



**Washington State  
Department of Transportation**

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# **Geotechnical Design Manual**

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**Environmental and Regional Operations**  
Construction Division  
Geotechnical Office

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## 9-1      **Overview and Data Needed**

This chapter addresses the design and construction of rock embankments, bridge approach embankments, earth embankments, and light weight fills. Static loading as well as seismic loading conditions are covered, though for a more detailed assessment of seismic loading on embankment performance, see Chapter 6. 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:

- Rock embankments, defined as fills in which the material in all or any part of an embankment contains 25 percent or more, by volume, gravel or stone 4 inches or more in diameter.
- 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 rock or bridge approach embankments, but that are constructed out of soil.
- 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.

### 9-1.1      **Site Reconnaissance**

General requirements for site reconnaissance are given in 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 geotechnical designer 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.

## 9-1.2 Field Exploration and Laboratory Testing Requirements

General requirements for the development of the field exploration and laboratory testing plans are provided in 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 9-1 provides a summary of site characterization needs and field and laboratory testing considerations for embankment design.

**Table 9-1 Summary of Information Needs and Testing Considerations for Embankments  
(Adapted From Sabatini, Et Al ., 2002)**

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Embankments and Embankment Foundations	<ul style="list-style-type: none"> <li>• settlement (magnitude &amp; rate)</li> <li>• bearing capacity</li> <li>• slope stability</li> <li>• lateral pressure</li> <li>• internal stability</li> <li>• borrow source evaluation (available quantity and quality of borrow soil)</li> <li>• required reinforcement</li> <li>• liquefaction</li> <li>• delineation of soft soil deposits</li> <li>• potential for subsidence (karst, mining, etc.)</li> <li>• constructability</li> </ul>	<ul style="list-style-type: none"> <li>• subsurface profile</li> <li>• (soil, ground water, rock)</li> <li>• compressibility parameters</li> <li>• shear strength</li> <li>• parameters unit weights</li> <li>• time-rate</li> <li>• consolidation parameters</li> <li>• horizontal</li> <li>• earth pressure</li> <li>• coefficients</li> <li>• interface friction parameters</li> <li>• pullout resistance geologic mapping</li> <li>• including</li> <li>• orientation and</li> <li>• characteristics of rock discontinuities</li> <li>• shrink/swell/</li> <li>• degradation of soil</li> <li>• and rock fill</li> </ul>	<ul style="list-style-type: none"> <li>• nuclear density</li> <li>• plate load test</li> <li>• test fill</li> <li>• CPT (w/ pore pressure measurement)</li> <li>• SPT</li> <li>• PMT</li> <li>• dilatometer</li> <li>• vane shear</li> <li>• rock coring (RQD)</li> <li>• geophysical testing</li> <li>• piezometers</li> <li>• settlement plates</li> <li>• slope inclinometers</li> </ul>	<ul style="list-style-type: none"> <li>• 1-D Oedometer</li> <li>• triaxial tests</li> <li>• unconfined compression</li> <li>• direct shear tests</li> <li>• grain size distribution</li> <li>• Atterberg Limits</li> <li>• specific gravity</li> <li>• organic content</li> <li>• moisture-density relationship</li> <li>• hydraulic conductivity</li> <li>• geosynthetic/soil testing</li> <li>• shrink/swell</li> <li>• slake durability</li> <li>• unit weight</li> <li>• relative density</li> </ul>

### 9-1.3 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.

Generally, embankments 10 feet or less in height, constructed over average to good soil conditions (e.g., non-liquefiable, medium dense to very dense sand, silt or gravel, with no signs of previous instability) will require only a basic level of site investigation. A geologic site reconnaissance (see Chapter 2), combined with widely spaced test pits, hand holes, or a few shallow borings to verify field observations and the anticipated site geology may be sufficient, especially if the geology of the area is well known, or if there is some prior experience in the area.

For larger embankments, or for any embankment to be placed over soft or potentially unstable ground, geotechnical explorations should in general be spaced no more than 500 feet apart for uniform conditions. In non-uniform soil conditions, spacing should be decreased to 100 to 300 foot intervals with at least one boring in each major landform or geologic unit. A key to the establishment of exploration frequency for embankments is the potential for the subsurface conditions to impact the construction of the embankment,

the construction contract in general, and the long-term performance of the finished project. The exploration program should be developed and conducted in a manner that these potential problems, in terms of cost, time, and performance, are reduced to an acceptable level. The boring frequency described above may need to be adjusted by the geotechnical designer to address the risk of such problems for the specific project.

All embankments over 10 feet in height, embankments over soft soils, or those that could impact adjacent structures (bridge abutments, buildings etc.), will generally require geotechnical borings for the design. The more critical areas for stability of a large embankment are between the top and bottom of the slopes. This is where base stability is of most concern and where a majority of the borings should be located, particularly if the near-surface soils are expected to consist of soft fine-grained deposits. At critical locations, (e.g., maximum embankment heights, maximum depths of soft strata), a minimum of two exploration points in the transverse direction to define the existing subsurface conditions for stability analyses should be obtained. More exploration points to define the subsurface stratigraphy, including the conditions within and below existing fill, may be necessary for very large fills or very erratic soil conditions.

Embankment widening projects will require careful consideration of exploration locations. Borings near the toe of the existing fill are needed to evaluate the present condition of the underlying soils, particularly if the soils are fine-grained.

In addition, borings through the existing fill into the underlying consolidated soft soil, or, if overexcavation of the soft soil had been done during the initial fill construction, borings to define the extent of removal, should be obtained to define conditions below the existing fill.

In some cases, the stability and/or durability of the existing embankment fill may be questionable because the fill materials are suspect or because slope instability in the form of raveling, downslope lobes, or slope failures have been observed during the site reconnaissance phase. Some embankments constructed of material that is susceptible to accelerated weathering may require additional borings through the core of the embankment to sample and test the present condition of the existing fill.

Borings are also needed near existing or planned structures that could be impacted by new fill placement. Soil sampling and testing will be useful for evaluating the potential settlement of the existing structure foundations as the new fill is placed.

The depth of borings, test pits, and hand holes will generally be determined by the expected soil conditions and the depth of influence of the new embankment. Explorations will need to be sufficiently deep to penetrate through surficial problem soils such as loose sand, soft silt and clay and organic materials, and at least 10 feet into competent soil conditions. In general, all geotechnical borings should be drilled to a minimum depth of twice the planned embankment height.

Understanding of the underlying soil conditions requires appropriate sampling intervals and methods. As for most engineering problems, testing for strength and compression in fine-grained soils requires the need for undisturbed samples. The SPT is useful in cohesionless soil where it is not practical or possible to obtain undisturbed samples for laboratory engineering tests. SPT sampling is recommended at wet sand sites where liquefaction is a key engineering concern.

On larger projects, cone penetration test (CPT) probes can be used to supplement conventional borings. Besides being significantly less expensive, CPT probes allow the nearly continuous evaluation of soil properties with depth. They can detect thin layers of soil, such as a sand lens in clay that would greatly reduce consolidation time that may be missed in a conventional boring. In addition, CPT probes can measure pore pressure dissipation responses, which can be used to evaluate relative soil permeability and consolidation rates. Because there are no samples obtained, CPT probes shall be used in conjunction with a standard boring program. Smaller projects that require only a few borings generally do not warrant an integrated CPT/boring field program.

#### 9-1.4 Groundwater

At least one piezometer should be installed in borings drilled in each major fill zone where stability analysis will be required and groundwater is anticipated. Water levels measured during drilling are often not adequate for performing stability analysis.

This is particularly true where drilling is in fine-grained soils that can take many days or more for the water level to equalize after drilling (see Chapter 2). Even in more permeable coarse grained soils, the drilling mud used to drill the boring can obscure detection of the groundwater level. Notwithstanding, water levels should be 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.

For embankment widening projects, piezometers are generally more useful in borings located at or near the toe of an existing embankment, rather than in the fill itself. Exceptions are when the existing fill is along a hillside or if seepage is present on the face of the embankment slope.

The groundwater levels should be monitored periodically to provide useful information regarding variation in levels over time. This can be important when evaluating base stability, consolidation settlement or liquefaction. As a minimum, the monitoring should be accomplished several times during the wet season (October through April) to assess the likely highest groundwater levels that could affect engineering analyses.

If practical, a series of year-round readings taken at 1 to 2 month intervals should be accomplished in all piezometers.

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 of the soil and its consolidation behavior under loading. The geotechnical designer should identify the location of the groundwater table and determine the range in seasonal fluctuation.

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, more than one piezometer should be installed to estimate the gradient. Also, 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.

## 9-2 Design Considerations

### 9-2.1 Typical Embankment Materials and Compaction

General instructions for embankment construction are discussed in the WSDOT *Construction Manual* Section 2.3.3, and specific construction specifications for embankment construction are provided in WSDOT Construction Specifications Section 2-03. The geotechnical designer should determine during the exploration program if any of the material from planned earthwork will be suitable for embankment construction (see Chapter 10). Consideration should be given to whether the material is moisture sensitive and difficult to compact during wet weather.

#### 9-2.1.1 Rock Embankments

The WSDOT *Standard Specifications* define rock embankment as “all or any part of an embankment in which the material contains 25 percent or more by volume of gravel or stone 4 inches or greater in diameter.” Compaction tests cannot be applied to coarse material with any degree of accuracy; therefore, a given amount of compactive effort is specified for rock embankments, as described in *Standard Specifications* Section 2-03.3(14)A.

Special consideration should be given to the type of material that will be used in rock embankments. In some areas of the state, moderately weathered or very soft rock may be encountered in cuts and used as embankment fill. On projects located in southwestern Washington, degradable fine grained sandstone and siltstone are often encountered in the cuts. The use of this material in embankments can result in significant long term settlement and stability problems as the rock degrades, unless properly compacted with heavy tamping foot rollers (Machan, et al., 1989).

The rock should be tested by the Washington Degradation Test (WSDOT Test Method 113) and the slake durability test (see Chapter 5) if there is suspicion that the geologic nature of the rock source proposed indicates that poor durability rock is likely to be encountered. When the rock is found to be non-durable, it should be physically broken down and compacted as earth embankment provided the material meets or exceeds common borrow requirements. Special compaction requirements may be needed for these materials. In general, tamping foot rollers work best for breaking down the rock fragments. The minimum size roller should be about 30 tons. Specifications should include the maximum size of the rock fragments and maximum lift thickness. These requirements will depend on the hardness of the rock, and a test section should be incorporated into the contract to verify that the Contractor’s methods will achieve compaction and successfully break down the material. In general, both the particle size and lift thickness should be limited to 12 inches.

#### 9-2.1.2 Earth Embankments and Bridge Approach Embankments

Three types of materials are commonly used in WSDOT earth embankments, including common, select, and gravel borrow. Bridge approach embankments should be constructed from select or gravel borrow, although common borrow may be used in the drier parts of the State, provided it is not placed below a structure foundation or immediately behind an abutment wall. Common borrow is not intended for use as foundation material beneath structures or as wall backfill due to its tendency to be more compressible and due to its poor drainage characteristics.

Requirements for common, select and gravel borrow are in Section 9-03.14 of the WSDOT *Standard Specifications*. The suggested range of soil properties for each material type to be used in design is discussed in Chapter 5. The common and select borrow specifications are intended for use where it is not necessary to strictly control the strength properties of the embankment material and where all weather construction is not required.

