Chapter 7
SEEPAGE AND HYDROSTATIC UPLIFT ANALYSES

Introduction

This chapter provides information to use when evaluating the seepage and hydrostatic uplift potential at a waste containment facility in Ohio. Both of these forces can cause significant damage to the landfill foundation layers, engineered components, and structures at a waste containment facility.

Seepage and hydrostatic uplift may undermine the foundation of engineered components and those ancillary structures that usually have small basal foot prints when the ground water level (phreatic or potentiometric) rises above the bottom elevation of the engineering component or structure. This condition may lead to pressure build up beneath the engineered component or structure that can simply lift the engineering component or damage the foundation soils under the structure.

Hydrostatic uplift is a floatation condition that is caused by the displaced volume of the rising water table. This condition can be easily corrected by counteracting and equalizing the hydrostatic uplift force by building heavier structures, anchoring, or by placing overburden material. A properly designed filter accompanied by an increase in the vertical stress from the constructed liner or emplaced waste may prevent damage associated with hydrostatic uplift or reduce the likelihood of soil boils from forming by keeping the sand particles in their original positions. However, it should be noted that such measures will have no measurable impact on the reduction of pore water pressure buildup within the soil and hence cannot be employed to reduce the seepage potential and associated damage.

Seepage is the flow of water through soils caused by the difference in head. This difference in head is a measure of the energy lost in overcoming the resistance provided by the soils and other underground obstructions. Seepage damage can be classified into three broad categories.

1. Uplift
2. Heaving
3. Piping or internal erosion.
Uplift Concept and Analysis

Water percolation through a soil layer affects hydrostatic uplift force. As a result, considering seepage may theoretically be a more accurate approach. The shear resistance of the soil could also be theoretically taken into account. However, for practical purposes, a conservative evaluation of the resistance created by a soil layer against hydrostatic uplift can be accomplished by calculating a maximum uplift force based on a maximum measured piezometric head and comparing it to the normal stress created by the overlying soil layers. This is especially true when checking an interface between a subbase and a clay (or plastic) liner, where any significant seepage through the liner material is not anticipated nor wanted.

When selecting the scenarios for analysis of the hydrostatic uplift or seepage potential, it must be ensured that the worst-case interactions of the excavation and of the construction grades with the phreatic and piezometric surfaces are selected. Temporal changes in phreatic and piezometric surfaces must be taken into account. The highest temporal phreatic and piezometric surfaces must be used in the analyses. Using average depth of excavation or average elevation for the phreatic and piezometric surfaces is not acceptable (see Figure 7-3). The goal of the analyses is to identify all areas within the facility where liners or other structures will be constructed that have a factor of safety less than 1.4 for hydrostatic uplift or 1.1 for seepage potential.

Figure 7-1 is an example of piping through the wall of excavation caused by high hydrostatic pressures at an Ohio landfill creating flow through more than 20 feet of heavy in situ clay materials causing flooding of the excavation.
Figure 7-2 illustrates a situation where a clay liner (or another soil layer) is constructed above a saturated layer. The piezometric head ($H_P$) is applying upward pressure on the liner.

![Diagram of uplift pressure acting below the liner system](image)

Figure 7-2 Example of uplift pressure acting below the Liner System

Factor of safety is commonly calculated as a ratio between a resisting (available or stabilizing) force and a driving (attacking or destabilizing) force. The factor of safety against hydrostatic uplift for the condition described in Figure 7-2 can be expressed as:

Equation 7-1

$$FS_{Uplift} = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{\gamma_{RSL} H_{RSL} + \gamma_{UNS} H_{UNS}}{\gamma_w H_P}$$

Where,

- $\gamma_{RSL} =$ field density of clay liner, pcf
- $\gamma_w =$ density of water, pcf
- $\gamma_{UNS} =$ field density of unsaturated foundation soil, pcf
- $H_{RSL} =$ thickness of recompacted soil liner, ft
- $H_{UNS} =$ thickness of unsaturated foundation soil, ft
- $H_P =$ piezometric level above the unsaturated foundation soil (head), ft

An unstable condition caused by hydrostatic uplift may develop when the hydrostatic uplift force acting along the line of saturation overcomes the downward stabilizing force created by the weight of the soil layer(s) above the line of saturation. If an area acted upon by the hydrostatic force is sufficiently great, excess water pressure may cause overlying soil to rise, creating a failure known as “heave.” Although heave can take place in any soil, it will most likely occur at an interface between a relatively impervious layer (such as a clay liner) and a saturated, relatively pervious base.
Figure 7-3 Example of Seepage through the Foundation

Figure 7-3 is another example of high hydrostatic pressures at an Ohio landfill causing flow through more than 20 feet of heavy in situ clay materials resulting in flooding of the excavation. The upward flow or seepage water is evidenced by a cloudy disturbance in the flooded excavation.

