GEOTEXTILE WRAP-FACE WALL USING MARGINAL BACKFILL

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ABSTRACT

A concrete retaining wall was constructed during October/November 2005. The height of the wall was 9 feet with a stem width of 0.83 ft, while the width of the base was 1.83 ft. The backfill was a low plasticity clay (CL). As a result of this design, the wall was not able to withstand the lateral pressures from the backfill and noticeable cracks in the wall developed within one month after backfilling. The backfill soil was then removed to relieve pressures on the wall until a remediation scheme could be developed and implemented. A geotextile wrap-face wall was chosen to reinforce the soil mass behind the existing concrete wall, which now acts as a facade. The geotextile wrap-face wall was designed with a high strength woven geotextile with seven layers of reinforcement. The in-situ soil (CL), a marginally suitable material, was used for the backfill. Extensive drainage was incorporated in the design and construction of the geotextile wrap-face wall to decrease backfill pore pressures. A gap between the face of the geotextile wall and the back of the concrete wall allows for deformation of the wrap-face wall without contacting the concrete wall. Index and compaction tests were performed on the backfill soil and interface shear tests were conducted with the geotextile and soil to provide design parameters. The geotextile wrap-face wall was constructed in July 2006, and the performance was monitored. During the monitoring period, four months post construction, no significant lateral, nor vertical movements have occurred, and the drainage system experienced significant flows.
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Chapter 1: Introduction

1.1 Overview/Background

Throughout the world there is an ever increasing lack of available prime development space due to the increasing population and associated land development. As a result, structures enabling maximum land utilization, such as retaining walls and mechanically stabilized earth (MSE), have proven very popular.

Retaining walls are built to accomplish exactly what their name implies, retain, or reinforce, a mass of soil, rock, or other material. Many different types of retaining walls exist, and include gravity and cantilever concrete retaining walls, and mechanically stabilized earth (MSE) retaining structures constructed using geosynthetics (Figure 1.1).

![Various types of retaining walls available](image)

The earliest retaining structures were massive gravity walls (Figure 1.1a). They were often made of mortared stones, masonry, or unreinforced concrete.
Lateral loads due to the adjacent backfill soil are resisted by virtue of the walls’ large mass, or externally stabilized (Coduto, 2001). Cantilever retaining structures (Figure 1.1b) are a refinement of the massive gravity wall concept. These walls have a much thinner stem, and utilize the weight of the backfill soil to provide most of the resistance to sliding and overturning (Coduto, 2001).

Internally stabilized systems reinforce the soil to provide the necessary stability. Soil is strong in compression, but has virtually no tensile strength. The inclusion of tensile reinforcing members in soil can significantly increase the strength and load-bearing capacity. One variation of internally stabilized reinforcement systems is mechanically stabilized earth (MSE). Mechanically stabilized earth reinforced walls can have many different types of facings, but all utilize tension members, i.e., geotextile, geogrid, and metal reinforcement, to retain the backfill soil. Segmental retaining walls (SRW) utilize facing units, or block segments (Figure 1.1c), that are stacked vertically. Each block has an attached tension member extending back into the backfill soil.

Another variation of an MSE wall is a geosynthetics wrap-face retaining wall (Figure 1.1d), and is the main topic of this thesis. Wrap-face retaining walls also have tension members, via geotextile, extending into the backfill soil. Instead of facing units, the geotextile reinforcement is wrapped around the face of the wall and embedded into the backfill soil for each successive layer of tensile reinforcement.

In critical segmental retaining wall applications high quality backfill is essential, but in many non-critical applications “marginal” quality backfill can be substituted for high quality backfill, resulting in a significant cost reduction (Figure 1.2). Typical geotextile reinforced walls can be constructed at 25 to 50 percent of cantilever concrete
retaining wall construction costs (Koerner, 1998). As a result, MSE retaining walls have become common on jobsites throughout the world. This retaining wall expense can be further reduced if native soils can be substituted for high quality granular backfill.

According to MSE wall design manuals by FHWA (1997) and AASHTO (2000), MSE wall backfill soil must be of high quality, i.e., contain less than 15 percent fines (particles passing #200 (0.075 mm) sieve). The small amount of fines allowed ensures proper drainage throughout the wall, and mitigates the buildup of pore water pressures in the backfill soil thus dramatically reducing the stress on the wall face. In many cases, such high quality backfill soil is difficult and expensive to obtain.

![Construction cost comparison of different types of retaining walls (Koerner, 1998)](image)

**Figure 1.2 - Construction cost comparison of different types of retaining walls (Koerner, 1998)**

High quality or permeable backfill can cost two to three times that of low quality, high fines backfill (Christopher and Stulgis, 2006). Examples of high quality and marginal backfill are shown in Figure 1.3.
As a result, the use of MSE wall backfill soil located on-site can have many positive attributes, such as close proximity to the construction site and minimal cost. Although the use of in-situ backfill soils is convenient, many times they are of marginally suitable quality, which can be defined as having greater than 35 percent fines, or a plasticity index greater than 20 (Sandri, 2006). This being so, it is postulated that MSE walls can be constructed using marginally suitable backfill soil and still perform properly if adequate drainage is incorporated in the design and construction of the structure.

1.2 Study Project

A cantilever reinforced concrete retaining wall was constructed during October 2005. The wall was nine feet (2.74 m) tall with a stem width of 10 inches (0.25 m) and a base width of approximately two feet (0.6) m. The backfill was a low plasticity clay (CL). As a result of this design, the wall was not able to withstand the pressures from the backfill and noticeable cracks developed in the wall. The backfill was removed within one month after construction to relieve pressures on the wall until a remediation scheme could be developed and implemented.
Remediation of the site consisted of the construction of a geotextile wrap-face wall directly behind the existing concrete wall to carry the applied soil pressures. On-site marginal soil was chosen as the backfill soil. A geotextile wrap-face wall using marginal backfill was then designed, constructed, and monitored to assess the performance of the structure.

1.3 Objective

The objective of this project was to design, construct, and assess the performance of a geotextile wrap-face wall using marginally suitable backfill soil. Performance assessment will help determine if marginally suitable backfill soil should be incorporated in future MSE wall design.

1.4 Scope

The complete analysis, design, construction, and performance monitoring of the geotextile wrap-face wall requires the completion of many tasks. These tasks include:

- Analyze literature on marginal backfill
- Analyze stability of existing concrete wall
- Measure soil/geotextile properties
- Design geotextile wrap-face wall
- Construct wall in field
- Monitor performance
- Assess performance
1.5 Thesis Layout

The layout of the thesis is as follows: A literature review of previous research on MSE retaining walls using marginally suitable backfill soil is presented in Chapter two. The existing concrete retaining wall is described in detail in Chapter three. The geotextile wrap-faced wall design is included in Chapter four. Construction issues accompanying the wrap-face wall are presented and discussed in Chapter five. A field construction photo documentary is presented along with as-built construction details. The field monitoring system is described and the performance data obtained using this system are presented. Conclusions and recommendations based on the findings from the project are presented in Chapter seven. Detailed calculations and results from the lab testing are provided in the appendices.
Chapter 2: Literature Review

2.1 Introduction

A literature review was conducted on reinforced earth walls constructed using marginally suitable backfill soils. A wide variety of literature was found on this topic and ranged from parametric analyses comparing different backfill soils to drainage design and materials used in reinforced earth walls. All of the papers presented below provide useful background information for the topic of this thesis.

2.2 Christopher and Stulgis (2006)

The state-of-practice in North America for the use of low permeability backfill soils in geosynthetic reinforced walls is emphasized in this paper. A survey was taken of the state departments of transportation and in general they defined a marginally suitable backfill soil as having greater than 15 percent fines or a plasticity index greater than six. A definition for marginally suitable backfill soil was containing fines up to 35 percent, which resulted in poor MSE wall performance. The backfill soil properties that impact design and performance of reinforced soil walls were permeability, strength, deformation, moisture-density relationship, tension cracks, and environmental effects.

Permeability of the backfill soil plays a major role in the development of excess pore water pressures in backfill. A low permeability backfill soil cause positive pore water pressures that produce a horizontal seepage force on the reinforced fill that decrease the stability of the retaining wall. Positive excess pore water pressures also reduce the shear resistance in the backfill soil.
The shear strength of the backfill soil can also impact the design and performance of an MSE wall. Research has shown that for every 1 degree reduction in frictional strength of the reinforced fill, there is a five percent increase in the reinforcement tensile stress. As a result, soils containing high fines will require a greater amount of reinforcement as compared to backfill soils containing low fines.

Deformation before and after construction of an MSE retaining wall will be greater for backfill soils containing high fines content as compared to backfill soils containing low fines content. Retaining wall deformation affects wall alignment, deformation of supported structures, downdrag on facing, and the develop of tension cracks.

The moisture-density relationship, or degree of compaction, of the backfill soil can have a major impact on MSE wall performance. Consolidation will occur in clay backfill soil if it is compacted on the wet side of the optimum moisture content.

Tension cracks developing at the back of the reinforced soil zone also affect performance of the MSE wall. The crack creates a zone for which water can infiltrate the reinforced soil mass and increase the positive excess pore water pressures.