Procedures for constructing earth embankments are described in Section 2-03.3(14)B of the *Standard Specifications*. Compaction is specified in accordance with Method A, Method B, or Method C. Method A consists of routing hauling equipment over the embankment and is not normally used on WSDOT projects. Method B limits the thickness of the lifts to 8 inches and requires that 90 percent of maximum dry density be achieved in all but the upper 2 feet of the embankment. In the upper two feet of the embankment the lift thickness is limited to 4 inches and the required compaction is 95 percent of maximum dry density. Method B is used on all embankments on WSDOT projects unless another method is specified.

Method C differs from Method B in that the entire embankment must be compacted to 95 percent of maximum dry density. Method C is required when the structural quality of the embankment is essential. Method C is required in bridge approach embankments as defined in Section 1-01.3 of the WSDOT *Standard Specifications*. Method C shall also be required on any foundation material beneath structures.

Because foundation stresses are transferred outward as well as downward into the bearing soils, the limits of the foundation material should extend horizontally outward from each edge of the footing a distance equal to the thickness of the fill below the foundation.

The maximum density and optimum moisture content for soil placed in earth embankments are determined by testing in accordance with WSDOT Test Method No. 606 (Method of Test for Compaction Control of Granular Materials) or AASHTO T 99 Method A (standard Proctor) as prescribed in Section 2-03.3(14)D of the *Standard Specifications*. Test method 606 is used if 30 percent or more of the material consists of gravel size particles (retained on the No. 4 sieve).

### **9-2.1.3 Fill Placement Below Water**

If material will be placed below the water table, material that does not require compaction such as Quarry Spalls, Foundation Material Class B, Shoulder Ballast, or light loose rip rap should be specified. Once above the water table, other borrow materials should be used. Quarry spalls and rip rap should be choked with Shoulder Ballast or Foundation Material Class A or B before placement of borrow. Alternately, construction geosynthetic for soil stabilization may be used to prevent migration of the finer borrow into the voids spaces of the coarser underlying material.

## 9-2.2 Embankments for Detention/Retention Facilities

Embankments for detention/retention facilities impounding over 10 acre-feet of water come under the jurisdiction of the Dam Safety Office (DSO) of the Washington State Department of Ecology and shall be designed as a small dam in accordance with DSO requirements.

Embankments for detention/retention facilities impounding 10 acre feet of water or less are not regulated by the DSO, but they should be designed using the DSO guidelines as the basis for design. Unlined 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.

## 9-2.3 Stability Assessment

In general, embankments 10 feet or less in height with 2H:1V 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 such as Seattle clay, estuarine deposits, or peat. Embankments over 10 feet in height or any embankment on soft soils, in unstable areas/soils, or those comprised of light weight fill require more in depth stability analyses, as do any embankments with side slope inclinations steeper than 2H:1V.

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 geotechnical designer. Slope stability analysis shall be conducted in accordance with Chapter 7.

Prior to the start of the stability analysis, the geotechnical designer 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 2H:1V 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 Chapter 6) and ground improvement may be needed.

Several methodologies for analyzing the stability of slopes are detailed or identified by reference in Chapter 7 and are directly applicable to earth embankments.

### 9-2.3.1 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 Chapters 8 and 15. 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 Washington 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 Chapter 6 for specific requirements regarding seismic design of embankments.

### 9-2.3.2 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 Chapter 5 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 such as Seattle Clay.

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 Section 9.3.1.

## 9-2.4 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 Section 8.11.3.2, 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 Chapter 5 for additional information). For granular soils, these parameters can be estimated empirically (see Section 8.11.3.2). Oedometer tests should be completed to at least twice the preconsolidation pressure with at least three, and preferably four, points on the virgin consolidation curve (i.e., at stresses higher than the preconsolidation pressure). The coefficient of consolidation value for the virgin curve can be ten times higher than that for the test results below the preconsolidation pressure.

### 9-2.4.1 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 Chapter 8 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.

## 9-2.4.2 Settlement Analysis

### 9-2.4.2.1 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 Chapters 5 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.

The detailed methodology to estimate primary consolidation settlement is provided in Section 8.11.3.2, except that the stress distribution below the embankment should be calculated as described in Section 9.2.4.3. The soil profile is typically divided into layers for analysis, with each layer reflecting changes in soils properties. In addition, thick layers with similar properties are often subdivided for refinement of the analysis since the settlement calculations are based on the stress conditions at the midpoint of the layer (i.e. it is typically preferable to evaluate a near-surface, 20-foot thick layer as two 10-foot thick layers as opposed to one 20-foot thick layer). The total settlement is the sum of the settlement from each of the compressible layers.

If the pre-consolidation pressure of any of the soil layers being evaluated is greater than its current initial effective vertical stress, the settlement will follow its rebound compression curve rather than its virgin compression curve (represented by  $C_c$ ). In this case  $C_{re}$ , the recompression index, should be used instead of  $C_{ce}$  in Equation 8-8 up to the point where the initial effective stress plus the change in effective stress imposed by the embankment surpasses the pre-consolidation pressure. Pre-consolidation pressures in excess of the current vertical effective stress occur in soils that have been overconsolidated, such as from glacial loading, preloading, or desiccation.

### 9-2.4.2.2 Secondary Compression

For organic soils and highly plastic soils determined to have an appreciable secondary settlement component, the secondary compression should be determined as described in Section 8.11.3.2.2, Equation 8-13. 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.

Similar to estimating the total primary consolidation, the contribution from the individual layers are summed to estimate the total secondary compression. 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.

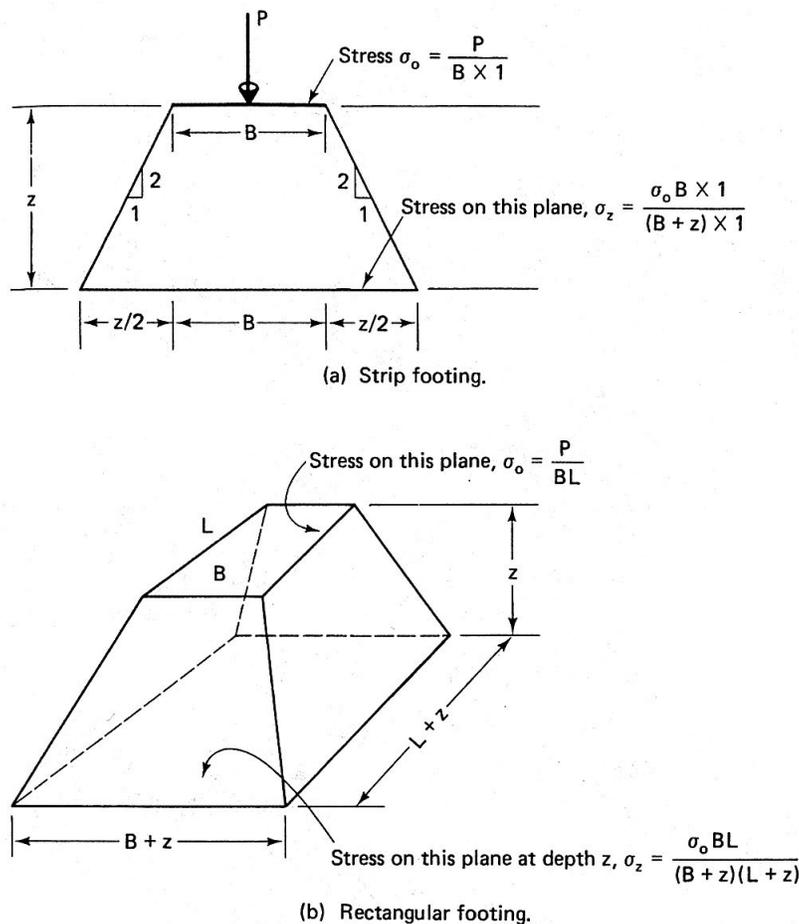
### 9-2.4.3 Stress Distribution

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.

#### 9-2.4.3.1 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 9-1. Since the same vertical load is spread over a much larger area at depth, the unit stress decreases.

Figure 9-1 2V:1H Method to Estimate Vertical Stress Increase as a Function of Depth Below Ground (After Holtz and Kovacs, 1981)



### 9-2.4.3.2 Theory of Elasticity

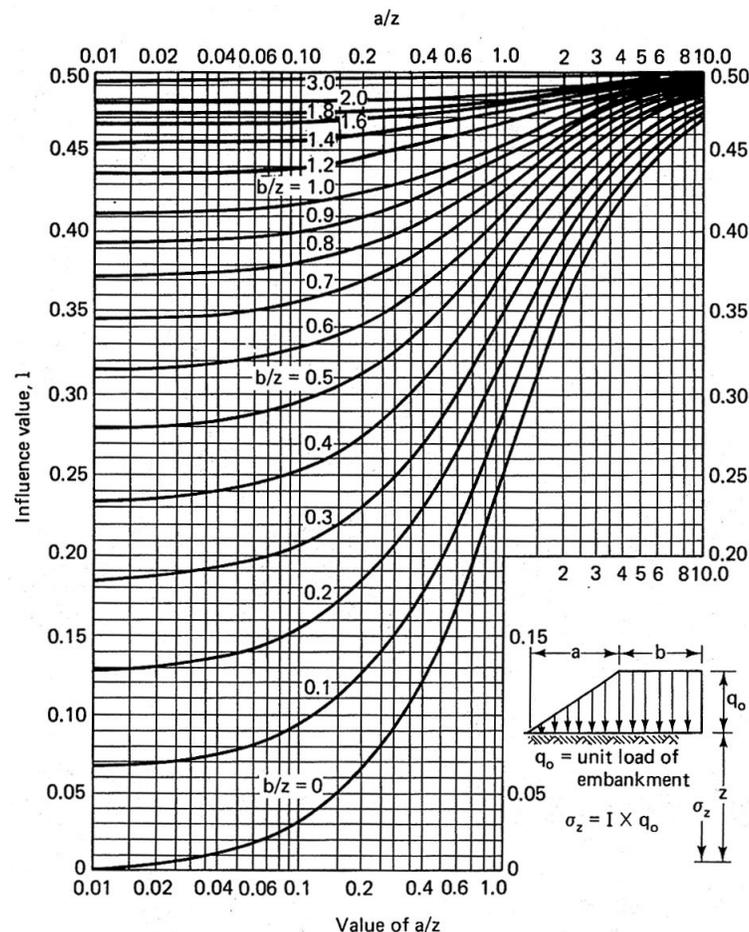
Boussinesq (1885) developed equations for evaluating the stress state in a homogenous, isotropic, linearly elastic half-space for a point load acting perpendicular to the surface. Elasticity based methods should be used to estimate the vertical stress increase in subsurface strata due to an embankment 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 (Westergaard, 1938), which also include Poisson's ratio (relates the ratio of strain applied in one direction to strain induced in an orthogonal direction).