**Heaving Concept and Analysis**

Heaving occurs when the effective stress in the soil approaches zero. At this point there will not be any surface contact between the soil particles, leading to the formation of surface cracking or more severe soil breaking at or below the surface. Heaving typically is analyzed by comparing the seepage force exerted by the groundwater with the effective or buoyant unit weight of the overburden counteracting materials.

Heaving may also occur at the bottom of excavations due to bearing capacity failure. The factor of safety against heaving may be obtained from the modified Terzaghi’s bearing capacity theory as presented below.

**Equation 7-2**

\[ FS = \frac{11c}{\gamma H} \]

Where,

- \( c \) = soil cohesion (\( \Phi = 0 \) concept), psf
- \( H \) = depth of excavation, ft
- \( \gamma \) = bulk unit weight of soil, pcf
Note that a value less than a constant of eleven will need to be used for trenches with small widths.

**Piping or Internal Erosion Concept and Analysis**

Piping or internal erosion, on the other hand, usually occurs when the drag or seepage forces, due to the water movement, exceed the cohesive resisting forces between the soil particles. This phenomenon will ultimately lead to the formation of soil erosion channels called soil pipes. Once soil piping starts, the flow in the soil pipe will continue to increase due to the decreased resistance to the flow or friction loss, and the increase in the velocity head, ultimately resulting in a collapse or other structural damage. Soil piping has been experienced at a number of excavation sites in Ohio. At those sites, higher seepage pressures or hydraulic gradients had caused the soil near the surface to form hairline cracks and soil boils. The internal erosion process continued to worsen with time and apparently progresses in a backward fashion until a number of soil pipes were formed between the surface and the source of the groundwater pressure, undermining the integrity of the foundation soil and liner system, and providing a direct conduit between the bottom of the landfill and the aquifer system.

This chapter discusses three methods (analytical method, flow net method, and finite element analysis method) to determine the seepage potential at a site.

**Analytical Method**

In order to evaluate if internal erosion or soil piping will be experienced during excavation and if soil boils will form, the internal and exit hydraulic gradient should be compared with the critical hydraulic gradient using the following equation:

**Equation 7-3**

\[
i_{cr} = \frac{\gamma'_w}{\gamma'_w - \frac{1}{G_s}} = \frac{G_s - 1}{e + 1}
\]

Where,

- \(i_{cr}\) = critical hydraulic gradient, unitless
- \(\gamma'_w\) = submerged unit weight, pcf
- \(\gamma_w\) = unit weight of water, pcf
- \(G_s\) = soil specific gravity, unitless
- \(e\) = soil void ratio, percent
The rate of total head loss or energy dissipation through the soil matrix is defined as the hydraulic gradient, $i$.

**Equation 7-4:** for zones of saturation

$$i_{actual} = \frac{\Delta h}{\Delta L} = \frac{\text{top of potentiometric surface elev.} - \text{top of clay liner elev.}}{\text{top of clay liner elev.} - \text{top of waer baring unit elev.}}$$

**Equation 7-5:** for upper most aquifers

$$i_{actual} = \frac{\Delta h}{\Delta L} = \frac{\text{top of potentiometric surface elev.} - \text{bottom of excavation elev.}}{\text{bottom of excavation elev.} - \text{top of waer baring unit elev.}}$$

Where,

- $\Delta h$ = head loss between two points, ft
- $\Delta L$ = apparent flow distance at which the hydraulic gradient is being measured, ft

The head loss increases linearly with increasing velocity when the flow is in the laminar and transition phases. The relationship becomes nonlinear in the turbulent phase. Once the turbulent phase is reached, even if the velocity is reduced, the flow will remain turbulent in part of the transition zone until the laminar zone is reached again. This explains why once the seepage damage begins, it will not be stopped or reduced without engineering intervention.

When considering the effect of the hydraulic gradient, the effective vertical stress may be defined by:

**Equation 7-6**

$$\sigma'_v = \sigma_v - i u$$

As the hydraulic gradient increases, $\sigma_v$ approaches $iu$ and $\sigma'_v$ approaches zero. At this point, the gradient approaches the critical gradient, $i_{cr}$, which can be demonstrate to be equal to $i_{cr} = \frac{\sigma_v}{u} = \frac{\gamma'}{\gamma_w}$.

The internal and exit hydraulic gradients can be calculated using a finite element or a flow net method. The answer obtained from these methods should be compared to the critical hydraulic gradient, $i_{cr}$, calculated using the above formulas.