Throughout the life of the MSE wall, environmental effects play a major role in the performance of the wall. Wetting and drying of the backfill produce micro-cracks and shrink-swell behavior in the reinforced soil backfill. These cracks generate increases in the positive pore water pressure, thus produce an additional lateral load on the reinforced soil mass. Wetting of high fines backfill soil that are compacted dry of optimum cause swelling and strength reduction that increase the potential of deformation or movements in the retaining wall. The sodium and chloride content of the soil must
also be evaluated so that degradation of the soil does not occur over time, and decrease
the strength of the backfill soil.
When these properties are properly evaluated, by laboratory testing and field quality
control, and accounted for, replacing standard AASHTO backfill materials with
marginally suitable soils can decrease construction costs of a reinforced soil wall by 20-
30 percent.

2.3 Sandri (2006)

Marginal backfill soils, defined by Sandri (2006), contain greater than 35 percent
fines or have a plasticity index greater than 20. Conditions of concern resulting from this
type of material are the generation of positive pore pressure within the reinforced fill,
wetting front advancing into the reinforced fill, and a seepage configuration established
within the reinforced fill. These conditions of concern result in poor drainage, which led
to the following drainage recommendations to be implemented when designing an MSE
wall to be constructed using marginally suitable backfill soil.

- Drainage should be incorporated in all directions in the reinforced soil.
- A drainage blanket on the back of the reinforced fill will reduce advancement of
  the wetting front.
- Systems to divert water from infiltrating the reinforced soil is better than internal
  drainage when fine grained soils are incorporated in the design of the MSE wall.
- When compacting reinforced soil, the water content should be in the range of plus
  or minus one percent of the optimum moisture content, use a kneading type
  compaction equipment, and grade the site so water flows away from the
  reinforced soil.
2.4 Lawson (2006)

The strength of a MSE wall comes from the fill and the reinforcement. The fill provides 80-85 percent of the wall’s strength, while the reinforcement provides 15-20 percent of the strength. As a result, the type of fill chosen for construction has a great effect on the overall strength of the MSE wall. According to Lawson (2006) fine-grained soils can be classified into three groups, fine frictional fills, fine cohesive fills, and cohesive frictional fills. A fine cohesive fill is usually an in-situ soil with low to medium plasticity and has an effective stress angle of internal friction that ranges from 27 – 37 degrees when well compacted. Lawson (2006) recommends:

- An MSE wall constructed of fine cohesive fill requires good drainage.
- Aspects pertaining to the performance of the fill material include the soil internal environment and the interaction between the soil and reinforcement.
- For fine-grained fills, the peak shear resistance occurs at 10-17 percent strain.
- The particular fill chosen should be at or below the A-Line on Casagrande’s soil classification chart (ASTM D2487).
- Drainage should take place before water infiltrates the reinforced fill, not through it.

2.5 Hatami and Bathurst (2006)

A parametric analysis of reinforced soil walls with different backfill material properties was performed by Hatami and Bathurst (2006). Four full-scale (3.6 m tall) reinforced soil walls were constructed with various mesh/grids. Using the data from these walls, a numerical model was developed that can model different backfill designs. Using this model the influence of backfill strength properties was analyzed.
It was found that that a cohesive strength component as low as 10 kilopascals can significantly decrease lateral displacement. As a result, fine-grained soils can be used with efficiency if placed using adequate drainage and installing performance instrumentation.

2.6 Koerner et al. (2006)

Segmental retaining walls are often constructed with low permeability backfill that can cause failure if not properly drained (Koerner et al., 2006). When the backfill does not have adequate drainage, seepage pressures develop and exhibit a lateral force on the wall that must be accounted for in the design of the wall. To avoid the buildup of seepage pressures the physical mass and internal strength of the reinforced zone should be increased to resist additional seepage forces. Another way to avoid failure of the wall due to seepage forces is to provide proper drainage of the seepage arriving at the reinforced soil zone so that the excessive increased stresses are avoided.


Failures or deformation occurring in MSE walls using high fines or high plasticity material often result from the lack of drainage that causes the buildup of positive pore water pressures (Stulgis, 2006). When MSE walls are constructed properly with a controlled high fines material a savings of 20-30 percent can result. Four full scale field MSE test walls were constructed of different backfill materials to demonstrate that MSE walls can perform properly even if a high fines material are used. A silty soil backfill (50 percent fines), performed well, although aggressive drainage must be installed to decrease the loads on the wall due to pore water pressure buildup. Monitoring of the walls will continue until the spring of 2007.
2.8 Summary of Literature Review

In critical segmental retaining wall applications high quality backfill is essential, but in many non-critical applications “marginal” quality backfill can be substituted for high quality backfill, resulting in a significant cost reduction. Marginal quality backfill can be defined as having greater than 35 percent fines, or having a plasticity index greater than 20. In many situations, native on-site soils can be used as the backfill material, which not only decreases construction costs, but speeds up construction time as well. When backfill of marginal quality is used in MSE wall construction, an aggressive drainage system must be designed and constructed to allow for the dissipation of pore water pressures in the reinforced soil mass. To achieve proper drainage, the reinforced soil mass should have drainage features on all faces. A result of such aggressive drainage is that water is diverted away from the reinforced soil mass, not through it. When a good drainage system is installed in an MSE wall, as well as proper construction techniques such as adequate soil compaction and a proper soil moisture-density relationship, or the soil water content field compacted at plus or minus one percent of the optimum moisture content, a high performance can be achieved. Numerous lab and field tests have been conducted to analyze the performance of segmental retaining walls using marginal quality backfill versus high quality backfill. Results of these analyses all indicate a satisfactory segmental retaining wall performance can be achieved by using marginal quality, or high fines, backfill with aggressive drainage and proper construction techniques. A summary of key points from above are tabulated in Table 2.1.
<table>
<thead>
<tr>
<th>Reference</th>
<th>Procedure performed</th>
<th>Focus</th>
<th>Backfill Key points</th>
<th>Geosynthetic Key points</th>
<th>Drainage Key points</th>
</tr>
</thead>
<tbody>
<tr>
<td>Christopher and Stulgis (2006)</td>
<td>Survey of state agencies and private sector groups regarding marginal backfill soil</td>
<td>Determining the factors that affect performance of geosynthetics reinforced soil walls</td>
<td>Backfill soil properties that impact design and performance of reinforced soil walls were permeability, strength, deformation, moisture-density relationship, tension cracks, and environmental effects.</td>
<td>N/A</td>
<td>Drainage system should divert water away from reinforced soil mass, not through it.</td>
</tr>
<tr>
<td>Sandri (2006)</td>
<td>Research of literature on marginal soil</td>
<td>Drainage for MSE walls using marginal fill</td>
<td>Compact backfill with kneading action at plus or minus one percent optimum moisture content</td>
<td>Internal geosynthetic should promote drainage</td>
<td>Drainage features should include: drainage in all directions, back drainage blanket, and divert water away</td>
</tr>
<tr>
<td>Lawson (2006)</td>
<td>Lab testing</td>
<td>Establish guidelines for different types of MSE backfill soil</td>
<td>80-85% of MSE wall strength is from the backfill. Peak shear resistance for fine grained soil does not occur until 10-17% strain</td>
<td>Bond resistance is the fundamental parameter of reinforced soil design.</td>
<td>Drainage is desired to take place before water enters the reinforced soil mass, not through it.</td>
</tr>
<tr>
<td>Hatami and Bathurst (2006)</td>
<td>Computer modeling</td>
<td>Develop a numerical model to predict lateral displacements of MSE walls</td>
<td>A cohesive strength component as low as 10 kPa can significantly decrease lateral deflections, thus fine grained soils can be used as backfill if placed correctly.</td>
<td>Geosynthetic reinforcement loads are greater for weaker backfill soils.</td>
<td>If appropriate care is taken to construction technique and drainage, MSE wall construction costs can be decreased if marginal fills are used.</td>
</tr>
<tr>
<td>Koerner et al (2006)</td>
<td>Establish design parameters and numerical modeling of seepage forces</td>
<td>Drainage pressures beneath, behind, and within SRW’s</td>
<td>The lateral earth pressure behind a poorly draining backfill can be twice that of a SRW with properly draining backfill soil.</td>
<td>Many geocomposites are available to facilitate drainage within the SRW reinforced zone</td>
<td>Proper drainage control is critical when fines are allowed in the backfill soil. The top of the SRW should be covered to mitigate infiltration, and drainage in front, behind and within the SRW should be installed.</td>
</tr>
<tr>
<td>Stulgis (2006) &amp; Geosynthetics June/July (2006)</td>
<td>Full-scale field tests</td>
<td>Develop parameters for MSE walls that will allow for use of a wider range of reinforced fill materials.</td>
<td>MSE walls can perform properly even if high fines materials are used.</td>
<td>N/A</td>
<td>An essential component of an MSE wall that uses reinforced fill with “high fines” soil is aggressive drainage, to prevent the buildup of pore water pressures.</td>
</tr>
</tbody>
</table>
Chapter 3: Description of Existing Wall and Site Details

3.1 Introduction

A concrete retaining wall was constructed at the residence of Dr. Allen Thompson during October of 2005. The project site is located in Boone County, Missouri (Figure 3.1 – 3.3).
The wall was constructed with number four (0.5 in. diameter) vertical and horizontal rebar spaced vertically and laterally every two feet. The wall has a height of nine feet (2.74 meters) (Figure 3.4).
The nine foot tall retaining wall section ties into the existing wing wall on the northwest corner of the house and extends westward approximately 29 feet. The second portion of the nine foot section extends approximately 14 feet to the north after two 45 degree angles approximately seven feet apart (Figure 3.5). The existing concrete wall continues to extend northward approximately 10 feet with a height of five feet, before a short five foot tall section extends toward the east.
The wall was backfilled approximately one month before noticeable cracks developed on the south face on the east side of the wall. The backfill was then removed to protect the wall from further deformation due to the backfill loading (Figure 3.6).
A preliminary stability analysis of the existing concrete wall was performed to confirm the condition of the wall. Detailed stability calculations and numerical analyses are provided in Appendix A. The results are tabulated in Table 3.1.

