.1.2 Use of Plastic Soils......................................................... 12-13  
    12.2.2.1.3 Use of Shales................................................................. 12-15  
  12.2.2.2 Fill Placement Below Water....................................................... 12-16  
12.2.3 Embankments for Detention/Retention Facilities.............................. 12-16  
  12.2.3.1 Earth Dams................................................................................ 12-16  
    12.2.3.1.1 Hazard Classification...................................................... 12-17  
    12.2.3.1.2 Conduits........................................................................ 12-17  
    12.2.3.1.3 Geometry....................................................................... 12-17  
  12.2.3.2 Effective Stress Under Seepage................................................ 12-20  
  12.2.3.3 Flow Nets .............................................................................. 12-20  
    12.2.3.3.1 Flow Net Properties......................................................... 12-21  
    12.2.3.3.2 Information Derived from Flow Nets............................... 12-22  
12.2.4 Stability Assessment ......................................................................... 12-26  
  12.2.4.1 Safety Factors........................................................................... 12-26  
  12.2.4.2 Strength Parameters............................................................... 12-27  
  12.2.4.3 Overstress Analysis............................................................... 12-27  
    12.2.4.3.1 Applicability................................................................. 12-28  
    12.2.4.3.2 Procedure .................................................................... 12-28  
12.2.5 Embankment Settlement Assessment............................................... 12-30  
  12.2.5.1 Settlement Impacts..................................................................... 12-31  
  12.2.5.2 Settlement Analysis............................................................... 12-31  
    12.2.5.2.1 Primary Consolidation.................................................... 12-31  
      12.2.5.2.1.1 Amount of Settlement............................................... 12-32  
      12.2.5.2.1.2 Time for Settlement................................................ 12-41  
    12.2.5.2.2 Secondary Compression................................................... 12-42  
    12.2.5.2.3 Estimating Settlement in Organic Soils............................ 12-44  
    12.2.5.2.4 Settlement of Granular Soils.......................................... 12-49  
  12.2.5.3 Stress Distribution...................................................................... 12-54  
    12.2.5.3.1 Simple 2V:1H Method...................................................... 12-54  
    12.2.5.3.2 Theory of Elasticity......................................................... 12-55  
    12.2.5.3.3 Empirical Charts............................................................ 12-56
## Table of Contents

12.2.5.3.4 Rate of Settlement................................................................. 12-58  
12.2.5.4 Analytical Tools........................................................................... 12-58  

### 12.3 STABILITY MITIGATION ................................................................. 12-58  
12.3.1 Staged Construction........................................................................ 12-59  
12.3.1.1 Design Parameters...................................................................... 12-63  
12.3.1.2 In-Situ Shear Strength and Determination of Stability Assuming  
Undrained Loading .............................................................................. 12-64  
12.3.1.3 Total Stress Analysis................................................................... 12-66  
12.3.1.4 Effective Stress Analysis ............................................................. 12-71  
12.3.2 Base Reinforcement ....................................................................... 12-74  
12.3.3 Ground Improvement..................................................................... 12-74  
12.3.3.1 Undercut and Backfill ............................................................... 12-75  
12.3.4 Lightweight Fills ............................................................................ 12-77  
12.3.4.1 Expanded Polystyrene Fill ......................................................... 12-77  
12.3.4.2 Lightweight Aggregates ............................................................. 12-78  
12.3.4.3 Tire Shreds .............................................................................. 12-79  
12.3.4.4 Lightweight Concrete Fill .......................................................... 12-79  
12.3.4.5 Toe Berms and Shear Keys ....................................................... 12-80  

### 12.4 SETTLEMENT MITIGATION .............................................................. 12-83  
12.4.1 Acceleration Using Wick Drains .................................................... 12-83  
12.4.2 Acceleration Using Surcharges ..................................................... 12-84  
12.4.3 Lightweight Fills .......................................................................... 12-86  
12.4.4 Over-excavation ........................................................................... 12-86  
12.4.4.1 Unsuitable Removal ................................................................. 12-86  
12.4.4.2 Close Sequence Excavation and Backfill ................................ 12-89  
12.4.4.3 Removal of Soil by Displacement .......................................... 12-93  

### 12.5 CONSTRUCTION CONSIDERATIONS AND PS&E DEVELOPMENT ...... 12-94  
12.5.1 Winter Earthwork Operations ....................................................... 12-97  
12.5.2 Settlement and Pore Pressure Monitoring .................................... 12-99  
12.5.3 Instrumentation ........................................................................... 12-99  
12.5.4 PS&E Considerations ................................................................. 12-99  
12.5.5 PS&E Checklist ............................................................................ 12-100  

### 12.6 REFERENCES .................................................................................. 12-100
APPENDIX 12-A    EXAMPLES ILLUSTRATING STAGED FILL CONSTRUCTION DESIGN
                     ................................................................................. 12-104
12-A.1 PROBLEM SETUP .......................................................................................... 12-104
12-A.2 DETERMINATION OF MAXIMUM STABLE FIRST STAGE FILL HEIGHT .... 12-105
12-A.3 TOTAL STRESS ANALYSIS PROCEDURE EXAMPLE.................................... 12-106
12-A.4 EFFECTIVE STRESS ANALYSIS PROCEDURE EXAMPLE............................ 12-112
12.1 OVERVIEW AND DATA NEEDED

This chapter addresses the design and construction of bridge approach embankments, earth embankments, and lightweight fills. Static loading as well as seismic loading conditions are covered, though for a more detailed assessment of seismic loading on embankment performance, see NYS DOT GDM Chapter 9. The primary geotechnical issues that impact embankment performance are overall stability, internal stability, settlement, materials, and construction.

For the purposes of this chapter, embankments include the following:

- Bridge approach embankments, defined as fill beneath a bridge structure and extending 100 feet beyond a structure’s end at subgrade elevation for the full embankment width, plus an access ramp on a 10H:1V slope from subgrade down to the original ground elevation. The bridge approach embankment also includes any embankment that replaces unsuitable foundation soil beneath the bridge approach embankment.
- Earth embankments are fills that are not classified as bridge approach embankments.
- Lightweight fills contain lightweight fill or recycled materials as a significant portion of the embankment volume, and the embankment construction is usually by special provision. Lightweight fills are most often used as a portion of the bridge approach embankment to mitigate settlement or in landslide repairs to reestablish roadways.

The construction of a bridge approach embankment, earth embankment, and (depending on the type of material used) lightweight fills consists of a series of compacted layers or lifts of material placed on top of each other until the level of the subgrade surface is reached. The subgrade surface is the top of the embankment and the surface upon which the subbase is placed.

With the exception of lightweight fills, any suitable material may be used to construct a bridge approach or earth embankment. The maximum dimension of any particle of the material may not be greater than \( \frac{2}{3} \) the loose lift thickness. Any particles that are larger than \( \frac{2}{3} \) the loose lift thickness must be removed and disposed of, or may be put in the embankment side slope.

12.1.1 Site Reconnaissance

General requirements for site reconnaissance are given in NYS DOT GDM Chapter 2.

The key geotechnical issues for design and construction of embankments include stability and settlement of the underlying soils, the impact of the stability and settlement on the construction staging and time requirements, and the impact to adjacent and nearby structures, such as buildings, bridge foundations, and utilities. Therefore, the Departmental Geotechnical Engineer should perform a detailed site reconnaissance of the proposed construction. This should include a detailed site review outside the proposed embankment footprint in addition to within the embankment footprint. This reconnaissance should extend at least two to three times the width of the embankment on either side of the embankment and to the top or bottom of slopes adjacent to
the embankment. Furthermore, areas below proposed embankments should be fully explored if any existing landslide activity is suspected.

### 12.1.2 Field Exploration and Laboratory Testing Requirements

General requirements for the development of the field exploration and laboratory testing plans are provided in NYSDOT GDM Chapter 2. The expected project requirements and subsurface conditions should be analyzed to determine the type and quantity of information to be obtained during the geotechnical investigation. During this phase it is necessary to:

- Identify performance criteria (e.g. allowable settlement, time available for construction, seismic design requirements, etc.).
- Identify potential geologic hazards, areas of concern (e.g. soft soils), and potential variability of local geology.
- Identify engineering analyses to be performed (e.g. limit equilibrium slope stability analyses, liquefaction susceptibility, lateral spreading/slope stability deformations, settlement evaluations).
- Identify engineering properties required for these analyses.
- Determine methods to obtain parameters and assess the validity of such methods for the material type.
- Determine the number of tests/samples needed and appropriate locations for them.

The goal of the site characterization for embankment design and construction is to develop the subsurface profile and soil property information needed for stability and settlement analyses. Soil parameters generally required for embankment design include:

- Total stress and effective stress strength parameters;
- Unit weight;
- Compression indexes (primary, secondary and recompression); and
- Coefficient of consolidation.

Table 12-1 provides a summary of site characterization needs and field and laboratory testing considerations for embankment design.
### 12.1.3 Soil Sampling and Stratigraphy

The size, complexity and extent of the soil sampling program will depend primarily on the type, height and size of embankment project as well as the expected soil conditions.

The data needed for an embankment design shall be as described in NYSDOT GDM Chapter 4.

### 12.1.4 Groundwater

The location of the groundwater table is particularly important during stability and settlement analyses. High groundwater tables result in lower effective stress in the soil affecting both the shear strength characteristics or the soil and its consolidation behavior under loading. The Departmental Geotechnical Engineer should identify the location of the groundwater table and determine the range in seasonal fluctuation.

---

<table>
<thead>
<tr>
<th>Geotechnical Issues</th>
<th>Engineering Evaluations</th>
<th>Required Information for Analyses</th>
<th>Field Testing</th>
<th>Laboratory Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankments and Embankment Foundations</td>
<td>settlement (magnitude &amp; rate)</td>
<td>subsurface profile (soil, ground water, rock)</td>
<td>nuclear density</td>
<td>1-D Oedometer</td>
</tr>
<tr>
<td></td>
<td>bearing capacity</td>
<td>compressibility parameters</td>
<td>plate load test</td>
<td>triaxial tests</td>
</tr>
<tr>
<td></td>
<td>slope stability</td>
<td>shear strength parameters</td>
<td>test fill</td>
<td>unconfined compression</td>
</tr>
<tr>
<td></td>
<td>lateral pressure</td>
<td>unit weights</td>
<td>CPT (w/ pore pressure measurement)</td>
<td>direct shear tests</td>
</tr>
<tr>
<td></td>
<td>internal stability</td>
<td>time-rate consolidation parameters</td>
<td>SPT</td>
<td>grain size distribution</td>
</tr>
<tr>
<td></td>
<td>borrow source evaluation (available quantity and quality of borrow soil)</td>
<td>horizontal earth pressure coefficients</td>
<td>PMT</td>
<td>Atterberg Limits</td>
</tr>
<tr>
<td></td>
<td>required reinforcement</td>
<td>interface friction parameters</td>
<td>dilatometer</td>
<td>specific gravity</td>
</tr>
<tr>
<td></td>
<td>liquefaction</td>
<td>pullout resistance</td>
<td>vane shear</td>
<td>organic content</td>
</tr>
<tr>
<td></td>
<td>delineation of soft soil deposits</td>
<td>geologic mapping including orientation and characteristics of rock discontinuities</td>
<td>rock coring (RQD)</td>
<td>moisture-density relationship</td>
</tr>
<tr>
<td></td>
<td>potential for subsidence (karst, mining, etc.)</td>
<td>shrink/swell/ degradation of soil and rock fill</td>
<td>geophysical testing</td>
<td>hydraulic conductivity</td>
</tr>
<tr>
<td></td>
<td>constructability</td>
<td></td>
<td>piezometers</td>
<td>geosynthetic/soil testing</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>settlement plates</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>slope inclinometers</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Water levels are recorded during drilling in all borings or test pits. Information regarding the time and date of the reading and any fluctuations that might be seen during drilling should be included on the field logs as described in NYSDOT Chapter 4.

If there is a potential for a significant groundwater gradient beneath an embankment or surface water levels are significantly higher on one side of the embankment than the other, the effect of reduced soil strength caused by water seepage should be evaluated. In this case, piezometers may be installed to estimate the gradient. See NYSDOT Chapter 23 for information regarding piezometers.

Seepage effects must be considered when an embankment is placed on or near the top of a slope that has known or potential seepage through it. A flow net or a computer model (such as MODFLOW) may be used to estimate seepage velocity and forces in the soil. This information may then be used into the stability analysis to model pore pressures.

12.2 DESIGN CONSIDERATIONS

Embankments, except for the upper few feet which may be considered as subgrade, are not seriously affected by traffic loads. The primary design requirements are slope stability and, to a lesser degree, compressibility or consolidation. The primary load is the weight of the overlying embankment and pavement material. When vertical displacement due to volume change of the embankments is critical, the effects can be minimized by allowing the embankment to consolidate prior to constructing the pavement. However, except for high embankments, the evaluation of the compressibility of the compacted materials is not generally performed in highway embankment design.

The strength requirements for slope stability can be based on high strain levels and therefore are not as dependent on compaction method. Currently, standard highway practices generally specify density requirements without determination of the strength requirements of an embankment of given height and slope. For low embankments this may be satisfactory, but problems may be encountered in high embankments.

12.2.1 Embankment Design in New York State

The Departmental Geotechnical Engineer seldom designs, in the strict sense, highway embankments. It is the position of the Geotechnical Engineering Bureau that strict adherence to the standard specification requirements will produce internally stable embankments constructed with side slopes of 1V on 2H.

Many hundreds of miles of embankments have been constructed with few serious problems, indicating for the most part that the earthwork specifications currently in use are effective. This can be attributed primarily to the following factors:

- Many of the embankments constructed in New York State are built out of granular materials.
• Contractors will often opt to waste highly plastic excavation materials and substitute granular or low-plasticity materials for embankment construction whenever economically feasible.
• The majority of the embankments constructed do not exceed 20 ft. (6.1 m) in height. Although embankments exceed this height at many bridge approaches, the material used at the abutment locations is mostly a select granular material.

12.2.2 Typical Embankment Materials and Compaction

General instructions for embankment construction are discussed in Geotechnical Engineering Manual (GEM-12) Guidelines for Embankment Construction, and specific embankment construction requirements are provided in the Standard Specifications. The Departmental Geotechnical Engineer should determine during the exploration program if any of the material from planned earthwork will be unsuitable for embankment construction. Consideration should be given to whether the material contains vegetable or organic matter, such as muck, peat, organic silt, topsoil or sod. Certain man-made deposits of industrial waste, sludge or landfill may be determined to be unsuitable.
Figure 12-1 Typical Cut / Embankment Section
12.2.2.1 Earth Embankments and Bridge Approach Embankments

The purpose of performing good earthwork construction is to provide a stable, uniform support for pavement and structures. This is accomplished by using suitable material of a proper gradation and compacting it to a required density. Once in-place, it must be protected against damage (specifically, wheel loads) until the pavement structure is placed.

The difficulty in earthwork construction is that the Contractor has a great deal of freedom in performing the work.

1. Selects Soil: The Contractor selects the soil to be used as long as it is (a) suitable and (b) of a conforming gradation.
2. Moisture Content: The Contractor is entirely responsible for the moisture content. Attempts to compact the soil may be made at whatever moisture content is deemed appropriate. If the results are unsatisfactory, the Contractor will have to manipulate the soil to get better results. (Add water or, most likely, dry the soil out). In any case, if the Contractor cannot achieve satisfactory results, the soil will become waste and the Contractor will have to bring in better material.
3. Equipment: The Contractor chooses the compaction equipment (within the constraints of the specifications). There is a list of pre-approved rollers on the Departments website. Some unapproved rollers may be evaluated through the formula in the Standard Specifications. Others may be tested on-grade by building a test section of embankment.

Once the compactor is chosen, the maximum loose lift thickness is determined by the specifications.

12.2.2.1.1 Material Requirements

Terms used in embankment construction:

- Suitable Material: A material whose composition is satisfactory for use in embankment construction is a suitable material. The moisture content of the material has no bearing upon such designation. In general, any mineral (inorganic) soil, blasted or broken rock and similar materials of natural or man-made (i.e. recycled) origin, including mixtures thereof, are considered suitable materials. Recycled materials that the Department has evaluated and approved for general use shall be considered to be suitable material for embankment construction subject to the conditions for use as determined by the Department. The Departmental Geotechnical Engineer can provide guidance on the use of such materials. In general, the use of recycled materials must be also sanctioned by the Department of Environmental Conservation, usually in the form of a Beneficial Use Determination (BUD).
- Unsuitable Material: Any material containing vegetable or organic matter, such as muck, peat, organic silt, topsoil or sod, or other material that is not satisfactory for use in embankment construction as defined by suitable material is designated as an unsuitable material. Certain man made deposits of industrial waste, toxic or contaminated materials,
sludge, landfill or other material may also be determined to be unsuitable materials, based on an evaluation by a Departmental Geotechnical Engineer, Office of Environment, and the Department of Environmental Conservation.

- Unstable Material: A suitable material whose moisture content causes it to become unstable under load. Unstable material may be made suitable.

- Construction and Demolition Debris (C&D): Management of C&D debris in New York State is regulated by Department of Environmental Conservation under 6 NYCRR Part 360 Solid Waste Management Facilities, Title 6 of the Official Compilation of Codes, Rules and Regulations. Exempted waste may be buried on NYSDOT-owned properties.
  - Uncontaminated stone, asphalt, brick, soil, & concrete
  - Trees, stumps, wood chips & yard waste
Non-Exempt waste shall be removed to authorized facility for disposal, recovery, or recycling.

The types of compaction equipment used are the prerogative of the Contractor. In general, however, the smooth steel-wheeled, vibrating drum and pneumatic tired rollers work well on coarse grained cohesionless material. The sheepsfoot and pneumatic tired rollers usually work well on cohesive or sticky material.

It should be remembered that the loose lift thickness requirements for each type of roller as indicated in these specifications are maximum. Density tests are performed to ensure the Contractors operations conform to the compaction criteria identified in the Standard Specifications. Geotechnical Test Method (GTM-6) Test Method for Rapid Earthwork Compaction Control and (GTM-9) Test Method for Earthwork Compaction Control by Sandcone or Volumeter Apparatus provide the procedures for the tests. If adequate compaction is not being attained, the thickness of the lift may have to be reduced or the size of the roller or number of passes may have to be increased.

12.2.2.1.2 Use of Plastic Soils

Notable problems have all been related to embankment construction using plastic soils. The problems have ranged from the "during construction" type, such as difficulty in handling and placing material, rutting, and weaving, to internal instability of the embankment soon after construction is completed.

Experiences in recent years have caused the Geotechnical Engineering Bureau to modify its operating procedure concerning the use of low- to high-plasticity soils as embankment construction material. Embankments that are to be constructed out of plastic soils are now more closely evaluated out, if deemed necessary, designed to prevent problems during construction. Provided right-of-way is not a problem, flattening the embankment side slope is a common approach. Other methods might be the inclusion of geosynthetic reinforcement, stabilization of embankment soil with chemicals, or zones construction (placement of granular materials in selected areas of the embankment).

The Geotechnical Engineering Bureau has documented several embankment problems.
associated with the use of plastic soils as an embankment construction material in a study titled "Evaluation of New York State Department of Transportation Clay Embankment Problems" (1987). Historically, the Geotechnical Engineering Bureau seldom designed embankments. The events identified dictated that the operating procedures be modified and strengthened in the area of the evaluation and design of embankments to be constructed with plastic soils.

**Design Procedure**

**PHASE I-IV**

The Departmental Geotechnical Engineer will study and evaluate all Design Reports with the following in mind:

1. Is the project in an area of known plastic soils?
2. Are there designed borrow areas shown in the report? Are these in plastic soil area?
3. What are the anticipated embankment heights (16 ft. or more)? Could the gradeline be lowered?
4. If there are cuts on the project, are they in plastic soil areas? Would the material logically be used in high fills?

During the site inspection, the Departmental Geotechnical Engineer will observe existing embankments, structure, etc., to evaluate their condition and note any problems which are apparent. If applicable, explorations in the designated borrow and cut areas should be progressed early in the design process. The samples should be investigated for plasticity and moisture content.

**PHASE V-VI**

Once the Departmental Geotechnical Engineer has determined that a strong probability exists that embankments 16 ft. or more in height can be expected to be constructed of plastic material, the Geotechnical Engineering Bureau should be requested to investigate the stability.

The Departmental Geotechnical Engineer should alert the Regional Project Manager that additional right-of-way may be necessary if slope are flattened or that additional money may be necessary if lightweight or granular fill or other special material must be used.

The Geotechnical Engineering Bureau will request the necessary explorations and testing, analyze the internal stability of the embankment and recommend special materials or construction procedures as necessary.