### 9-2.4.3.3 Empirical Charts

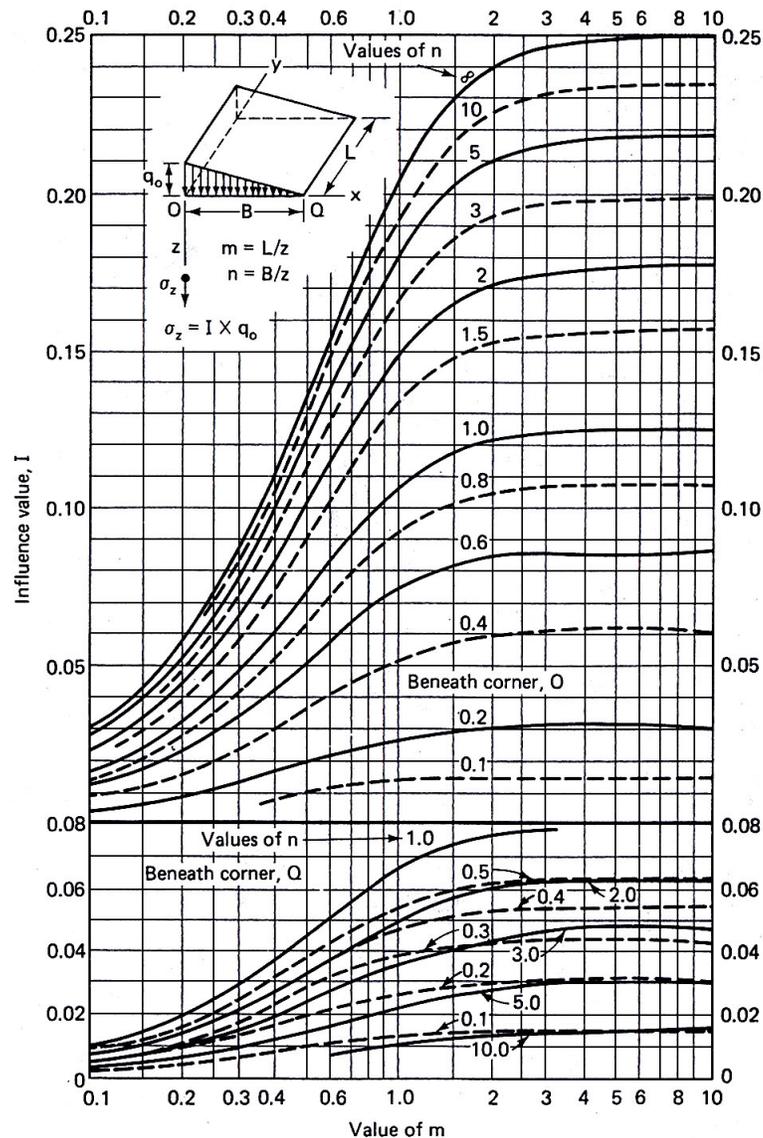
The equations for the theory of elasticity have been incorporated into design charts and tables for typical loading scenarios, such as below a foundation or an embankment.

Almost all foundation engineering textbooks include these charts. For convenience, charts to evaluate embankment loading are included as Figures 9-2 and 9-3.

**Figure 9-2 Influence Factors for Vertical Stress Under a Very Long Embankment (After NAVFAC, 1971 as Reported in Holtz and Kovacs, 1981)**



**Figure 9-3 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)**



#### 9-2.4.3.4 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 Section 8.11.3.2.

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.

#### 9-2.4.4 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.

### 9-3 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.

#### 9-3.1 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. Construction of the second and subsequent stages commences when the strength of the compressible soils is sufficient to maintain stability. 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 ( $K_0$ ), 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. Alternatively, numerical modeling programs, such as FLAC and PLAXIS, can be used to assess staged construction, subject to the approval of the WSDOT State 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 occurred, 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 if the desired amount of consolidation has occurred, since the rate of consolidation may still be on the linear portion of the curve at 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 ( $ru$ ), 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.

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 Appendix 9-A.

### 9-3.1.1 *Design Parameters*

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

The geotechnical designer 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 geotechnical designer 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 geotechnical designer 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 ( $P_o'$ ) for the sample as determined from the unit weight and the depth that the sample was obtained.

$$P_o' = D\gamma' \quad (9-1)$$

Where:

$$\begin{aligned} D &= \text{Sample Depth in feet} \\ \gamma' &= \text{Effective Unit Weight (pcf)} \end{aligned}$$

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:

$$A = \Delta U / \Delta\sigma_1 \quad (9-2)$$

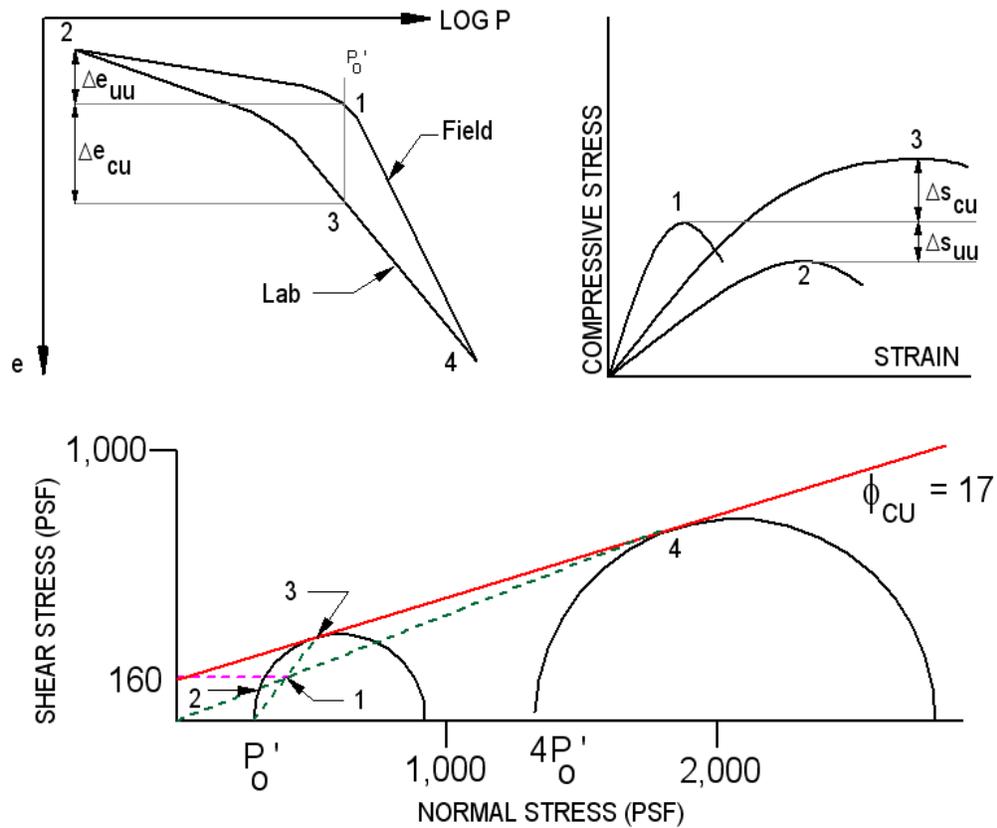
$$B = \Delta U / \Delta\sigma_3 \quad (9-3)$$

### 9-3.1.2 *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 geotechnical designer should be aware that sample disturbance can result in incorrect values of strength for normally consolidated fine grained soils. Figure 9-4 shows how to correctly obtain the cohesive strength for short term, undrained loading.

Figure 9-4 Determination of Short Term Cohesive Shear Strength From the CU Envelope



When a normally consolidated sample is obtained, the initial effective stress ( $P'_0$ ) and void ratio correspond to position 1 on the  $e - \log P$  curve shown in Figure 9-4. 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 ( $P'_0$ ) 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 ( $P'_0$ ) and as close as practical to  $4P'_0$ . On the Mohr diagram draw a line from the ordinate to point 4, and draw a second line from  $P'_0$  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 9-4, the cohesion intercept for the CU strength envelope (solid line) is 150 psf. The corrected strength based on the construction procedure in Figure 9-4 would be 160 psf. While the difference is slight in this example, it may be significant for other projects.

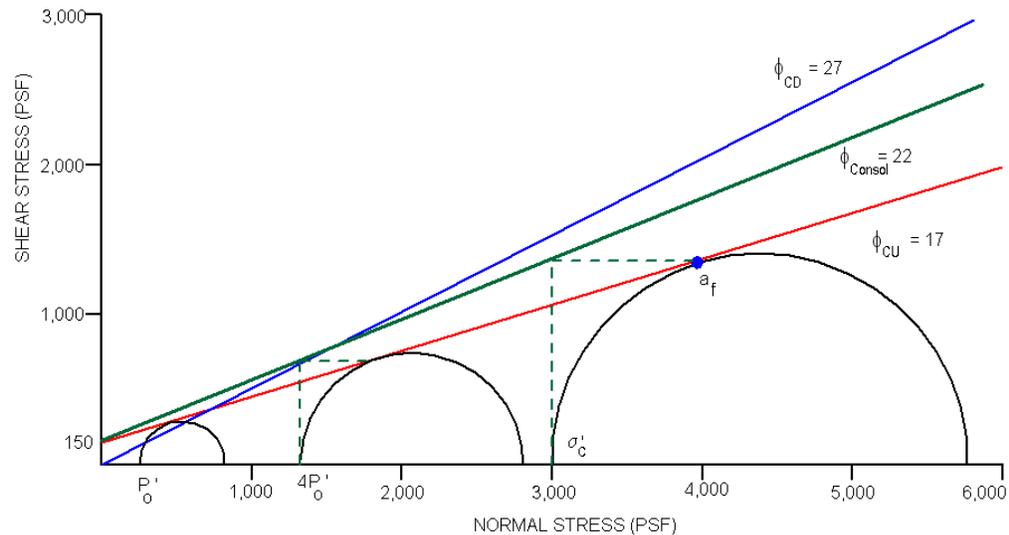
Once the correct shear strength data has been obtained, the embankment stability can be assessed. If the embankment stability is inadequate, proceed to performing a total stress or effective stress analysis, or both.

### 9-3.1.3 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 9-5 illustrates how to determine the correct strength due to consolidation and partial pore pressure dissipation.

**Figure 9-5 Consolidated Strength Construction From Triaxial Data**



To correct  $\phi_{cu}$  for the effects of consolidation use the following (see Ladd, 1991):

$$a_f/\sigma'_c = \tan \phi_{consol} \quad (9-4)$$

$$\tan \phi_{consol} = \sin \phi_{cu} / (1 - \sin \phi_{cu}) \quad (9-5)$$

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

$$\Delta C_{uu} = \Delta \sigma_v \tan \phi_{consol} \quad (9-6)$$

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:

$$C_{uu_{u\%}} = C_{uu_i} + U(C_{uu_i}) = C_{uu_i} + U\Delta\sigma_v \tan\phi_{\text{consol}} \quad (9-7)$$

The consolidation is dependent 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:

$$T = tC_v/H^2 \quad (9-8)$$

Where:

$$T = 0.25\pi U^2; \text{ for } U < 60\% \quad (9-9)$$

and,

$$T = 1.781 - 0.933\log(100 - U\%); \text{ for } U > 60\% \quad (9-10)$$

The geotechnical designer 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 Section 9.3.1.1.2 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 Section 9.2.3.1. 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 9-2 and 9-3). 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.