<table>
<thead>
<tr>
<th>Factory of Safety Condition</th>
<th>Desired Value</th>
<th>Calculated Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overturning</td>
<td>&gt;2.0</td>
<td>0.18</td>
</tr>
<tr>
<td>Sliding</td>
<td>&gt;2.0</td>
<td>0.4</td>
</tr>
<tr>
<td>Bearing Capacity</td>
<td>&gt;2.0</td>
<td>5.7</td>
</tr>
<tr>
<td>Slope Failure</td>
<td>&gt;1.5</td>
<td>1.6</td>
</tr>
</tbody>
</table>

The stability analysis verifies that the existing concrete wall is not stable in the fully backfilled condition. The factors of safety for overturning and sliding are much too low for the current existing concrete wall. To mitigate the potential for overturning failure of the concrete wall, a geotextile wrap-faced wall was designed to reinforce the backfill soil and reduce the load on the existing concrete wall, which will be used as a façade (non-load bearing). Global slope failure was investigated through the use of a slope stability
computer modeling program, SlopeW®. The retaining structure was modeled using undrained and drained strength parameters. Results shown in Figures 3.7 and 3.8 indicate a factor of safety of 1.6 and 1.9 for the undrained and drained case, respectively.

Figure 3.7 SlopeW® global slope failure analysis results of existing concrete wall (undrained condition)
3.2 Issues of Concern for Existing Wall

As can be seen in Figure 3.4a, the footing width of the existing wall is approximately two feet, and is too small for a concrete retaining wall with a height of nine feet. According to Coduto (2001), preliminary trial footing widths for concrete cantilever retaining walls should range between half the wall height to the full height of the retaining wall. The footing width for the concrete cantilever retaining wall in question is approximately one quarter of the wall height.
The computed factors of safety (Table 3.1) indicate the potential for sliding, and overturning of the wall are great in the backfilled position. While in the backfilled state, several cracks developed throughout the concrete wall (Figures 3.9 – 3.12).

**Figure 3.9** Crack in existing concrete wall (July 2006)

**Figure 3.10** Existing crack in concrete wall (July 2006)
Figure 3.11 Existing crack in concrete wall (July 2006)

Figure 3.12 Existing corner crack in concrete wall (July 2006)
When the cracks were noticed, the backfill soil was removed so further damage to the wall did not take place. The concrete wall provides a pleasing aesthetic appearance but is not functional in its current state. A remediation design was needed to reinforce the backfill soil, while saving the existing concrete wall. A solution to this challenge is to utilize a geotextile wrap-face wall that will reinforce the backfill soil and allow drainage while carrying the lateral load due to the soil so that the existing concrete wall remains stable.
3.3 Summary of Existing Concrete Wall

The existing concrete wall in the backfilled state has unacceptably low factors of safety against sliding and overturning. Evidence in the form of several cracks forming along the wall illustrates the condition of the concrete wall’s stability. A remediation plan including a geotextile wrap-face wall will aid in relieving the soil pressures on the existing concrete wall and allow the concrete wall to stay in place, which maintains the aesthetics of the structure while remaining stable.
Chapter 4: Geotextile Wrap-Face Wall Design and Design Parameters

4.1 Introduction

A wrap-face retaining wall was chosen to reinforce the soil mass immediately behind the existing concrete retaining wall. A wrap-face retaining wall consists of layers of compacted backfill with a reinforcing fabric or grid between each layer that wraps around the exposed face of the layer and is embedded within the soil mass to form a wrap-face (Figure 4.1).

Many different materials can be utilized as the reinforcing members of a wrap-face wall. Such materials include, but are not limited to polymeric materials (geotextiles and geogrids), or metallic grids (reinforcing wire fabric). The reinforcing material and earthen backfill are two integral components that are key in the retaining wall performance. The strength of an MSE wall comes from the fill and the reinforcement.
The fill provides 80-85 percent of the wall’s strength, while the reinforcement provides 15-20 percent of the strength. As a result, the type of fill chosen for construction has a great effect on the overall strength of the MSE wall (Lawson, 2006).

The backfill material chosen for this project was the on-site soil. The native soil is a light brown silty clay with low plasticity, and was classified as marginal quality backfill. The reinforcement material was a woven polypropylene geotextile (Propex 2044), with properties shown in Table 4.1. Two nonwoven geotextiles (Typar® and Geosyn 451®), HDPE geonet, and four inch perforated plastic pipe were also chosen as drainage material to be incorporated in the design and construction of the geotextile wrap-face retaining wall.

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Method</th>
<th>Minimum Average Roll Value (English)</th>
<th>Minimum Average Roll Value (Metric)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grab Tensile</td>
<td>ASTM-D-4632</td>
<td>600/500 lbs</td>
<td>2.67/2.22 kN</td>
</tr>
<tr>
<td>Grab Elongation</td>
<td>ASTM-D-4632</td>
<td>15 %</td>
<td>15 %</td>
</tr>
<tr>
<td>Wide Width Tensile</td>
<td>ASTM-D-4595</td>
<td>400/400 lbs/in</td>
<td>70.1/70.1 kN/m</td>
</tr>
<tr>
<td>Wide Width Elongation</td>
<td>ASTM-D-4595</td>
<td>10/8 %</td>
<td>10/8 %</td>
</tr>
<tr>
<td>Mullen Burst</td>
<td>ASTM-D-3786</td>
<td>1350 psi</td>
<td>9300 kPa</td>
</tr>
<tr>
<td>Puncture</td>
<td>ASTM-D-4833</td>
<td>180 lbs</td>
<td>0.800 kN</td>
</tr>
<tr>
<td>Trapezoidal Tear</td>
<td>ASTM-D-4533</td>
<td>250 lbs</td>
<td>1.11 kN</td>
</tr>
<tr>
<td>UV Resistance</td>
<td>ASTM-D-4355</td>
<td>80 % at 500 hr</td>
<td>80 % at 500 hr</td>
</tr>
<tr>
<td>AOS (1)</td>
<td>ASTM-D-4751</td>
<td>30 sieve</td>
<td>0.600 mm</td>
</tr>
<tr>
<td>Permittivity</td>
<td>ASTM-D-4491</td>
<td>0.15 sec⁻¹</td>
<td>15 sec⁻¹</td>
</tr>
<tr>
<td>Flow Rate</td>
<td>ASTM-D-4491</td>
<td>10 gal/min/ft²</td>
<td>405 L/min/m²</td>
</tr>
</tbody>
</table>

A geotextile wrap-face wall generates strength when the friction between the backfill and geotextile is mobilized. This interface friction enables the layers of geotextile to remain embedded within the backfill layers. Using this friction, two different design concepts are commonly utilized.
The first design is based on the gravity wall concept (Figure 4.2). The reinforced soil mass acts like a gravity wall to retain the adjacent soil. The geotextile reinforcement takes the lateral load of the soil within the reinforcing zone and carries it in tension.

![Reinforced mass acts as a gravity wall](image)

Figure 4.2 Reinforced soil mass acting as a gravity wall design concept

A second wrap-face wall design concept assumes that the failure plane acts at an angle from the horizontal in relation to the angle of internal friction of the backfill soil (Figure 4.3). Soil behind this line is assumed stable while soil toward the wall face needs reinforcement. The geotextile reinforcement extends through the failure plane and is anchored within the retained soil by the soil/geotextile friction. If properly designed pullout resistance is greater than the lateral load that is transferred to the geotextile through the geotextile/soil interface friction (Figure 4.4).
4.2 Backfill Soil Laboratory Testing and Results

Establishing proper wrap-face wall design parameters required laboratory testing on the native backfill soil. Testing on the backfill included establishing the Atterberg limits and moisture-density relationship.
Bulk soil samples were obtained from the site after backfill directly behind the existing concrete wall was excavated. Atterberg limit values obtained for the native backfill soil can be found below in Table 4.2 (Appendix B).