**Construction Procedure**

The special construction requirements of any designed embankments should be explained in detail at the preconstruction meeting. If a designated borrow area of plastic material is part of the contract, the characteristics of the material should be explained (e.g. moisture contents and their meaning, gray and brown material, etc.).

At this time, the Regional Geotechnical Engineer should designate the geotechnical test procedures to be used on the project. The project personnel should be made aware that there will be frequent field inspections of earthwork operations by geotechnical program personnel. In
addition, the project personnel should be instructed on the following:

- best type of compaction equipment for embankment materials like thickness,
- moisture influences on compaction,
- rutting under compactor,
- rutting under construction traffic,
- crowning and sealing surface daily,
- ability to have Contractor remove part of lift so that compaction test may be done in lower portion of lift,
- compaction testing frequency,
- when to ask for help from the Regional Geotechnical Engineer.

As soon as the Contractor selects the source of material for the high embankments, the Engineer should notify the Regional Geotechnical Engineer. If necessary, these areas should then be explored to determine moisture contents, plasticity and compaction control curves as determined by the Geotechnical Engineering Bureau.

During construction of the embankment, the Departmental Geotechnical Engineer will conduct frequent inspections. They will verify that the embankment, if designed, is being properly constructed or, if not designed, the lower lifts are being properly constructed. The Departmental Geotechnical Engineer will check SiteManager for results of compaction testing.

### 12.2.2.1.3 Use of Shales

Shales have been used as an embankment material in several States, without problems, when appropriate procedures for placement and compaction of embankments are followed. The major problem with shales is the lack of a method to properly classify the shale and predict its behavior. When exposed to water, shales can deteriorate to a low strength clay or silt. Some shales may also have expansive characteristics. Those problems can be aggravated where the climate is humid. Shales interbedded with limestone may prevent adequate compaction. Other factors that may contribute to poor embankment performance are:

- excessive lift thickness,
- inadequate compaction practices,
- steep side slopes,
- water infiltration,
- lack of sidehill benching,
- inadequate drainage.

The Materials Bureau does not consider shale as a construction material, therefore there is no specific testing procedure performed by the Materials Bureau to evaluate shales.

According to the FHWA study, many sites have successfully used shale as embankment material by restricting lift thickness and implementing appropriate compaction procedures. Other recommended procedures adopted by other states include:

- flatter slopes,
- encapsulated shale,
• stage construction,
• extensive drainage.

The use of all recommended measures may not be economically feasible. Criteria are need to determine when specific measures are required based on the existing conditions. Other than highways, the Corps of Engineers has used shales in earth and rockfill dams. The successful compaction and performance of shale materials are attributed to the use of heavy compaction equipment and compaction procedures stipulated for each type of shale. For weathered or soft shales which are easily broken down, thin lifts (6-8 in) and heavy tempting roller were used. For harder shales containing chunks and fines, thicker lifts (12-24 in. and maximum particle size of ⅔ of the lift thickness) and 50-ton rubber-tired rollers were used.

If shale were chosen to be used as embankment material, it would be advisable to have an evaluation performed by a Departmental Engineering Geologist to identify the shale encountered in the project area and carry out lab testing to estimate its durability. The shale then can be treated as soil-like or rock-like material and proper compaction procedure can be applied. Instrumentation to monitor the performance of the shale embankment is recommended.

12.2.2.2 Fill Placement Below Water

If material will be placed below the water table, material that does not require compaction such as Select Borrow or Select Fill should specified. Once above the water table, other appropriate material should be specified. The Select Borrow or Select Fill should be choked with finer material before placement of the overlying material. Alternately, geotextile separation or stabilization may be used to prevent migration of the finer overlying material into the void spaces of the coarser underlying material.

12.2.3 Embankments for Detention/Retention Facilities

12.2.3.1 Earth Dams

Dams are defined as any artificial barrier and its appurtenant works constructed for the purpose of holding water or any other fluid. Dams are regulated by the NYS Department of Environmental Conservation under Part 608 Use and Protection of Waters, as established under the Environmental Conservation Law, Article 15-0503. A permit to construct a dam is not required if:

1. The construction of a dam that has a height less than 15 feet, and a maximum impoundment capacity less than three million gallons; or
2. The construction of a dam with a height equal to or less than 6 feet, regardless of its maximum impoundment capacity, or a dam with a maximum impoundment capacity equal to or less than one million gallons, regardless of its height.

Embankments for detention/retention facilities not regulated by the NYSDEC should still be designed using the NYSDEC Guidelines for Design of Dams as the basis for design.
drainage facilities shall be analyzed for seepage and piping through the embankment fill and underlying soils. Stability of the fill and underlying soils subjected to seepage forces shall have a minimum safety factor of 1.5. Furthermore, the minimum safety factor for piping stability analysis shall be 1.5.

12.2.3.1.1 Hazard Classification

- Class “A”: dam failure will damage nothing more than isolated farm buildings, undeveloped lands or township or country roads.
- Class “B”: dam failure can damage homes, main highways, minor railroads, or interrupt use or service of relatively important public utilities.
- Class “C”: dam failure can cause loss of life, serious damage to homes, industrial or commercial buildings, important public utilities, main highways, and railroads.

12.2.3.1.2 Conduits

- Only two types of conduits are permitted on Hazard Class “B” structures, precast reinforced concrete pipe and cast-in-place reinforced concrete.
- On Hazard Class “A” structures, welded steel pipe or corrugated metal pipe may be used providing the depth of fill over the pipe does not exceed 15 ft. and the pipe diameter does not exceed 2 ft.
- All outlet conduits shall be designed for internal pressure equal to the fill reservoir head and for the superimposed embankment loads, acting separately.
- The minimum size diameter conduits shall be made watertight.
- Any pipe or conduit passing through an embankment shall have features constructed into the embankment whereby seepage occurring along the pipe or conduit is collected and safely conveyed to the downstream toe of the embankment. This can be accomplished by using a properly designed and constructed filter and drainage diaphragm. The filter and drainage diaphragm will be required unless it can be shown that anti-seep collars will adequately serve the purpose.
- Anti-seep collars will not be permitted for dams with a Dwight in excess of 20 ft. If anti-seep collars are used in lieu of a drainage diaphragm, they shall have a watertight connection to the pipe. Collar material shall be compatible with pipe materials. The anti-seep collars shall increase the seepage path along the pipe by at least 15%.
- A means of dissipating energy shall be provided at the outlet end of all conduits 1 ft. or more in diameter. If a plunge pool is used, the conduit should be cantilevered 8 ft. over a concrete, steel or treated timber support located near or at the downstream toe of the embankment. The plunge pool should be riprap-lined if a conduit 18 in. or more in diameter is used. The forgoing may apply to smaller pipe if the embankment’s downstream slope is steep and the soil erodible.

12.2.3.1.3 Geometry

- The downstream slope of earth dams without seepage control measures should be no steeper than 1V on 3H. If seepage control measures are provided, the downstream slope
CHAPTER 12
Embankments

should be no steeper than 1V on 2H.

• The upstream slope of earth dams should be no steeper than 1V on 3H.

• The sideslopes of homogeneous earth dams may have to be made flatter based on the results of design analyses or if the embankment material consists of fine grained plastic soils such as CL, MH or CH soils as described by the Unified Soil Classification System.

• The minimum allowable top width (W) of the embankment shall be the greater dimension of 10 ft. or W, as calculated by the following equation:

Equation 12-1

\[ W = 0.2H + 2.1 \]

Where H is the height of the embankment (meters)

• The top of the dam should be sloped to promote drainage and minimize surface infiltrations and should be cambered so that the design freeboard is maintained after post-construction settlement takes place.

• Earth dams, in general, should have seepage control measures, such as interior drainage blankets in order to keep the line of seepage from emerging on the downstream slope, and to control foundation seepage. Hazard Class “a” dams less than 20 ft. in height and Hazard Class “B” dams less than 10 ft. in height, if constructed on and of erosion-resistant materials do not require special measures to control seepage.

• In zoned embankments, consideration should be given to the relative permeability and gradation of embankment materials. No particle greater in size than 6 in. in maximum dimension should be allowed to be placed in the impervious zone of the dam.

• 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 foundation. The cutoff or key trench should be excavated to a depth of at least 3 ft. into the foundation soils and backfilled with compacted embankment material. Where the final dept of cutoff cannot be established with certainty during design, a note should appear on the plans stating that the final depth of the cutoff trench will be determined by the Engineer during the time of construction. Backfilling of the cutoff or key trench should be performed in the dry, unless special construction procedures are used. The bottom of the trench should be at least 8 ft. deep and should be increased in the case of dams more than 20 ft. high. The widths of complete cutoffs may be made considerably less if the cutoff is extended vertically a minimum distance of 4 ft. into impervious material. In the case of a cutoff or key trench extending into bedrock, the trench does not have to extend into rock. However, all shattered and disintegrated rock should be removed and surface fissures filled with cement grout. The need for pressure grouting rock foundations should evaluated and, if necessary, adequately provided for.

• Backfill around conduits should be placed in layers not thicker than 4 in. before compaction with particle size limited to 3 in. in greatest dimension and compacted to a density equal to that of the adjacent portion of the dam embankment regardless of compaction equipment.
Figure 12-2 Causes of Earth Dam Structural Failures
12.2.3.2 Effective Stress Under Seepage

A flow-net is a representation of the direction of flow and the hydraulic head existing at any point in a cross section of the soil through which water is flowing. It consists of two families.
of curves, one of which is known as flow lines, which define the direction of flow, and the other family known as equi-potential lines, which are contours of equal heads consisting of pressure and elevation heads. The velocity head being very small is neglected.

![Figure 12-5 Flow Net – Infinite Slope](image)

### 12.2.3.3.1 Flow Net Properties

1. Flow lines and equipotential lines intersect at right angles to form similar squares (for $k_\alpha = kH$). The sides of the squares are curved, but by repeated subdivision, true square areas are approached.
2. The quantity of water flowing between any two adjacent flow lines is the same throughout the flow-net.
3. The head loss between any two adjacent equipotential lines is the same throughout the flow-net.
4. The velocity of flow and hydraulic gradient are inversely proportional to the spacing to the flow-net lines and therefore seepage forces are maximum at points where flow lines or potential lines tend to converge.
5. Every flow-net has four boundaries, two of which are flow lines and two are potential lines. In cases of seepage beneath sheet pile cutoffs, masonry dams, and through the foundation below an impervious dam section, all boundaries are known or can be assumed. The impervious contacts constitute the upper and lower flow lines while the entrance and discharge faces constitute the potential boundaries. In other cases the boundary conditions are not completely defined. For example, in considering seepage through dams or dikes, the location of the upper flow boundary or the line of seepage is not directly known, but
must be determined mathematically, by model studies, or by graphical methods.

6. For conditions of steady seepage, there exists only one possible flow-net which fulfills the boundary conditions and gives the correct solution for the basic seepage equations.

### 12.2.3.3.2 Information Derived from Flow Nets

1. Quantity of Seepage

**Equation 12-2**

\[ Q = k h \frac{N_f}{N_p} \]

where:
- \( k \) = coefficient of permeability
- \( h \) = total hydrostatic head
- \( N_p \) = number of equal drops in potential
- \( N_f \) = number of flow channels

2. The Hydrostatic Head at any Point

The potential lines represent contours of equal head so that open-ended standpipes or piezometers installed at any point along a particular potential line will register the same height of water after equilibrium has been established. The hydrostatic head \( h \), at any point, can be determined graphically, or computed by the following equation:

**Equation 12-3**

\[ h_i = \frac{n_p h_p}{N_p} \]

Where \( N_p \) is the number of equal potential drops between the point under construction and zero potential.

3. The Average Hydraulic Gradient

**Equation 12-4**

\[ i = \frac{h_2}{L} \]

Where \( h_2 \) is the average difference in head between the approaching and receding faces of any element and \( L \) is the average length of the flow path. When the element consists of square of the flow-net, \( h_2 = \Delta h \) which is defined as the head loss between the two potential boundaries of the square and equal to a constant fraction of the total head loss, \( h \). If the element under consideration is at the discharge face, then the computed gradient is termed the discharge or escape gradient.
Maximum gradients are at points in the flow-net where the "squares" are smallest. Thus, with the aid of a flow-net, it is possible to determine at a glance critical points in the section where forces due to seeping water are maximum. The escape gradient is of great importance since it determines safety against "piping" and "boils" and indicates the relative importance and necessity for filters or other drainage treatment in the zone of seepage emergence.

4. The Seepage Force

**Equation 12-5**

\[ F = iA\gamma \]

Where \( \Delta h \), again, is the average difference in head between the entrance and discharge faces of the segment, \( a \) is the average width, \( A \) is the area of the segment and \( \gamma_w \) is the unit weight of water. The seepage forces obtained from a well-constructed flow-net can be combined with gravity forces for stability analyses.

*Figure 12-6 Flow Net*
Figure 12-7 Flow Net

Figure 12-8 Flow Net
Figure 12-9 Flow Net

Figure 12-10 Flow Net
12.2.4 Stability Assessment

In general, embankments 10 feet or less in height with 1V on 2H or flatter side slopes, may be designed based on past precedence and engineering judgment provided there are no known problem soil conditions such as liquefiable sands, organic soils, soft/loose soils, or potentially unstable soils (e.g. clay, estuarine deposits, peat). Embankments over 10 feet in height or any embankment on soft soils, in unstable areas/soils, or those comprised of lightweight fill require more in depth stability analyses, as do any embankments with side slope inclinations steeper than 1V on 2H. Moreover, any fill placed near or against a bridge abutment or foundation, or that can impact a nearby buried or above-ground structure, will likewise require stability analyses by the Departmental Geotechnical Engineer. Slope stability analysis shall be conducted in accordance with NYSDOT GDM Chapter 10.

Prior to the start of the stability analysis, the Departmental Geotechnical Engineer should determine key issues that need to be addressed. These include:

- Is the site underlain by soft silt, clay or peat? If so, a staged stability analysis may be required.
- Are site constraints such that slopes steeper than 1V on 2H are required? If so, a detailed slope stability assessment is needed to evaluate the various alternatives.
- Is the embankment temporary or permanent? Factors of safety for temporary embankments may be lower than for permanent ones, depending on the site conditions and the potential for variability.
- Will the new embankment impact nearby structures or bridge abutments? If so, more elaborate sampling, testing and analysis are required.
- Are there potentially liquefiable soils at the site? If soil, seismic analysis to evaluate this may be warranted (see NYSDOT GDM Chapter 9) and ground improvement may be needed.

Several methodologies for analyzing the stability of slopes are detailed or identified by reference in Chapter 10 and are directly applicable to earth embankments.

12.2.4.1 Safety Factors

Embankments that support structure foundations or walls or that could potentially impact such structures should be designed in accordance with the AASHTO LRFD Bridge Design Specifications and NYSDOT GDM chapters 11 and 17. If an LRFD design is required, a resistance factor is used in lieu of a safety factor. However, since slope stability in the AASHTO LRFD Bridge Design Specifications is assessed only for the service and extreme event (seismic) limit states, the load factors are equal to 1.0, and the resistance factor is simply the inverse of the factor of safety (i.e., 1/FS) that is calculated in most slope stability analysis procedures and computer programs. The resistance factors and safety factors for overall stability under static conditions are as follows:

- All embankments not supporting or potentially impacting structures shall have a minimum safety factor of 1.25.
Embankments supporting or potentially impacting non-critical structures shall have a resistance factor for overall stability of 0.75 (i.e., a safety factor of 1.3).

All Bridge Approach Embankments and embankments supporting critical structures shall have a resistance factor of 0.65 (i.e., a safety factor of 1.5). Critical structures are those for which failure would result in a life threatening safety hazard for the public, or for which failure and subsequent replacement or repair would be an intolerable financial burden to the citizens of New York State.

Under seismic conditions, only those portions of the new embankment that could impact an adjacent structure such as bridge abutments and foundations or nearby buildings require seismic analyses and an adequate overall stability resistance factor (i.e., a maximum resistance factor of 0.9 or a minimum factor of safety of 1.1). See NYSDOT GDM Chapter 9 for specific requirements regarding seismic design of embankments.

12.2.4.2 Strength Parameters

Strength parameters are required for any stability analysis. Strength parameters appropriate for the different types of stability analyses shall be determined based on NYSDOT GDM Chapter 6 and by reference to FHWA Geotechnical Engineering Circular No. 5 (Sabatini, et al., 2002).

If the critical stability is under drained conditions, such as in sand or gravel, then effective stress analysis using a peak friction angle is appropriate and should be used for stability assessment. In the case of over-consolidated fine grained soils, a friction angle based on residual strength may be appropriate. This is especially true for soils that exhibit strain softening or are particularly sensitive to shear strain.

If the critical stability is under undrained conditions, such as in most clays and silts, a total stress analysis using the undrained cohesion value with no friction is appropriate and should be used for stability assessment.

For staged construction, both short (undrained) and long term (drained) stability need to be assessed. At the start of a stage the input strength parameter is the undrained cohesion. The total shear strength of the fine-grained soil increases with time as the excessive pore water dissipates, and friction starts to contribute to the strength. A more detailed discussion regarding strength gain is presented in NYSDOT GDM Section 12.3.1.

12.2.4.3 Overstress Analysis

When the shear stress due to embankment loading exceeds the undrained shear strength of the foundation soil the foundation soil is said to be in a state of over stress. When over stress occurs uncontrollable and unpredictable undrained deformations take place in the foundation soil due to loss in strength in the overstressed zone and transfer of shear stress to zones of higher shear strength. If there is insufficient strength in the transferred zone total foundation failure will result.
The normal safety factor used to guard against an embankment foundation failure is 1.25. In most cases this safety factor is high enough to guard against an over-stress condition. However, in the case where a highly overconsolidated clay soil or dense granular material exists over a thin normally consolidated or slightly granular material exists over a thin normally consolidated or slightly overconsolidated clay soil such as exists in the Buffalo area, the overall safety factor may have to be increased somewhat to guard against this consideration. The load transfer capacity of the still surface soil in reducing shear stresses.

### 12.2.4.3.1 Applicability

Perform overstress analysis when clay is sensitive and shearing stress at any point exceeds shearing strength.

### 12.2.4.3.2 Procedure

1. **Check for Clay Sensitivity**
   - High moisture content clays with liquid limit exceeding moisture content by at least 10%.
   - Unusually sharp break in the shear stress-strain curve.
   - Ultimate strength is less than 80 percent of peak strength.
   - Larger than normal pore pressure build-up during shear strength test.

2. **Check for Overstress**
   - Plot shear stress with depth (See Figure 12-12) and compare to shearing strength with depth. The zone of overstress at time equals zero after embankment construction is where the shearing stress exceeds the shearing strength. This zone of overstress increases with time due to loss of shearing strength (ultimate strength) in the overstressed zone. The loss in strength continues until the net difference between the shearing stress and ultimate shearing strength is equal to zero (see Figure 12-11).
   - From experience, ultimate shearing strength has been found to be in the order of 0.75 times the maximum shearing strength for sensitive New York clays. The limits of overstress increase with time as indicated above. However, consolidation and strength gain also increase with time depending on the rate of embankment construction and the soils coefficients of consolidation. This tends to decrease the zone of overstress with time. These two rate processes are opposite in effect. To complicate matters, comparatively large strains occur in the overstressed zone causing a decrease in permeability, which affects consolidation and ultimate strength gain.

   Since sensitive soils are usually very slow consolidating and it is difficult to determine ultimate strength gain effects, strength increase due to consolidation should be neglected in the final analysis.

3. **Progress overall stability analyses by the Bishop Method utilizing design shear strength as obtained from Figure 12-11.**
Figure 12-11 Overstress Analysis on Sensitive Clays
12.2.5 Embankment Settlement Assessment

New embankments, as is true of almost any new construction, will add load to the underlying soils and cause those soils to settle. As discussed in NYSDOT GDM Chapter 8, the total settlement has up to three potential components: 1) immediate settlement, 2) consolidation settlement, and 3) secondary compression.
Settlement shall be assessed for all embankments. Even if the embankment has an adequate overall stability factor of safety, the performance of a highway embankment can be adversely affected by excessive differential settlement at the road surface.

Settlement analyses for embankments over soft soils require the compression index parameters for input. These parameters are typically obtained from standard one-dimensional oedometer tests of the fine-grained soils (see NYSDOT GDM Chapter 6 and Chapter 8 for additional information).