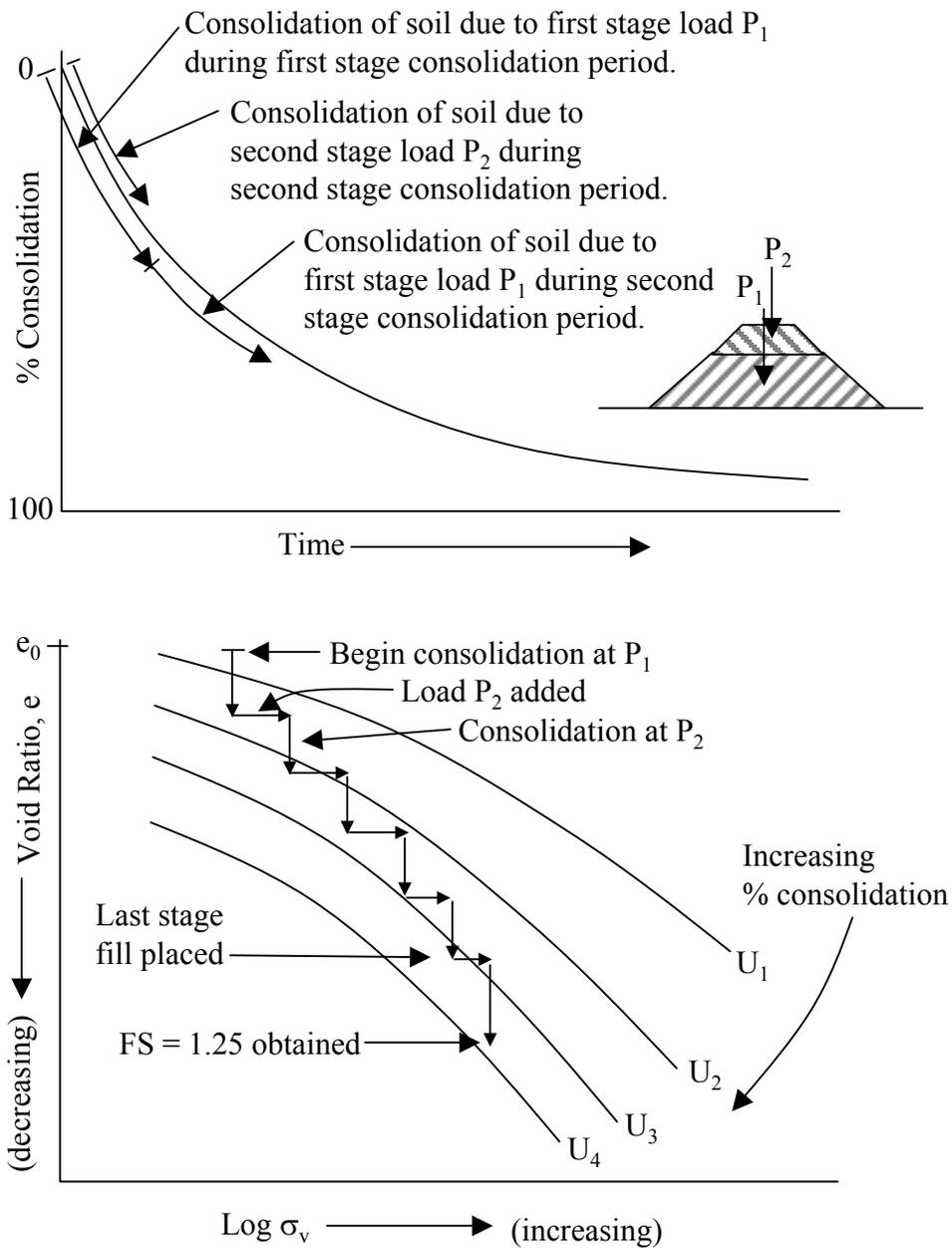
Determine the strength gain in each layer/section of soft subsoil by multiplying the consolidation stress increase by the tangent of  $\phi_{\text{consol}}$  (see Equation 9-6), where  $\phi_{\text{consol}}$  is determined as shown in Figure 9-5 and Equation 9-5. This will be an undrained strength. Multiply 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 9-7, 9-8, and 9-9 or 9-10. 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 9-6. 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 feet, and delay period increments range from 10 to 30 days.

Figure 9-6 Concepts Regarding the Percent Consolidation Resulting From Placement of Multiple Fill Stages

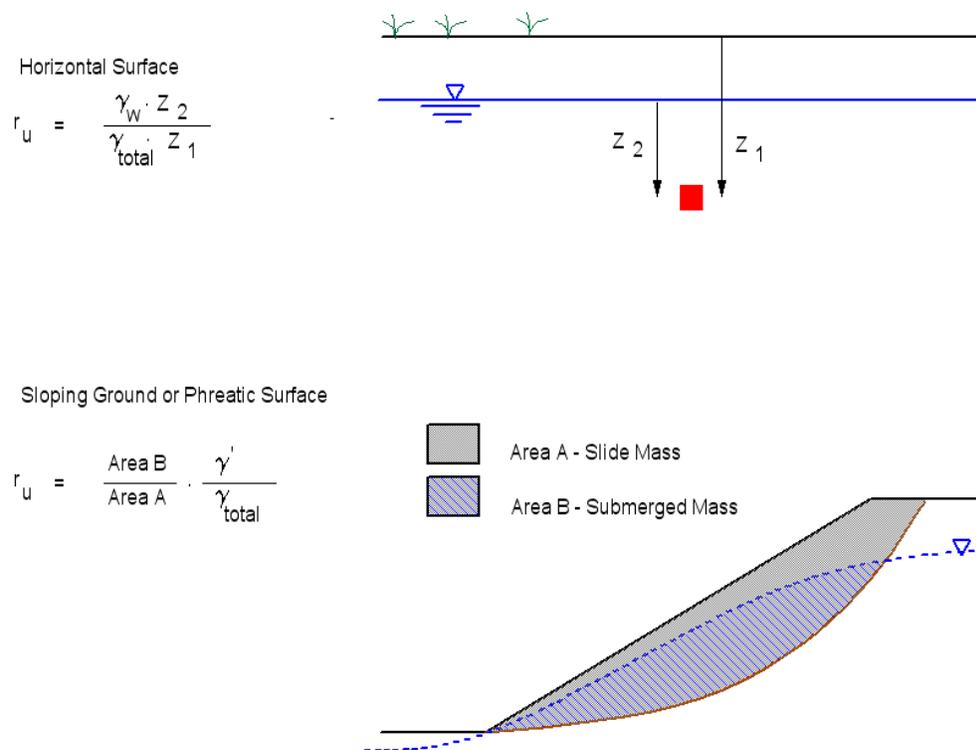


### 9-3.1.4 Effective Stress Analysis

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 Chapter 7). 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 9-7.

**Figure 9-7 Pore Pressure Ratio Concepts**



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 9-2 and 9-3), 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  $K_0$ , 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:

$$\Delta u_p = B(\Delta\sigma_{\text{oct}} + a\Delta\tau_{\text{oct}})(1-U) \quad (9-11)$$

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

$$\Delta\sigma_{\text{oct}} = (\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3)/3 = (\Delta\sigma_v + K_0\Delta\sigma_v + K_0\Delta\sigma_v)/3 = (1 + 2K_0)\Delta\sigma_v/3 \quad (9-12)$$

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.
- $\Delta\sigma_{\text{oct}}$  = 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.
- $\Delta\tau_{\text{oct}}$  = 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.

$$\Delta\tau_{\text{oct}} = [(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_3 - \Delta\sigma_1)^2]^{1/2} \quad (9-13)$$

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

$$\Delta\tau_{\text{oct}} = 1.414\Delta\sigma_v(1 - K_0) \quad (9-14)$$

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 9-11 simplifies to:

$$\Delta u_p = B[(1 + 2K_0)/3]\Delta\sigma_v(1-U) \quad (9-15)$$

where,  $K_0 = 1 - \sin \phi_{CD}$  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 Section 9.3.1.3).

Now determine  $r_u$  as follows:

$$r_u = \Delta u_p / \Delta\sigma_v = B[(1 + 2K_0)/3]\Delta\sigma_v(1-U) / \Delta\sigma_v = B[(1 + 2K_0)/3](1-U) \quad (9-16)$$

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.

### 9-3.2 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 Section 9.3.1). 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 determined per WSDOT Standard Practice T925 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. Alternatively, the values of  $T_{al}$  provided in the WSDOT Qualified Products List (QPL) could be used, but will be conservative for this application. 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 provided in the QPL or as otherwise determined using T925 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 Chapter 15). 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 gravel borrow, instead of common or select borrow, may also be appropriate in order to increase the embankment shear strength.

### 9-3.3 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. Chapter 11 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 Chapter 11 and Section 9.4.1) 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 Chapter 11 for additional guidance and references to use if this technique is to be implemented.

### 9-3.4 Light Weight 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 polystyrene blocks (geofoam), light weight aggregates (rhyolite, expanded shale, blast furnace slag, fly ash), wood fiber, shredded rubber tires, and other materials. Lightweight fills are infrequently used due to either high costs or other disadvantages with using these materials.

#### 9-3.4.1 Geof foam

Geofoam is approximately 1/100<sup>th</sup> the weight of conventional soil fill (typically with a unit weight of 1 to 3 pcf) and, as a result, is particularly effective at reducing driving forces or settlement potential. Typical geofoam embankments consist of the foundation soils, the geofoam fill, and a pavement system designed to transfer loads to the geofoam. Geofoam dissolves readily in gasoline and other organic fluids/vapors and therefore must be encapsulated where such fluids can potentially reach the geofoam. Other design considerations for geofoam include creep, flammability, buoyancy, moisture absorption, photo-degradation, and differential icing of pavement constructed over geofoam. Furthermore, geofoam should not be used where the water table could rise and cause buoyancy problems, as geofoam will float. Design guidelines for geofoam embankments are provided in the NCHRP document titled *Geofoam Applications in the Design and Construction of Highway Embankments* (Stark et al., 2004). Additional information on the design properties and testing requirements are provided in Chapter 5.

### 9-3.4.2 **Light Weight Aggregates**

Mineral aggregates, such as expanded shales, rhyolite, fly ash, or blast furnace slags, can also be used as lightweight fill materials. Expanded shales and rhyolite materials consist of inert mineral aggregates that have similar shear strengths to many conventional fill materials, but weigh roughly 50 to 70 pcf. The primary disadvantage with expanded shales and rhyolite 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 State Geotechnical Engineer is required before such materials may be considered for use in embankments.

### 9-3.4.3 **Wood Fiber**

Wood fibers may also be used for lightweight fill. For permanent applications, only fresh wood fiber should be used to prolong the life of the fill. Wood fiber fills typically have unit weights between about 35 to 55 pcf. To mitigate the effects of leachate, the amount of water entering the wood should be minimized. Wood fiber fill will experience creep settlement for several years and some pavement distress should be expected during that period. See Chapter 5 for more information regarding wood fiber fills.

### 9-3.4.4 **Scrap (Rubber) Tires**

In 1996, a moratorium on the use of scrap tires as embankment fill was put into effect due to several instances where the tire fills caught fire due to some type of exothermic reaction which has yet to be fully defined. A report to the Washington State legislature was published in 2003 to address whether or not, and under what circumstances, the moratorium on the use of scrap tires as fill should be lifted (Baker, et al., 2003). Based on that report, scrap tire fills up to 10 feet in thickness may be considered, provided that they are designed and specified as described in Baker, et al. (2003).

### 9-3.4.5 **Light Weight Cellular Concrete**

Large quantities of air can be entrained into concrete to produce a very light weight porous concrete, commonly referred to as cellular concrete, that can be poured in place of soil to reduce the driving force to improve stability or reduce settlement. Typical range of unit weights is from 25 to 65 pcf, and relative to soil, its shear strength is fairly high. Actual compressive strengths averaging 100 psi are expected for the 27-30 pcf density, the most commonly used density of cellular concrete. However, if significant differential settlement is still anticipated in spite of the use of the light weight concrete, due to its relatively brittle nature, the concrete could crack, losing much of its shear strength at the crack. This should be considered if using light weight cellular concrete.