<table>
<thead>
<tr>
<th>Soil Description</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light brown silty clay</td>
<td>42</td>
<td>26</td>
<td>16</td>
</tr>
</tbody>
</table>

The Atterberg limit values were plotted on Casagrande’s plasticity chart to show the results (Coduto, 1999) (Figure 4.5). The native backfill soil classifies as a low plasticity silty lean clay (CL) according to the Unified Soil Classification System (USCS) criteria (ASTM D2487).

A moisture-density relationship was developed by performing a modification of the standard Proctor test (ASTM D698) (Appendix C).
Fifty (50) percent standard Proctor compaction energy was used while performing this test because sub-standard Proctor compaction energy was anticipated in the field. To achieve 50 percent standard Proctor energy, the number of blows per lift of soil was reduced by half the standard amount. The moisture-density relationship of the native CL clay compacted with 50 and 100 percent standard Proctor energy is shown in Figure 4.6.

![Figure 4.6 Moisture-density relationship of silty lean clay backfill using 50% and 100% standard Proctor energy](image-url)
The Proctor curve indicates that at 50 and 100 percent standard Proctor energy, the maximum dry unit weight of the backfill soil is 94 and 105 pounds per cubic foot, and the optimum moisture content is 23 and 16 percent, respectively.

<table>
<thead>
<tr>
<th>Compaction Energy (ft-lbs)</th>
<th>Max dry unit weight, $\gamma_{d_{\text{max}}}$ (pcf)</th>
<th>Optimum water content, $w_{\text{opt}}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12400</td>
<td>105</td>
<td>16</td>
</tr>
<tr>
<td>6200</td>
<td>94</td>
<td>23</td>
</tr>
</tbody>
</table>

### 4.3 Geotextile/Clay Interface Testing and Results

The interface friction between the backfill soil and geotextile was assessed. The interface friction is the resistance provided when the geotextile and backfill soil slide in relation to one another. The interface friction is necessary to evaluate the internal strength of the wrap-face retaining wall. A pullout failure can occur if the interface friction resistance is less than the tensile forces produced by the soil in the reinforced zone. Preliminary tests to determine the soil to geotextile interface friction angle were performed using a tilt table (Figure 4.7) (Appendix D).
The tilt table is a device in which a crank arm is attached to a platform by a cable. When the crank arm is rotated the platform, which is attached to the base by a pivot point at the opposite end, tilts and increases the angle from the horizontal at which the platform rests. A layer of geotextile was placed and secured on the platform. A magnetic inclinometer was placed on the platform to identify the angle from the horizontal plane at which the platform raised. A picture of the inclinometer can be seen in Figure 4.8.
A plastic mold that was three inches tall and 11 inches in diameter was used to compact and place the soil in contact with the geotextile (Figure 4.9).

The soil was placed and compacted using a five and a half pound drop hammer. Once compacted to the desired density, the mold was shifted upward approximately one half inch so that friction between mold and geotextile does not occur.
To allow for the placement of a normal force, in the form of steel plates, a spacer block covering the entire compacted soil specimen, and that lifted the plates above the mold, was inserted. Once the setup was complete, the crank arm was turned slowly at constant rate and the platform was lifted. The platform was lifted until the interface friction between the soil and geotextile was less than the sliding force experienced by the compacted soil specimen. The angle at which the interface friction failure occurred was then recorded for each particular normal force applied.

Using the moisture-density relationship of the silty clay for 50 percent standard Proctor energy, the amount of energy generated using a five and a half pound hammer was correlated to the size of the compaction mold. To obtain a dry unit weight of 94 pounds per cubic foot at a moisture content of 23 percent, 62 blows with one lift were required.

The normal force was varied three times and the friction angle was recorded. No steel plates were used for the first trial, while steel plates weighing 45 and 90 pounds were used in the remaining two trials. These three weights correlate to a normal force of zero, 31.3, and 63.6 pounds. The normal force is the portion of the load which acts perpendicular to the ground surface. These normal forces are less than the minimum overburden weight that the geotextile will experience. Results of these three tests are tabulated in Table 4.5.
Table 4.4  Tilt table specimen data

<table>
<thead>
<tr>
<th></th>
<th>Target Value</th>
<th>Actual Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_d$ (pcf)</td>
<td>94</td>
<td>97</td>
</tr>
<tr>
<td>w (%)</td>
<td>23</td>
<td>21</td>
</tr>
</tbody>
</table>

Table 4.5 - Tilt table input parameters and results

<table>
<thead>
<tr>
<th>Test #</th>
<th>Weight (lb)</th>
<th>$\delta$ (degrees)</th>
<th>Normal Force (lb)</th>
<th>$\sigma_n$ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>45</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>45</td>
<td>46</td>
<td>31.3</td>
<td>47.7</td>
</tr>
<tr>
<td>3</td>
<td>90</td>
<td>45</td>
<td>63.6</td>
<td>96.4</td>
</tr>
</tbody>
</table>

The data in Table 4.4 indicate that the actual compacted dry unit weight and moisture content values are near the desired target values. The actual dry unit weight has a percent difference of three percent, while an eight percent difference was observed in the actual moisture content. Only one specimen was compacted due to the fact that sliding failure never occurred before the maximum tilt capabilities of the apparatus were reached in all three tests.

Results of the tilt table tests indicate that the interface friction angle between the compacted soil and geotextile is high. Using this method an interface friction angle of approximately 45 degrees was obtained, and did not vary under the range of normal loads tested. These tests indicate a relatively high interface friction angle between the backfill and geotextile reinforcement. These values may also differ from a specimen prepared using the target moisture parameters. A water content difference of eight percent between the actual and target specimen moisture content may produce somewhat different results. An alternate testing method was utilized to confirm the interface friction angle.
The interface friction angle between the geotextile reinforcement and backfill soil was further researched by performing direct interface shear tests. Tests were performed using an automated GeoJack® direct shear device with automatic data collection (Figure 4.10).

The machine setup was modified so that the soil/geotextile interface was accurately modeled during the interface shear tests. An aluminum disk and clamping system were machined to precisely fit the existing shear plates so that geotextile fabric could be secured to the shear plate. The shear plates with machined geotextile testing components are shown in Figure 4.11.
The geotextile fabric was cut to fit the shear plates and clamped to the bottom shear plate on each end (Figure 4.12).
Once the bottom geotextile and shear plate were clamped, the top shear plate was applied to allow compaction of a clay specimen on top of the geotextile (Figure 4.13). To achieve compaction, two separate lifts were compacted using a miniature drop hammer to deliver 50 percent standard Proctor energy with a calibrated blow count. The soil specimen was then leveled off and placed in the direct shear box (Figure 4.14). A filter paper and porous disk were placed on top of the compacted specimen to aid in the dissipation of any excess pore water pressure during the testing.
Figure 4.13 - Assembled direct shear plates with geotextile and clamping system

Figure 4.14 - Prepared soil specimen in direct shear box
Direct shear tests were performed using three different configurations, with three different normal loads. The testing configurations included shear in the geotextile machine direction, geotextile cross-machine direction, and geotextile cross-machine direction submerged in water. The normal loads used for each configuration were one, five, and 10 pounds per square inch. As in the tilt table test, these normal loads were chosen to model the forces expected in the field.

The direct shear test is performed in two phases: consolidation phase and shear phase. Once the specimen is positioned correctly in the direct shear device, the normal load is applied and the specimen is allowed to consolidate. The consolidation is monitored and a real time consolidation plot can be viewed on the computer screen. Full consolidation took many days to complete. As a result, each specimen tested was allowed to consolidate a standard amount of time of one hour. After consolidation, the top shear plate with specimen was lifted away from the bottom shear plate approximately one-sixteenth of an inch using screw pins. Once the two plates were separated, the shearing phase was initiated. The shear rate for each specimen was 0.00833 inches per minute. The shear rate calculations are shown in Appendix E. A direct shear test in the shear phase is shown in Figure 4.15.
During the direct shear test, two direct current displacement transducers (DCDT) measure the horizontal and vertical displacement that occur while the bottom shear plate moves in relation to the top shear plate. A load cell, attached to the shear box, measures the lateral shear force. The displacement and force gauges are shown Figures 4.17 - 4.19.
Figure 4.17 - Measurement devices on direct shear device

Figure 4.18 Close-up of external vertical DCDT
Each direct shear test was performed until peak friction between the soil and geotextile occurred or until horizontal movement limitations (approximately 0.7 inches) of the testing apparatus governed. The data were recorded in tabular format, and further data reduction was required to produce graphical interface friction information.

As previously mentioned, testing configurations included direct shear in the geotextile machine direction, geotextile cross-machine direction, and in the geotextile cross-machine direction with the specimen submerged in water while subjected to one, five, and 10 pounds per square inch normal loads, respectively. A graphical representation of a set of direct shear tests with the specimen in the cross-machine direction while submerged in water is shown in Figure 4.20 (all results shown in Appendix F).
Figure 4.20 shows that as the normal stress increases the shear stress also increases until peak shear stress is reached. Failure occurs when peak shear stress is reached and the shear stress begins to reduce. When a line is drawn through the failure points for each normal load, the interface friction angle can be determined for a particular testing configuration. The interface angle of friction was found to be 30 degrees when submerged and sheared in the geotextile cross machine direction. The interface friction angles determined for each testing configuration are shown in Table 4.6.