As long the internal and exit hydraulic gradients are less than the critical gradient, $i_{cr}$, the seepage is expected to be in accordance with the principle of Darcy’s law and the permeability of the soil should remain constant. This suggests that there will be minimal
disturbance to the soil structures and that internal erosion will not occur. However, when the hydraulic internal or exit gradient exceeds the critical gradient, \( i_{cr} \), the hydraulic conductivity of the soil is expected to increase, ultimately resulting in a loss of strength of the foundation soils, leading to the formation of soil piping tunnels and soil boils on the surface. This condition, when it occurs in cohesionless soil, is termed as a quick condition. Simple testing and visual inspection of the flow coming from the sand boils can shed a very important light on the severity of the soil piping condition. If fine materials are being carried along with the flow from the sand boils, this may be an indication that a severe piping condition is developing. In this situation, immediate action will need to be implemented to remedy the cause of the problem.

The tractive stress concept may also be used to determine the magnitude of the critical gradient. Khilar et al. reported the critical gradient necessary to cause soil piping to be:

Equation 7-7

\[
i_{cr} = \frac{\tau_{cr}}{2.878 \gamma_w \left( \frac{n_o}{K_o} \right)^{0.5}}
\]

Where,

\( \tau_{cr} \) = critical tractive stress (dynes/cm\(^2\)) = 0.001 \((S_v + \alpha_u) \tan (30 + 1.73 PI) \) (Dunn, 1959; Abt et al., 1996; Philip et al. 2006).

\( S_v \) = the saturated shear strength, N/m\(^2\), lb/ft\(^2\)

\( \alpha_u \) = unit conversion constant, 8630 N/m\(^2\), 180 lb/ft\(^2\)

\( PI \) = Plasticity Index from the Atterberg limits

\( n_o \) = initial porosity, percent

\( K_o \) = initial hydraulic conductivity (cm/sec) (One order of magnitude less than the lab permeability)

Philip’s recommends using “unconfined compressive test (ASTM D211-66-76) to determine the saturated shear strength, \( S_v \).

Flow Net Method

Seepage problems can also be estimated through the use of the continuity equation which will result in a special form of equation that is called the “Laplace Equation.” In three dimensions, the equation will take the following form:
Equation 7-8

\[ \frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} + \frac{\delta^2 h}{\delta z^2} = 0 \]

Graphical representation of this equation will yield ellipses for the flow lines and hyperbolas for the equipotential lines. The two families of curves will intersect at 90 degree angles to form a pattern of peculiar square figures. Using Darcy’s Law the following form of equation can be easily used to determine the amount of seepage per unit time per unit distance from the groundwater to an excavation or from an elevated ground reservoir or sedimentation pond through the embankment.

Equation 7-9

\[ q = K h_t \frac{N_f}{N_d} \]

Where,

- \( q \) = seepage flow rate, cfs
- \( k \) = soil hydraulic conductivity, ft²/sec
- \( h_t \) = total head loss, ft
- \( N_f \) = total number of flow channels in the flow net, unitless
- \( N_d \) = total number of equipotential drop lines in the flow net, unitless

**Finite Element Analysis Method (FEM)**

Finite element models are available to perform seepage analysis for saturated or unsaturated steady and unsteady state flow conditions. The basic steps involved in most finite element programs are:

1. Selection of the cross section that is intended to be modeled. The section geometry will need to be entered into the finite element program.

2. The cross section is discretized by dividing it into smaller sections or elements. This option is usually automatic in most finite element programs. Some finite element programs may allow you to select the type of element to be used in the finite element analysis. The materials’ hydraulic conductivity and initial fluid head or potential will need to be specified. The boundary conditions will need to be defined accurately otherwise the results may be questionable. The finite element
programs that deal with seepage analysis contain flow, potential, gradient, and velocity relationships that will allow you to solve for these unknown at each node established by the discretizing step using element stiffness matrices and equations. The programs will assemble the stiffness matrices; account for the known boundary conditions and solve for the unknowns.

It should be noted that FEM programs are used to solve the partial differential flow equation which is undefined at points of singularity. It is meaningless to compute the gradients at points of singularity. Therefore, unless the surface has sharp and abrupt changes, attempts must be modeled surfaces as they are encountered in the field (i.e., transition between the floor and side slope should be modeled as a curve instead sharp angle).

Calculation of Seepage Factor of Safety

The calculation for factor of safety concerning seepage is calculated as follows:

\[ FS = \frac{i_{\text{critical}}}{i_{\text{actual}}} \]

Soil Gradation and Piping Damage

Formation of soil piping is very favorable in a permeable sandy/gravelly formation. Soil formation is considered to be prone to piping damage when the following conditions are met:

1. There is more than 10% by weight of particles finer than 0.25 mm; and
2. There is a lack of particles with a grain size in the range of 0.5 to 2 mm; and
3. The coefficient of uniformity is greater than 20; and
4. The coefficient of curvature is greater than 3.
When two soils with similar grain size distribution curves are being compared, the soil with the higher friction angle will tend to exhibit a higher critical gradient and therefore, will be less prone to piping damage.