12.2.5.1 Settlement Impacts

Because primary consolidation and secondary compression can continue to occur long after the embankment is constructed (post construction settlement), they represent the major settlement concerns for embankment design and construction. Post construction settlement can damage structures and utilities located within the embankment, especially if those facilities are also supported by adjacent soils or foundations that do not settle appreciably, leading to differential settlements. Embankment settlement near an abutment could create an unwanted dip in the roadway surface, or downdrag and lateral squeeze forces on the foundations. See NYSDOT GDM Chapter 11 for more information regarding the use of bridge approach slabs to minimize the effects of differential settlement at the abutment, and the methodology to estimate downdrag loads on foundations.

If the primary consolidation is allowed to occur prior to placing utilities or building structures that would otherwise be impacted by the settlement, the impact is essentially mitigated. However, it can take weeks to years for primary settlement to be essentially complete, and significant secondary compression of organic soils can continue for decades. Many construction projects cannot absorb the scheduling impacts associated with waiting for primary consolidation and/or secondary compression to occur. Therefore, estimating the time rate of settlement is often as important as estimating the magnitude of settlement.

To establish the target settlement criteria, the tolerance of structures or utilities to differential settlement that will be impacted by the embankment settlement shall be determined. Lateral movement (i.e., lateral squeeze) caused by the embankment settlement and its effect on adjacent structures, including light, overhead sign, and signal foundations, shall also be considered. If structures or utilities are not impacted by the embankment settlement, settlement criteria are likely governed by the long-term maintenance needs of the roadway surfacing. In that case, the target settlement criteria shall be established with consideration of the effect differential settlement will have on the pavement life and surface smoothness.

12.2.5.2 Settlement Analysis

12.2.5.2.1 Primary Consolidation

The key parameters for evaluating the amount of settlement below an embankment include knowledge of:
• The subsurface profile including soil types, layering, groundwater level and unit weights;
• The compression indexes for primary, rebound and secondary compression from laboratory test data, correlations from index properties, and results from settlement monitoring programs completed for the site or nearby sites with similar soil conditions. See NYSDOT GDM Chapters 6 and 8 for additional information regarding selection of design parameters for settlement analysis.
• The geometry of the proposed fill embankment, including the unit weight of fill materials and any long term surcharge loads.

Primary consolidation settlement ($S_c$) occurs when the increase in load on a soil results in the generation of excess pore pressures within the soil voids. Depending on the type of soil, the time to reduce the excess pore pressures to some steady state level may be rapid (cohesionless soils) or may require long periods of time (cohesive soils). Therefore, primary consolidation settlement is comprised of two components, the amount of settlement and the time for settlement to occur. The amount of time required for cohesionless soil to settle is relatively short, typically occurring during construction, and the amount of settlement can be determined using immediate or elastic settlement theory. Therefore, the remainder of this Section will exclusively look at cohesive (clay and plastic silt) soils. Typically, normally consolidated (OCR = 1) soils undergo primary consolidation settlement. For the purpose of this Chapter, all plastic cohesive soils that have an OCR of less than four shall have the primary consolidation settlement determined.

The determination of primary consolidation settlement is based on six steps as presented in Table 12-2.

<table>
<thead>
<tr>
<th>Step</th>
<th>Item</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Computation of the initial vertical effective stress ($σ'<em>{vo}$) [total vertical stress ($σ</em>{vo}$) and pore water pressure ($u_o$)] of the layer(s) midpoint.</td>
</tr>
<tr>
<td>2</td>
<td>Determination of preconsolidation stresses ($σ'_p$ or $p'_c$).</td>
</tr>
<tr>
<td>3</td>
<td>Computation of changes in vertical effective stress ($Δσ'$) [associated with changes in both total stress ($Δσ_v$) and pore water pressures ($Δu$)] due to the construction.</td>
</tr>
<tr>
<td>4</td>
<td>Determination of compressibility of the clay or plastic silt.</td>
</tr>
<tr>
<td>5</td>
<td>Computation of layer compressions ($S_c$).</td>
</tr>
<tr>
<td>6</td>
<td>Computation of time for compressions.</td>
</tr>
</tbody>
</table>

Table 12-2 Primary Consolidation Settlement Steps
(Modified from Shallow Foundations – June 2001)

12.2.5.2.1.1 Amount of Settlement

In cohesive soils, loads are carried first by the pore water located in the interstitial space between the soil grains and then by the soil grains. The pore water pressure increases proportional to the load applied at that depth. As the excess pore water pressures reduce through drainage, the load is transferred to the soil grains. This drainage causes the settlement of cohesive soils. Therefore,
the settlement is directly proportional to the volume of water drained from the soil layer. Typically, the area loaded is large, resulting in the flow of water vertically (either up or down) and not horizontally. Therefore, one-dimensional consolidation theory may be used to determine settlements of cohesive soils.

The one-dimensional consolidation test is used to determine the parameters for use in one-dimensional consolidation theory. These parameters are indicated in Table 12-3.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>( C_c ) or ( C_{ec} )</td>
<td>Compression Index</td>
</tr>
<tr>
<td>( C_r ) or ( C_{er} )</td>
<td>Recompression Index</td>
</tr>
<tr>
<td>( C_\alpha ) or ( C_{\alpha\alpha} )</td>
<td>Secondary Compression Index</td>
</tr>
<tr>
<td>( \sigma'_p ) or ( p'_c )</td>
<td>Effective Preconsolidation Stress</td>
</tr>
<tr>
<td>( c_v )</td>
<td>Coefficient of Consolidation</td>
</tr>
<tr>
<td>( m_v )</td>
<td>Coefficient of Vertical Compression</td>
</tr>
</tbody>
</table>

**Table 12-3 Consolidation Parameters and Symbols**

The Coefficient of Consolidation and the Coefficient of Vertical Compression are required when determining the rate of settlement and will be discussed in the next Section. The effective preconsolidation stress is extremely important because it is used to determine if a soil is normally consolidated (NC) or overconsolidated (OC). In normally consolidated soils, the effective preconsolidation stress is equal to the existing effective overburden stress (i.e. \( \sigma'_{vo} = \sigma'_p \)) (see Figure 12-13). Normally consolidated soils tend to have large settlements. Overconsolidated soils tend to have an effective preconsolidation stress greater than the existing effective overburden stress (i.e. \( \sigma'_{vo} < \sigma'_p \)) (see Figure 12-14). Overconsolidated soils do not tend to have large settlements. One-dimensional consolidation tests shall be performed in accordance with NYSDOT GDM Chapter 6. In some locations, under consolidated soils (i.e. \( \sigma'_{vo} > \sigma'_p \)) (see Figure 12-15) have been known to exist. These soils are still consolidating under the weight of the soil and should be anticipated to have very large amounts of settlement.

Casagrande (1936) developed a graphical procedure for determining the preconsolidation stress. The Casagrande procedure for determining preconsolidation stress is outlined in Table 12-4.

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Locate the point of sharpest curvature on the e-log ( p ) or e-log ( p' ) curve.</td>
</tr>
<tr>
<td>2</td>
<td>From this point (a) (see Figures 12-16 or 12-17), draw a horizontal line (b) and a tangent (b) to the curve.</td>
</tr>
<tr>
<td>3</td>
<td>Bisect the angle formed by these two lines (c).</td>
</tr>
<tr>
<td>4</td>
<td>Extend the virgin curve (d) backward to intersect the bisector (c).</td>
</tr>
<tr>
<td>5</td>
<td>The point where these lines (d and c) cross determines the preconsolidation pressure (( \sigma'_p ) or ( p'_c )).</td>
</tr>
</tbody>
</table>
Table 12-4 Determination of Preconsolidation Stress
(Duncan and Buchignani, 1976)

Figure 12-13 Normally Consolidated
(Duncan and Buchignani, 1976)

Figure 12-14 Overconsolidated
(Duncan and Buchignani, 1976)

Figure 12-15 Underconsolidated
(Duncan and Buchignani, 1976)
Figure 12-16 Determination of Preconsolidation Stress from e-log p
(Duncan and Buchignani, 1976)
One-dimensional consolidation tests are sensitive to sample disturbance; therefore, the results of the test must be corrected. This method is applied to test results presented as ε-log p and ε-log p curves. Duncan and Buchignani (1976) provide methods for correcting both ε-log p and ε-log p for both normally consolidated and overconsolidated soils. The procedures for correcting the ε-log p curves (normally consolidated and overconsolidated) are presented in Table 12-5.
<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Locate point A at the intersection of $e_o$ and $\sigma'_p$ ($P_p$)</td>
</tr>
<tr>
<td>2</td>
<td>Locate point B on the virgin curve or extension where $e = 0.4e_o$.</td>
</tr>
<tr>
<td>3</td>
<td>Connect points A and B with a straight line – this is the corrected virgin curve.</td>
</tr>
</tbody>
</table>

**Overconsolidated Soil ($\sigma'_v < \sigma'_p$) (Figure 12-19)**

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Locate point A at the intersection of $e_o$ and $\sigma'_v$ ($P'_o$)</td>
</tr>
<tr>
<td>2</td>
<td>Draw a line from point A parallel to the rebound curve and locate point B where this line intersects $\sigma'_p$ ($P_p$)</td>
</tr>
<tr>
<td>3</td>
<td>Locate point C on the virgin curve or extension where $e = 0.4e_o$.</td>
</tr>
<tr>
<td>4</td>
<td>Connect points B and C with a straight line – this is the corrected virgin curve.</td>
</tr>
</tbody>
</table>

**Table 12-5 Correction of the e-log p Curve for Disturbance**  
(modified from Duncan and Buchignani, 1976)

![Figure 12-18 Corrected e-log p Normally Consolidated Curve](image)

(Duncan and Buchignani, 1976)
The Duncan and Buchignani (1976) procedures for correcting the $\varepsilon$-log $p$ curves (normally consolidated and overconsolidated) are presented in Table 12-6.

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Locate point A at the intersection of $\varepsilon$=0 and $\sigma'_p$ ($P_p$)</td>
</tr>
<tr>
<td>2</td>
<td>Locate point B on the virgin curve or extension where $\varepsilon$ = 0.4.</td>
</tr>
<tr>
<td>3</td>
<td>Connect points A and B with a straight line – this is the corrected virgin curve.</td>
</tr>
</tbody>
</table>

**Overconsolidated Soil ($\sigma'_{vo} < \sigma'_p$) (Figure 12-21)**

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Locate point A at the intersection of $\varepsilon$=0 and $\sigma'_{vo}$ ($P'_o$)</td>
</tr>
<tr>
<td>2</td>
<td>Draw a line from point A parallel to the rebound curve and locate point B where this line intersects $\sigma'_p$ ($P_p$)</td>
</tr>
<tr>
<td>3</td>
<td>Locate point C on the virgin curve or extension where $\varepsilon$ = 0.4.</td>
</tr>
<tr>
<td>4</td>
<td>Connect points B and C with a straight line – this is the corrected virgin curve.</td>
</tr>
</tbody>
</table>

**Table 12-6 Correction of the $\varepsilon$-log $p$ Curve for Disturbance (modified from Duncan and Buchignani, 1976)**
The compression ($C_c$ or $C_{cc}$) and recompression ($C_r$ or $C_{cr}$) indices are determined from the corrected curves. The compression ($C_c$ or $C_{cc}$) index is the slope of the virgin portion of the corrected curve, either $\varepsilon$-log $p$ ($C_c$) or $\varepsilon$-log $p$ ($C_{cc}$), over a full cycle. The recompression index is the slope of the recompression portion of the corrected curve, either $\varepsilon$-log $p$ ($C_r$) or $\varepsilon$-log $p$ ($C_{cr}$) over a full logarithmic cycle. If the slope of either portion of the curve does not extend over a full logarithmic cycle extend the line in both directions to cover a full logarithmic cycle.
The determination of the amount of settlement is dependent on whether the soil is normally consolidated, overconsolidated or a combination of both. The amount of settlement for underconsolidated soils is determined the same as normally consolidated soil. In addition, the way the data is presented, either e-log p or ε-log p curves, will also determine which equation is used.

Presented in Table 12-7 are the equations for determining the total primary consolidation settlement ($S_c$).

<table>
<thead>
<tr>
<th>e-log p</th>
<th>Equation 12-6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma'<em>{vo} = \sigma'</em>{p}$</td>
<td>$S_c = \sum_i H_o \frac{C_c}{1+e_o} \left(\log \frac{\sigma'<em>{f}}{\sigma'</em>{vo}}\right)$</td>
</tr>
<tr>
<td>$\sigma'<em>{f} &lt; \sigma'</em>{p}$</td>
<td>$S_c = \sum_i H_o \frac{C_r}{1+e_o} \left(\log \frac{\sigma'<em>{f}}{\sigma'</em>{vo}}\right)$</td>
</tr>
<tr>
<td>$\sigma'<em>{vo} &lt; \sigma'</em>{p} &lt; \sigma'_{f}$</td>
<td>$S_c = \sum_i H_o \left[\frac{C_c}{1+e_o} \left(\log \frac{\sigma'<em>{f}}{\sigma'</em>{vo}}\right) + \frac{C_r}{1+e_o} \left(\log \frac{\sigma'<em>{p}}{\sigma'</em>{vo}}\right)\right]$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>e-log p</th>
<th>Equation 12-9</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma'<em>{vo} = \sigma'</em>{p}$</td>
<td>$S_c = \sum_i H_o C_g (\log \frac{\sigma'<em>{f}}{\sigma'</em>{vo}})$</td>
</tr>
<tr>
<td>$\sigma'<em>{f} &lt; \sigma'</em>{p}$</td>
<td>$S_c = \sum_i H_o C_g (\log \frac{\sigma'<em>{f}}{\sigma'</em>{vo}})$</td>
</tr>
<tr>
<td>$\sigma'<em>{vo} &lt; \sigma'</em>{p} &lt; \sigma'_{f}$</td>
<td>$S_c = \sum_i H_o [C_g (\log \frac{\sigma'<em>{f}}{\sigma'</em>{vo}}) + C_g (\log \frac{\sigma'<em>{p}}{\sigma'</em>{vo}})]$</td>
</tr>
</tbody>
</table>

### Table 12-7 Primary Consolidation Settlement Equations

Where:
- $H_o$ = Thickness of $i$th layer
- $e_o$ = Initial void ratio of $i$th layer
- $\sigma'_{f}$ = Final pressure on $i$th layer

**Equation 12-12**

$$\sigma'_{f} = \sigma'_{vo} + \Delta \sigma_v$$

Where:
- $\sigma'_{vo}$ = Initial vertical effective stress on the $i$th layer
- $\Delta \sigma_v$ = Change in stress on the $i$th layer
12.2.5.2.1.2 Time for Settlement

As indicated previously, consolidation settlement occurs when a load is applied to a saturated cohesive soil squeezing water out from between the soil grains. The length of time for primary consolidation settlement to occur is a function of compressibility and permeability of the soil. The Coefficient of Consolidation ($C_v$) is related to the permeability ($k$) and the Coefficient of Vertical Compression ($m_v$) as indicated in the following equations:

**Equation 12-13**

$$C_v = \frac{1}{\gamma_w} \frac{k}{m_v}$$

**Equation 12-14**

$$m_v = \frac{\Delta \varepsilon_v}{\Delta \sigma'_v}$$

**Equation 12-15**

$$m_v = \frac{\Delta e}{\Delta \sigma'_v (1 + e_{av})}$$

Where:
- $\gamma_w =$ Unit weight of water
- $\Delta \varepsilon_v =$ Change in sample height
- $\Delta \sigma'_v =$ Change in effective stress
- $\Delta e =$ Change in void ratio
- $e_{av} =$ Average void ratio during consolidation

The Coefficient of Consolidation is typically provided as part of the results of consolidation testing. A $C_v$ is provided for each load increment applied during the test. The $C_v$ used to determine the time for primary consolidation settlement should be the one at the stress (load increment) closest to the anticipated field conditions. If $C_v$ is not provided, it should be determined using the procedures outlined in *Shallow Foundations*, FHWA NHI-01-023.

The time ($t$) for primary consolidation settlement is determined using the equation listed below.

**Equation 12-16**

$$t = \frac{TH^2}{C_v}$$

Where:
- $t =$ Time for settlement
CHAPTER 12
Embankments

T = Time factor from Equation 12-17
H_o = Maximum distance pore water must flow through
c_v = Coefficient of Consolidation

The distance the pore water must flow through is affected by the permeability of the materials above and below the cohesive material. If the cohesive material is between two cohesionless materials (i.e. highly permeable materials) then the thickness of the cohesive material is cut in half. This is also known as two-way or double drainage. If the cohesive material is bordered by an impermeable material, then the drainage path is the full thickness of the layer. This is also called one-way or single drainage. According to Das (1990), the time factor (T) is related to the degree of consolidation (U) in the following equations:

**Equation 12-17**

\[ T = \frac{\pi}{4} \left( \frac{U\%}{100} \right)^2 \leq 2.5 \]

\[ \left[ 1 - \left( \frac{U\%}{100} \right)^{5.6} \right]^{0.357} \]

for

**Equation 12-18**

\[ 0 \leq U\% \leq 100\% \]

Where:
U\% = Degree of consolidation in percent

Where U equals 100\%, T approaches infinity (\(\infty\)). The limit of T indicated in equation 12-17 results in a U of 99.3%. If the amount of settlement at a U of 99.3% exceeds the acceptable performance limits, contact the Geotechnical Engineering Bureau for guidance.

**12.2.5.2.2 Secondary Compression**

For organic soils and highly plastic soils determined to have an appreciable secondary settlement component, the secondary compression should be determined. Note the secondary compression is in general independent of the stress state and theoretically is a function only of the secondary compression index and time.

Secondary compression settlement occurs after the completion of primary consolidation settlement (i.e. U=99.3%). This type of compression settlement occurs when the soil continues to vertically displace despite the fact that the excess pore pressures have essentially dissipated. Secondary compression typically occurs in highly plastic (PI>21) or organic (percent organics>30%) cohesive soils. Secondary compression settlement is evident on both the e-log p and \(e\)-log p curves (see Figure 12-22).
The Coefficient of Secondary Compression \( C_\alpha \) can be determined using the slope of the corrected curve over one full logarithmic cycle. As with primary consolidation settlement, the amount of secondary compression settlement can be determined using either the \( e/\log p \) or \( \varepsilon/\log p \) curves. Presented in Table 12-8 are the equations for determining secondary compression settlement.

<table>
<thead>
<tr>
<th>( e/\log p )</th>
<th>( S_s = \sum H_o \frac{C_\alpha}{1+e_o} \log \left( \frac{t_2}{t_1} \right) )</th>
<th>Equation 12-19</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E/\log p )</td>
<td>( S_s = \sum H_o C_\alpha \log \left( \frac{t_2}{t_1} \right) )</td>
<td>Equation 12-20</td>
</tr>
</tbody>
</table>

Table 12-8 Secondary Compression Settlement Equations

Where:
- \( H_o \) = Thickness of \( i^{th} \) layer
- \( e_o \) = Initial void ratio of \( i^{th} \) layer
- \( t_1 \) = Time when secondary compression begins (i.e. \( U=99.3\% \))
- \( t_2 \) = Time when secondary compression is desired, usually the service life of structure
Secondary compression settlement is sometimes confused with creep. As indicated previously, secondary compression settlement occurs after the pore pressures have achieved a steady state condition and the settlement is the result of particle movement or realignment. Creep occurs after the pore pressures have achieved a steady state condition and there is no volume change. Creep is related to shear strength rather than compressibility. In many cases, it is not possible to distinguish between creep and secondary compression settlement.

Since secondary compression is not a function of the stress state in the soil but rather how the soil breaks down over time, techniques such as surcharging to pre-induce the secondary settlement are sometimes only partially effective at mitigating the secondary compression. Often the owner must accept the risks and maintenance costs associated with secondary compression if a cost/benefit analysis indicates that mitigation techniques such as using lightweight fills or overexcavating and replacing the highly compressible soils are too costly.

### 12.2.5.2.3 Estimating Settlements in Organic Soils

Secondary compression settlement for organic soils may be estimated using the following procedure:

1. Enter Figure 12-23 with the $\frac{\sigma_f}{\sigma_{vo}}$ ratio and moisture content, determine $S_c$.
2. Enter Figure 12-24 with the $\frac{\sigma_f}{\sigma_{vo}}$ ratio and moisture content, determine $e_f$.
3. Enter Figure 12-25 with moisture content and the time in years after completion of primary settlement, determine $\Delta e$.
4. The amount of total settlement is computed using the following equation:

**Equation 12-21**

$$S = S_c H + \frac{\Delta e H}{1 + e_f}$$

Where:
- $S =$ Total settlement
- $H =$ Total layer thickness
- $S_c =$ The amount of primary settlement in a 1 ft. layer
- $\Delta e \over 1 + e_f =$ The amount of secondary settlement in a 1 ft. layer for a given time.