<table>
<thead>
<tr>
<th>Testing Configuration</th>
<th>Cohesion (psi)</th>
<th>Interface friction angle b/n soil and geotextile (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear in geotextile machine direction</td>
<td>2.5</td>
<td>50</td>
</tr>
<tr>
<td>Shear in geotextile cross-machine direction</td>
<td>3.5</td>
<td>32</td>
</tr>
<tr>
<td>Shear in geotextile cross-machine direction while submerged in water</td>
<td>0</td>
<td>30</td>
</tr>
</tbody>
</table>
Based on the laboratory data, the interface friction angle between the low plasticity clay and geotextile varies among the configurations analyzed and ranges from 30 to 50 degrees. When the geotextile is placed in the cross-machine direction the interface friction angle is reduced from 50 to 35 degrees. This angle is further reduced, from 32 to 30 degrees, when the cross-machine direction specimen is submerged in water for approximately 24 hours and while testing.

4.4 Geotextile Wrap-Face Wall Design

Many factors must be accounted for when designing a geotextile wrap-face retaining wall. Such factors include the strength properties of the construction materials, e.g.’s, the backfill soil and geotextile reinforcement, interaction between construction materials, and the known and assumed forces that will be exerted on the wall. Using these material properties and forces, the internal and external stability of the wrap-face retaining wall can be analyzed and a design produced.

Internal stability of the wall was first addressed. To achieve internal stability, sufficient geotextile spacing, geotextile length, and overlap distance must be determined. Some assumptions in the wrap-face wall design included a backfill soil’s total unit weight of 115 pounds per cubic foot, and an angle of internal friction of zero degrees. The interface friction angle was assumed to be 15 degrees. This value is lower than the values that were found during direct shear lab testing; however, it is assumed that construction practices will not be as controlled as in the laboratory, thus a reduction in the interface friction angle might result in the field case. It should be noted that these values are fictitious due to the face that the angle of internal friction is less than the interface friction angle.
If the angle of internal friction is zero, then the interface friction angle can not be greater than zero. Nonetheless, these values were used in the initial design which increase the safety factor. A cohesion value of two pounds per square inch (2 psi) was also assumed, which falls within the range of cohesion values obtained during the direct shear tests. The ultimate tension for the geotextile reinforcement (Propex 2044) was assumed to be 400 lb/in (70 kN/m), which was obtained from the Propex Fabrics product specification data (Propex Fabrics, Inc. 2005). This value was obtained from a wide-width tension test.

To determine the geotextile layer vertical separation distances, earth pressures were assumed to be linearly distributed using Rankine active earth pressure conditions for the soil backfill (Koerner, 1998). No surcharge loads were assumed in the design because it was anticipated that the area above the wall will be open space. This assumption is un-conservative due to the fact that the future is uncertain. The coefficient of lateral earth pressure must first be determined using Equation 4.1.

\[ K_a = \tan^2 (45 - \frac{\phi}{2}) \]  

\( K_a \) = coefficient of active lateral earth pressure (unitless)  
\( \phi \) = angle of internal friction (degrees)

Since the angle of internal friction of the soil was assumed to be zero, the coefficient of lateral earth pressure (K_a) was determined to be 1.0, proving to be a very conservative assumption. The horizontal earth pressure was then calculated using Equation 4.2.

\[ \sigma_h = \gamma * K_a \times z = 115 * z \text{ psf} \]  

\( \sigma_h \) = total horizontal earth pressure (psf)  
\( \gamma \) = total unit weight (pcf)  
\( z \) = depth (ft)  
\( K_a \) = coefficient of lateral earth pressure (unitless)
The allowable geotextile tension strength must also be calculated for the design by applying reduction factors to the ultimate tension value. These reduction factors take into account ways in which the ultimate tension strength can be reduced, and are defined and shown in Table 4.7.

<table>
<thead>
<tr>
<th>Strength reduction mode</th>
<th>Reduction factor ranges</th>
<th>Reduction factor used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Installation damage</td>
<td>1.1 to 2.0</td>
<td>1.2</td>
</tr>
<tr>
<td>Creep</td>
<td>2.0 to 4.0</td>
<td>2.5</td>
</tr>
<tr>
<td>Chemical damage</td>
<td>1.0 to 1.5</td>
<td>1.1</td>
</tr>
<tr>
<td>Biological damage</td>
<td>1.0 to 1.3</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Using the reduction factors shown in Table 4.7, and Equation 4.3, the allowable tensile strength \( T_{allow} \) of the geotextile was determined to be 120 lb/in.

\[
T_{allow} = T_{ult} \left( \frac{1}{RF_{ID} \ast RF_{CE} \ast RF_{CD} \ast RF_{BD}} \right) = 120 \text{ lb/in} \quad \text{(Equation 4.3)}
\]

\( T_{allow} \) = allowable geotextile tensile strength (lb/in)

\( T_{ult} \) = ultimate geotextile tensile strength (lb/in)

The vertical geotextile spacing \( (S_v) \) represents the maximum distance between each geotextile layer at different depths throughout the wall profile. The vertical geotextile spacing dictates the number of layers used to construct the wrap-face wall. The vertical spacing is determined using Equation 4.4.

\[
S_v = \frac{T_{allow}}{\sigma_h \ast (FS)} \quad \text{(Equation 4.4)}
\]

\( T_{allow} \) = allowable tensile strength (lb/in)

\( \sigma_h \) = horizontal stress at depth \( z \)

FS = factor of safety against failure of geotextile breakage \( \approx 1.4 \)
Using Equation 4.4 a total of seven layers of geotextile reinforcement were required to construct a wall with a height of nine feet, using the backfill and reinforcing materials previously discussed. The layer vertical spacings calculated range from one foot, near the bottom, to two feet, at the top of the wall.

The length of each geotextile layer is determined by the angle at which the Rankine failure plane extends through the backfill soil. The inclination of the Rankine failure plane can be represented by Equation 4.5.

\[
\text{Inclination of Rankine Failure Plane} = 45 + \frac{\phi}{2} \quad \text{(Equation 4.5)}
\]

Since the angle of internal friction was assumed to be zero, the inclination of the failure plane is forty-five degrees from the toe of the existing concrete footing (Figure 4.21).

![Figure 4.21 Schematic of Rankine failure plane and geotextile embedment lengths](image)

The stress in the geotextile reinforcement reaches a maximum near the failure plane and falls off sharply to either side.
The geotextile fabric length between the wall face and Rankine failure plane is defined as the non-acting lengths \( L_r \). The lengths of geotextile required beyond the failure surface, labeled the active zone, are defined as the embedment lengths \( L_e \). The total length of each geotextile layer required is the sum of the non-acting length and the embedment length, which was calculated using Equations 4.6 and 4.7 (Koerner, 1998), using the required geotextile length, vertical spacing, and overlap. The geotextile dimensions were calculated and the design parameters are presented in Table 4.8 and Figure 4.22.

\[
L_e = \frac{S_v \sigma_h FS}{2(c_a + \gamma z \tan \delta)}
\]

**Equation 4.6**

- \( S_v \) = vertical layer spacing (ft)
- \( \sigma_h \) = horizontal stress at depth \( z \)
- \( FS \) = factor of safety against failure of geotextile breakage \( \approx 1.4 \)
- \( c_a \) = soil adhesion between soil and geotextile (psf)
- \( \gamma \) = total unit weight of backfill soil (pcf)
- \( \delta \) = angle of shearing friction between soil & geotextile (degrees)
- \( z \) = depth of geotextile layer (ft)

\[
L_r = (H - z) \tan(45 - \frac{\delta}{2})
\]

**Equation 4.7**

- \( H \) = wrap-face wall height (ft)
Table 4.8  Geotextile reinforcement design dimensions Sv, Lr, Le, and Lo

<table>
<thead>
<tr>
<th>Layer #</th>
<th>Depth (ft)</th>
<th>Spacing, Sv (ft)</th>
<th>Lr + Le (ft)</th>
<th>Lo (ft)</th>
<th>Total Geotextile Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (Bottom)</td>
<td>9</td>
<td>1</td>
<td>4</td>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>1</td>
<td>4</td>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>1</td>
<td>5</td>
<td>3</td>
<td>9</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>1</td>
<td>6</td>
<td>3</td>
<td>10</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>1.5</td>
<td>7</td>
<td>3</td>
<td>11.5</td>
</tr>
<tr>
<td>6</td>
<td>3.5</td>
<td>1.5</td>
<td>9</td>
<td>3</td>
<td>13.5</td>
</tr>
<tr>
<td>7 (Top)</td>
<td>2</td>
<td>2</td>
<td>10</td>
<td>3</td>
<td>15</td>
</tr>
</tbody>
</table>

Figure 4.22 - Profile view of wrap-face wall as-designed
Although the active zone overlap length, $L_o$, was determined to be less than one foot, it is recommended that an overlap length of three feet be specified (Koerner, 1998). Using Equation 4.8, the overlap length was compared to the minimum three foot overlap recommendation.

$$L_o = \frac{S_v \cdot \sigma_h \cdot FS}{4(c_a + \gamma \cdot z \cdot \tan \delta)}$$  \hspace{1cm} \text{(Equation 4.8)}

The maximum overlap length for the upper layer, at $z = 2$ ft, was verified as a conservative approach. At this depth, an overlap length of 0.5 ft was calculated, which is less than the 3 ft overlap design length. Therefore, using a three foot overlap should be more than adequate. All design calculations are found in Appendix G.