It is recommended that a seepage analysis be performed if the potentiometric head on the bottom of excavation exceeds 5% of the thickness/distance of the soil between the source of groundwater pressure and the bottom of excavation. Soil piping will very likely occur if the head on the bottom of excavation exceed 20% of the soil thickness/distance between the source of groundwater pressure and the bottom of excavation.

The potentiometric or phreatic heads in the saturated soil foundation layer can be measured with the aid of piezometers, water levels in borings, or other techniques, and compared to the thickness/distance of the soil between the source of groundwater pressure and the bottom of excavation to evaluate if the 20% criterion has been exceeded. If this screening criterion is not satisfied (i.e., exceeding the 20% criterion), a more detailed and accurate calculation using facility’s own specific values must be included with any request to construct a liner system or install an engineered structure.

Special attention must be paid to the quality and level of compaction of the backfill material that will be placed around seep collars or underground conduits. If the backfill material placed is poorly compacted or was left exposed to develop cracks, a seepage pathway potential will be very likely to occur. Again, once this phenomenon occurs, seepage velocity will increase leading to even eroding more impervious and well compacted material such as recompacted soil liner system.

**Determination of Total Head and the Concept of Seepage Forces**

Flow of water through soils is governed by the total head. Bernoulli’s Equation defines the total head for steady state of non-viscous incompressible fluids as follow:

Total Head = Pressure Head (piezometric) + Elevation Head (Potential) + Velocity Head

Or in a mathematical form:

\[
\text{Total Head (feet)} = \frac{P}{\gamma_w} + Z + \frac{V^2}{2g}
\]

The elevation head or potential head is the distance between an arbitrarily selected datum and the point in question. Elevation head can be negative if the point in question falls below the datum line. Bernoulli’s equation can be used to determine the gradient and the pore water pressure at a point.

Water seeping through the soil imparts energy to soil grains in the form of friction. The seepage force can be represented by:
Equation 7-10

\[ F = 62.4 i V = 62.4 A \Delta h \]

Where,

- \( F \) = seepage force
- \( i \) = average hydraulic gradient
- \( V \) = volume of which the hydraulic gradient is acting upon
- \( \Delta h \) = head loss between two points
- \( A \) = Area

For a given area and energy loss, the seepage force is constant regardless of the distance over which it travels. To illustrate this concept, the following review is provided.

\[ F_1 = 62.4 i_1 V_1 \]
\[ i_1 = \frac{\Delta h}{H_1} \]
\[ V_1 = A_1 H_1 \]
\[ \vdots \]
\[ F_1 = 62.4 \frac{\Delta h}{H_1} A_1 H_1 = 62.4 \Delta h A_1 \]

Assume \( A_1 = A_2 = A \)

\[ F_1 = 62.4 \Delta h A \]
Similarly, 

\[ F_2 = 62.4 i_2 V_2 \]

\[ i_2 = \frac{\Delta h}{H_2} \]

\[ V_2 = A_2 H_2 \]

\[ .: F_2 = 62.4 \frac{\Delta h}{H_2} A_2 H_2 = 62.4 \Delta h A_2 \]

Assume \( A_1 = A_2 = A \)

\[ F_2 = 62.4 \Delta h A \]

This will conclude that \( \overline{F_1} = \overline{F_2} \)

This demonstrates that the seepage force will not be reduced by the thickness of the layer it travels through.

**Minimum Factors of Safety**

The following factors of safety should be used, unless superseded by rule, when demonstrating that a facility will resist the hydrostatic uplift and seepage potential.

Hydrostatic Uplift Analysis: \( FS > 1.4 \)

Seepage Analysis: \( FS > 1.1 \)

The use of a higher factor of safety against hydrostatic uplift or seepage potential may be warranted whenever:

1. A failure would have a catastrophic effect upon human health or the environment, uncertainty exists regarding the accuracy, consistency, or validity of data, and no opportunity exists to conduct additional testing to improve or verify the quality of the data. Designers may want to consider increasing the required factor of safety if repairing a facility after a failure would create a hardship for the responsible parties or the waste disposal customers.

2. Large uncertainty exists about the effects that changes to the site conditions over time may have on the phreatic or piezometric surfaces, and no engineered controls can be implemented that will significantly reduce the uncertainty.

3. The soil is classified to have moderately rapid to extremely rapid erosion rate.
A facility must be designed to prevent failures due to hydrostatic uplift. A factor of safety against hydrostatic uplift and seepage potential lower than 1.4 and 1.1, respectively; is not considered a sound engineering practice in most circumstances. This is due to the uncertainties in calculating a factor of safety against hydrostatic uplift and seepage potential, and any failure of the waste containment facility due to hydrostatic uplift or seepage potential is likely to increase the potential for harm to human health and the environment. If a facility has a factor of safety against hydrostatic uplift less than 1.4, it may be necessary to lower the groundwater table to an acceptable level until enough stabilizing material is placed above the liner system to result in a factor of safety greater than 1.4. However, if it is determined that the factor of safety against seepage potential is less than 1.1, mitigation to reduce the uplift or seepage pressures, redesigning the facility to achieve the required factor of safety, or using another site not at risk of a failure due to seepage potential will be necessary.