Table 12-9 provides the assumptions for Figures 12-23, 24, and 25.
### Table 12-9 Estimating Settlements in Organic Soils – Assumptions for Figures 12-23, 24, and 25

<table>
<thead>
<tr>
<th>Moisture Content (%)</th>
<th>Initial Void Ratio (e₀)</th>
<th>Coefficient of Secondary Consolidation (Cₐ)</th>
<th>Compression Index (Cₖ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>2.2</td>
<td>0.05</td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>4.2</td>
<td>0.10</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td>6.0</td>
<td>0.15</td>
<td></td>
</tr>
<tr>
<td>400</td>
<td>8.0</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>500</td>
<td>10.0</td>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>600</td>
<td>11.6</td>
<td>0.27</td>
<td></td>
</tr>
<tr>
<td>700</td>
<td>13.0</td>
<td>0.30</td>
<td></td>
</tr>
</tbody>
</table>

Note: It was assumed that the primary settlement will be completed in 1 year. Make necessary computations for time periods less than 1 year.
Figure 12-23 Estimating Settlements in Organic Soils
Figure 12-24 Estimating Settlements in Organic Soils
Figure 12-25 Estimating Settlements in Organic Soils
12.2.5.2.4 Settlement of Granular Soils

Even if it were possible to obtain undisturbed samples of granular soils, the variability of granular deposits would still lead to considerable uncertainty. Water-laid sands and gravels, having been deposited in flowing water, are commonly more variable and irregular than silts and clays, which have settled out in almost still water.

Consequently, the Geotechnical Engineer, not having a clear idea of the settlement mechanism in granular soils and not being able to measure their compressibility, is left with empirical methods. Empirical methods attempt to infer a property, in this case the compressibility of the soil, by correlating it against an easily measurable property to which it is thought to be related, such as the sampler spoon penetration resistance. A problem with empirical methods is that the determining factors are not necessarily the same for the inferred as for the measured property or they may affect the two properties to different degrees. An example of such a factor is precompression, which has a great effect on compressibility but a minor one on penetration resistance. Consequently, empirical methods are valid only for those conditions under which they have been derived. Their use under conditions that may be different introduces considerable uncertainty.

Considering the above, the Geotechnical Engineering Bureau has had remarkable success in estimating settlements in granular soils. Review of settlement readings that pass through the GEB indicates that the calculated settlements are generally within 50% of the observed magnitude. Exceptions to the rule have occurred in some medium compact sand-gravel mixtures, where the observed settlements have been considerably smaller than estimated, and in very loose and loose silts and sandy silts, where the use of “Hough’s Charts” has been found to underestimate the settlement. It may be that our settlement computation method is well suited for most granular soils with little or no precompression and that the settlement overestimates have occurred in precompressed soils. It can be seen, however, that settlement computations in granular soils are far from exact and that cases where we have closely predicted the actual settlement involve probably a considerable amount of luck.

Hough’s Method

Hough’s Method of computing settlement in granular soils is used by few Geotechnical Engineers. The Geotechnical Engineering Bureau has used it since the mid-1960’s for the following reasons:

- The conservativeness of the unmodified Terzaghi-Peck Method, the prevalent method at the time, and
- The ability to include the effects of soil layers with different compressibilities in the analysis.

A modification that the GEB has made to Hough’s Method is the adjustment of sampler penetration resistance for overburden pressure intensity.
The main weakness of Hough’s Method is the lack of information regarding its origin. Another is the implied assumption that settlement in granular materials is the result of one-dimensional compression and that shear displacement under foundations has no effect on settlement. If Hough’s Method were strictly valid, load-settlement curves for plate load tests would be concave-up on an arithmetic plot with settlement increasing down. Because of shear displacements, this is not the case.

The following recommendations should be observed when using Hough’s Method:

1. Hough’s Method should not be used in cohesive (clayey or organic) materials. The moisture content, in combination with geologic or strength information, is a much more reliable indicator of the compressibility of these soils. Neither should Hough’s Method be used for granular soils containing significant amounts of organic material (topsoil; alluvium containing organic material). While for these soils, use of the moisture content to determine $C_c$ is conservative, the use of Hough’s $C$ based on penetration resistance would be unconservative.

2. Do not use blow count to determine $C$ (“Bearing capacity index”) in uncompacted fill material. These materials have been deposited in a much different manner than natural soils and are often in an extremely loose state. The penetration resistance in uncompacted fill may yield an unconservative indication of compressibility because of the presence of coarse particles and the possibility of it exiting at bulking density. Fine sandy fill material with no cinders, ashes or other foreign material and having a sampler penetration resistance (unadjusted) of 5 to 15 blows per foot has been found on two projects to act as if $C=50\pm10$.

3. Hough’s $C$ vs. $N$ charts appear to be unconservative for normally consolidated silts and sandy silts with unadjusted penetration resistance ($N_{NY}$) of 5 blows or less per foot. For such soils, $C=10$ should be used at $N_{NY}=1$, increasing to $C=25$ at $N_{NY}=5$.

4. The use of Hough’s charts, as well as the use of other methods that do not distinguish between normally consolidated and precompressed sands, has been found to be very conservative in Colonie sands. In the few cases where settlements have been monitored, measured settlement have been about 20% of calculated settlements. This may be a result of precompression of the sands soon after deposition by wind-driven traveling sand dunes. It has been found that even where $N_{NY}$ is 1 or 2 blows per foot, $C=200$ can be used. This pertains only to the naturally deposited Colonie sands, not to sand that has been used, with little or no compaction, to fill excavations or natural depressions.

5. The NYSDOT’s method for adjusting or correcting the blow count is based on Alpan’s “Estimating the Settlements of foundations on Sand”. This method results in a considerably greater increase in the blow count at shallow depths than subsequently recommended by Peck.

6. The settlement of structures constructed over embankment material to be placed in the same contract cannot be predicted because the type of material the Contractor will use is generally not known. If only a thin layer of embankment material is involved, settlements will be small. In the case of high embankments (more than 20 ft. of embankment material under footing), settlements and lateral movements may occur in the embankment if the embankment material contains silt or clay and is placed and compacted at a moisture
content higher than optimum. In the case of silty clay or clay as embankment material, problems may appear even with lower embankment heights. If it seems likely that wet or clayey soils will be used for embankment construction, consideration should be given to requiring material providing support to the structure to be compacted dry of optimum. An alternate solution is to increase the depth of select structural fill.

7. If the foundation for a structure is to be constructed below groundwater level (or the level of water in an adjacent body of water), there is a possibility that uplift or incipient piping may loosen the soil in the bottom of the excavation. If this were to happen, the penetration resistance observed before construction would no longer be valid. This generally is not a problem for shallow open excavations, unless the soil stratification is adverse (highly pervious soils or rock underlying less pervious soil) or the soils are difficult to drain and easily disturbed (silt and fine silty sands).

8. In selecting a representative blow count on which to base C, high blows in samples containing gravel should be disregarded. In a deposit of cohesionless soil, the blows obtained for the zones of the deposit with the least amount of grave should be used to determine C.

9. Some types of soils have been deposited under conditions such that apparent cohesion resulting from capillary forces (in uncompacted fill) or inter-particle attraction (in loess: wind-deposited silt, not found in New York State) permit the development of a very loose soil structure. Saturation of the soil by water destroys the apparent cohesion and a large part of the inter-particle attraction, thereby causing settlements. This is something that should be considered when dealing with uncompacted fill as a foundation material.
Figure 12-26 Bearing Capacity Index “C”
(Hough, 1959)
Figure 12-27 Bearing Capacity Index “C”
(Hough, 1959)
12.2.5.3 Stress Distribution

One of the primary input parameters for settlement analysis is the increase in vertical stress at the midpoint of the layer being evaluated caused by the embankment or other imposed loads. It is generally quite conservative to assume the increase in vertical stress at depth is equal to the bearing pressure exerted by the embankment at the ground surface. In addition to the bearing pressure exerted at the ground surface, other factors influencing the stress distribution at depth include the geometry (length and width) of the embankment, inclination of the embankment side slopes, depth below the ground surface to the layer being evaluated, and horizontal distance from the center of the load to the point in question. Several methods are available to estimate the stress distribution.

12.2.5.3.1 Simple 2V:1H Method

Perhaps the simplest approach to estimate stress distribution at depth is using the 2V:1H (vertical to horizontal) method. This empirical approach is based on the assumption that the area the load acts over increases geometrically with depth as depicted in Figure 12-28. Since the same vertical load is spread over a much larger area at depth, the unit stress decreases.
Boussinesq (1885) developed equations for evaluating the stress state in a homogenous, isotropic, linearly elastic half-space for a point load acting perpendicular to the surface. Elasticity based methods should be used to estimate the vertical stress increase in subsurface strata due to an embankment loading, or embankment load in combination with other surcharge loads. While most soils are not elastic materials, the theory of elasticity is the most widely used methodology to estimate the stress distribution in a soil deposit from a surface load. Most simplifying charts and the subroutines in programs such as SAF-1 and EMBANK are based on the theory of elasticity. Some are based on Boussinesq theory and some on Westergaard’s equations.
(Westegaard, 1938), which also include Poisson’s ratio (relates the ratio of strain applied in one direction to strain induced in an orthogonal direction).

### 12.2.5.3.3 Empirical Charts

The equations for the theory of elasticity have been incorporated into design charts and tables for typical loading scenarios, such as below a foundation or an embankment. Almost all foundation engineering textbooks include these charts. For convenience, charts to evaluate embankment loading are included as Figures 12-29 and 12-30.

![Figure 12-29 Influence Factors for Vertical Stress Under a Very Long Embankment](after NAVFAC, 1971 as reported in Holtz and Kovacs, 1981)
Figure 12-30 Influence Values for Vertical Stress Under the Corners of a Triangular Load of Limited Length (after NAVFAC, 1971 as reported in Holtz and Kovacs, 1981)
12.2.5.3.4 Rate of Settlement

The time rate of primary consolidation is typically estimated using equations based on Terzaghi’s one-dimensional consolidation theory. The time rate of primary consolidation shall be estimated as described in NYSDOT GDM Section 12.2.4.2.1.2.

The value of $C_v$ should be determined from the laboratory test results, piezocone testing, and/or back-calculation from settlement monitoring data obtained at the site or from a nearby site with similar geologic and soil conditions.

The length of the drainage path is perhaps the most critical parameter because the time to achieve a certain percentage of consolidation is a function of the square of the drainage path length. This is where incorporating CPTs into the exploration program can be beneficial, as they provide a nearly continuous evaluation of the soil profile, including thin sand layers that can easily be missed in a typical boring exploration program. The thin sand lenses can significantly reduce the drainage path length.

It is important to note some of the assumptions used by Terzaghi’s theory to understand some of its limitations. The theory assumes small strains such that the coefficient of compressibility of the soil and the coefficient of permeability remain essentially constant. The theory also assumes there is no secondary compression. Both of these assumptions are not completely valid for extremely compressible soils such as organic deposits and some clays. Therefore, considerable judgment is required to when using Terzaghi’s theory to evaluate the time rate of settlement for these types of soil. In these instances, or when the consolidation process is very long, it may be beneficial to complete a preload test at the site with sufficient monitoring to assess both the magnitude and time rate of settlement for the site.

12.2.5.4 Analytical Tools

The primary consolidation and secondary settlement can be calculated by hand or by using computer programs such as SAF-1 (Prototype Engineering Inc., 1993) or EMBANK (FHWA, 1993). Alternatively, spreadsheet solutions can be easily developed. The advantage of computer programs such as SAF-1 and EMBANK are that multiple runs can be made quickly, and they include subroutines to estimate the increased vertical effective stress caused by the embankment or other loading conditions.

12.3 STABILITY MITIGATION

A variety of techniques are available to mitigate inadequate slope stability for new embankments or embankment widenings. These techniques include staged construction to allow for the underlying soils to gain strength, base reinforcement, ground improvement, use of lightweight fill, and construction of toe berms and shear keys. A summary of these instability mitigation techniques is presented below along with the key design considerations.
12.3.1 Staged Construction

Where soft compressible soils are present below a new embankment location and it is not economical to remove and replace these soils with compacted fill, the embankment can be constructed in stages to allow the strength of the compressible soils to increase under the weight of new fill. An increase in effective stress in the subsoil results in an increase in the shear strength. Thus, when the soil strength is initially too low to ensure stability of the final design height of the embankment, its strength may be gradually increased by constructing the embankment in stages, with sufficient time between each increment of embankment height to ensure adequate porewater pressure dissipation and gain in shear strength to support the subsequent load increment.

The method is applicable when the permeability of the subsoil is large enough, or the total dept of compressible strata sufficiently thin, to permit dissipation of excess porewater pressures within available construction time.

In situations where these conditions are not fulfilled, the method may be used in conjunction with other techniques such as the installation of vertical drains or the use of berms.
In order to define the allowable height of fill for each stage and maximum rate of construction, detailed geotechnical analysis is required. This analysis typically requires consolidated undrained (CU), consolidated drained (CD) or consolidated undrained with pore pressure measurements (CU_p), and initial undrained (UU) shear strength parameters for the foundation soils along with the at-rest earth pressure coefficient (K_o), soil unit weights, and the coefficient of consolidation (c_v).

The analysis to define the height of fill placed during each stage and the rate at which the fill is placed is typically completed using a limit equilibrium slope stability program along with time.
rate of settlement analysis to estimate the percent consolidation required for stability. Stability analyses are conducted to determine the factors of safety for various heights of embankment and various degrees of consolidation throughout the subsurface soil profile. The maximum safe height of embankment that can be placed with no rate restrictions is first determined in order that the maximum consolidation may be obtained during the waiting period. The time interval between fill increments and the height increments is next determined. As each increment is placed, the factor of safety should not decrease below an established minimum value.

Figure 12-32 Flow Chart for Stage Construction Control

Alternatively, numerical modeling programs, such as FLAC and PLAXIS, can be used to assess staged construction, subject to the approval of the NYSDOT Departmental Geotechnical Engineer. Numerical modeling has some advantages over limit equilibrium approaches in that both the consolidation and stability can be evaluated concurrently. The disadvantages of numerical modeling include the lack of available field verification of modeling results, and most
geotechnical engineers are more familiar with limit equilibrium approaches than numerical modeling. The accuracy of the input parameters can be critical to the accuracy of numerical approaches. Steps for using a limit equilibrium approach to evaluate staged construction are presented below.

For staged construction, two general approaches to assessing the criteria used during construction to control the rate of embankment fill placement to allow the necessary strength gain to occur in the soft subsoils are available. The two approaches are total stress analysis and effective stress analysis:

• For the total stress approach, the rate of embankment construction is controlled through development of a schedule of maximum fill lift heights and intermediate fill construction delay periods. During these delay periods the fill lift that was placed is allowed to settle until an adequate amount of consolidation of the soft subsoil can occur. Once the desired amount of consolidation has occurs, placement of the next lift of fill can begin. These maximum fill lift thicknesses and intermediate delay periods are estimated during design. For this approach, field measurements such as the rate of settlement or the rate of pore pressure decrease should be obtained to verify that the design assumptions regarding rate of consolidation are correct. However, if only a small amount of consolidation is required (e.g., 20 to 40% consolidation), it may not be feasible to determine of the desired amount of consolidation has occurred, since the rate of consolidation may still be on the linear portion of the curve to this point. Another approach may be to determine if the magnitude of settlement expected at that stage, considering the degree of consolidation desired, has been achieved. In either case, some judgment will need to be applied when interpreting such data and deciding whether or not to reduce or extend the estimated delay period during fill construction.

• For the effective stress approach, the pore pressure increase beneath the embankment in the soft subsoil is monitored and used to control the rate of embankment construction. During construction, the pore pressure increase is not allowed to exceed a critical amount to insure embankment stability. The critical amount is generally controlled in the contract by use of the pore pressure ratio \( r_u \), which is the ratio of pore pressure to total overburden stress. To accomplish this pore pressure measurement, pore pressure transducers are typically located at key locations beneath the embankment to capture the pore pressure increase caused by consolidation stress. As is true of the total stress approach, some judgment will need to be applied when interpreting such data and deciding whether or not to reduce or extend the estimated delay period during fill construction, as the estimate of the key parameters may vary from the actual values of the key parameters in the field. Also, this approach may not be feasible if the soil contains a high percentage of organic material and trapped gases, causing the pore pressure readings to be too high and not drop off as consolidation occurs.

Since both approaches have limitations and uncertainties, it is generally desirable to analyze the embankment using both approaches, to have available a backup plan to control the rate of fill placement, if the field data proves difficult to interpret. Furthermore, if the effective stress method is used, a total stress analysis should in general always be conducted to obtain an estimate of the time required to build the fill for contract bidding purposes.
Site investigation must be extremely thorough as the construction sequence is dependent on the rate of gain of shear strength. Thus a good knowledge of the relations between effective stress and shear strength as well as the consolidation parameters for the substrata are needed if stability is to be assured and differential settlement avoided without exceeding the time allowed for the contract. Field instrumentation is an absolute requirement to ensure success.

It is usually necessary to limit the total time of stage construction to ensure that sufficient time is available following completion of the embankment, to allow consolidation settlements to be largely completed while staying within the typical duration of a contract. In some circumstances, a temporary pavement may be required to accommodate differential settlements continuing after the end of construction.

Detailed procedures for both approaches are provided in the sections that follow. These procedures have been developed based on information provided in Ladd (1991), Symons (1976), Skempton and Bishop (1955), R. D. Holtz (personal communication, 1993), S. Sharma (personal communication, 1993), and R. Cheney (personal communication, 1993). Examples of the application of these procedures are provided in NYSDOT GDM Appendix 9-A.

### 12.3.1.1 Design Parameters

First, define the problem in terms of embankment geometry, soil stratigraphy, and water table information.

The Departmental Geotechnical Engineer must make some basic assumptions regarding the fill properties. Typically, the designer assumes presumptive values for the embankment fill, since the specific source of the fill material is usually not known at the time of design. However, specialized soils laboratory tests should be performed for the soft underlying soils. From undisturbed samples, the Departmental Geotechnical Engineer should obtain Unconsolidated Undrained (UU) triaxial tests and Consolidated Undrained (CU) triaxial tests with pore pressure measurements. These tests should be used to determine the initial undrained shear strength available. The CU test with pore pressure measurements should also be used to determine the shear strength envelope needed for total or effective stress analyses. In addition, the Departmental Geotechnical Engineer should obtain consolidation test data to determine compressibility of the soft underlying soils as well as the rate of consolidation for the compressible strata ($C_v$). $C_v$ will be an important parameter for determining the amount of time required during consolidation to gain the soil shear strength needed.

In general triaxial tests should be performed at the initial confining stress ($P_0'$) for the sample as determined from the unit weight and the depth that the sample was obtained.
Equation 12-22

\[ P_o' = D \gamma' \]

D = Sample Depth in feet
\( \gamma' \) = Effective Unit Weight (pcf)

The third point in the triaxial test is usually performed at \( 4P_o' \). During the triaxial testing it is important to monitor pore pressure to determine the pore pressure parameters A and B. Note that A and B are not constant but change with the stress path of the soil. These parameters are defined as follows:

Equation 12-23

\[ A = \frac{\Delta U}{\Delta \sigma_1} \]

Equation 12-24

\[ B = \frac{\Delta U}{\Delta \sigma_3} \]

12.3.1.2 In-Situ Shear Strength and Determination of Stability Assuming Undrained Loading

The first step in any embankment design over soft cohesive soils is to assess its stability assuming undrained conditions throughout the entire fill construction period. If the stability of the embankment is adequate assuming undrained conditions, there is no need to perform a staged construction design. The UU shear strength data, as well as the initial shear strength from CU tests, can be used for this assessment.

The Departmental Geotechnical Engineer should be aware that sample disturbance can result in incorrect values of strength for normally consolidated fine grained soils. Figure 12-33 shows how to correctly obtain the cohesive strength for short term, undrained loading.
When a normally consolidated sample is obtained, the initial effective stress ($P'_o$) and void ratio correspond to position 1 on the $e$ - Log $P$ curve shown in Figure 12-33. As the stress changes, the sample will undergo some rebound effects and will move towards point 2 on the $e$ – Log $P$ curve. Generally, when a UU test is performed, the sample state corresponds to position 2 on the $e$ – Log $P$ curve. Samples that are reconsolidated to the initial effective stress ($P'_o$) during CU testing undergo a void ratio change and will generally be at point 3 on the $e$ – Log $P$ curve after reconsolidation to the initial effective stress. It is generally assumed that consolidating the sample to 4 times the initial effective stress prior to testing will result in the sample closely approximating the field “virgin” curve behavior.