4.4a. Compaction of Backfill soil

Compaction of the backfill was achieved using a skid loader (Bobcat® model number 763). The operating weight of this machine is 5368 pounds with tire pressures of 30 pounds per square inch. While moving the soil around, the weight from the machine will result in some compaction of the backfill soil.

4.5 External Stability of Wrap-Face Wall Design

External stability of the geotextile wrap-face wall was analyzed by determining the factor of safety with respect to overturning, sliding, and bearing capacity. The external stability of the wrap-face wall is significantly higher than the external stability of the existing concrete wall when in the backfilled position. Results of the external stability analysis regarding the geotextile wrap-face retaining wall constructed according to the above design specifications are given in Table 4.9 (calculations are provided in Appendix H).
Table 4.9 Calculated and desired external stability of the designed geotextile wrap-face wall

<table>
<thead>
<tr>
<th>Factor of safety condition</th>
<th>Desired Value</th>
<th>Calculated Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overturning</td>
<td>&gt;2.0</td>
<td>5.0</td>
</tr>
<tr>
<td>Sliding</td>
<td>&gt;2.0</td>
<td>1.5</td>
</tr>
<tr>
<td>Bearing capacity</td>
<td>&gt;2.0</td>
<td>4.8</td>
</tr>
<tr>
<td>Slope Failure</td>
<td>&gt;1.5</td>
<td>1.5</td>
</tr>
</tbody>
</table>

The factor of safety with respect to overturning for the reinforced mass was found to be five. This value is significantly increased when compared to the same value for the existing concrete retaining wall. The factor of safety with respect to sliding was calculated at 1.5. Although this value is less than the desired value of two, a conservative analysis was utilized while performing the calculation. The actual wrap-face wall will have a greater sliding resistance due to the base of the concrete wall located directly in front of the reinforced mass (not included in the calculation). Total slope failure was again investigated using the slope stability software SlopeW®. Geotextile reinforcement and the existing concrete wall were incorporated into the slope model and evaluated for undrained and drained conditions (Figures 4.23 and 4.24). Safety factors of 1.5 and 2.0 were calculated for undrained, and drained conditions, respectively. After performing the external stability analysis of the reinforced mass, the geotextile wrap face wall should perform well under the assumed conditions.
Figure 4.23 SlopeW® analysis results of existing concrete wall with reinforced soil mass (undrained condition)
4.6 Assumptions Used in Wrap-Face Wall Design

Many assumptions were used in the design of the geotextile wrap-face wall. The assumptions can be found in the internal stability calculations, as well as the external stability analyses. A conservative approach was taken when making each assumption. As a result of incorporating conservative values, it is expected that higher factors of safety were established.
Many of the assumed parameters were used in the internal stability design. These parameters include backfill unit weight, cohesion, interface angle of internal friction, coefficient of lateral earth pressure, and geotextile overlap lengths (Table 4.10).

Table 4.10  Parameters assumed/approximated and the respective range, mean, and COV

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value used in design</th>
<th>Approximated /Measured value</th>
<th>Range</th>
<th>Mean</th>
<th>COV (%)</th>
<th>&gt; or &lt; % of range values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Backfill unit weight</td>
<td>115 pcf</td>
<td>Avg ~ 112 pcf (Measured)</td>
<td>102 – 108 pcf</td>
<td>108 pcf</td>
<td>1.0</td>
<td>&lt; 100</td>
</tr>
<tr>
<td>Backfill friction angle</td>
<td>0°</td>
<td>0° &gt; (\Phi) &gt; 30°</td>
<td>24° – 40°</td>
<td>33.3°</td>
<td>13°</td>
<td>&lt; 83</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(Coduto, 1999)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Interface friction angle</td>
<td>15°</td>
<td>30° – 50° (Measured)</td>
<td>19.2° – 32°</td>
<td>26.6°</td>
<td>13°</td>
<td>&lt; 83 (assuming (\delta)=30°)</td>
</tr>
<tr>
<td>Coefficient of lateral earth</td>
<td>1.0</td>
<td>0.6 (Lambe &amp; Whitman,</td>
<td>0 – 1.0</td>
<td>--</td>
<td>--</td>
<td>&lt; 100</td>
</tr>
<tr>
<td>pressure</td>
<td></td>
<td>1969)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\(a\) = values from Phoon & Kulhawy (1999)

A backfill total unit weight of 115 pounds per cubic foot was used in the design of the geotextile wrap-face wall. Nuclear density tests during geotextile wall construction indicate an average total unit weight of 112 pounds per cubic foot. Although the assumed design backfill unit weight was a good approximation of the actual field conditions, it is slightly greater. A possible range for silt and clay total unit weight is 102 to 108 pounds per cubic foot (Phoon & Kulhawy, 1999). Using this range the measured total unit weight is greater than 100 percent of the estimated ranged values. Using a greater unit
weight than expected, a conservative assumption, increases the lateral stresses resulting
in increased geotextile embedment lengths required for each layer of geotextile
reinforcement.

The angle of internal friction of the soil backfill was assumed to be zero degrees
in the wrap-face wall design. Although silts and clays are not generally known for high
angles of internal friction, some friction angle value is common. Typical effective
friction angles for soils with similar plasticity can range up to 30 degrees (Coduto, 1999).
A range for the effective friction angle of silts and clays found by direct shear is 24 to 40
degrees (Phoon & Kulhawy, 1999). Using this range, the design value was greater than
83 percent of the range values. An underestimation of the angle of internal friction of the
backfill soil has implications that affect other parameters such as the coefficient of lateral
earth pressure and the Rankine failure plane, which will be discussed later. The
geotextile non-acting lengths are also increased, and the geotextile layer spacing is
decreased when a conservative backfill friction angle is assumed.

The interface angle of internal friction between the geotextile and backfill soil
was measured in the lab tests and found to range between 30 and 50 degrees. An
estimation for the interface friction angle in clays with geotextile contact can be
estimated using 83 percent of the angle of internal friction of the clay (Koerner, 1998).
Using this value, 83 percent of the provided range values are less than the estimated
value. When this value is underestimated, the geotextile embedment lengths are
increased. The interface angle of internal friction used in design was 15 degrees, and is a
conservative assumption.
The coefficient of lateral earth pressure is affected by the backfill soil’s angle of internal friction. When a friction angle of zero degrees is assumed, the coefficient of lateral earth pressure is 1.0. For example, when backfill friction angle of 15 degrees is assumed, this coefficient becomes approximately 0.6, thus greatly reducing the lateral stress potential induced by the backfill soil. When the coefficient of lateral earth pressure is reduced, the horizontal stress on the geotextile reinforcement is reduced. This allows for greater geotextile layer spacing. A decreased coefficient of lateral earth pressure also reduces the required geotextile embedment lengths.

The maximum geotextile overlap required is found in the top lift, approximately two feet from the top of wall. The required overlap length was found to be less than one foot for a factor of safety of 1.4. Although an overlap length of one foot was calculated, three foot overlaps were specified in the design. This assumption decreases the potential for pullout of the wrap-face and makes it constructible.

Many assumptions were incorporated within the design of the geotextile wrap-face wall. These parameters, regarding the backfill soil, geotextile reinforcement, and the interaction between the two materials, generate a wrap-face wall design with a factor of safety that is greater than the required. A comparison of these safety factor values are discussed in Chapter 6, section 3.

4.7 Summary of Geotextile Wrap-Face Wall and Design Parameters

A geotextile wrap-face wall was built directly behind the existing concrete wall retaining wall, with a gap between the back of the concrete wall and the face of the geotextile wrap-face wall, to resist the active earth pressure from the retained backfill.
The objective is to reduce the load on the existing concrete wall so that it remains stable, and can be used as a façade to protect the geotextile face of the reinforced soil mass.

Laboratory testing of the native backfill soil and geotextile reinforcement established design parameters that are unique to the project site. The backfill soil is a silty low plasticity clay (CL) that when compacted with 50 percent standard Proctor energy has a maximum dry density of 94 pounds per cubic foot, and optimum water content of 23 percent. The interface angle of friction (GT/Soil), when tested in the submerged cross machine direction, was found to be 30 degrees.