The factors of safety specified in this policy are based on the assumptions contained in this policy. Those assumptions include, but are not limited to, the use of conservative, site-specific, higher quality data; proper selection of worst-case geometry; and the use of calculation methods that are demonstrated to be valid and appropriate for the facility. If different assumptions are used, these factors of safety may not be appropriately protective of human health and the environment. For instance, using the average depth of excavation (double-dot dashed line in Figure 7-4) and the average elevation of the piezometric surface (large dashed line) result in the conclusion that hydrostatic uplift will not occur, which is not appropriate. Note that the temporal high piezometric surface (small dashed line) does intersect the liner system (hashed area) creating the potential for hydrostatic uplift that must be analyzed. The factors of safety specified in this policy are based on the assumption that the soil will poses a moderately slow to extremely slow erosion rate \( I_{\text{erosion}} \) greater than 5.0.

The erosion characteristic of a soil can be described by a parameter called the erosion rate index. The erosion index rate measures the rate of erosion and the critical shear stress of the soil when erosion is expected to begin. The rate of erosion appears to be dependent on the soil gradation, amounts of fines and clay, Atterberg limits, in situ water and density, construction specifications, and to a certain degrees on the soil mineralogy and its cementations property.
The erosion rate index for plastic soils may be estimated using the following equation developed by Wan and Fell, 2004:

Equation 7-12

\[
I_{erosion} = 0.153\gamma_d - 0.042RD + 0.1\omega + 0.097\Delta\omega_r - 0.056F - 0.09LL + 0.11I_p + 0.44P - 10
\]

Where,

\( I_{erosion} \) = erosion rate index
\( \gamma_d \) = soil dry density, lb/ft\(^3\)
\( RD \) = relative density of the soil, % (use 95 for undisturbed foundation soil)
\( \omega \) = water content, %
\( \Delta\omega_r \) = water content ratio, % = \( \frac{\omega - OMC}{OMC} \times 100 \)
\( OMC \) = optimum water content, %
\( F \) = percent of fines in the soil, %
\( Clay^* \) = percent of clay (0.002mm) in the soil, %
\( LL \) = liquid limit, %
\( I_p \) = plasticity index, %
\( P \) = pinhole test classification value

*Wan and Fell define it to be percent passing 0.005mm
The following table provides guidance for selecting the pinhole test classification value.

<table>
<thead>
<tr>
<th>Dispersiveness Category</th>
<th>Pinhole Test Classification Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>1</td>
<td>Dispersive clays that fail rapidly under 2-in head</td>
</tr>
<tr>
<td>D2</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>ND4</td>
<td>3</td>
<td>Slightly to moderately dispersive clays that erode slowly under 2-in or 7-in head</td>
</tr>
<tr>
<td>ND3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>ND2</td>
<td>5</td>
<td>Nondispersive clay with very slight to no colloidal erosion under 15-in or 40 in head</td>
</tr>
<tr>
<td>ND1</td>
<td>6</td>
<td></td>
</tr>
</tbody>
</table>

The dispersiveness category is based on ASTM D 4647, Methods A&C.

Qualitative terms for representing the erosion rate index were suggested as follows:

<table>
<thead>
<tr>
<th>Erosion Rate Index (I_{\text{erosion}})</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;2</td>
<td>Extremely rapid</td>
</tr>
<tr>
<td>2 – 3</td>
<td>Very rapid</td>
</tr>
<tr>
<td>3 – 4</td>
<td>Moderately rapid</td>
</tr>
<tr>
<td>4 – 5</td>
<td>Moderately slow</td>
</tr>
<tr>
<td>5 – 6</td>
<td>Very slow</td>
</tr>
<tr>
<td>&gt;6</td>
<td>Extremely slow</td>
</tr>
</tbody>
</table>

The responsible party should ensure that the design and specifications in all authorizing documents and the QA/QC plan clearly require that the assumptions and specifications used in the hydrostatic uplift and seepage analyses for the facility will be followed and confirmed during and before construction, operations, and closure.

From time to time, changes to the facility design may be needed that will alter the assumptions and specifications used in the hydrostatic uplift or seepage analyses. If this
occurs, a request to change the facility design is required to be submitted for Ohio EPA approval in accordance with applicable rules. The request to change the facility design must include a new hydrostatic uplift and seepage analyses that uses assumptions and specifications appropriate for the change request.