To determine the correct shear strength for analysis, perform a CU triaxial test at the initial effective stress ($P'_o$) and as close as practical to $4P'_o$. On the Mohr diagram draw a line from the ordinate to point 4, and draw a second line from $P'_o$ to point 3. Where the two lines intersect, draw a line to the shear stress axis to estimate the correct shear strength for analysis. In Figure 12-33 Determination of Short-Term Cohesive Shear Strength from the CU Envelope.
12-33, the cohesion intercept for the CU strength envelope (solid line) is 150 psf. The corrected strength based on the construction procedure in Figure 12-33 would be 160 psf. While the difference is slight in this example, it may be significant for other projects.

Once the correct shear strength data has been obtained, the embankment stability can be assessed. If the embankment stability is inadequate, proceed to performing a total stress or effective stress analysis, or both.

**12.3.1.3 Total Stress Analysis**

The CU triaxial test is ideally suited to staged fill construction analysis when considering undrained strengths. A CU test is simply a series of UU tests performed at different confining pressures. In the staged construction technique, each embankment stage is placed under undrained conditions (i.e., “U” conditions). Then the soil beneath the embankment stage is allowed to consolidate under drained conditions, which allows the pore pressure to dissipate and the soil strength to increase (i.e., “C” conditions).

In most cases, the CU envelope cannot be used directly to determine the strength increase due to the consolidation stress placed on the weak subsoil. The stress increase from the embankment fill is a consolidation stress, not necessarily the normal stress on potential failure planes in the soft soil, and with staged construction excess pore pressures due to overburden increases are allowed to partially dissipate. Figure 12-34 illustrates how to determine the correct strength due to consolidation and partial pore pressure dissipation.

**Figure 12-34 Consolidated Strength Construction from Triaxial Data**
To correct $\varphi_{cu}$ for the effects of consolidation use the following (see Ladd, 1991):

**Equation 12-25**

$$\frac{a_f}{\sigma_c} = \tan \phi_{\text{consol}}$$

**Equation 12-26**

$$\tan \phi_{\text{consol}} = \frac{\sin \varphi_{cu}}{(1 - \sin \varphi_{cu})}$$

Determine the strength gain ($\Delta C_{uu}$) by multiplying the consolidation stress increase ($\Delta \sigma_v$) by the tangent of $\varphi_{\text{consol}}$. The consolidation stress increase is the increased effective stress in the soft subsoil caused by the embankment fill.

**Equation 12-27**

$$\Delta C_{uu} = \Delta \sigma_v \tan \phi_{\text{consol}}$$

This is an undrained strength and it is based on 100% consolidation. When constructing embankments over soft ground using staged construction practices, it is often not practical to allow each stage to consolidate to 100%. Therefore, the strengths used in the stability analysis need to be adjusted for the consolidation stress applied and the degree of consolidation achieved in the soft soils within the delay period between fill stages. The strength at any degree of consolidation can be estimated using:

**Equation 12-28**

$$C_{uu \%} = C_{uu_i} + (U C_{uu})$$

$$= C_{uu_i} + U \Delta \sigma_v \tan \phi_{\text{consol}}$$

The consolidation is dependant upon the time ($t$), drainage path length ($H$), coefficient of consolidation ($C_v$), and the Time Factor ($T$). From Holtz and Kovacs (1981), the following approximation equations are presented for consolidation theory:

**Equation 12-29**

$$T = \frac{t C_v}{H^2}$$

where,

**Equation 12-30**

$$T = 0.25 \pi U^2$$

for $U < 60\%$
Equation 12-31

\[
T = 1.781 - 0.933 \log(100 - U\%) \quad \text{for } U > 60\%
\]

The Departmental Geotechnical Engineer should use these equations along with specific construction delay periods (t) to determine how much consolidation occurs by inputting a time (t), calculating a Time Factor (T), and then using the Time Factor (T) to estimate the degree of consolidation (U).

Once all of the design parameters are available, the first step in a total stress staged fill construction analysis is to use the initial undrained shear strength of the soft subsoil to determine the maximum height to which the fill can be built without causing the slope stability safety factor to drop below the critical value. See NYSDOT GDM Section 12.3.1.1.2 for determination of the undrained shear strength needed for this initial analysis.

In no case shall the interim factor of safety at any stage in the fill construction be allowed to drop below 1.15. A higher critical value should be used (i.e., 1.2 or 1.25) if uncertainty in the parameters is high, or if the soft subsoil is highly organic. At the end of the final stage, determine the time required to achieve enough consolidation to obtain the minimum long-term safety factor (or resistance factor if structures are involved) required, as specified in NYSDOT GDM Section 12.2.3.1. This final consolidation time will determine at what point the embankment is considered to have adequate long-term stability such that final paving (assuming that long-term settlement has been reduced during that time period to an acceptable level) and other final construction activities can be completed. In general, this final consolidation/strength gain period should be on the order of a few months or less.

Once the maximum safe initial fill stage height is determined, calculate the stress increase resulting from the placement of the first embankment stage using the Boussinesq equation (e.g., see Figures 12-29 and 12-30). Note that because the stress increase due to the embankment load decreases with depth, the strength gain also decreases with depth. To properly account for this, the soft subsoil should be broken up into layers for analysis just as is done for calculating settlement. Furthermore, the stress increase decreases as one moves toward the toe of the embankment. Therefore, the soft subsoil may need to be broken up into vertical sections as well. Determine the strength gain in each layer/section of soft subsoil by multiplying the consolidation stress increase by the tangent of \(\varphi_{\text{consol}}\) (see Equation 12-27), where \(\varphi_{\text{consol}}\) is determined as shown in Figure 12-34 and Equation 12-36. This will be an undrained strength. Multiply this UU strength by the percent consolidation that has occurred beneath the embankment up to the point in time selected for the fill stage analysis using Equations 12-28, 12-29, and 12-30 or 12-31. This will be the strength increase that has occurred up to that point in time. Add to this the UU soil strength existing before placement of the first embankment stage to obtain the total UU strength existing after the selected consolidation period is complete. Then perform a slope stability analysis to determine how much additional fill can be added with consideration to the new consolidated shear strength to obtain the minimum acceptable interim factor of safety.
Once the second embankment stage is placed, calculation of the percent consolidation and the strength gain gets more complicated, as the stress increase due to the new fill placed is just starting the consolidation process, while the soft subsoil has already had time to react to the stress increase due to the previous fill stage. Furthermore, the soft subsoil will still be consolidating under the weight of the earlier fill stage. This is illustrated in Figure 12-35. For simplicity, a weighted average of the percent consolidation that has occurred for each stage up to the point in time in question should be used to determine the average percent consolidation of the subsoil due to the total weight of the fill.

Continue this calculation process until the fill is full height. It is generally best to choose as small a fill height and delay period increment as practical, as the conservatism in the consolidation time estimate increases as the fill height and delay time increment increases. Typical fill height increments range from 2 to 4 ft, and delay period increments range from 10 to 30 days.
Figure 12-35 Concepts Regarding the Percent Consolidation Resulting from Placement of Multiple Fill Stages
12.3.1.4 Effective Stress Analysis

In this approach, the drained soil strength, or $\phi_{CD}$, is used to characterize the strength of the subsoil. Of course, the use of this soil strength will likely indicate that the embankment is stable, whereas the UU strength data would indicate that the embankment is unstable (in this example). It is the buildup of pore pressure during embankment placement that causes the embankment to become unstable. The amount of pore pressure buildup is dependent on how rapidly the embankment load is placed. Given enough time, the pore pressure buildup will dissipate and the soil will regain its effective strength, depending on the permeability and compressibility of the soil.

The key to this approach is to determine the amount of pore pressure buildup that can be tolerated before the embankment safety factor drops to a critical level, using $\phi_{CD}$ for the soil strength and conducting a slope stability analysis (see NYSDOT GDM Chapter 10). A slope stability computer program such as XSTABL can be used to determine the critical pore pressure increase directly. This pore pressure increase can then be used to determine the pore pressure ratio, $r_u$, which is often used to compare with in-situ pore pressure measurements. The pore pressure ratio, $r_u$, is defined as shown in Figure 12-36.

![Figure 12-36 Pore Pressure Ratio Concepts](image-url)
For XSTABL, the critical pore pressure increase is input into the program as a “pore pressure constant” for each defined soil unit in the soil property input menu of the program. This pore pressure is in addition to the pore pressure created by the static water table. Therefore, a water table should also be included in the analysis. Other slope stability programs have similar pore pressure features that can be utilized.

To determine the pore pressure increase in the soft subsoil to be input into the stability analysis, calculate the vertical stress increase created by the embankment at the original ground surface, for the embankment height at the construction stage being considered. Based on this, determine the vertical stress increase, $\Delta \sigma_v$, using the Boussinesq stress distribution (e.g., Figures 12-29 and 12-30), at various depths below the ground surface, and distances horizontally from the embankment centerline, in each soil unit which pore pressure buildup is expected (i.e., the soft silt or clay strata which are causing the stability problem). Based on this, and using $K_o$, the at-rest earth pressure coefficient, to estimate the horizontal stress caused by the vertical stress increase, determine the pore pressure increase, $\Delta u_p$, based on the calculated vertical stress increase, $\Delta \sigma_v$, as follows:

\begin{equation}
\Delta u_p = B(\Delta \sigma_{oct} + a\Delta \tau_{oct})(1 - U)
\end{equation}

The octahedral consolidation stress increase at the point in question, $\Delta \sigma_{oct}$, is determined as follows:

\begin{equation}
\Delta \sigma_{oct} = \Delta \sigma_1 + \Delta \sigma_2 + \Delta \sigma_3 = \frac{\Delta \sigma_v + K_o \Delta \sigma_v + K_o \Delta \sigma_v}{3} = \frac{(1 + 2K_o)\Delta \sigma_v}{3}
\end{equation}

where,

- $B$ = pore pressure parameter which is dependent on the degree of saturation and the compressibility of the soil skeleton. $B$ is approximately equal to 1.0 for saturated normally consolidated silts and clays.
- $\Delta \sigma_{oct}$ = the change in octahedral consolidation stress at the point in the soil stratum in question due to the embankment loading.
- $a$ = Henkel pore pressure parameter that reflects the pore pressure increase during shearing. “$a$” is typically small and can be neglected unless right at failure. If necessary, “$a$” can be determined from triaxial tests and plotted as a function of strain or deviator stress to check if neglecting “$a$” is an acceptable assumption.
- $\Delta \tau_{oct}$ = the change in octahedral shear stress at the point in the soil stratum in question due to the embankment loading.
- $U$ = the percent consolidation, expressed as a decimal, under the embankment load in question.
Equation 12-34

$$\Delta \tau_{oct} = [(\Delta \sigma_1 - \Delta \sigma_2)^2 + (\Delta \sigma_2 - \Delta \sigma_3)^2 + (\Delta \sigma_3 - \Delta \sigma_1)^2]^{0.5}$$

In terms of vertical stress, before failure, this equation simplifies to:

Equation 12-35

$$\Delta \tau_{oct} = 1.414 \Delta \sigma_v (1 - K_o)$$

In this analysis, since only consolidation stresses are assumed to govern pore pressure increase, and strength gain as pore pressure dissipates (i.e., the calculation method is set up to not allow failure to occur), it can be assumed that “a” is equal to zero. Therefore, Equation 12-27 simplifies to:

Equation 12-36

$$\Delta u_p = B \frac{(1 + 2K_o)}{3} \Delta \sigma_v (1 - U)$$

where, $K_o = 1 - \sin \phi_{CD}$ for normally consolidated silts and clays.

Estimate the slope stability factor of safety, determining $\Delta u_p$ at various percent consolidations (i.e., iterate) to determine the maximum value of $\Delta u_p$ that does not cause the slope stability interim safety factor to drop below the critical value (see NYSDOT GDM Section 12.3.1.3).

Now determine $r_u$ as follows:

Equation 12-37

$$r_u = \frac{\Delta u_p}{\sigma_v} = \frac{B \frac{(1 + 2K_o)}{3} \Delta \sigma_v (1 - U)}{\Delta \sigma_v} = B \frac{(1 + 2K_o)}{3} (1 - U)$$

The pore pressures measured by the piezometers in the field during embankment construction are the result of vertical consolidation stresses only (Boussinesq distribution). Most experts on this subject feel that pore pressure increase due to undrained shearing along the potential failure surface does not occur until failure is actually in progress and may be highly localized at the failure surface. Because of this, it is highly unlikely that one will be able to measure pore pressure increase due to shearing along the failure surface using piezometers installed below the embankment unless one is lucky enough to have installed a piezometer in the right location and happens to be taking a reading as the embankment is failing. Therefore, the pore pressure increase measured by the piezometers will be strictly due to consolidation stresses.
Note that $r_u$ will vary depending on the embankment height analyzed. $r_u$ will be lowest at the maximum embankment height, and will be highest at the initial stages of fill construction. Therefore, $r_u$ should be determined at several embankment heights.

### 12.3.2 Base reinforcement

Base reinforcement may be used to increase the factor of safety against slope failure. Base reinforcement typically consists of placing a geotextile or geogrid at the base of an embankment prior to constructing the embankment.

Base reinforcement is particularly effective where soft/weak soils are present below a planned embankment location. The base reinforcement can be designed for either temporary or permanent applications. Most base reinforcement applications are temporary, in that the reinforcement is needed only until the underlying soil’s shear strength has increased sufficiently as a result of consolidation under the weight of the embankment (see NYSDOT GDM Section 12.3.1). Therefore, the base reinforcement does not need to meet the same design requirements as permanent base reinforcement regarding creep and durability. For example, if it is anticipated that the soil will gain adequate strength to meet stability requirements without the base reinforcement within 6 months, then the creep reduction factor could be based on, say, a minimum 1 year life, assuming deformation design requirements are met. Other than this, only installation damage would need to be addressed, unless unusual chemical conditions exist that could cause rapid strength degradation. However, if it is anticipated that the soil will never gain enough strength to cause the embankment to have the desired level of stability without the base reinforcement, the long-term design strengths for a minimum 75 year life shall be used.

The design of base reinforcement is similar to the design of a reinforced slope in that limit equilibrium slope stability methods are used to determine the strength required to obtain the desired safety factor (see NYSDOT GDM Chapter 17). The detailed design procedures provided by Holtz, et al. (1995) should be used for embankments utilizing base reinforcement.

Base reinforcement materials should be placed in continuous longitudinal strips in the direction of main reinforcement. Joints between pieces of geotextile or geogrid in the strength direction (perpendicular to the slope) should be avoided. All seams in the geotextiles should be sewn and not lapped. Likewise, geogrids should be linked with mechanical fasteners or pins and not simply overlapped. Where base reinforcement is used, the use of crushed stone instead of fill material may also be appropriate in order to increase the embankment shear strength.

### 12.3.3 Ground Improvement

Ground improvement can be used to mitigate inadequate slope stability for both new and existing embankments, as well as reduce settlement. The primary ground improvement techniques to mitigate slope stability fall into two general categories, namely densification and altering the soil composition. NYSDOT GDM Chapter 14, Ground Improvement, should be reviewed for a more detailed discussion and key references regarding the advantages and disadvantages of these techniques, applicability for the prevailing subsurface conditions, construction considerations,
and costs. In addition to the two general categories of ground improvement identified above, wick drains (discussed in NYSDOT GDM Chapter 14 and Section 12.4.1) may be used in combination with staged embankment construction to accelerate strength gain and improve stability, in addition to accelerating long-term settlement. The wick drains in effect drastically reduce the drainage path length, thereby accelerating the rate of strength gain. Other ground improvement techniques such as stone columns can function to accelerate strength gain in the same way as wick drains, though the stone columns also reduce the stress applied to the soil, thereby reducing the total strength gain obtained. See NYSDOT GDM Chapter 14 for additional guidance and references to use if this technique is to be implemented.

12.3.3.1 Undercut and Backfill

Undercutting is the process of excavating below the usual cut limits in order to correct a natural deficiency. If the deficiency is anticipated during the design stage, then the undercuts will be designated on the plans, the backfill material specified and the payment quantities provided. Unanticipated deficiencies encountered during construction which need corrective undercutting require an Order On Contract.

The material used to backfill undercuts and the placement requirements depend on the reason for the undercut and the conditions existing when the work is performed. Each case is separate and materials which are good for one case may not be satisfactory for another.

There are two basic conditions under which undercut and backfill work are necessary: to minimize damages caused by differential frost action and to provide a stable platform to support the embankment or pavement structure.

Frost Heaves

Differential frost heaves are probably the most dangerous conditions encountered by the traveling public in the Winter and early Spring. Uniform frost heaves can be tolerated, but large differential ones cannot. The conditions necessary for a frost heave to develop are water, freezing temperatures and frost susceptible soils. Water is controlled to some degree by side ditches and underdrains, but it can be drawn up from the water table by capillary action. There is no effective control over the freezing temperatures. The frost susceptible subgrade soils can be removed and replace with broken rock or granular materials which are less susceptible to frost action. If this is done, the soils within the subgrade area may freeze but ice lenses, which cause the differential heaving, will not form.

Differential frost heaves have also been experienced in layered gravel deposits. These are excellent foundation soils. However, due to the way they were deposited, differential frost heaves may develop. Undercutting and backfilling with the same material is a possible solution. The purpose of the undercut and backfill work is then to mix up the layers to produce a uniform foundation. Uniform foundation conditions will minimize any differential heavings.
**Weak Subgrade Soils**

Instability of the natural subgrade soil is the other reason for corrective undercut and backfill work. In cuts, the subbase course is generally placed directly on the natural soil excavated to subgrade elevation. Sometimes this natural soil may not have enough strength to support the weight of the proof roller or other construction equipment. In these cases, the undercut and backfill work is performed to provide a stable platform for support.

Wet, silty soils are responsible for most subgrade stability problems. Groundwater emerging in the floor of the excavation can saturate this fine grained soil, reducing its strength. As the construction equipment passes over the area, excess pore pressures are created and rutting occurs. The vibrations of the machinery will tend to draw water up to the surface further reducing the soil strength. Sometimes, the conditions get so bad that the wet soils have to be excavated with a drag line.

The requirements for the placement and compaction of backfill material in undercut areas depend on the condition of the soils at the bottom of the excavation. When the bottom is solid, the backfill must be placed and compacted according to NYSDOT *Standard Specifications*. The area between the subgrade surface and the bottom of undercut is defined as the subgrade area, resulting in a minimum compaction requirement of 95% of Standard Proctor Maximum Density. Therefore, close inspection of the compaction operation is a must. If, however, the bottom of the excavation is soft and unstable, the NYSDOT *Standard Specifications* gives the Engineer permission to adopt or approve any placement/compaction procedure for the undercut backfill that will produce a satisfactory job. Thus, for soft foundation conditions, the selection of lift thickness and compaction equipment may be based upon conditions at the site and do not necessarily have to conform to the normal compaction specifications. Generally, the use of vibratory rollers to compact the backfill in soft foundation areas should be discouraged, as the vibrations may cause the silt to be pumped into the backfill.

Areas which are designed to be undercut should not be changed in the field. The design was prepared to correct a natural deficiency that was known in advance. For example, a particular soil may provide good subgrade support, but may be very frost susceptible. Therefore, it must be removed.

The severity of subgrade stability problems depends on the water conditions when the cut is made. It is very difficult to determine during design. The limits for undercut and backfill work are generally left to be determined in the field by the Regional Geotechnical Engineer. These problems may be reduced in magnitude in some cases if the cut is allowed to drain after it has been completed to the original pay lines. This waiting period will allow the groundwater table to stabilize and the material beneath the subgrade to dry. A combination of these two effects may allow the extent and depth of the undercut to be reduced, thus resulting in a significant cost savings.
12.3.4 Lightweight Fills

Lightweight embankment fill is another means of improving embankment stability. Lightweight fills are generally used for two conditions: the reduction of the driving forces contributing to instability, and reduction of potential settlement resulting from consolidation of compressible foundation soils. Situations where lightweight fill may be appropriate include conditions where the construction schedule does not allow the use of staged construction, where existing utilities or adjacent structures are present that cannot tolerate the magnitude of settlement induced by placement of typical fill, and at locations where post-construction settlements may be excessive under conventional fills.

Lightweight fill can consist of a variety of materials including expanded polystyrene fill ("geofoam" blocks), lightweight aggregates (expanded shale (solite, norlite), pumice, blast furnace slag, fly ash), shredded rubber tires, and other materials. Lightweight fills are infrequently used due to either high costs or other disadvantages with using these materials.