A design of the geotextile wrap-face wall was developed using the above design parameters. The new wall will be nine feet in height and have seven layers of geotextile reinforcement that range in vertical spacing from one to two feet. The total length of geotextile reinforcement layers range from eight to 15 feet with the embedment lengths increasing with the height of the wall (Table 4.11a,b, & c). Backfill compaction will be achieved by tracking of a skid loader, (Bobcat 763), over the reinforced mass. Under these design specifications, the active earth pressure will be absorbed by the reinforced soil mass, and the existing concrete wall should experience little to no loading, while being utilized as a façade.
Table 4.11 Design specifications of geotextile wrap-face wall

<table>
<thead>
<tr>
<th>Layer #</th>
<th>Thickness (ft)</th>
<th>Total Length of Geotextile (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>9</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>10</td>
</tr>
<tr>
<td>5</td>
<td>1.5</td>
<td>11.5</td>
</tr>
<tr>
<td>6</td>
<td>1.5</td>
<td>13.5</td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>15</td>
</tr>
</tbody>
</table>

*a all geotextile lateral overlaps ≥ 1 ft
*b all geotextile overlaps ≥ 3 ft

<table>
<thead>
<tr>
<th></th>
<th>Dry Unit Weight ($\gamma_d$), pcf</th>
<th>94</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Content (w), %</td>
<td></td>
<td>23 ± 2%</td>
</tr>
</tbody>
</table>

c) Drainage System

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Underdrain</td>
<td>Geotextile/Geonet/Geotextile</td>
</tr>
<tr>
<td>Sidewall drain</td>
<td>Geotextile against east and west walls</td>
</tr>
<tr>
<td>Pipe drain</td>
<td>4 inch diameter perforated pipe located on top of concrete wall footing</td>
</tr>
<tr>
<td>Internal drainage</td>
<td>2 inch sand layer on top of layer #1</td>
</tr>
<tr>
<td>Grading</td>
<td>3 percent slope to the North from wall face</td>
</tr>
</tbody>
</table>
Chapter 5: Geotextile Wrap-Face Wall Construction Issues

5.1 Introduction

Construction issues, or challenges, arise with every engineered design. Although some issues may stem from the general design process, a majority of the major construction issues arise from the site conditions that are unique to each project. Many construction issues were considered while designing the wrap-face retaining wall. These are addressed in the following sections.

5.2 Construction Issues

Several construction issues were encountered when designing the wrap-face retaining wall. The very reason why the project exists, the existing poorly constructed concrete wall, is the major cause for most of the construction issues faced in the design. The existing concrete wall restricts access to the wrap-face and increases construction difficulties. Some of these construction difficulties include temporary fascia support, compaction, drainage; geotextile overlaps, and fill between the geotextile wrap-face wall and the existing concrete wall.

5.3 Temporary Fascia Support

Conventional wrap-face walls utilize an L-bracket that slides in between successive layers of geotextile reinforcement and compacted backfill soil enabling compaction of the wall face (Figure 5.1).
Figure 5.1 Construction sequence for geotextile walls (Koerner, 1998)

The space limitations resulting from the existing concrete wall at this particular project site do not allow for a compaction form of this nature (Figure 5.2).
Backfill compaction, especially at the wrap face, is a vital component that influences wrap-face wall performance and future wall deformation. In order to facilitate compaction, a temporary fascia support, or timber face form, was constructed along the length of each soil lift in order to form and compact the wrap face. A two inch by 12 inch timber with four inch spacing blocks along the back side enabled the wrap-face wall to be constructed away from the existing concrete wall. Although the timber face form was braced against the existing concrete wall, the lateral forces exerted on the concrete wall due to compaction procedures were calculated to be tolerable by the existing concrete wall (calculations provided in Appendix I). A schematic of the temporary fascia support is shown in Figure 5.3.
Once a soil lift is completed, the compaction timber is removed and placed above the most recent layer completed. The process enables the wrap-face to be constructed away from the existing concrete wall, with a gap between the back of the concrete wall and the face of the geotextile wrap-face wall (Figure 5.2).
A gap between the two walls allows room for deformation that might occur in the geotextile wrap-face wall and maintains no loading on the concrete wall.

### 5.4 Backfill Compaction

Backfill compaction greatly affects the performance of a geotextile wrap-face wall. Excessive deformation can occur when insufficient backfill compaction occurs. Unraveling of the wrap-face can occur when adequate compaction is not performed in the area directly behind the wrap-face, termed the “hand compaction zone.” This zone is labeled hand compaction because construction loads are desired to be minimized near the wall face (Simac, 2006). Although machine compaction is not performed in this area, some type of compaction device, e.g. tamper, jumping jack, or small vibratory roller, should be utilized to ensure that wall face unraveling does not occur.

Backfill compaction was performed using a full size skid-loader (Bobcat® 763) (Figure 5.4).

![Figure 5.4 Backfill compaction using skid loader (July 24, 2006)](image)
Although kneading compaction, using a sheepsfoot roller, of the clay backfill is preferred, tracking of the rubber-tired skid loader produces some kneading compaction. The skid-loader has an operating weight of approximately 5400 pounds and a tire pressure of 30 pounds per square inch. Compaction of the clay backfill resulted from tracking over the fill with the skid-loader. Calculations verify that the skid-loader can work within one foot of the concrete wall face, previously termed the hand compaction zone, without failure occurring in the existing concrete wall (Appendix I). The lateral loads shown in Table 5.1 were calculated following the lateral loading procedures found in Koerner (1998). It is assumed that the skid-loader weighs a total of 8500 pounds (5400 lb x 1.6 FS). As a result, a safety factor of 1.6, as compared to the actual condition, was incorporated in the calculated lateral loads. Lateral loads exerted on the wall act within two feet by one foot area on the wall face, and the skid-loader is located one foot away in the horizontal direction from the concrete wall face (Figure 5.5).
Figure 5.5  Elevation view of lateral loading of concrete wall due to skid loader

Figure 5.6  Plan view of lateral loading to the skid loader point load
Table 5.1 Calculated lateral point loads on existing concrete wall due to skid-loader compaction

<table>
<thead>
<tr>
<th>Depth from top of wall (ft)</th>
<th>Calc’d lateral pt. load req’d for overturning failure (lbs)</th>
<th>Calc’d lateral pt. load due to skid loader acting at specified depth (lbs) (weight assumed 8500 lbs)</th>
<th>Calc’d lateral pt. load due to skid loader acting at specified depth (lbs) (weight assumed 5400 lbs)</th>
<th>Safety factor against overturning using skid-loader weight of 5400 lbs</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>164</td>
<td>105</td>
<td>67</td>
<td>2</td>
</tr>
<tr>
<td>2.75</td>
<td>210</td>
<td>209</td>
<td>133</td>
<td>2</td>
</tr>
<tr>
<td>4.25</td>
<td>276</td>
<td>158</td>
<td>100</td>
<td>3</td>
</tr>
<tr>
<td>5.5</td>
<td>375</td>
<td>74</td>
<td>47</td>
<td>8</td>
</tr>
<tr>
<td>6.5</td>
<td>525</td>
<td>52</td>
<td>33</td>
<td>16</td>
</tr>
<tr>
<td>7.5</td>
<td>875</td>
<td>31</td>
<td>20</td>
<td>44</td>
</tr>
<tr>
<td>8.5</td>
<td>2625</td>
<td>19</td>
<td>12</td>
<td>219</td>
</tr>
</tbody>
</table>

The lateral loads due to the skid-loader are less than the lateral loads required for overturning of the concrete wall. The actual operating weight of the skid-loader used (5400 lbs) is less than the assumed weight (8500 lbs) in the calculations. The factor of safety against overturning for each depth below the top of the wall is the ratio of the lateral load required for overturning and the lateral load due to the skid-loader. These factors of safety values range from two to 219, with a minimum value located at approximately three feet below the top of wall. A graphical representation of this load distribution can be seen in Figure 5.7.
The data shown in Figure 5.7 indicate that the maximum lateral load is found at approximately thirty percent of the total wall height measured from the wall top. This value corresponds to the maximum lateral pressure calculated to act on the nine feet retaining wall to act 2.75 ft below the top of the concrete.

5.5 Drainage

Adequate drainage is one of the most important aspects of a geotextile wrap-face wall, especially when marginal backfill incorporated (Sandri, 2005). A properly constructed geotextile wrap-face wall incorporates drainage on all sides of the reinforced soil mass. A schematic of the as-designed drainage features for the wrap-face wall is shown in Figure 5.8.
A drainage blanket located on the back side, sidewalls, and beneath the reinforced soil will prevent moisture from migrating upward into the soil and provide a conduit for moisture that has infiltrated the soil to escape without buildup of pore water pressures. A perforated pipe drain is to be located at the bottom of the concrete wall footing, and contact between the perforated pipe and drainage blanket will allow water to escape the reinforced soil mass. Drainage features should be designed so that water is diverted away from the reinforced soil mass, not through it (Lawson, 2005). After completion of the wrap-face wall, the surface soils should be graded at a slope so that water is diverted away from the reinforced soil mass.
5.6 Geotextile Overlaps

Due to the geometry of the existing concrete wall, layout of the geotextile reinforcement included many overlaps. Cutting and placing large pieces of geotextile is required so that the geotextile’s machine direction is perpendicular to the existing concrete wall face. Placing geotextile in the machine direction is not necessitated by the tensile strength, as the strengths in the machine and cross machine direction are equal. Geotextile roll width dictated the direction of geotextile placement, when the layers were greater than nine feet. Minimum overlap between all pieces of fabric is specified to be one foot in length (Figure 5.9).
Problem areas found within the geotextile layout arise in the angled portions of the wrap-face wall. In these areas as much as three layers of geotextile reinforcement is present. This amount of material will make constructing a clean and smooth wrap face difficult. Proper care must be taken when placing the geotextile so that adequate overlapping joints and angles are obtained and a tight wrap face is constructed.
5.7 Void Between Back of Concrete Wall and Wrap-Face Wall

As previously mentioned, timber face forms are removed once a soil layer is complete. This procedure leaves a void between the geotextile wall and concrete wall. It is desired to have a void or highly compressible inclusion (fill), so that lateral deformation of the geotextile wall does not load the existing concrete wall (Figure 5.10).