**REPORTING**

This section describes the information that should be submitted to demonstrate that a facility is not susceptible to hydrostatic uplift and seepage damage. Ohio EPA recommends that the following information be included in its own separate section of a geotechnical and stability analyses report that will be submitted for Ohio EPA review:

1. A narrative and tabular summary of the results of the hydrostatic uplift and seepage analyses,

2. A summary and discussion of the results of the subsurface investigation that apply to the hydrostatic uplift and seepage analysis and how they were used in the analyses,

3. A summary of the worst-case scenarios used to analyze the hydrostatic uplift and seepage potential at the facility,

4. Isopach maps comparing the excavation and construction grades, depicting the temporal high phreatic and piezometric surfaces and showing the limits of the waste containment unit(s),

5. Drawings showing the cross sections analyzed. The cross sections should include:
   a. The engineered components and excavation limits of the facility,
   b. The soil stratigraphy and their properties such as thickness, porosity, hydraulic conductivity and degree of saturation, and
   c. The locations of the temporal high phreatic and piezometric surfaces.

6. The detailed hydrostatic uplift and seepage calculations, and

7. Any figures, drawings, or references relied upon during the analysis marked to show how they relate to the facility.

8. All electronic files and relevant information.
Hydrostatic pressure can cause in situ materials to fracture and allow the passage of the underlying ground water into an excavation, causing flooding of the excavation and weakening the in situ materials. The two delta formations in the above picture are obvious evidence of flow through the in situ materials, which at this Ohio landfill are over 20 feet thick.

Hydrostatic pressures are causing ground water to pipe into an excavation of an Ohio landfill. This may have been caused by fracturing of the in situ materials, piping, or from an improperly abandoned boring.

Any drawings or cross sections referred to in this policy that are already present in another part of the geotechnical and stability analyses report can be referenced rather than duplicated in each section. It is helpful if the responsible party ensures the referenced items are easy to locate and marked to show the appropriate information.
Seepage Analysis using FEM and Traditional Methods - Example Calculation

To assess the seepage potential, the gradient due to upward seepage force at the base of excavation will need to be evaluated at least on a 100 ft by 100 ft grid and at the toe of the side slopes. For illustrative purpose, the seepage potential is evaluated at point “A” where the following information is available.

The specific gravity of the foundation soil = 2.75, void ration = 0.5, saturated unit weight of the soil = 139.2 pcf, and the geometry and hydrostatic conditions are shown in the figure below.

The results of the vertical hydraulic gradients can be determined using a finite element program. The following is the results obtained using “Slide” program.
Enlarging the view near the area of concern at point “A” for this example, the following is obtained.

Therefore, the expected exit vertical gradient at point “A” is 1.04.

Performing the same problem using the Equation 7-4, one can obtain the actual gradient at point “A”:

\[
\frac{\Delta h}{\Delta L} = \frac{\text{potentiometric surface elev} - \text{water table elev}}{\text{subgrade surface elev} - \text{top of UAS elev}} = \frac{798 - 782.5}{782.5 - 766.5} = 0.97
\]

This compares well with the results obtained from the finite element program of 1.04.

The critical gradient is the smaller of:

\[
\begin{align*}
i_{cr} &= \frac{\gamma'}{\gamma_w} = \frac{\gamma_{sat} - \gamma_w}{\gamma_w} = \frac{135.2 - 62.4}{62.4} = 1.16 \quad \leftarrow \text{controls} \\
&= \frac{G_s - 1}{e + 1} = \frac{2.75 - 1}{0.5 + 1} = 1.17
\end{align*}
\]
Hydrostatic Uplift of a Pump Station - Example Calculation

Find the factor of safety for the pumping station against uplifting.

Assume the pumping station walls are coated with an impermeable epoxy coating layer.

\[
FS_{\text{Uplift}} = \frac{\text{Pullout Resistance}}{\text{Uplift Pressure}}
\]

\[
FS_{\text{Uplift}} = \frac{\text{Wall Friction + Weight of Soil above Extended Base + Deadmen Resistance + Structure Weight}}{\text{Uplift Pressure}}
\]
In this case, we do not have an extended base or deadmen, therefore,

\[
FS_{Uplift} = \frac{Wall \ Friction + Structure \ Weight}{Uplift \ Pressure}
\]

Uplift Pressure\(=\gamma_w A_o H_w\)

\[
Uplift \ Pressure = \gamma_w A_o H_w
\]

\[
Uplift \ Pressure = (62.4 \ pcf) \left( \frac{\pi (6 \ ft)^2}{4} \right) (9 \ ft) = 15,871 \text{ lbs}
\]

Wall Friction \(= K_o \gamma_b \frac{H}{2} A_o \tan\phi + A_o C_s\)

Weight of Soil above Extended Base \(= (A_o - A_i) H \gamma_b\)

Where,

\(K_o = \text{at rest pressure coefficient}\)
\(\gamma_b = \text{bouyant soil weight}\)
\(A_o = \text{outside area of manhole shaft}\)
\(A_i = \text{inside area of manhole shaft}\)
\(C_s = \text{saturated cohesion}\)