If a unit weight of under 30 pcf is specified, placement technique can cause the bubbles in the concrete, which gives the cellular concrete its reduced unit weight, to collapse under its self-weight, resulting in concrete shrinkage and higher unit weight. To prevent

this, the concrete should be placed in lifts that are 2 ft or less if the unit weight is greater than 30 pcf, and pours limited to about one hour at a time. If the specified unit weight is under 25 pcf, it is difficult to maintain consistency and quality, to prevent the concrete from collapsing on itself during construction, or collapsing in the long-term if exposed to water passing through the porous concrete. To ensure consistency and quality of the final, as cured, product, a Class III light weight cellular concrete with a minimum unit weight of 30 pcf and a minimum compressive strength of 80 psi shall be used. For smaller, lower height, light weight cellular concrete fills, Class II light weight cellular concrete with a minimum unit weight of 25 pcf and minimum compressive strength of 40 psi, installed using a maximum lift thickness of 2 ft pours limited to about 1 hour at a time, may be considered, subject to approval by the State Geotechnical Engineer.

### 9-3.4.6 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. 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.

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 quarry spalls 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.

## 9-4 Settlement Mitigation

### 9-4.1 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 Section 9.3.3, 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 Chapter 11. Section 2-03.3(14)H of the WSDOT *Standard Specifications* addresses installation of prefabricated vertical drains.

## 9-4.2 Acceleration Using Surcharges

Surcharge loads are additional loads placed on the fill embankment above and beyond the design height. 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 Section 9.3. 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 is 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.

### 9-4.3 Lightweight Fills

Lightweight fills can also be used to mitigate settlement issues as indicated in Section 9.3.4. Lightweight fills reduce the new loads imposed on the underlying compressible soils, thereby reducing the magnitude of the settlement. See Chapter 5 and Section 9.3.4 for additional information on light weight fill.

### 9-4.4 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 feet 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 over-excavated unsuitable soils.

## 9-5 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. Chapter 530 of the WSDOT *Design Manual* provides guidance for the use of geotextile for separation or soil stabilization (see also Chapter 16). Note that for extremely soft and wet soil, a site specific design should be performed for the geotextile.

Hillside Terracing is specified in Section 2-03.3(14) of the WSDOT *Standard Specifications*. 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 3H:1V or steeper should be terraced to improve stability. However there may be specific cases where terracing may be waived during design, such as when the existing slope is steeper than 1H:1V and benching would destabilize the existing slope.

The compaction requirements in the WSDOT *Standard Specifications* apply to the entire embankment, including near the sloping face of the embankment.

For embankment slopes of 2H:1V 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 1.5H:1V. Surficial stability of embankments (See Chapter 7) should be evaluated during design for embankment slopes of 2H:1V or steeper. The embankment design shall include the use of techniques that will improve embankment face slope stability for embankment slopes steeper than 1.7H:1V, and should consider the use of such techniques for slopes of 2H:1V or steeper.

Approaches typically used to address compaction and surficial stability of embankment slopes include:

- Over-build the embankment laterally at the slope face approximately 2 feet, compact the soil, and then trim off the outer 2 feet of the embankment to produce a well compacted slope face.
- Use strips of geosynthetic placed in horizontal layers at the slope face as a compaction and surficial stability aid (see Elias, et al., 2001). The strips should generally be a minimum of 4 feet wide (horizontally into the slope) and spaced vertically at 1 to 1.5 feet (1.5 feet maximum). The specific reinforcement width and vertical spacing will depend on the soil type. The reinforcement strength required depends on the coarseness and angularity of the backfill material and the susceptibility of the geosynthetic to damage during placement and compaction. See Elias, et al. (2001) for specific guidance on the design of geosynthetic layers as a compaction and surficial stability aid.

Even if good compaction can be obtained using one of these techniques, the potential for erosion and surficial instability should be addressed through appropriate use of slope vegetation techniques such as seeding and mulching, temporary or permanent turf reinforcement mats, or for deeper surficial stability problems, bioengineering. Note that

if geosynthetic layers are placed in the soil as a compaction aid or to improve overall embankment slope stability, the typical practice of cultivating the upper 1 feet of the soil per the WSDOT *Standard Specifications*, Section 8-02, should not be conducted. Instead, the landscape architect who is developing the slope vegetation plan should consult with the HQ Geotechnical Division to insure that the slope vegetation plan (either per the WSDOT *Standard Specifications* or any special provisions developed) does not conflict with the slope geosynthetic reinforcement and the need for good compaction out to the slope face.

### 9-5.1 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 geotechnical report should 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 Section 9.3.1, 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.

### 9-5.2 Instrumentation

The following discussion of monitoring equipment typically used for embankment construction monitoring provides an overview of the typical equipment available. 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). Additional information on WSDOT policies regarding instrumentation installation and standards is provided in Chapter 3.

### 9-5.2.1 *Piezometers*

Three types of piezometers are commonly used to monitor embankment construction: open standpipe, pneumatic and vibrating wire. Each type of piezometer has advantages and disadvantages. The sections below describe the various piezometer types.

**Open Standpipe Piezometers** – These piezometers are installed in a drilled borehole. A porous zone or screen is installed in the soil layer of interest. For embankment settlement purposes it is necessary to completely seal the porous zone against the inflow of water from shallower zones. Open standpipe piezometers are relatively simple to install and the water level readings are easy to obtain. However, standpipes may interfere with or be damaged by construction activities and the response time for changes in water pore pressure in low permeability soils is slow. This type of piezometer is generally not very useful for monitoring the pore pressure increase and subsequent decrease due to consolidation in staged construction applications.

**Pneumatic Piezometers** – Pneumatic piezometers are usually installed in drilled boreholes in a manner similar to standpipe piezometers, but they can be sealed so that increases in pore water pressure result in a smaller volume change and a more rapid response in instrument measurement. Pneumatic piezometers do not need open standpipes. However, crimping or rupture of the tubes due to settlement of the embankment can cause failure.

**Vibrating Wire Piezometers** – Vibrating wire piezometers are usually installed in drilled boreholes; although, models are available for pushing into place in soft soils. The cables can be routed long distances and they are easily connected to automatic data acquisition systems.

### 9-5.2.2 *Instrumentation for Settlement*

#### 9-5.2.2.1 **Settlement Plates**

Settlement plates are used to monitor settlement at the interface between native ground and the overlying fill. They consist of a steel plate welded to a steel pipe. An outer pipe consisting of steel or PVC pipe is placed around the pipe and the embankment is built up around it. Both pipes are extended to the completed surface. The outer pipe isolates the inner pipe from contact with the fill. As the embankment and soil surface settle, the top of the inner pipe can be monitored with standard survey equipment. These devices are simple to use, but provide data at only one point and are subject to damage during construction.

#### 9-5.2.2.2 **Pneumatic Settlement Cells**

These cells are generally placed at the interface between the embankment fill and native ground. A flexible tube is routed to a reservoir, which must be located away from the settlement area. The reservoir must be kept at a constant elevation. The precision of the cells is about 0.75 inches.

### 9-5.2.2.3 Sondex System

The Sondex System can be used for monitoring settlement at several points at depth. The system is installed in a borehole and consists of a series of stainless steel wire loops on a plastic corrugated pipe. The plastic pipe is placed over an access casing and grouted in the borehole. The locations of the stainless steel loops are determined by electrical induction measurements from a readout unit. The loops can be located to about 0.05 inches and displacements of up to 2 inches can be measured. Accurate measurement of settlement depends on the compatibility of the soil and grout.

Therefore, if the grout mix has a higher strength than the surrounding soil, not all the settlement will be measured.

### 9-5.2.2.4 Horizontal Inclinometer

Horizontal inclinometers are used to measure vertical deflections in a grooved guide casing, placed horizontally beneath the embankment. The probe is pulled through the casing and readings of inclination relative to horizontal are obtained. The inclinometer is a highly accurate system for obtaining settlement data. Because the length of the inclinometer probe is typically about 2 feet, large displacements of the casing caused by settlement may stop passage of the probe.

## 9-5.3 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 geotechnical designer should analyze and interpret the data.

## 9-5.4 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.
- Hillside terracing 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

## 9-5.5 Requirements for Temporary Fills for Construction Facilitation

Temporary fills for haul roads, construction equipment access, and other temporary construction activities shall be designed in accordance with this GDM, in particular this chapter (Chapter 9), except as noted in the following subsections.

### 9-5.5.1 Design Requirements

The design of the temporary fill/fill slope shall address the stability and settlement of the temporary fill itself as well as the impact of the temporary fill on the global stability and deformation of the of the overall slope on which the fill is located. The stability and movement of any temporary structures and construction equipment (e.g., cranes, compaction equipment, etc.) placed on the temporary fill shall also be addressed in the design. Temporary fills and fill slopes shall be designed such that the risk to health and safety of workers and the public is kept to an acceptable level and that adjacent facilities are not damaged. Seismic design of temporary fills and fill slopes is not required.

If temporary fills are placed on or adjacent to permanent or temporary structures, the impact of the temporary fill on those structures, both with regard to stability and lateral and vertical movements, shall be assessed. The functioning and design life of those structures shall not be compromised by the placement of the temporary fill.

If temporary walls are used to support the temporary fill, the impact of the temporary fill on the wall stability and deformations shall be addressed, and the design of the temporary wall shall meet the requirements in Chapter 15 and the AASHTO LRFD Bridge Design Specifications.

As a minimum, the design of temporary fill slopes for stability by or under the supervision of a registered professional engineer shall include geotechnical calculations to address slope stability (i.e., Chapter 7). If the fill is placed over relatively soft to very soft ground, the deformation of the fill shall also be determined through engineering calculations (i.e., Chapter 9) that are based on a knowledge of the subsurface conditions present and engineering data that can be used to estimate soil and rock properties. Such calculations shall also address the effect of ground water conditions and the loading conditions on or above the slope that could affect its stability and deformation. The design shall be conducted in accordance with the requirements in this GDM and referenced documents. Engineering recommendations based upon field observations alone shall not be considered to be an engineering design, unless the fill is a low height (less than 10 feet high) granular, cohesionless well-compacted fill without concentrated loads from large equipment or structure supports, and the fill is placed over dense to very dense soil or rock, in which the supporting soil or rock is not affected by fissures, slickensides, or other localized weaknesses.

### **9-5.5.2 Safety Factors and Design Life Considerations**

For temporary fill slopes, the safety factors specified in Section 9.2.3.1 are applicable. If the soil properties are well defined and shown to have low variability, a lower factor of safety may be justified through the use of the Monte Carlo simulation feature available in slope stability analysis computer programs. In this case, a probability of failure of 0.01 or smaller shall be targeted (Santamarina, et al., 1992). However, even with this additional analysis, in no case shall a slope stability safety factor less than 1.2 be used for design of the temporary fill slope.

### **9-5.5.3 Design Loads**

The design of temporary fills and fill slopes shall address the actual construction- related loads that could be imposed on the temporary fill. As a minimum, the temporary fill shall be designed for a live load surcharge of 250 psf to address routine construction equipment traffic on the fill. For unusual temporary loadings resulting from large cranes or other large equipment placed on the fill, the loading imposed by the equipment shall be specifically assessed and taken into account in the design of the fill. For the case where large or unusual construction equipment loads will be applied to the fill, the construction equipment loads shall still be considered to be a live load, unless the dynamic and transient forces caused by use of the construction equipment can be separated from the construction equipment weight as a dead load, in which case, only the dynamic or transient loads carried or created by the use of the construction equipment need to be considered live load.