12.3.4.1 Expanded Polystyrene Fill

Since the early 1970’s, polystyrene foams have been used in a wide variety of geotechnical applications. Expanded polystyrene is the most common and versatile of the polystyrene foams in use today. Expanded Polystyrene Fill (EPS) (a.k.a. “geofoam”) has a density that is less than 1% of typical soil fills. A typical block measuring 2 ft x 4 ft x 8 ft (0.6m x 1.2m x 2.4m) at a density of 1.25 pcf (0.2 kN/m$^3$) weighs 80 lbs (36 kg). Therefore, EPS is particularly effective at reducing driving forces or settlement potential. By using this extremely lightweight material in an embankment section over a deep, soft organic or clay soil deposit, significant time and cost savings may be achieved as compared to other foundation stabilization and settlement mitigation techniques.

Typical EPS embankments consist of the foundation soils, the EPS blocks, a subsurface drainage system, a protective cap and a pavement structure. Typically, a subsurface drainage system includes the installation of a layer of graded crushed stone placed behind and below the EPS fill, connected to a positive outlet. The protective cap is a design aspect to be examined, which will protect the EPS from accidental petroleum spills and concentrated loads. EPS will dissolve upon encountering petroleum products such as gasoline or diesel fuel. The protective cap may be either a geomembrane or concrete (typically at least 4 in. (100 mm) thick). When concrete is utilized, the protective cap may also serve to enhance the overall performance by aiding in the distribution of live and dead loads.
Other design considerations for EPS include creep, flammability, buoyancy, moisture absorption, photo-degradation, and differential icing of pavement constructed over EPS. Furthermore, EPS should not be used where the water table could rise and cause buoyancy problems, as EPS will float. Design guidelines for EPS embankments are provided in the Geotechnical Engineering Manual (GEM-24) Guidelines for Design and Construction of Expanded Polystyrene Fill as a Lightweight Soil Replacement. An additional reference includes the NCHRP document titled Geofoam Applications in the Design and Construction of Highway Embankments (Stark et al., 2004).

Geotechnical Test Procedure (GTP-7) Expanded Polystyrene Fill Sampling and Specimen Preparation Procedure provides the method used by the NYSDOT Geotechnical Engineering Bureau in selecting representative specimens from large blocks of EPS. The objective of this method is to select nine (9) representative specimens for Density Testing (ASTM D1622) and Compressive Strength testing (ASTM D1621). The nine specimens are taken from three columns cut out of the large EPS block.

12.3.4.2 Lightweight Aggregates

Mineral aggregates, such as expanded shale (norlite, solite), pumice, fly ash, or blast furnace slags, can also be used as lightweight fill materials. Expanded shales and pumice materials consist of inert mineral aggregates that have similar shear strengths to many conventional fill materials, but weigh roughly half as much. The primary disadvantage with expanded shales and pumice is that these materials are expensive. Fly ash can also be used for lightweight fill; however, fly ash is difficult to place and properly control the moisture condition. Blast furnace slag is another waste material sometimes used for lightweight fill. Due to the weight of blast furnace slag, it is not as effective as other lightweight fill materials. Also, slag materials have been documented to swell when hydrated, potentially damaging improvements founded above the slag. The chemical composition of fly ash and blast furnace slag should be investigated to confirm that high levels of contaminants are not present. Due to the potential durability and chemical issues associated with some light weight aggregates, approval from the Departmental Geotechnical Engineer is required before such materials may be considered for use in embankments.
12.3.4.3 Tire Shreds

Tire shreds are defined by NYSDOT as pieces of scrap tire between 2 in. and 12 in. (50 mm and 300 mm) in size. Because of its light weight, tire shreds can be used in normal (routine) embankment construction or in a situation where a lightweight embankment material is required due to a potential settlement or stability problem.

The use of tire shreds in embankment construction involves meeting special design requirements as well as attention to environmental issues. Note that the Department of Environmental Conservation (DEC) has issued a generic Beneficial Use Determination (BUD) for using tire shreds. That means that as far as DEC is concerned, tire shreds are not considered a solid waste, and are a viable construction material. As such, they may be transported and handled without the need for special permits. Tire shreds are addressed in Section 360-1.15 (6) of 6 NYCRR Part 360, Title 6 of the Official Compilation of Codes, Rules and Regulations, effective September 29, 1997.

Design guidelines for tire shred embankments are provided in the Geotechnical Engineering Manual (GEM-20) Guidelines for Project Selection, Design, and Construction of Tire Shreds in Embankments.

The condition and quality of tire shreds varies depending on the shredding methods and the condition of the shredding equipment. In addition, existing piles of tire shreds are possible hiding places for undesirable materials and objects. The Statewide process for evaluating and assuring the quality of tire shreds intended for use on NYSDOT projects is provided in Geotechnical Control Procedure (GCP-19) Sampling and Testing of Tire Shreds. This manual also establishes the criteria tire shreds must meet in order to be acceptable to the Department.

12.3.4.4 Lightweight Concrete Fill

Lightweight Concrete Fill is an engineered geotechnical material with a unique strength / density relationship which can be used to reduce loads on soft foundation soils, buried structures, or against retaining walls. Lightweight Concrete Fill consists of a Portland cement matrix containing uniformly distributed, non-interconnected air voids introduced by a foaming agent. The flowability and cementitious properties of this material provide a product that is self leveling and does not require compaction.

Lightweight Concrete Fill has a density that is 25% - 35% of that of typical soil fills. By using this lightweight material in an embankment section over a deep, soft organic or clay soil deposit, significant time and cost savings may be achieved as compared to other foundation stabilization and settlement mitigation techniques. In addition, the flowability characteristic of the Lightweight Concrete Fill makes it an ideal material to fill voids in areas which would be inaccessible to other lightweight material.
If significant differential settlement is still anticipated in spite of the use of the lightweight concrete fill, due to its relatively brittle nature, the concrete could crack, losing much of its shear strength. This should be considered if using lightweight concrete fill. Its cost can be quite high, being among the most expensive of the lightweight fill materials mentioned herein.

### 12.3.4.5 Toe Berms and Shear keys

Toe berms and shear keys are each methods to improve the stability of an embankment by increasing the resistance along potential failure surfaces. Toe berms are typically constructed of granular materials that can be placed quickly, do not require much compaction, but have relatively high shear strength. As implied by the name, toe berms are constructed near the toe of the embankment slopes where stability is a concern. The toe berms are often inclined flatter than the fill embankment side slopes, but the berm itself should be checked for stability. The use of berms may increase the magnitude of settlements as a consequence of the increased size of the loaded area.

![Figure 12-38 Toe Berm](image_url)
Toe berms increase the shearing resistance by:

- Adding weight, and thus increasing the shear resistance of granular soils below the toe area of the embankment;
- Adding high strength materials for additional resistance along potential failure surfaces that pass through the toe berm; and
- Creating a longer failure surface, thus more shear resistance, as the failure surface now must pass below the toe berm if it does not pass through the berm.

The required dimensions of berms depend on the heights and width of the embankment, soil shear strength and depth of soft soil. The stability of various combinations of berm widths and heights are found to determine the most economical and satisfactory solution (see Figure 12-39). The toe of the berm should extend beyond the arc of the most critical circle. Also, the berm itself should be checked for stability to ascertain that the berm itself will not fail. There have been cases where stepped berms have been sued to achieve stability for the entire berm and embankment.
The use of berms may increase the magnitude of the settlements as a consequence of the increased size of the loaded area. Moreover, the duration of the settlement period may be extended as a result of the increased length of horizontal drainage path produced by the berms. However, undrained deformations are likely to be reduced.

Toe berms are to be construed concurrently with the embankment. As the berms normally serve only as a counterweight, compaction is not essential except to avoid any problems of
trafficability or when culverts are to go through the berm. If an existing unstable slope is to be stabilized by a berm, a process by which the material is pushed off the edge of the fill and allowed to roll down the slope is not to be used. The construction is to start at the toe of the slope and work its way upwards. Virtually any material can be employed as fill for the berms, provided it can be made sufficiently stable and durable to from the berms.

Shear keys function in a manner similar to toe berms, except instead of being adjacent to and incorporating the toe of the fill embankment, the shear key is placed under the fill embankment—frequently below the toe of the embankment. Shear keys are best suited to conditions where they key can be embedded into a stronger underlying formation. Shear keys typically range from 5 to 15 feet in width and extend 4 to 10 feet below the ground surface. They are typically backfilled with Select Fill or similar materials that are relatively easy to place below the groundwater level, require minimal compaction, but still have high internal shear strength. Like toe berms, shear keys improve the stability of the embankment by forcing the potential failure surface through the strong shear key material or along a much longer path below the shear key.

12.4 SETTLEMENT MITIGATION

12.4.1 Acceleration Using Wick Drains

Wick drains, or prefabricated drains, are in essence vertical drainage paths that can be installed into compressible soils to decrease the overall time required for completion of primary consolidation. Wick drains typically consist of a long plastic core surrounded by a geotextile. The geotextile functions as a separator and a filter to keep holes in the plastic core from being plugged by the adjacent soil, and the plastic core provides a means for the excess pore water pressures to dissipate. A drainage blanket is typically placed across the ground surface prior to installing the wick drains and provides a drainage path beneath the embankment for water flowing from the wick drains.

The drains are typically band-shaped (rectangular) measuring a few inches wide in plan dimension. They are attached to a mandrel and are usually driven/pushed into place using either static or vibratory force. After the wick drains are installed, the fill embankment and possibly surcharge fill are placed above the drainage blanket. A key consideration for the use of wick drains is the site conditions. If obstructions or a very dense or stiff soil layer is located above the compressible layer, pre-drilling may be necessary. The use of wick drains to depths over about 60 feet require specialized equipment.

The primary function of a wick drain is to reduce the drainage path in a thick compressible soil deposit. As noted in NYSDOT GDM Section 12.2.4.2.1.2, a significant factor controlling the time rate of settlement is the length of the drainage path. Since the time required for a given percentage consolidation completion is related to the square of the drainage path, cutting the drainage path in half would reduce the consolidation time to one-fourth the initial time, all other parameters held constant. However, the process of installing the wick drains creates a smear zone that can impede the drainage. The key design issue is maximizing the efficiency of the spacing of
the drains, and one of the primary construction issues is minimizing the smear zone around the drains. A full description of wick drains, design considerations, example designs, guideline specifications, and installation considerations are provided by reference in NYSDOT GDM Chapter 14.

12.4.2 Acceleration Using Surcharges

Surcharges are additional loads placed on the fill embankment above and beyond the design height. The effectiveness of the surcharge is dependent upon several factors which should be analyzed, such as the time-settlement characteristics of the foundation soil, and the ratio between surcharge height and the final fill height. As the surcharge height-fill height ratio decreases, the effectiveness of the surcharge also decreases. The loading intensity of the surcharge increment on the compressible layers should be checked by the usual pressure distribution methods. If the fill is high and the compressible layer is deep, then the surcharge will be relatively ineffective.

Also, the effect of a surcharge loading on the stability of the embankment for a foundation shear failure should be checked. Many cases have been encountered where a surcharge would have been desirable form the standpoint of settlement problems, but would have made the stability critical.

Surcharges have been used effectively in the areas of bridge abutments located on soils such as loose silts, fine sand, and clayey silts that consolidate rapidly. By preloading the abutment area, it is possible to reduce the structure settlement to an amount where a spread footing foundation may be used instead of piles.

The primary purpose of a surcharge is to speed up the consolidation process. The surcharges speed up the consolidation process because the percentage of consolidation required under a surcharge will be less than the complete consolidation under the design load. As noted previously, it is customary to assume consolidation is essentially complete at the theoretical 90% completion stage, where $T = 0.848$. In comparison, $T = 0.197$ for 50% consolidation. Therefore it takes less than one-fourth the time to achieve an average of 50% consolidation in a soil layer than it does to achieve 90%. In this example, the objective would be to place a surcharge sufficiently large such that 50% of the total settlement estimated from the fill embankment and the surcharge is equal to or greater than 100 percent of the settlement estimated under the fill embankment alone at its design height. Based on previous experience, the surcharge fill needs to be at least one-third the design height of the embankment to provide any significant time savings.
In addition to decreasing the time to reach the target settlement, surcharges can also be used to reduce the impact of secondary settlement. Similar to the example presented above, the intent is to use the surcharge to pre-induce the settlement estimated to occur from primary consolidation and secondary compression due to the embankment load. For example, if the estimated primary consolidation under an embankment is 18 inches and secondary compression is estimated at an additional 6 inches over the next 25 years, then the surcharge would be designed to achieve 24 inches of settlement or greater under primary consolidation only. The principles of the design of surcharges to mitigate long-term settlement provided by Cotton, et al. (1987) should be followed.

Using a surcharge typically will not completely eliminate secondary compression, but it has been successfully used to reduce the magnitude of secondary settlement. However, for highly organic soils or peats where secondary compression is expected to be high, the success of a surcharge to reduce secondary compression may be quite limited. Other more positive means may be needed to address the secondary compression in this case, such as removal.

Two significant design and construction considerations for using surcharges include embankment stability and re-use of the additional fill materials. New fill embankments over soft soils can result in stability problems as discussed in NYSDOT GDM Section 12.3. Adding additional surcharge fill would only exacerbate the stability problem. Furthermore, after the settlement objectives have been met, the surcharge will need to be removed. If the surcharge material cannot
be moved to another part of the project site for use as site fill or as another surcharge, it often not economical to bring the extra surcharge fill to the site only to haul it away again. Also, when fill soils must be handled multiple times (such as with a “rolling” surcharge), it is advantageous to use gravel borrow to reduce workability issues during wet weather conditions.

### 12.4.3 Lightweight Fills

Lightweight fills can also be used to mitigate settlement issues as indicated in NYSDOT GDM Section 12.3.4. Lightweight fills reduce the new loads imposed on the underlying compressible soils, thereby reducing the magnitude of the settlement. See NYSDOT GDM Section 12.3.4 for additional information on lightweight fill.

### 12.4.4 Over-excavation

Over-excavation simply refers to excavating the soft compressible soils from below the embankment footprint and replacing these materials with higher quality, less compressible soil. Because of the high costs associated with excavating and disposing of unsuitable soils as well as the difficulties associated with excavating below the water table, over-excavation and replacement typically only makes economic sense under certain conditions. Some of these conditions include, but are not limited to:

- The area requiring overexcavation is limited;
- The unsuitable soils are near the ground surface and do not extend very deep (typically, even in the most favorable of construction conditions, over-excavation depths greater than about 10 ft are in general not economical);
- Temporary shoring and dewatering are not required to support or facilitate the excavation;
- The unsuitable soils can be wasted on site; and
- Suitable excess fill materials are readily available to replace the overexcavated unsuitable soils.

#### 12.4.4.1 Unsuitable Removal

This treatment is used for swamp deposits that are predominantly organic. A typical peat or muck may contain, by volume, nine parts water and one part organic and inert soil. This material has a very low shear strength and is highly compressible under embankment loads. It is very difficult to satisfactorily stabilize this soil in place so as to obtain a smooth pavement to safely carry high speed traffic. Explorations through old county and town roads constructed across peat deposits have indicated that extensive maintenance has been required in order to keep the roadways above high water.

Sufficient explorations should be taken to indicate the depth of excavation on the contract cross-sections and to predetermine quantities. A contour plan of the depth of unsuitable material is made available to the project engineer for large projects where the swamp bottom is irregular.
The width of excavation should be such that the embankment slopes will not be unstable resulting in settlements or lateral movement of the roadway shoulder; Typical sections for various cases of excavation are shown on Figure 12-42. Some designers become concerned that the stability of the backfill against the adjacent peat or muck may be critical and that the backfill will spread and crack. A circular arc stability analysis often indicates that this condition should be critical. However, from experience on many projects, it has been found that backfill with vertical side slopes underground will be stable against this type of movement. It is possible that the organic soils supply more passive resistance against lateral movement than present analyses indicate. Also, explorations on several completed projects indicated that the backfill below ground surface remained in a vertical plane and that there was practically no bulging of the backfill.
Figure 12-42 Unsuitable Material Removal Procedure

On large excavation projects, the disposal of unsuitable material may be a problem. The specifications should indicate that the material is to be placed in spoil banks outside the right-of-way or used on construction to flatten slopes. Soils with high organic contents may shrink up to 50% on drying.

During construction, the vertical excavation slope in peat or muck will be stable for several days before backfilling, provided that the excavation is kept full of water. In cases where the water has been pumped out of excavations, the sides have caved-in resulting in needless additional excavation. This is a basic slope stability problem. When the organic material is under water, it has a submerged unit weight of 5 to 10 pcf. When the excavation is drained, the saturated unit weight of the adjacent soil increases by approximately 62.4 pcf from the submerged weight. The increase in effective weight greatly decreases the stable height of excavation. This fact is often overlooked by Engineers in the field. Frequently, Engineers require the Contractor to lower the water level so that the excavation may be inspected. When this is done, sloughing problems will develop at the sides of the excavation.

For fills with widths in the order of 200 ft., it is not practical to carry the entire width of backfill across the swamp at one time. The usual procedure is to progress in strips 50 to 70 ft. wide. Advancing fill fronts are skewed away from the completed portions to help eliminate the possibility of unsuitable material being entrapped between the fills. This method has been used successfully on several projects with no adverse post-construction settlements.
Frequently, the unsuitable organic soils are underlain by very loose silt or soft clayey silt that appear to be unsuitable material for an embankment foundation when uncovered by excavation equipment. However, when backfill is placed on this material, it will consolidate rapidly and provide an adequate foundation. A very difficult problem to control in the field is the problem of over-excavation when the underlying soil is loose or soft.

The organic material under the widened section is excavated and backfilled with suitable granular soil. This method has been used successfully to depths of 15 ft. Failure of the existing roadway into the open excavation is prevented by keeping a minimum length of open excavation parallel to the roadway centerline.

Also, no traffic should be allowed within 10 ft. of the open excavation. After completion of excavation and backfill, a surcharge may be used, if necessary, to decrease differential settlements across the roadway and the excavated material may be used to flatten slopes.

A special problem of embankment stabilization by excavation is encountered for widening projects on secondary roads crossing swamps. These old roads are usually floating on fills that have penetrated, by displacement and settlement, to a considerable depth below the swamp surface. The existing roadway is reasonably stable, since the underlying soil is well compressed after 30 years or more of loading. Any new embankment material placed on the adjacent swamp surface will undergo settlement in the order of several feet, thus creating a differential settlement problem across the highway section.

12.4.4.2 Close Sequence Excavation and Backfill

Close sequence excavation and backfill is a coordinated operation which limits the extent and time of an open unsupported excavation prior to backfilling. The control criteria for the procedure are established on a project specific basis.

Construction for widenings over soft soils, toe of slope shear key installations and swamp crossings are examples of situations which may require close sequence excavation and backfill to prevent collapse of the adjacent ground. Frequently, the procedure also includes a requirement for maintaining the water level at a specific elevation during the operation. Utilization of this process enables the foundation treatment to be constructed safely, faster and more economically, (i.e., without sheeting or bracing).

The objective of this procedure is that of maintaining slope stability and preventing subsidence of the adjacent ground without the use of sheeting or bracing until backfill for the new construction is safely placed. Experience has shown that short lengths of unsupported excavations will not fail or slough during short time intervals, even if the computed safety factor for this condition indicates otherwise. However, where such experience is lacking and stability analyses are used to determine short term dimensional criteria, a minimum safety factor of 1.1 should be used. Special considerations for a given project may warrant a higher short term safety factor. Note that maintaining the water level in the excavation where possible exerts an important
positive influence in this type of analysis.

A steep backfill face facilitates excavation and also enables the length of open excavation to be kept to a minimum. A 1V on 1H backfill end slope could be maintained using very angular stone in still water or in the dry if apparent cohesion is present. Rounded gravel or stone will spread to 1V on 1.5 or 1V on 2H. Sand or sandy gravel backfill placed in moving water would spread to a flatter slope. A heavier backfill material such as rock would be required to maintain a steep face in this situation.

The key aspects or considerations for contract presentation are:

- Direction of Operation.
- Critical dimensions for and allowable time periods of open excavation.
- Water level maintenance requirements.
- Restrictions on locations for disposal of excavated material.
- Type of granular backfill material.
- Required steepness of backfill face.
- Due regard for construction equipment requirements, i.e., backhoe, dragline or dredge.
- Surcharge and waiting period requirements.

Note that a decision to not use the technique might be in order if, after analyses are made, valid concerns remain as to being able to successfully execute this procedure. Engineering judgement plays a key role here. These situations require a proper assessment of the reasons for doubting complete success. There are, however, many design and installation situations which warrant its consideration and possible use. The most frequent types of applications are shown conceptually in Figure 12-43.

A. Unsuitable Material Removal

Unsuitable material removal for roadway construction is most frequently accomplished by excavation when the deposit in question is not more than 6 meters deep. A roadway on a new location where unsuitable material exists can be built following the requirements of Standard Sheet 203-1 Construction Details Unsuitable Material Excavation and Backfill. This sheet has been used for many years as the basis for this construction procedure. Note that wide embankments may dictate that the procedure be carried out in consecutive strips.