Therefore, the wall friction along the different layers ➔

Wall friction = friction along gravel + friction along sand + friction along silty clay + friction along clay

Friction along gravel \(= K_o \gamma_b \frac{H}{2} A_o \tan\phi + A_o C_s\)

\[
= (1 - \sin\phi') (\gamma_{sat} - \gamma_w) \frac{H}{2} (2 \pi r H) \tan\phi + A_o C_s
\]

\[
= (1 - \sin 37) (128 - 62.4) \frac{3}{2} (2 \pi (3))(3) \tan 37 + A_o (0)
\]

\(= 1,670 \text{ lbs}\)
Friction along sand = \( (1 - \sin 35) \left[ 126 - 62.4 \right] \frac{3}{2} \left( 2 \pi \right) \left( \frac{3}{3} \right) \tan 35 + A_o(0) = 1,611 \text{ lbs} \)

Friction along silty clay = \( (1 - \sin 32) \left[ 127 - 62.4 \right] \frac{3}{2} \left( 2 \pi \right) \left( \frac{3}{3} \right) \tan 32 = 1,610 \text{ lbs} \)

Friction along clay = \( (1 - \sin 28) \left[ 130 - 62.4 \right] \frac{3}{2} \left( 2 \pi \right) \left( \frac{3}{3} \right) \tan 28 + \left( 1 - \sin 28 \right) \left( 108 \frac{2}{2} \right) \left( 2 \pi \right) \left( \frac{3}{3} \right) \tan 28 \)
\[ = 2,766 \text{ lbs} \]

Weight of the Structure = \( (A_o - A_i) H \theta_{\text{concrete}} \)
\[ = \left( \frac{\pi D_o^2}{4} - \frac{\pi D_i^2}{4} \right) H \theta_{\text{concrete}} \]
\[ = \frac{\pi}{4} \left( D_o^2 - D_i^2 \right) H \theta_{\text{concrete}} \]
\[ = \frac{\pi}{4} \left( 6^2 \text{ ft}^2 - 5^2 \text{ ft}^2 \right) \left( 19 \text{ ft} \right) \left( 150 \text{ pcf} \right) \]
\[ = 24,622 \text{ lbs} \]

\[ \therefore \quad FS = \frac{1,670 + 1,611 + 1,610 + 2,766 + 24,622}{15,871} = 2.0 \]

Factor of safety not considering wall friction is:

\[ FS = \frac{24,622}{15,871} = 1.6 \]
Hydrostatic Uplift at the Sump - Example Calculation

Given a sump with dimensions of 24 ft x 24 ft x 2 ft deep.

Floor and flow line slopes = 3%.

Piezometric head level is determined to be 1 ft above the sump rim.

Foundation soil has $\Phi = 22^\circ$ and $c = 100$ psf.

Since we know the piezometric head is 1 ft above the sump rim, this will mean it will extend a distance equal to 33 ft from the rim if the floor slope is 3%.

The volume of the area delineated by the piezometric line intersecting the sump area is the volume of the sump + volume of 3 full spectrum + the volume of a small spectrum between the sump and the 3:1 side slope.

Assume the volume of the small spectrum is equal to zero.

Volume of the sump = 24 ft x 24 ft x 2 ft = 1,152 ft$^3$
Volume of the spectrum

\[
\frac{(24\text{ft} \times 1\text{ft}) + (90\text{ft} \times 1\text{ft})(33\text{ft})}{2}
\]

\[
= 5,643 \text{ft}^3
\]

Total volume of the area subjected to uplifting = 5,643 + 1,152 = 6,795 ft\(^3\)

This will result in an uplift force = 6,795 ft\(^3\) \times 62.4 \text{pcf} = 424,008 \text{ lbs}

Volume of dry soil resisting uplift = \[90\text{ft} \times 90\text{ft} \times 4\text{ft}\] − 6,795 ft\(^3\)

\[
= 25,605 \text{ft}^3
\]

Assume \(\gamma_{\text{soil}} = 120 \text{ pcf}\)

Weight of soil in this wedge = 120 \text{ pcf} \times 25,605 \text{ ft}^3 = 3,072,600 \text{ lbs}

\[
\therefore \text{F.S} = \frac{3,072,600}{424,008} = 7.2
\]

Note that this is without considering the soil shear resistance to uplift.

However, if we redo the same analysis for the situation shown in the next figure where the phreatic head is only 1 ft below the bottom of the sump and using the same analysis as before.

Now the weight of soil resisting uplift is equal to 69,120 \text{ lbs}
resulting is a factor of safety of approximately 0.96.