If temporary structures (e.g., false work and formwork support) are placed on or adjacent to the temporary fill, the temporary fill shall be designed to carry the loads resulting from the temporary structures and to meet the stability and deformation requirements of those structures.

### **9-5.5.4 Design Property Selection**

In addition to the requirements in Chapter 9 for determination of design properties, the requirements for design property selection for temporary cuts and shoring in Chapters 5 and 15 shall also be considered applicable to temporary fills and fill slopes.

### **9-5.5.5 Performance Requirements for Temporary Fills and Fill Slopes**

Temporary fills and slopes shall be designed to prevent excessive deformation that could result in damage to adjacent facilities, both during fill construction and during the life of the temporary fill. An estimate of expected displacements or vibrations, threshold limits that would trigger remedial actions, and a list of potential remedial actions if thresholds are exceeded should be developed. Thresholds shall be established to prevent damage to adjacent facilities, as well as degradation of the soil properties due to deformation.

The removal of the temporary fill shall not adversely impact adjacent structures and facilities.

### 9-5.5.6 *Temporary Fill Submittal and Submittal Review Requirements*

Temporary Fill submittals shall generally meet the requirements in Section 2-09.3(3)B of the *Standard Specifications M 41-10*.

When performing a geotechnical review of a contractor temporary fill submittal, the following items should be specifically evaluated:

1. Performance objectives for the temporary fill
  - a. Is the anticipated length of time the temporary fill will be in place provided?
  - b. Are objectives regarding anticipated and allowed deformations of the fill and adjacent and supported structures provided?
  - c. Are the performance objectives compatible and consistent with contract and GDM/BDM requirements?
2. Subsurface conditions
  - a. Is the soil/rock stratigraphy consistent with the subsurface geotechnical data provided in the contract boring logs?
  - b. Did the contractor/fill designer obtain the additional subsurface data needed to meet the geotechnical exploration requirements fills and temporary fill walls as identified in Chapters 9 and 15, respectively?
  - c. Was justification for the soil, rock, and other material properties used for the design of the temporary fill provided, and is that justification, and the final values selected, consistent with Chapter 5 and the subsurface field and lab data obtained at the fill site?
  - d. Were ground water conditions adequately assessed through field measurements combined with the site stratigraphy to identify zones of ground water, aquitards and aquicludes, artesian conditions, and perched zones of ground water that could impact the stability and deformation of the fill and adjacent facilities that may be impacted by the presence of the temporary fill?
3. Temporary fill loading
  - a. Have the anticipated loads on or caused by the temporary fill been correctly identified, considering all applicable limit states?
  - b. If construction or public traffic near or on the temporary fill, has a minimum traffic live load surcharge of 250 psf been applied?
  - c. If larger construction equipment such as cranes will be placed on the temporary fill, have the loads from that equipment been correctly determined and included in the temporary fill design?

4. Temporary fill design
  - a. Have the correct design procedures been used (i.e., the GDM and referenced design specifications and manuals)?
  - b. Have all appropriate limit states been considered (e.g., global stability of slopes above and below wall, global stability of wall/slope combination, internal wall stability, external wall stability, bearing capacity, settlement, lateral deformation, piping or heaving due to differential water head, etc.)?
5. Are all safety factors, or load and resistance factors for LRFD temporary wall or structure design, identified, properly justified in a manner that is consistent with the GDM, and meet or exceed the minimum requirements of the GDM?
6. Have the effects of any construction activities adjacent to the temporary fill on the stability/performance of the fill been addressed in the shoring design (e.g., excavation or soil disturbance below the fill, excavation dewatering, vibrations and soil loosening due to soil modification/improvement activities, etc.)?
7. Temporary fill monitoring/testing
  - a. Is a monitoring/testing plan provided to verify that the performance of the fill and the structures it supports or impacts is acceptable throughout the design life of the system?
  - b. Have appropriate displacement or other performance triggers been provided that are consistent with the performance objectives of the fill and adjacent facilities?
8. Temporary fill removal
  - a. Have any portions of the temporary fill (including temporary fill walls used to support the fill) to be left in place after construction of the permanent structure is complete been identified?
  - b. Has a plan been provided regarding how to prevent the remaining portions of the temporary fill or walls from interfering with future construction and performance of the finished work (e.g., will the remaining portions impede flow of ground water, create a hard spot, create a surface of weakness regarding slope stability, etc.)?

## 9-6 References

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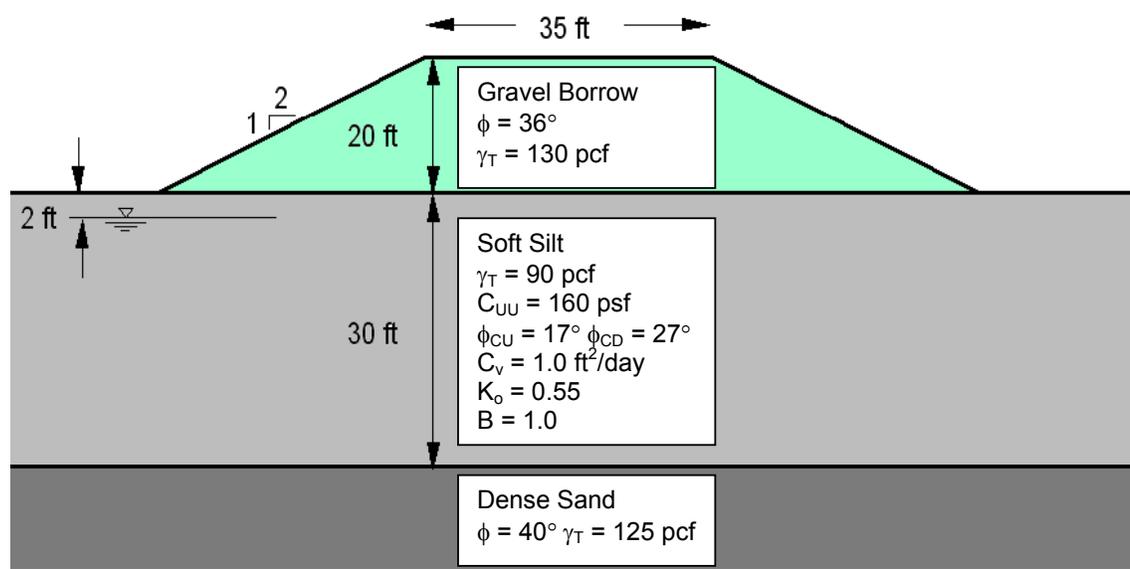
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# Appendix 9-A Examples Illustrating Staged Fill Construction Design

## 9-A-1 Problem Setup

First, the geotechnical designer 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 feet thick earth embankment from Gravel Borrow with 2H:1V side slopes. The embankment will have a roadway width of 35 feet and will be constructed over soft silt. The soft silt is 30 feet thick and overlies dense sand. Ground water was observed 2 feet below the existing ground surface during the field exploration.

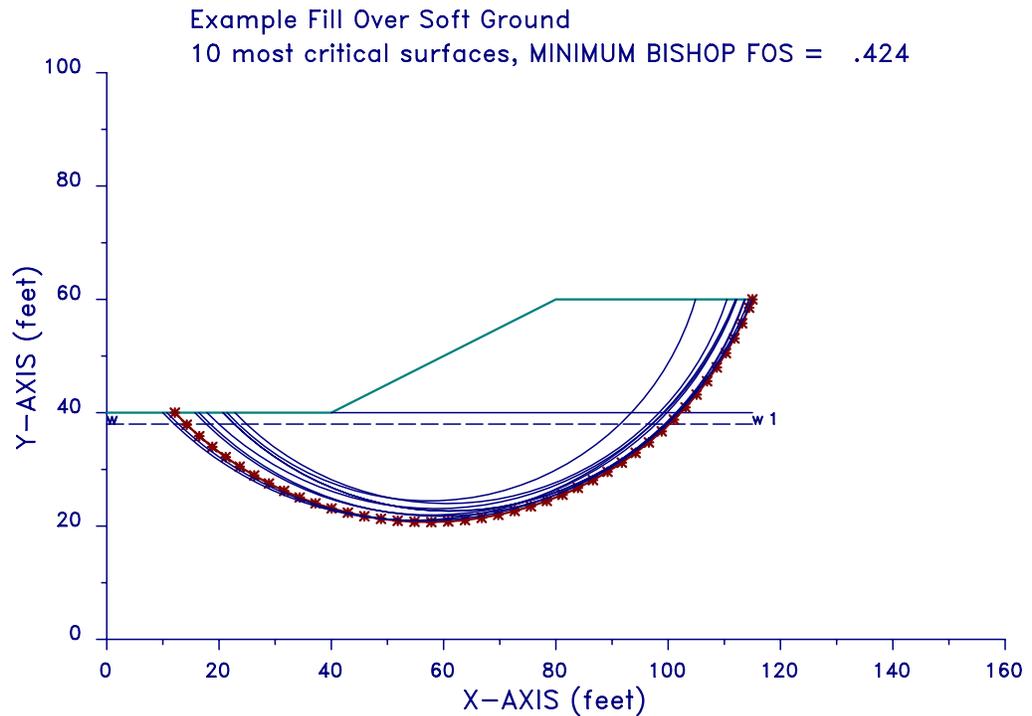
Figure 9-A-1 Embankment Geometry for Example



Using the test results, the geotechnical designer 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 short-term (undrained) stability using  $C_{uu} = 160$  psf (see Figures 9-4 and 9-5 for the specific strength envelopes used). Figure 9-A-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.

**Figure 9-A-2 Undrained Stability for the Example Geometry**

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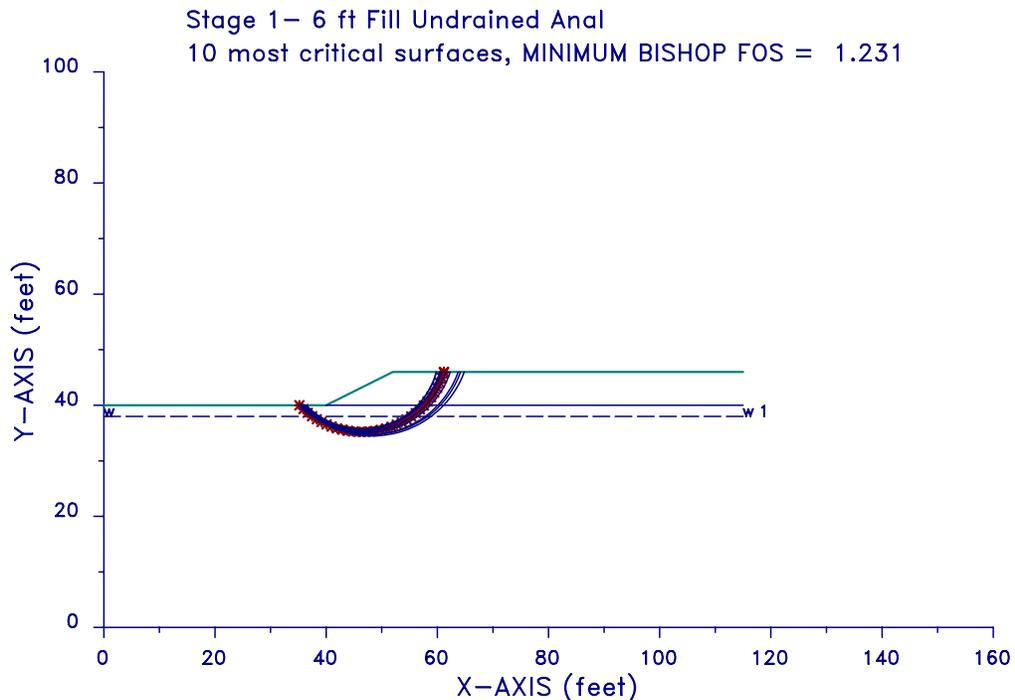


## 9-A-2 Determination of Maximum Stable First Stage Fill Height

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 9-A-3, the maximum initial fill height is 6 feet. This initial fill height is used as a starting point for both the total stress and the effective stress analyses.