Embankment widenings, accompanied by a change in grade, are occurring on an increasing frequency. These projects must be approached carefully so as to insure that a failure of the existing roadway is prevented, especially if traffic is to be maintained on it during construction. Here again, stability analyses may be necessary to determine dimensional controls parallel to centerline where experience is lacking or possibly where the unsuitable material is in turn underlain by a relatively weak stratum.
B. Shear Key

The results of stability studies often indicate that slope stability or embankment foundation failures can be best prevented or corrected by use of a shear key treatment. The shear key can be for a cut slope or for a new or existing embankment. The shear key dimensions in cross-section are dictated by stability analysis studies. Dimensional controls in a longitudinal direction may be based on experience or additional stability studies.

C. Utility and Drainage Line Installations

It is not uncommon for new utility or drainage lines to be installed along the toe of slope in wide trenches. This construction situation is analogous to the shear key condition and can be evaluated in the same manner.

It may be difficult to restrict the time and safe length of an open excavation for this application during construction. This is due to the time and working space required for bedding and joining successive lengths of pipe, box culvert, etc. Consequently, these excavations are frequently braced. However, close sequence excavation and backfill should be considered, particularly when the width of trench is large and individual segments or lengths of conduit to be installed are relative short.

It is crucial for the success of this method to have well defined construction controls, which are implemented and enforced by qualified personnel.
Figure 12-43 Typical Applications of Close Sequence Excavation
12.4.4.3 Removal of Soil by Displacement

The displacement procedure (Figure 12-44) is accomplished by placing sufficient embankment material on the foundation soil to cause the underlying soil to displace by shear failure in the direction of least resistance. The essential design features for a successful displacement operation are to have sufficient embankment weight to force out the underlying soil and to have a sufficient depth of mudwave excavation before the advancing fill front so that the direction of displacement can be controlled and the fill will sink to the desired depth. Also, the advancing fill front should have a steep front face. In some cases, the height of fill to insure effective displacement will be greater than final grade. The displacement method is used for peat and muck deposits greater than thirty feet in depth, where complete excavation may become difficult, and for very soft clays or organic silts that would not stand on a steep excavation slope under water.

![Figure 12-44 Sinking an Embankment by Displacement Due to its Weight](image)

In swamps where firm bottom is irregular it is difficult to obtain complete displacement if the mudwaves are directed against a rising firm bottom surface. The front of the fill should be skewed where necessary to direct the displacement towards the deeper portion of the swamp. Frequently, a drawing is included in the plans indicating the direction of successive fill fronts as the embankment is constructed across a swamp area.

In deep swamps, culverts should be located where the organic material is shallow, usually near the edge of the swamp. This minimizes the possibility of culvert settlement due to consolidation of deep backfill and possible pockets of entrapped organic material.

Where the organic material is underlain by very soft clays that increase in strength with depth, it is necessary to estimate the depth to which the displacement will progress into the clay deposit. This requires detailed laboratory testing and stability analyses.
It is desirable to provide a waiting period before paving to allow the backfill and any entrapped organic soil to consolidate. Sometimes a surcharge (Figure 12-45) is used to eliminate any potential post-construction settlement.

The rate of backfill placement should not exceed the rate of removal of a mudwave material. If this rule is not enforced, the backfill can trap pockets of unsuitable material resulting in undesirable settlements after paving.

Figure 12-45 Sinking a Fill by Displacement Using a Surcharge

The surface root mat is removed at least 30 to 40 ft. ahead of the backfill to promote displacement. All mudwave material is excavated that rises above a designated elevation for a 30 to 40 ft. distance in front of the backfill in order to insure continuing displacement to the desired depth. The critical elevation for excavation may be determined form stability analyses. The water elevation in the exaction at the time of construction is important and often cannot be predicted. Any mudwave material rising above water will have a much greater counterweight effect than submerged material and should be removed. All excavated material should be cast behind the advancing fill front or removed from the site.

12.5 CONSTRUCTION CONSIDERATIONS AND PS&E DEVELOPMENT

Consideration should be given to the time of year that construction will likely occur. If unsuitable soil was encountered during the field investigation, the depth and station limits for removal should be provided on the plans. NYSDOT GDM Chapter 18 provides guidance for the use of
geotextile for separation or soil stabilization. Note that for extremely soft and wet soil, a site specific design should be performed for the geotextile.

Benching is specified on Standard Sheet 203-2 *Earthwork Transition and Benching Details*. Where embankments are built on existing hillsides or existing embankment slopes, the existing surface soil may form a plane of weakness unless the slope is terraced or stepped. Terracing breaks up the plane, increasing the strength of the entire system. Generally slopes that are 1V on 3H or steeper should be terraced to improve stability. However, there may be specific cases where benching may be waived during design, such as when the existing slope is steeper than 1V on 1H and benching would destabilize the existing slope.

Bench widths are variable, depending on the slope, with the height being held to approximately 4 ft. Standard Sheet 203-2 *Earthwork Transition and Benching Details* sets a predetermined payment quantity of 0.6 yd$^3$/ft. of bench to compensate the Contractor for the work of excavating the material to form the bench and replacing it with embankment.

The compaction requirements in the NYSDOT Standard Specifications apply to the entire embankment, including near the sloping face of the embankment. For embankment slopes of 1V on 2H or steeper, depending on the embankment soil properties, getting good compaction out to the embankment face can be difficult to achieve, and possibly even unsafe for those operating the compaction equipment. The consequences of poor compaction at the sloping face of the embankment include increased risk of erosion and even surficial slope instability. This issue becomes especially problematic as the embankment slope steepness approaches 1V on 1.5H. Surficial stability of embankments (See NYSDOT GDM Chapter 10) should be evaluated during design for embankment slopes of 1V on 2H or steeper. The embankment design shall include the use of techniques that will improve embankment face slope stability for embankment slopes steeper than 1V on 1.7H, and should consider the use of such techniques for slopes of 1V on 2H or steeper.

Table 12-10 summarizes the general guidelines recommended for safe, stable embankment sideslopes.

<table>
<thead>
<tr>
<th>Side Slope</th>
<th>Slope Treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1V on 2H</td>
<td>None Needed (generally).</td>
</tr>
<tr>
<td>1V on 2H to 1V on 1.5H</td>
<td>Stone Blanket, Grouted Rip-Rap, Geosynthetically Reinforced Soil System (GRSS).</td>
</tr>
<tr>
<td>1V on 1.5H to 1V on 1H</td>
<td>Geosynthetically Reinforced Soil System (GRSS), and other Reinforced Slope Systems, Grouted Rip-Rap (only for low slopes).</td>
</tr>
<tr>
<td>Steeper than 1V on 1H</td>
<td>An Engineered Wall or Slope Reinforcement System</td>
</tr>
</tbody>
</table>

*Table 12-10 Slope Treatments for Various Embankment Sideslopes*
To ensure uniformity under load, proof rolling is part of the NYSDOT Standard Specifications and thererby required by the Contractor. The purpose of proof rolling embankments is to find areas of non-uniformity of compaction, and in cuts, areas of the subgrade which will not satisfactorily support the proof roller. Proof rolling is performed prior to the placement of the subbase course. If the compaction of the upper embankment layers, or the natural soil conditions, are not uniform and dense, these non-uniformities will be reflected in the finished pavement which will result in poor performance and high maintenance activities/costs.

For embankments, the stress level to which the proof roller must be loaded is determined by the type of soil in the subgrade. Materials which are inherently stronger will require a higher stress level: whereas for weaker soils, a lower level is required. However, there is a minimum stress level of 30 tons gross load with the tires inflated to 40 psi. The NYSDOT Standard Specifications is used to determine the initial stress level. The type of subgrade soil (or the foundation soil if the embankment is less than 3 ft. in thickness), its gradation (well graded or poorly graded) and its relative support are the factors needed to select the initial stress level. The Regional Geotechnical Engineer can supply expert advice in this area. The Contractor has complete control over the construction of the embankments and proof rolling will check the workmanship. Therefore, it is in the NYSDOT’s interest to select the highest stress level consistent with the material, gradation and relative subgrade support.

The proof roller is operated briefly and the action of the embankment is watched closely. If there is “consistent lateral displacement of soil out of the wheel paths” then the stress level may be reduced to the next lower level. Lateral displacement is rutting and is due to the shearing of the soil. Proof rolling is not designed or intended to fail an embankment, but to point out areas of non-uniform compaction. If the roller stress is reduced one level and consistent rutting still occurs, then the stress level should be reduced again. This procedure may be followed until the roller does not consistently displace the soil.

Once the final roller stress has been determined, the roller makes two complete coverages on the subgrade surface within the outside edges of shoulders (roadway limits). Depressions are filled with material similar to the subgrade soil so that uniformity will be continued. Major deficiencies are corrected. The correction areas are then re-compactied in the normal manner and proof rolled again. All deficiencies are corrected at the Contractor’s expense until the subgrade surface shows a satisfactory uniform response to the proof roller. Since the Contractor constructed the embankments, it is the Contractors responsibility to provide a suitable foundation for the pavement structure. Until this suitable foundation is provided, the Contractors work is not complete.

Proof rolling embankments may only be eliminated when, in areas of limited access or maneuvering space, it might damage adjacent work, or when the proof roller may come within 5 ft. of a culvert, pipe, or other conduit.

Proof rolling cuts is performed in order to determine areas of the subgrade which will not adequately or uniformly support the pavement structure. Correction of these natural deficiencies is the NYSDOT’s responsibility and the work is called undercut and backfill. Once an area is
undercut and backfilled, proof rolling is not required (e.g. a designed undercut eliminates the proof rolling entirely and an area ordered to be undercut by the Engineer due to construction conditions will not require proof rolling). In other words, the undercut and backfill work will correct any problem areas and, therefore, proof rolling will be a waste of money.

The proof roller used in cuts is loaded to a single stress level of 30 tons gross load and have the tires inflated to 40 psi. This stress level may be different form the one required in embankment sections. The Contractor is required to use the specified stress level. It is not in the NYSDOT’s best interest to load the roller to a high stress level as this may falsely show more areas requiring corrective undercut and backfill work.

Two complete passes are required and the Engineer may require undercut and backfill work based on the action of the proof roller on the subgrade. Proof rolling in cut section is not required in areas where undercut and backfill work have been performed, in rock cuts or in areas where the Engineer determines it will be detrimental to work. As in embankment sections, proof rolling in cuts is not intended to destroy the subgrade, but only to point our areas of inadequate subgrade support.

12.5.1 Winter Earthwork Operations

The NYSDOT Standard Specifications state earthwork construction operations requiring compaction shall not be performed from November 1st thru April 1st, except with a Winter Earthwork submittal subject to the provisions of the Specification and approved by the Regional Director or his designated representative.

The Winter Earthwork Submittal process requires the Contractor to provide a submittal outlining the modification to the materials, methods for placement and methods for controlling weather effects on both the material and existing ground. The material requirements for winter earthwork are provided in the NYSDOT Standard Specifications.

Winter Earthwork is subject to the following restrictions:
- Transitioning from the normal construction season to the exempt winter earthwork months between November 1st and April 1st, the use of standard earthwork materials may continue to be permitted only under the conditions where the air temperature, ground temperature and material temperature are all above 32° F at the time of placement. Modifications to compaction procedures, including but not limited to the use of thinner lifts, may be required when the temperatures are above 32° F but below 40° F at the time of placement.
- Between November 1st and April 1st, if the air temperature, ground temperature, or material temperature is at or below 32° F at the time of placement, earthwork may only proceed using material that meets the winter earthwork material requirements of the NYSDOT Standard Specifications.

In all work incorporated into the final product, the Contractor shall not place material that is frozen, or place fill material on frozen ground regardless of the date.
Satisfactory compaction of most materials can only be achieved when the temperature is 32°F or higher. Compaction of both unclassified material and granular material becomes increasingly difficult as air and material temperatures approach freezing. This fact has been verified repeatedly through both laboratory and field testing. The increased difficulty is caused by the water in soil reacting less and less as a lubricant until it actually becomes ice, at which point compaction is totally inhibited.

Figure 12-46 Compaction Control Curves (Standard and Modified AASHTO) for:
A: Brown Sand, Some Gravel, Trace of Silt and Trace of Silt and
B: Brown Fine Sand, Trace of Silt
(NYSDOT Geotechnical Engineering Bureau Investigation)

The only exceptions are coarse materials that do not require water for compaction. Compaction of these materials is achieved by point to point mechanical interlocking that does not require particle lubrication.

Generally, all embankment, subbase, and trench backfilling operations (with the exception of rock embankment construction), shall cease on October 31st. Trench backfilling is a particularly critical operation. The proper compaction of the material is generally required to guarantee adequate support of the pipe and to minimize post-construction differential settlement of overlying pavement courses.
12.5.2 Settlement and Pore Pressure Monitoring

If settlement is expected to continue after embankment construction, some type of monitoring program should be provided. Settlement should be monitored, if post construction settlement will affect pavement performance or a settlement sensitive structure will be constructed on the embankment. The type of monitoring will depend on the magnitude and time frame of the settlement. For many monitoring programs, use of survey hubs or monuments and routine surveying methods are adequate. These methods are commonly used if paving should be delayed until embankment settlement is nearly complete. The contract documents will include the time period that the settlement should be monitored and the frequency of observations.

Settlement estimates provided in the contract should be conservative. Therefore, if another construction operation must be delayed until the settlement of the embankment is nearly complete, the time estimate should be the longest length of time that is likely to be necessary; then the Contractor will not be delayed longer than anticipated.

As discussed in NYSDOT GDM Section 12.3.1, embankments constructed over soft ground may require the use of staged construction to ensure the stability of the embankment. Geotechnical instrumentation is a vital part of construction to monitor field performance and provide information relevant to decisions regarding the rate of construction. The principal parameters monitored during embankment construction are pore water pressure and displacement, both vertical and lateral.

As discussed previously, in relatively impermeable, soft, saturated soil, the applied load from embankment construction increases the pore water pressure. With time, the excess pore water pressure will dissipate and the shear strength will increase. It is important to measure the pore water pressure to determine when it is safe to proceed with additional embankment construction. In such cases it is also useful to measure vertical deformation to assist in the interpretation of the data to assess the rate at which embankment construction should proceed.

12.5.3 Instrumentation

The monitoring equipment typically used for embankment construction are discussed in NYSDOT Chapter 23.

A more comprehensive discussion of monitoring techniques is available in Geotechnical Instrumentation for Monitoring Field Performance (Dunnicliff, 1993) and Geotechnical Instrumentation Reference Manual, NHI Course No. 13241 FHWA-HI-98-034 (Dunnicliff, 1998).

12.5.4 PS&E Considerations

Specifications for monitoring equipment that will be supplied by the Contractor should ensure that the equipment is compatible with the read out equipment that will be used during
construction. The specifications should also make clear who will provide the monitoring and analyze the data. If the Contractor’s survey crew will collect the settlement data, it should be indicated in the special provisions. It is also important to stipulate who will analyze the data and provide the final determination on when settlement is complete or when additional fill can be placed. In general, the Departmental Geotechnical Engineer should analyze and interpret the data.

12.5.5 PS&E Checklist

The following issues should be addressed in the PS&E regarding embankments:

- Slope inclination required for stability
- Embankment foundation preparation requirements, overexcavation limits shown on plans
- Plan details for special drainage requirements such as lined ditches, interceptor trenches, drainage blankets, etc.
- Benching requirements
- Evaluation of on-site materials
- Special embankment material requirements
- Special treatment required for fill placement such as non-durable rock, plastic soil, or lightweight fill
- Magnitude and time for settlement
- Settlement waiting period estimated in the Special Provisions (SP)
- Size and limits of surcharge
- Special monitoring needs
- If instrumentation is required to control the rate of fill placement, do the SP’s clearly spell out how this will be done and how the readings will be used to control the Contractor’s operation
- SP’s clearly state that any instrumentation damaged by contractor personnel will be repaired or replaced at no cost to the State
- Settlement issues with adjacent structures, should construction of structures be delayed during embankment settlement period
- Monitoring of adjacent structures

12.6 REFERENCES


Geotechnical Engineering Bureau, *Sampling and Testing of Tire Shreds*, Geotechnical Control Procedure GCP-19, New York State Department of Transportation, Office of Technical Services,


New York State Standard Sheets, Department of Transportation, Design Quality Assurance Bureau, 

Prototype Engineering, Inc., SAF-1, Soil Settlement Analyses Software Suite, Winchester, 
Massachusetts, 1993.

Engineering Circular 5 (GEC5) - Evaluation of Soil and Rock Properties, Report No FHWA-IF- 

Seepage Analysis and Control for Dams, Engineering and Design, EM1110-2-1901, Engineering 

Skempton, A. W., and Bishop, A. W., The Gain in Stability Due to Pore Pressure Dissipation in 


Stark, T., Arellano, D., Horvath, J. and Leshchinsky, D., Geofoam Applications in the Design 
and Construction of Highway Embankments, NCHRP Report 529, Transportation Research 
Board, 58 pp., 2004

Symons, I.F., Assessment and Control of Stability for Road Embankments Constructed on Soft 
Subsoils, Transport and Road Research laboratory, Crowthorne, Berkshire, TRRL Laboratory 
Report 711, 32 pp., 1976

Terzaghi, K, and Peck, R.B., Soil Mechanics in Engineering Practice, John Wiley and Sons, 
1948.

Tonkins, T. and Terranova, T., Instrumentation of Transportation Embankments Constructed on 


Material Reinforced by Numerous Strong Horizontal Sheets, in Contribution to the Mechanics of 
268-277, 1938.
Examples Illustrating
Appendix 12-A Staged Fill Construction Design

12-A.1 PROBLEM SETUP
First, the Departmental Geotechnical Engineer should define the problem in terms of embankment geometry, soil stratigraphy, and water table information. For this example the proposed construction entails constructing a 20 ft thick earth embankment from Gravel Borrow with 1V on 2H side slopes. The embankment will have a roadway width of 35 ft and will be constructed over soft silt. The soft silt is 30 ft thick and overlies dense sand. Ground water was observed 2 ft below the existing ground surface during the field exploration.