Now considering the foundation soil shear resistance properties:

\[
\tau = c + \sigma \tan \phi
\]

\[
\sigma = 120 \text{pcf} \times 1\text{ft} = 120 \text{psf}
\]

\[
\tau = 100 + 120 \tan 22^\circ = 148 \text{psf}
\]
The area resisting uplift below the sump = 24 ft x 1 ft x 4 sides = 96 ft$^2$

The corresponding uplift pressure at the bottom of the sump = (24 ft x 24 ft x 2 ft)(62.4 pcf) = 71,885 lbs.

The resisting force contributed by the soil shear strength = 96 ft$^2$ x 148 psf = 14,208 lbs

Therefore,

$$FS = \frac{69,120 + 14,208}{71,885} = 1.16$$

Ohio EPA does not recommend relying on the soil cohesion or friction properties to calculate the resisting force for the uplift pressure due to the uncertainty associated with the soil fracturing.
Estimation of the Soil Erosion Rate – Example Calculation

An embankment was constructed to the following specifications:

Average field density = $\gamma_d = 120$ pcf

Average moisture content = $MC = 12\%$

Find the predicted erosion rate index for an embankment that was constructed from soils that have the following properties:

Maximum dry density = $MDD = 130$ pcf

Optimum moisture content = $OMC = 10\%$

% passing the No. 200 sieve = $F = 65\%$

% passing the No. 200 sieve = $Clay = 35\%$

Liquid limit = $LL = 28\%$

Plastic limit = $PL = 15\%$

Pinhole test classification = $P = 3$

$$I_{erosion} = 0.153\gamma_d - 0.042RD + 0.1\omega + 0.0097\Delta\omega_r - 0.0056F + 0.042Clay - 0.09LL + 0.11I_p + 0.44P - 10.2$$

Where,

$$RD = \frac{\gamma_d}{MDD} \times 100 = \frac{120}{130} \times 100 = 92.3\%$$

$$\Delta\omega_r = \frac{\omega - OMC}{OMC} \times 100 = \frac{12 - 10}{10} \times 100 = 20\%$$

$$I_p = LL - PL = 28 - 15 = 13\%$$

Substituting,

$$I_{erosion} = 0.153(120) - 0.042(92.3) + 0.1(12) + 0.0097(20) - 0.0056(65) + 0.042(35) - 0.09(28) + 0.11(13) + 0.44(3) - 10.2$$

$$I_{erosion} = 7.0$$

This is greater than 5 $\Rightarrow$ the soil is not prone to erosion, and therefore the normal factors of safety applied for seepage will likely be appropriate.
Use of Flow net - Example Calculation

Note: This example is included as an illustration on how to use the flow net method and not as an acceptable method to deal with seepage problems from groundwater to the bottom of waste containment cells.

Determine the spacing required in the till layer for the underdrain collection pipe if it is used to intercept groundwater flowing to the surface. Also, determine the pore water pressure, gradient and velocity at point “A”. The permeability of the till layer is $1 \times 10^{-5}$ cm/sec.

The discharge from the bedrock aquifer to the perforated collection pipe can be found using the following formula:

$$q = K h_L \frac{N_f}{N_d}$$

Where,

- $h_L = 3.2$ m
- $N_f = 5$ (F1 through F5)
- $N_d = 7$ (E1 to E7)
Therefore,

\[ q = (1.0 \times 10^{-7} \text{ m/sec}) (3.2m) \left( \frac{5}{7} \right) = 2.29 \times 10^{-7} \text{ m}^2/\text{sec} = 5.2 \text{ gal/day per meter of pipe length} \]

The total discharge to each pipe will be twice the amount calculated above due to symmetry, and hence the total discharge is approximately 10.4 gal/day per meter length.

The approximate length of the aquifer that will discharge to the perforated pipe will be 2X ("X" can be scaled off the scale drawing). In this case "X" was determined to be 13m. Therefore the perforated pipe should be spaced at 26m on centers.

The pore water pressure at point "A" is:

Pressure Head at A = (Total Head - Elevation Head) \_A

\[ = (19m - N_d \times \Delta h) \_A - 6.5m \]

Where

\[ \Delta h = \frac{\text{Total Head Loss}}{N_d} = \frac{3.2}{7} = 0.46 \]

\[ \therefore \]

Pressure Head at A = (19m - 2.5 \times 0.46) - 6.5m = 11.4m

\[ \therefore \]

Pore Water Pressure at A = (11.4m)(9.81m/sec^2) = 112KPa = 16.2psi

Hydraulic gradient and velocity at point "A" are:

\[ i_A = \frac{\Delta h}{\text{Distance of flow travel}} = \frac{0.46}{3} = 0.153 \]

\[ V_A = K \times i_A = (10^{-5} \text{ cm/sec})(0.153) = 1.5 \times 10^{-6} \text{ cm/sec} \]
REFERENCES


Chi Fai et. al., 2004 Investigation of Rate and Erosion of Soils in Embankment Dams, Journal of Geoenvironmental Engineering.