**Figure 9-A-3 Stage 1 Fill Stability, Assuming no Strength Gain and a Fill Height of 6 Feet**

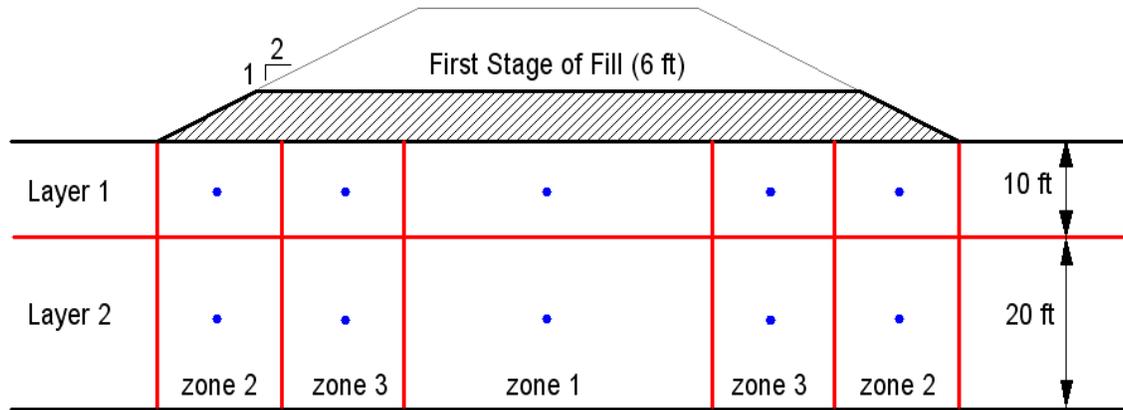
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## 9-A-3 Total Stress Analysis Procedure Example

In this approach, the undrained soil strength envelope, or  $\phi_{\text{consol}}$ , as determined in Figure 9-5, is used to characterize the strength of the subsoil. Next, the geotechnical designer 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 Section 9.3.1.3). The geotechnical designer calculates the stress increase resulting from the placement of the first embankment stage using the Boussinesq equation or those of Westergaard (see Figures 9-2 and 9-3). 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 9-A-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.

Figure 9-A-4 Division of Subsurface for Estimating Strength Increase and Consolidation



For the example geometry model the embankment as a continuous strip with a width of 103 feet ( $B = 35' + (4 \times 20) - (2 \times 6)$ ). 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:

Zone	Layer	Z	Z/B	I	$\sigma_v$ 6 feet $\times$ 130 pcf	$\Delta\sigma_v$ ( $I \times \sigma_v$ )
1	1	5 feet	0.049	0.98	780 psf	764 psf
	2	20 feet	0.190	0.93	780 psf	725 psf
2	1	5 feet	0.049	0.55	780 psf	429 psf
	2	20 feet	0.190	0.75	780 psf	585 psf

Once the geotechnical designer has the stress increase, the increase in strength due to consolidation can be estimated using Equations 9-6 and 9-7. 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 9-8 through 9-10, 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:

$$T = tC_v/H^2$$

$$T = 15 \text{ days}(1 \text{ feet}^2/\text{Day})/(30 \text{ feet}/2)^2 \text{ (assumed double draining)}$$

$$T = 0.067 = 0.25pU^2; \text{ for } U < 60\%$$

$$U = 0.292 \text{ or } 29\%$$

Therefore, at 15 days and 29% consolidation, using Equation 9-7, the strength gain would be as follows:

Zone	Layer	$\Delta\sigma_v$ ( $I \times \sigma_v$ )	$C_{uu}$	U	$\phi_{\text{consol}}$	$C_{uu \text{ 29\%}}$
1	1	764 psf	160 psf	0.29	22°	250 psf
	2	725 psf	160 psf	0.29	22°	245 psf
2	1	429 psf	160 psf	0.29	22°	210 psf
	2	585 psf	160 psf	0.29	22°	228 psf

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:

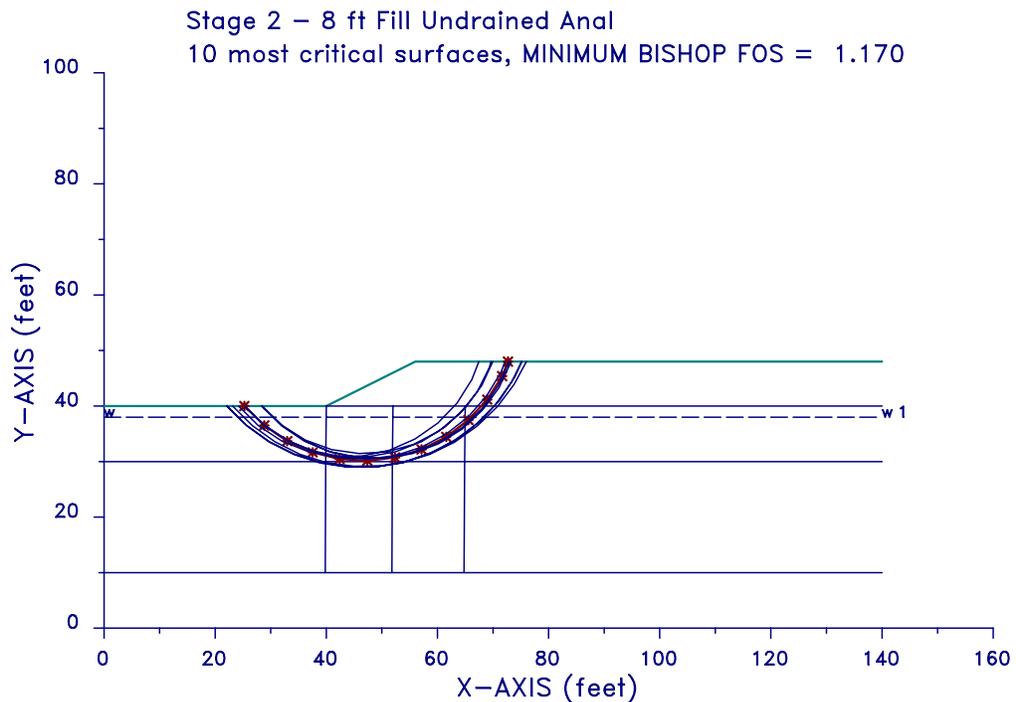
Zone	Layer	$\Delta\sigma_v$ ( $l \times \sigma_v$ )	$C_{uu}$	U	$\phi_{consol}$	$C_{uu\ 59\%}$
1	1	764 psf	160 psf	0.59	22°	342 psf
	2	725 psf	160 psf	0.59	22°	333 psf
2	1	429 psf	160 psf	0.59	22°	262 psf
	2	585 psf	160 psf	0.59	22°	299 psf

The geotechnical designer 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 geotechnical designer 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 9-A-5, the total fill height can be increased to 8 feet (2 feet of new fill is added) after the 15 day delay period.

**Figure 9-A-5 Stage 2 Undrained Analysis, Assuming 15 Day Delay Period After Atage 1, and a Total Fill Height of 8 Feet**

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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 feet ( $B = 35' + (4 \times 20) - (2 \times 8)$ ). 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:

Zone	Layer	Z	Z/B	I	$\sigma_v$ 8 feet $\times$ 130 pcf	$\Delta\sigma_v$ ( $I \times \sigma_v$ )
1	1	5 feet	0.049	0.98	1040 psf	1019 psf
	2	20 feet	0.190	0.93	1040 psf	967 psf
2	1	5 feet	0.049	0.55	1040 psf	231 psf
	2	20 feet	0.190	0.75	1040 psf	315 psf

Once the geotechnical designer has the stress increase, the increase in strength due to consolidation can be estimated. The geotechnical designer must now begin to use weighted averaging to account for the difference in consolidation times (see Figure 9-6). The first stage of fill was allowed to settle for 15 days prior to placing the additional 2 feet of fill in the second stage, bringing the total fill height up to 8 feet. 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:

$$T = tC_v/H^2$$

$$T = 30 \text{ days}(1 \text{ feet}^2/\text{Day})/(30 \text{ feet}/2)^2 \text{ (assumed double draining)}$$

$$T = 0.133 = 0.25\pi U^2; \text{ for } U < 60\%$$

$$\text{So, } U = 0.41 \text{ or } 41\%$$

The average consolidation of the 15 + 15 day delay period will be:  
 $[6 \text{ feet}(0.41) + 2 \text{ feet}(0.29)]/8 \text{ feet} = 0.38 \text{ or } 38\%$

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

Zone	Layer	$\Delta\sigma_v$ ( $I \times \sigma_v$ )	$C_{uu}$	U	$\phi_{\text{consol}}$	$C_{uu \text{ 38\%}}$
1	1	764 psf	160 psf	0.38	22°	317 psf
	2	725 psf	160 psf	0.38	22°	309 psf
2	1	429 psf	160 psf	0.38	22°	248 psf
	2	585 psf	160 psf	0.38	22°	280 psf

The geotechnical designer 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:

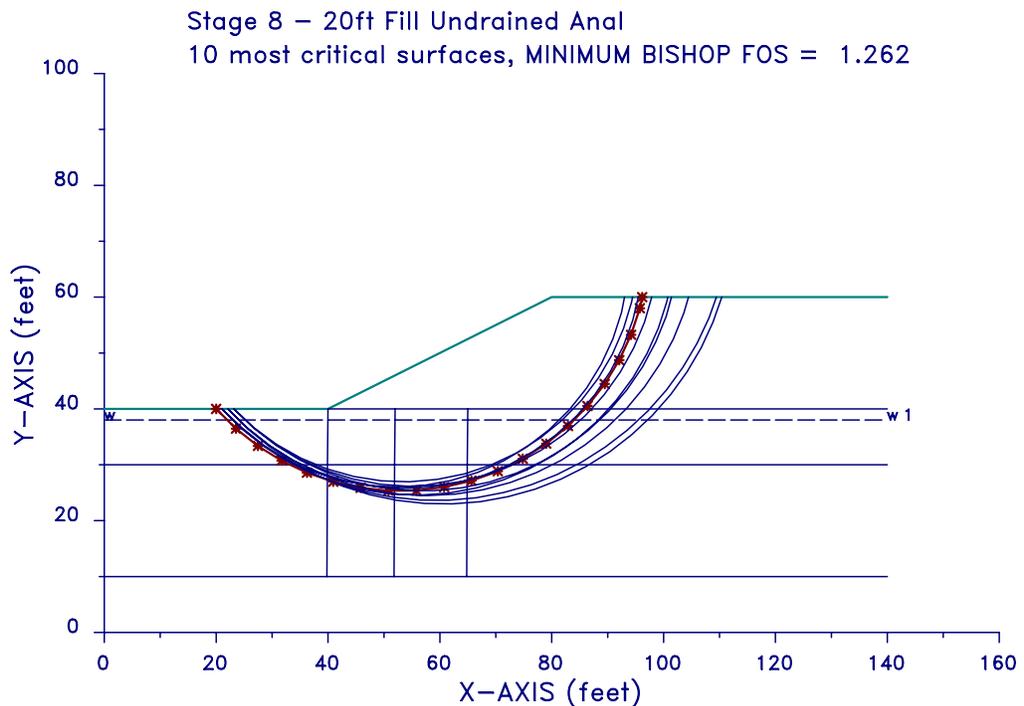
Zone	Layer	$\Delta\sigma_v$ ( $l \times \sigma_v$ )	$C_{uu}$	U	$\phi_{consol}$	$C_{uu, 38\%}$
1	1	2509 psf	160 psf	0.71	22°	880 psf
	2	780 psf	160 psf	0.71	22°	384 psf
2	1	2314 psf	160 psf	0.71	22°	824 psf
	2	962 psf	160 psf	0.71	22°	436 psf
3	1	1430 psf	160psf	0.71	22°	570 psf
	2	1560 psf	160 psf	0.71	22°	608 psf

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 9-A-6.