Figure 12A-1 Embankment Geometry for Example

Using the test results, the Departmental Geotechnical Engineer should first assess short term (undrained) strength of the embankment to determine if staged construction is required. For the example geometry, XSTABL was used to assess shortterm (undrained) stability using $C_{uu} = 160$ psf (see Figures 12-28 and 12-29 for the specific strength envelopes used). Figure 12A-2 provides the results of the stability analysis, and indicates that the factor of safety is well below the minimum long-term value of 1.25 required for an embankment without a structure. Therefore, staged construction or some other form or mitigation is required to construct the embankment. For this example, continue with a staged construction approach.
12-A.2 DETERMINATION OF MAXIMUM STABLE FIRST STAGE FILL HEIGHT

The analysis conducted in the previous section is conducted again, but this time limiting the fill height to that which has a factor of safety that is equal to or greater than the minimum acceptable interim value (use $FS = 1.15$ to 1.2 minimum for this example). As shown in Figure 12A-3, the maximum initial fill height is 6 ft. This initial fill height is used as a starting point for both the total stress and the effective stress analyses.
12-A.3 TOTAL STRESS ANALYSIS PROCEDURE EXAMPLE

In this approach, the undrained soil strength envelope, or $\phi_{consol}$, as determined in Figure 12-29, is used to characterize the strength of the subsoil. Next, the Departmental Geotechnical Engineer determines how much strength gain can be obtained by allowing the first stage of fill to consolidate the underlying soft soils, using total stresses and undrained strengths after consolidation (see NYSDOT GDM Section 12.3.1.3). The Departmental Geotechnical Engineer calculates the stress increase resulting from the placement of the first embankment stage using the Boussinesq equation or those of Westergaard (see Figures 12-24 and 12-25). Note that because the stress increase due to the embankment load decreases with depth, the strength gain also decreases with depth. To properly account for this, the soft subsoil should be broken up into layers and zones for analysis just as is done for calculating settlement. For the example, the subsurface is divided into the layers and zones shown in Figure 12A-4 to account for the differences in stress increase due to the embankment. The geotechnical designer will have to utilize judgment in determining the optimum number of layers and zones to use. If the division of zones is too coarse, the method may not properly model the field conditions during construction, and too fine of a division will result in excessive computational effort.
For the example geometry model the embankment as a continuous strip with a width of 103 ft \((B = 35' + (4\times20) - (2\times6))\). As zone 3 is located close to the center of the embankment the stress change in that zone will be close to that near the center of the embankment for the stage 1 loading. Therefore, zone 3 is not used in the analysis example yet. It will be used later in the example. The stress increases in the zones are as follows:

<table>
<thead>
<tr>
<th>Layer</th>
<th>Zone</th>
<th>Z</th>
<th>Z/B</th>
<th>I</th>
<th>(\sigma_v) 6 ft. x 130pcf</th>
<th>(\Delta\sigma_v) (1 x (\sigma_v))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>5 ft.</td>
<td>0.049</td>
<td>0.98</td>
<td>780 psf</td>
<td>764 psf</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>20 ft.</td>
<td>0.190</td>
<td>0.93</td>
<td>780 psf</td>
<td>725 psf</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>5 ft.</td>
<td>0.049</td>
<td>0.55</td>
<td>780 psf</td>
<td>429 psf</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>20 ft.</td>
<td>0.190</td>
<td>0.75</td>
<td>780 psf</td>
<td>585 psf</td>
</tr>
</tbody>
</table>

**Table 12A-1 Stress Increases for Example**

Once the Departmental Geotechnical Engineer has the stress increase, the increase in strength due to consolidation can be estimated using Equations 12-27 and 12-28. However, the strength increase achieved will depend on the degree of consolidation that occurs. The consolidation is dependant upon the time \((t)\), drainage path length \((H)\), coefficient of consolidation \((C_v)\), and the Time Factor \((T)\). Using Equations 12-29 through 12-31, assuming the stage 1 fill is allowed to consolidate for 15 days and assuming the soft soil layer is doubly drained, the percent consolidation would be:

\[
T = \frac{tC_v}{H^2}
\]

\[
T = 15 \text{ days}(1 \text{ ft}^2/\text{Day})/(30 \text{ ft}/2)^2 \text{ (assumed double draining)}
\]

\[
T = 0.067 = 0.25\pi U^2; \text{ for } U < 60\%
\]

\[
U = 0.292 \text{ or } 29\%
\]
Therefore, at 15 days and 29% consolidation, using Equation 12-28, the strength gain would be as follows:

<table>
<thead>
<tr>
<th>Layer</th>
<th>Zone</th>
<th>$\Delta\sigma_v$ (I x $\sigma_v$)</th>
<th>$C_{uui}$</th>
<th>U</th>
<th>$\varphi_{consol}$</th>
<th>$C_{uu 29%}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>764 psf</td>
<td>160 psf</td>
<td>0.29</td>
<td>22°</td>
<td>250 psf</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>725 psf</td>
<td>160 psf</td>
<td>0.29</td>
<td>22°</td>
<td>245 psf</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>429 psf</td>
<td>160 psf</td>
<td>0.29</td>
<td>22°</td>
<td>210 psf</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>585 psf</td>
<td>160 psf</td>
<td>0.29</td>
<td>22°</td>
<td>228 psf</td>
</tr>
</tbody>
</table>

**Table 12A-2 Strength Gain at 15 Days for Example**

Using the same procedure the strength gain at other time periods can be estimated. For example, at 60 days the percent consolidation would be 59%, and the strength gain would be as follows:

<table>
<thead>
<tr>
<th>Layer</th>
<th>Zone</th>
<th>$\Delta\sigma_v$ (I x $\sigma_v$)</th>
<th>$C_{uui}$</th>
<th>U</th>
<th>$\varphi_{consol}$</th>
<th>$C_{uu 59%}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>764 psf</td>
<td>160 psf</td>
<td>0.59</td>
<td>22°</td>
<td>342 psf</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>725 psf</td>
<td>160 psf</td>
<td>0.59</td>
<td>22°</td>
<td>333 psf</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>429 psf</td>
<td>160 psf</td>
<td>0.59</td>
<td>22°</td>
<td>262 psf</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>585 psf</td>
<td>160 psf</td>
<td>0.59</td>
<td>22°</td>
<td>299 psf</td>
</tr>
</tbody>
</table>

**Table 12A-3 Strength Gain at 60 Days for Example**

The Departmental Geotechnical Engineer should consider that as consolidation time increases the relative increase in strength becomes less as time continues to increase. Having a settlement delay period that would achieve 100% consolidation is probably not practical due to the excessive duration required. Delay period of more than 2 months are generally not practical. Continue the example assuming a 15 day settlement delay period will be required. Using the strength gained, the Departmental Geotechnical Engineer determines how much additional fill can be placed.

Determine the height of the second stage fill that can be constructed by using $C_{uu 29\%}$ and increasing the fill height until the factor of safety is approximately 1.2 but not less than 1.15. As shown in Figure 12A-5, the total fill height can be increased to 8 ft (2 ft of new fill is added) after the 15 day delay period.
For the second stage of fill, the effective footing width changes as the fill becomes thicker. The equivalent footing width for use with the Boussinesq stress distribution will be 99 ft ($B = 35' + (4 \times 20) - (2 \times 8)$). As zone 3 is located close to the center of the embankment the stress change in that zone will be close to that near the center of the embankment for the stage 1 and stage 2 loading. Therefore, zone 3 is not used in the analysis example yet. It will be used later in the example. The stress increases in the zones are as follows:

<table>
<thead>
<tr>
<th>Layer</th>
<th>Zone</th>
<th>Z</th>
<th>Z/B</th>
<th>I</th>
<th>$\sigma_v$ 8 ft. x 130 pcf</th>
<th>$\Delta\sigma_v$ (I x $\sigma_v$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>5 ft.</td>
<td>0.049</td>
<td>0.98</td>
<td>1040 psf</td>
<td>1019 psf</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>20 ft.</td>
<td>0.190</td>
<td>0.93</td>
<td>1040 psf</td>
<td>967 psf</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>5 ft.</td>
<td>0.049</td>
<td>0.55</td>
<td>1040 psf</td>
<td>231 psf</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>20 ft.</td>
<td>0.190</td>
<td>0.75</td>
<td>1040 psf</td>
<td>315 psf</td>
</tr>
</tbody>
</table>

Table 12A-4 Stress Increases for Example
Once the Departmental Geotechnical Engineer has the stress increase, the increase in strength due to consolidation can be estimated. The Departmental Geotechnical Engineer must now begin to use weighted averaging to account for the difference in consolidation times (see Figure 12-30). The first stage of fill was allowed to settle for 15 days prior to placing the additional 2 ft of fill in the second stage, bringing the total fill height up to 8 ft. If the second lift of soil is allowed to consolidate for another 15 days, the soil will actually have been consolidating for 30 days total. For 30 days, the Time Factor (T), would be:

\[
T = \frac{tC_v}{H^2} \\
T = 30 \text{ days}(1 \text{ ft}^2/\text{Day})/(30\text{ft}/2)^2 \text{ (assumed double draining)} \\
T = 0.133 = 0.25\pi U^2; \text{ for } U < 60\% \\
So, U = 0.41 \text{ or } 41\%
\]

The average consolidation of the 15 + 15 day delay period will be:
[6 ft(0.41) + 2 ft(0.29))/8 ft = 0.38 or 38%

The strength gain at 30 days and 38% average consolidation would be as follows:

<table>
<thead>
<tr>
<th>Layer</th>
<th>Zone</th>
<th>$\Delta \sigma_v$ (I x $\sigma_v$)</th>
<th>$C_{uu}$</th>
<th>U</th>
<th>$\Phi_{consol}$</th>
<th>$C_{uu \ 38%}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>764 psf</td>
<td>160 psf</td>
<td>0.38</td>
<td>22°</td>
<td>317 psf</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>725 psf</td>
<td>160 psf</td>
<td>0.38</td>
<td>22°</td>
<td>309 psf</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>429 psf</td>
<td>160 psf</td>
<td>0.38</td>
<td>22°</td>
<td>248 psf</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>585 psf</td>
<td>160 psf</td>
<td>0.28</td>
<td>22°</td>
<td>280 psf</td>
</tr>
</tbody>
</table>

Table 12A-5 Strength Gain at 40 Days for Example

The Departmental Geotechnical Engineer would continue this iterative process of adding fill, determining the weighted average consolidation, subsequent strength gain, and stability analysis to determine the next “safe” lift until the embankment is constructed full height.

Once the final stage fill is placed, it will continue to cause consolidation of the soft subsoil, increasing its strength. The calculations to determine the time required once the embankment is completed to cause the factor of safety to increase to the minimum long-term acceptable FS of 1.25 are summarized as follows:
Table 12A-6 Strength Gain for Factor of Safety of 1.25

The calculations tabulated above assume that 25 days after the final fill layer is has elapsed, resulting in an average degree of consolidation of 71%.

The final stability analysis, using the undrained shear strengths tabulated above, is as shown in Figure 12A-6.
In summary, the fill increments and delay periods are as follows:

<table>
<thead>
<tr>
<th>Stage</th>
<th>Fill Increment</th>
<th>Time Delay Prior to Next Stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6 ft.</td>
<td>15 days</td>
</tr>
<tr>
<td>2</td>
<td>2 ft.</td>
<td>15 days</td>
</tr>
<tr>
<td>3</td>
<td>2 ft.</td>
<td>15 days</td>
</tr>
<tr>
<td>4</td>
<td>2 ft.</td>
<td>15 days</td>
</tr>
<tr>
<td>5</td>
<td>2 ft.</td>
<td>30 days</td>
</tr>
<tr>
<td>6</td>
<td>2 ft.</td>
<td>30 days</td>
</tr>
<tr>
<td>7</td>
<td>3 ft.</td>
<td>10 days</td>
</tr>
<tr>
<td>8</td>
<td>1 ft.</td>
<td>25 days to obtain FS = 1.25</td>
</tr>
<tr>
<td>Totals</td>
<td>20 ft.</td>
<td>155 days</td>
</tr>
</tbody>
</table>

**Table 12A-7 Fill Increment and Delay Periods for Example**

Fewer stages can be selected by the Departmental Geotechnical Engineer, but longer delay periods are required to achieve more consolidation and the higher strength increases necessary to maintain stability. A comparable analysis using thicker fill stages and longer settlement delay periods yielded the following:

<table>
<thead>
<tr>
<th>Stage</th>
<th>Fill Increment</th>
<th>Time Delay Prior to Next Stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6 ft.</td>
<td>60 days</td>
</tr>
<tr>
<td>2</td>
<td>4.5 ft.</td>
<td>60 days</td>
</tr>
<tr>
<td>3</td>
<td>5.5 ft.</td>
<td>40 days</td>
</tr>
<tr>
<td>4</td>
<td>4 ft.</td>
<td>5 days to obtain FS = 1.25</td>
</tr>
<tr>
<td>Totals</td>
<td>20 ft.</td>
<td>165 days</td>
</tr>
</tbody>
</table>

**Table 12A-8 Fill Increment and Delay Periods for Fewer Stages for Example**

When using the total stress method of analysis it is often best to maximize the initial fill height. Doing this will produce the greatest amount of soil strength gain early in the construction of the fill. In addition, keeping the subsequent stages of fill as small as possible enables the fill to be constructed with the shortest total delay period, though in the end, the time required to achieve the final long-term safety factor is approximately the same for either approach.

**12-A.4 EFFECTIVE STRESS ANALYSIS PROCEDURE EXAMPLE**

In this approach, the drained soil strength, or $\phi_{CD}$, is used to characterize the strength of the subsoil. From Figure 12-29, $\phi_{CD}$ is 27°. However, it is the buildup of pore pressure during embankment placement that causes the embankment to become unstable. The amount of pore pressure buildup is dependent on how rapidly the embankment load is placed. Given enough time, the pore pressure buildup will dissipate and the soil will regain its effective strength, depending on the permeability and compressibility of the soil. The key to this approach is to determine the amount of pore pressure build up that can be tolerated before the embankment
safety factor drops to a critical level when using \( \phi_{CD} \) for the soil strength. A limit equilibrium stability program such as XSTABL should be used to determine the pore pressure increase that can be tolerated and result in the embankment having a safety factor of 1.15 to 1.2 during construction.

Many of the newer stability programs have the ability to accept \( r_u \) values directly or to calculate \( r_u \). The Departmental Geotechnical Engineer should be aware of how the stability program calculates \( r_u \). When using XSTABL, the Departmental Geotechnical Engineer should not input \( r_u \) directly. Instead, he should input excess pore pressures directly into the program and then run the stability analysis.

The rate of fill construction required to prevent \( r_u \) from being exceeded cannot be determined directly from the drained analysis, as embankment stability needs in addition to the subsoil consolidation rate affects the rate of construction. The total construction time cannot therefore be determined directly using \( C_v \) and the percent consolidation required for stability.

Using the example geometry shown in Figure 12A-1, the Departmental Geotechnical Engineer should divide the subsurface into layers and zones in a manner similar to that shown in Figure 12A-4. The Departmental Geotechnical Engineer then determines the stress increase due to the first stage of fill, 6 feet in this case.

The stress increases in the zones are as follows based on an equivalent strip footing width of 103 ft:

<table>
<thead>
<tr>
<th>Layer</th>
<th>Zone</th>
<th>Z</th>
<th>Z/B</th>
<th>I</th>
<th>( \sigma_v ) 6 ft. x 130 pcf</th>
<th>( \Delta \sigma_v ) (I x ( \sigma_v ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>5 ft.</td>
<td>0.049</td>
<td>0.98</td>
<td>780 psf</td>
<td>764 psf</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>20 ft.</td>
<td>0.190</td>
<td>0.93</td>
<td>780 psf</td>
<td>725 psf</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>5 ft.</td>
<td>0.049</td>
<td>0.55</td>
<td>780 psf</td>
<td>429 psf</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>20 ft.</td>
<td>0.190</td>
<td>0.75</td>
<td>780 psf</td>
<td>585 psf</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>5 ft.</td>
<td>0.049</td>
<td>0.98</td>
<td>780 psf</td>
<td>764 psf</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>20 ft.</td>
<td>0.190</td>
<td>0.93</td>
<td>780 psf</td>
<td>725 psf</td>
</tr>
</tbody>
</table>

Table 12A-9 Stress Increases for Example

Note that Zone 3 has the same stress increase as Zone 1.

As discussed previously in NYSDOT GDM Section 12.3.1.4, the pore pressure increase is dependent upon the load and the degree of consolidation. Using Equation 12-36 with an assumed percent consolidation, determine the pore pressure change to use in the stability analysis. It will be necessary to perform the analysis for several percent consolidations to determine what the critical pore pressure is for maintaining stability.
\[
K_0 = 1 - \sin \varphi_{CD} = 1 - \sin 27^\circ = 0.55
\]

\[
B = 1.0, \text{ assuming subsoil is fully saturated. For Layer 1, Zone 1, at 30\% consolidation,}
\]

\[
\Delta u_p = B[(1 + 2K_0)/3] \Delta \sigma_v (1-U) = 1.0[(1 + 2(0.55))/3](764 \text{ psf})(1-.30) = 374 \text{ psf}
\]

The remaining values are as follows:

<table>
<thead>
<tr>
<th>Layer</th>
<th>Zone</th>
<th>( \Delta \sigma_v ) (I x ( \sigma_v )) ( \text{(psf)} )</th>
<th>U (%)</th>
<th>( \Delta u_{p30%} ) ( \text{(psf)} )</th>
<th>U (%)</th>
<th>( \Delta u_{p35%} ) ( \text{(psf)} )</th>
<th>U (%)</th>
<th>( \Delta u_{p40%} ) ( \text{(psf)} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>764</td>
<td>30</td>
<td>374</td>
<td>35</td>
<td>346</td>
<td>40</td>
<td>320</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>725</td>
<td>30</td>
<td>354</td>
<td>35</td>
<td>329</td>
<td>40</td>
<td>303</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>429</td>
<td>30</td>
<td>209</td>
<td>35</td>
<td>194</td>
<td>40</td>
<td>179</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>585</td>
<td>30</td>
<td>286</td>
<td>35</td>
<td>265</td>
<td>40</td>
<td>245</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>764</td>
<td>30</td>
<td>373</td>
<td>35</td>
<td>346</td>
<td>40</td>
<td>320</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>725</td>
<td>30</td>
<td>354</td>
<td>35</td>
<td>329</td>
<td>40</td>
<td>303</td>
</tr>
</tbody>
</table>

**Table 12A-10 Pore Pressure Change for Example**

The slope stability results from XSTABL are provided in Figure 12A-7. For the two subsoil layers, all zones, a drained friction angle, \( \varphi_{CD} \), of 27° was used, and the pore pressure increases \( \Delta u_p \) from the tabulated summary of the calculations provided above were inserted into the soil zones shown in Figure 12A-7 as pore pressure constants. The results shown in this figure are for a percent consolidation of 35\%. 
Figure 12A-7 Stage 1 Drained Analysis at Percent Consolidation of 35% and a Fill Height of 6 ft.

Using Equation 12-37, $r_u$ at this stage of the fill construction is determined as follows:

$$r_u = B[(1 + 2K_o)/3] (1-U) = 1.0[(1 + 2(0.55))/3](1-0.35) = 0.45$$

Subsequent stages of fill construction are checked to determine the critical pore pressure ratio, up to the point where the fill is completed. The pore pressure ratio is evaluated at several fill heights, but not as many stages need to be analyzed as is the case for total stress analysis, as the rate of fill construction is not the focus of the drained analysis. All that needs to be achieved here is to adequately define the relationship between $r_u$ and the fill height. Therefore, one intermediate fill height (13.5 ft) and the maximum fill height (20 ft) will be checked.

For a fill height of 13.5 ft, the stress increases in the zones are as follows based on an equivalent strip footing width of 88 ft:
### Table 12A-11 Stress Increases for Example

Note that the stress increase in Zone 3 is now different than the stress increase in Zone 1, due to the fact that the embankment slope now is over the top of Zone 3.

The pore pressure increase resulting from a 13.5 ft high fill, assuming various percent consolidations, is recalculated using Equation 12-36 as illustrated earlier. The results of these calculations are as tabulated below:

<table>
<thead>
<tr>
<th>Layer</th>
<th>Zone</th>
<th>Z</th>
<th>Z/B</th>
<th>I</th>
<th>$\sigma_v$</th>
<th>$\Delta\sigma_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>13 ft. x 130 pcf</td>
<td>(I x $\sigma_v$)</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>5 ft.</td>
<td>0.049</td>
<td>0.97</td>
<td>1690 psf</td>
<td>1700 psf</td>
</tr>
<tr>
<td>2</td>
<td>20 ft.</td>
<td>0.190</td>
<td>0.90</td>
<td>1690 psf</td>
<td>1580 psf</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>5 ft.</td>
<td>0.049</td>
<td>0.40</td>
<td>1690 psf</td>
<td>702 psf</td>
</tr>
<tr>
<td>2</td>
<td>20 ft.</td>
<td>0.190</td>
<td>0.55</td>
<td>1690 psf</td>
<td>965 psf</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>5 ft.</td>
<td>0.049</td>
<td>0.75</td>
<td>1690 psf</td>
<td>1320 psf</td>
</tr>
<tr>
<td>2</td>
<td>20 ft.</td>
<td>0.190</td>
<td>0.70</td>
<td>1690 psf</td>
<td>1230 psf</td>
<td></td>
</tr>
</tbody>
</table>

Note that higher percent consolidations are targeted, as a higher percent consolidation is likely to have occurred by the time the fill is 13.5 ft high. The slope stability results from XSTABL are provided in Figure 12A-8. For the two subsoil layers, all zones, a drained friction angle, $\phi_{CD}$, of 27° was used, and the pore pressure increases $\Delta u_p$ from the tabulated summary of the calculations provided above were inserted into the soil zones shown in Figure 12A-8 as pore pressure constants. The results shown in this figure are for a percent consolidation of 60%.
Figure 12A-8 Stage 2 Drained Analysis at Percent Consolidation of 60% and a Fill Height of 13.5 ft.

Using Equation 12-37, $r_u$ at this stage of the fill construction is determined as follows:

$$r_u = B[(1 + 2K_o)/3](1-U) = 1.0[(1 + 2(0.55))/3](1-0.60) = 0.28$$

Similarly, these calculations were conducted for the full fill height of 20 ft, and for a minimum FS = 1.15 to 1.2, $r_u$ was determined to be 0.22 (U = 68%).

In summary, the pore pressure ratios that should not be exceeded during fill construction are as follows:

<table>
<thead>
<tr>
<th>Total Fill Height (ft.)</th>
<th>$r_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>0.45</td>
</tr>
<tr>
<td>13.5</td>
<td>0.28</td>
</tr>
<tr>
<td>20</td>
<td>0.22</td>
</tr>
</tbody>
</table>

Table 12A-13 Pore Pressure Ratio Not to be Exceeded for Example
Values of $r_u$ could be interpolated to estimate the critical $r_u$ at other fill heights. It should be assumed that if these values of $r_u$ are used to control the rate of fill construction, the time required to build the fill will be approximately as determined from the total stress analysis provided in the previous section.