**Figure 9-A-6 Final Stage Undrained Analysis, Assuming 25 Days Have Expired Since Last Fill Increment Was Placed, and a Total Fill Height of 20 Feet**

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In summary, the fill increments and delay periods are as follows:

Stage	Fill Increment	Time Delay Prior to Next Stage
1	6 feet	15 days
2	2 feet	15 days
3	2 feet	15 days
4	2 feet	15 days
5	2 feet	30 days
6	2 feet	30 days
7	3 feet	10 days
8	1 feet	25 days to obtain FS = 1.25
<b>TOTALS</b>	20 feet	155 days

Fewer stages can be selected by the geotechnical designer, 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:

Stage	Fill Increment	Time Delay Prior to Next Stage
1	6 feet	60 days
2	4.5 feet	60 days
3	5.5 feet	40 days
4	4 feet	5 days to obtain FS = 1.25
<b>TOTALS</b>	20 feet	165 days

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.

#### 9-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 9-5,  $\phi_{CD}$  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 geotechnical designer should be aware of how the stability program calculates  $r_u$ . When using XSTABL, the geotechnical designer should not input  $r_u$  program directly. Instead, he should input excess pore pressures directly into the 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 9-A-1, the geotechnical designer should divide the subsurface into layers and zones in a manner similar to that shown in Figure 9-A-4. The geotechnical designer 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 feet:

Zone	Layer	Z	Z/B	I	$\sigma_v$ 6 feet $\times$ 130 pcf	$\Delta\sigma_v$ (I $\times$ $\sigma_v$ )
1	1	5 feet	0.049	0.98	780 psf	764 psf
	2	20 feet	0.190	0.93	780 psf	725 psf
2	1	5 feet	0.049	0.55	780 psf	429 psf
	2	20 feet	0.190	0.75	780 psf	585 psf
3	1	5 feet	0.049	0.98	780 psf	764 psf
	2	20 feet	0.019	0.93	780 psf	725 psf

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

As discussed previously in Section 9.3.1.4, the pore pressure increase is dependent upon the load and the degree of consolidation. Using Equation 9-15 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.

$$K_0 = 1 - \sin \phi_{CD} = 1 - \sin 27^\circ = 0.55$$

$B = 1.0$ , 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:

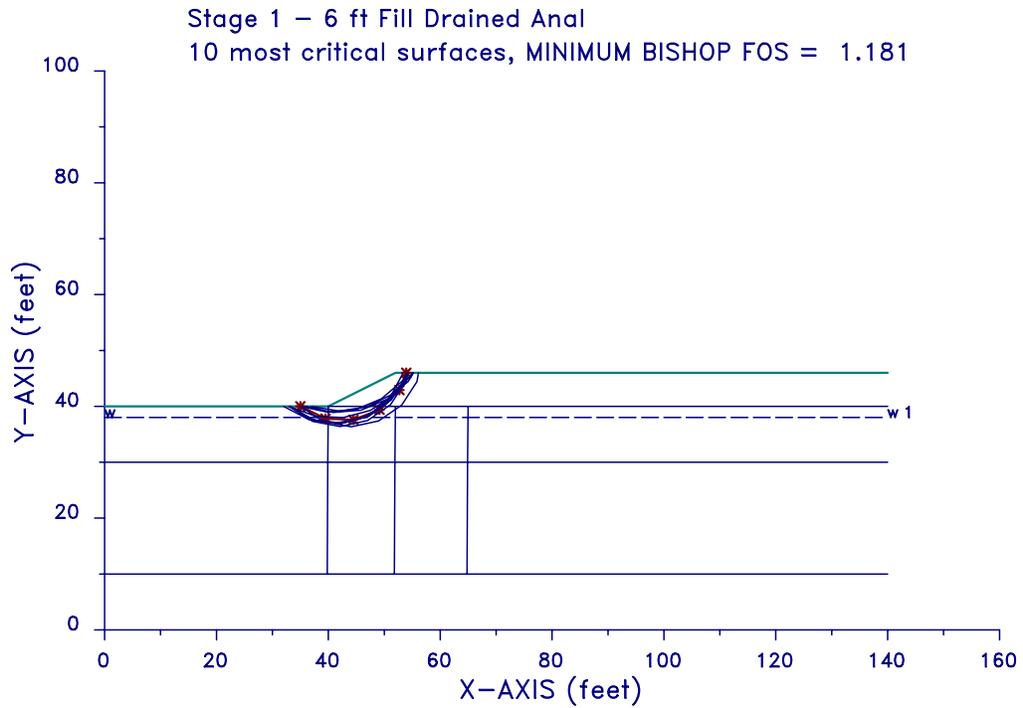
Layer	Zone	$\Delta\sigma_v$ (I $\times$ $\sigma_v$ ) (psf)	U (%)	$\Delta u_{p30\%}$ (psf)	U (%)	$\Delta u_{p35\%}$ (psf)	U (%)	$\Delta u_{p40\%}$ (psf)
1	1	764	30	374	35	346	40	320
	2	725	30	354	35	329	40	303
2	1	429	30	209	35	194	40	179
	2	585	30	286	35	265	40	245
3	1	764	30	373	35	346	40	320
	2	725	30	354	35	329	40	303

The slope stability results from XSTABL are provided in Figure 9-A-7. For the two subsoil layers, all zones, a drained friction angle,  $\phi_{CD}$ , of  $27^\circ$  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 9-A-7 as pore pressure constants.

The results shown in this figure are for a percent consolidation of 35%.

**Figure 9-A-7 Stage 1 Drained Analysis at Percent Consolidation of 35% and a Fill Height of 6 Feet**

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Using Equation 9-16,  $r_u$  at this stage of the fill construction is determined as follows:

$$r_u = B[(1 + 2K_0)/3](1-U) = 1.0[(1 + 2(0.55))/3](1-0.35) = 0.45$$

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 feet) and the maximum fill height (20 feet) will be checked.

For a fill height of 13.5 feet, the stress increases in the zones are as follows based on an equivalent strip footing width of 88 feet:

Zone	Layer	Z	Z/B	I	$\sigma_v$ 13 feet x 130 pcf	$\Delta\sigma_v$ (I x $\sigma_v$ )
1	1	5 feet	0.049	0.97	1,690 psf	1,700 psf
	2	20 feet	0.190	0.90	1,690 psf	1,580 psf
2	1	5 feet	0.049	0.40	1,690 psf	702 psf
	2	20 feet	0.190	0.55	1,690 psf	965 psf
3	1	5 feet	0.049	0.75	1,690 psf	1,320 psf
	2	20 feet	0.019	0.70	1,690 psf	1,230 psf

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.

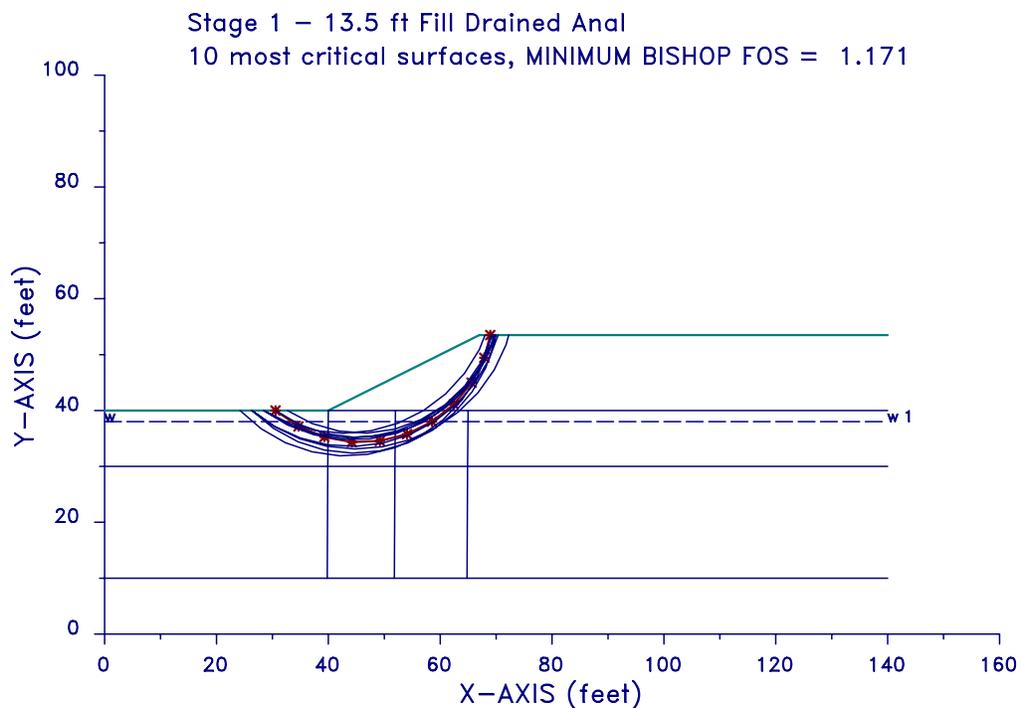
The pore pressure increase resulting from a 13.5 feet high fill, assuming various percent consolidations, is recalculated using Equation 9-15 as illustrated earlier. The results of these calculations are as tabulated below:

Zone	Layer	$\Delta\sigma_v$ ( $l \times \sigma_v$ ) (psf)	U (%)	$\Delta u_{p55\%}$ (psf)	U (%)	$\Delta u_{p60\%}$ (psf)	U (%)	$\Delta u_{p65\%}$ (psf)
1	1	1702	55	534	60	475	65	415
	2	1580	55	496	60	441	65	386
2	1	702	55	220	60	196	65	171
	2	695	55	218	60	194	65	170
3	1	1316	55	413	60	367	65	321
	2	1229	55	386	60	343	65	300

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 feet high. The slope stability results from XSTABL are provided in Figure 9-A-8. For the two subsoil layers, all zones, a drained friction angle,  $\phi_{CD}$ , of  $27^\circ$  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 9-A-8 as pore pressure constants. The results shown in this figure are for a percent consolidation of 60%.

**Figure 9-A-8 Stage 2 Drained Analysis at Percent Consolidation of 60% and a Fill Height of 13.5 Feet**

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Using Equation 9-16,  $r_u$  at this stage of the fill construction is determined as follows:

$$r_u = B[(1 + 2K_0)/3](1-U) = 1.0[(1 + 2(0.55))/3](1-0.60) = 0.28$$

Similarly, these calculations were conducted for the full fill height of 20 feet, 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:

Total Fill Height (ft)	$r_u$
6	0.45
13.5	0.28
20	0.22

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.