![Figure 5.10 Profile view of void space between concrete wall and wrap-face wall](image)

A cover plate must be placed over the gap between the top of the concrete wall and top of the geotextile wall. Again, the cover plate must be installed so that lateral load is not transferred from the deforming soil mass to the existing concrete wall. The cover plate should be made of a material that will perform throughout the life of the wrap-face wall, i.e., steel.

In addition to covering the void, the cover plate will protect the geotextile wrap-face from ultraviolet (UV) rays (wavelengths shorter than 400 nm) produced by the sun. UV light stimulates oxidation by which the molecular chains are cut off.
If this process starts, the molecular chains degrade continuously and the original molecular structure changes, resulting in a substantial reduction of the mechanical resistance and brittleness of the geosynthetic (Shukla and Yin, 2006). As the geotextile degrades, the strength decreases, which can decrease the performance of the geotextile wrap-face wall.

5.8 Summary of Construction Issues

The amount and degree of difficulty define the ease of which construction of a project occurs. Many construction issues are present in this particular geotextile wrap-face wall design and summarized in Table 5.2. Compaction of each soil lift is facilitated using a timber face-form braced against the existing concrete retaining wall. Calculations verify that compaction performed using these braces and skid-loader tracking will produce lateral loads tolerable by the concrete wall. Compaction of the backfill will occur due to the weight of the skid-loader, and some kneading compaction will occur due to rubber tires. The geotextile reinforcement will be placed with the machine direction perpendicular to the wall face. Minimum overlap lengths of one (1) ft are required, although extended geotextile overlaps will occur at the angled sections of the concrete wall and will increase construction difficulty. Once the wall is complete, the void located between the back of the existing concrete wall and geotextile wrap-face wall can filled with a compressible material. A cover plate over the area will provide protection against objects falling in the gap, and UV degradation of the geotextile face.
<table>
<thead>
<tr>
<th>Issue</th>
<th>Key points</th>
<th>Options to solve</th>
</tr>
</thead>
</table>
| **Temporary fascia support** | • Conventional L-bracket form cannot be utilized due to existing concrete wall.  
• A different method must be developed.  
  • Form must be removable.  
  • Form must generate a gap between the two walls after removal. | • Construct face-forms from 2” by 12” timber, with 4” spacers along the form length to generate a gap between walls.  
• Eye hooks in timbers allow for removal after the layer is complete. |
| **Compaction**                | • Degree of backfill compaction affects wall performance.  
  • Unraveling can occur at wall face if compaction is not performed in area termed “hand compaction zone”.  
  • Compaction technique must not cause failure of existing concrete wall. | • Compaction performed by tracking of a skid-loader (Bobacat® 763).  
• Skid-loader can operate less than 1 ft away from wall face, hand compaction zone, without causing overturning failure of the concrete wall.  
• Maximum lateral force due to skid-loader on concrete wall occurs at 3 ft below top of wall. |
| **Drainage**                  | • Adequate drainage is one of the most important aspects of geosynthetics wall construction, especially when using marginal backfill.  
  • Poor drainage can increase the buildup of positive poor water pressures in the backfill soil.  
  • Drainage system should divert water away from the reinforced soil mass, not through it. | • Drainage system incorporates drainage features on all sides of the geosynthetics wall.  
• Drainage blanket underneath the soil mass and on the side walls will transmit infiltrated water to perforated drain pipe.  
• Drain pipe will run along the footing top and will be wrapped in drainage blanket to allow flow of water away from the wall, downslope. |
| **Overlaps**                  | • Geometry of existing concrete wall generates multiple geotextile overlaps.  
  • Angled sections have as many as 3 layers of geotextile, which make constructing a clean and smooth wrap face difficult.  
  • Adequate overlaps generate a seamless geotextile wall. | • Proper care during geotextile cutting and placement aids in the construction of a smooth geotextile wall face.  
• Minimum overlaps of 1 ft aid in a seamless geotextile wall face and minimize the risk of unraveling. |
| **Void between geosynthetic and concrete wall** | • Timber face-forms generate a void between the two walls.  
  • Void can be left open or filled, but cannot transfer load from geosynthetic wall to concrete wall.  
  • A cover plate prevents objects falling into the void and geosynthetics UV exposure. | • A compressible inclusion filling the void space allows for movement of the geotextile wall without transferring load to the concrete wall.  
• A steel cover plate, or other non-degrading material, spans the gap between the walls and mitigates UV exposure of the geosynthetic reinforcement. |
Chapter 6: Documentation of the Field Construction of the Geotextile Wall

6.1 Introduction

Construction of the geotextile wrap-face wall using marginal backfill took place on July 17 – 24, 2006 at the residence of Dr. Allen Thompson, 12304 West Highway EE Rocheport, MO 65279. During this time, very hot and humid weather conditions were present, except for a brief rain shower on the morning of July 21, 2006 (Table 6.1).

Table 6.1 Daily weather conditions during construction of geotextile wall (Missouri Agricultural Experiment Station - Sanborn Field)

<table>
<thead>
<tr>
<th>Date - 2006</th>
<th>7/17</th>
<th>7/18</th>
<th>7/19</th>
<th>7/20</th>
<th>7/21</th>
<th>7/22</th>
<th>7/23</th>
<th>7/24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max Air Temp., °F</td>
<td>98.7</td>
<td>96.2</td>
<td>100.8</td>
<td>100.9</td>
<td>83.6</td>
<td>82.2</td>
<td>87.8</td>
<td>92.4</td>
</tr>
<tr>
<td>Min Air Temp., °F</td>
<td>76.6</td>
<td>79.4</td>
<td>77.6</td>
<td>76.4</td>
<td>67.4</td>
<td>64.1</td>
<td>64.4</td>
<td>65.6</td>
</tr>
<tr>
<td>Avg. Relative Humidity, %</td>
<td>60.4</td>
<td>65.7</td>
<td>66.7</td>
<td>57.7</td>
<td>72.8</td>
<td>69.2</td>
<td>62.5</td>
<td>54.6</td>
</tr>
<tr>
<td>Daily Precip. Total, inches</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.15</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Project site conditions encountered at the site varied slightly from expected conditions.

As a result, minor changes in construction as compared to the original design were needed. Overall, the geotextile wrap face wall was constructed as close to the original design specifications as possible.
6.2 Photo Documentary of Field Construction

After transporting the materials and tools needed to the project site, an initial survey of the existing conditions was performed. The soil where the geotextile wrap-face wall was to be constructed was excavated and placed in the yard between the house and the road (Figures 6.1 and 6.2). The footing of the concrete retaining wall was exposed, but required additional hand and skid-loader excavating to sufficiently clear the footing of additional soil. Once the footing was cleared, the existing perforated pipe drain was inspected (Figure 6.3). The perforated pipe drain was placed along the concrete wall footing adjacent to the house.

![Figure 6.1 Plan view of project site layout. Note excavated soil pile northeast of excavation.](image-url)
Figure 6.2  View of project site facing southwest  (July 17, 2006)

Figure 6.3  Existing perforated pipe drain placed along top of the concrete wall footing (July 18, 2006).  Note the sediment and gravel inside of the pipe.
Intrusion of rock and soil particles through the perforations resulted in partial clogging of the drainage pipe. When clogging occurs, the drainage system flow volume decreases, making the buildup of positive pore water pressures behind the wall more likely. This particular section of pipe was removed and replaced.

Construction methods are available to mitigate particle intrusion into the perforated drainage pipe. One method is to wrap the pipe in a geotextile, which acts as a filter. The geotextile allows water flow into the perforations in the pipe, while screening the soil and rock particles (ASTM D6707). Another method to mitigate clogging of drainage pipe is to construct a graded filter around the pipe. A graded filter consists of one or more layers of carefully graded soil placed between the potentially migrating soil and the drain. Each filter zone is fine enough to prevent significant migration of the upstream soil, yet coarse enough not to migrate into the downstream soil. The filter also must have a significantly high hydraulic conductivity to effectively transmit water to the drain.

Further inspection of the perforated pipe drainage system revealed that the flow line was nearly level and indirect. These drainage deficiencies were removed by installing an exit pipe beneath the concrete wall footing and exiting down to the adjacent driveway down slope (Figure 6.4).
Pipe fittings were purchased and installed to construct the new drainage system, as shown in Figures 6.5 and 6.6. After the drainage pipe system was completed, the backfilling began.
Figure 6.5  Preparing the excavated footing hole for pipe installation (July 17, 2006). Note the aggregate underdrain already in place.

Figure 6.6  Installation of drainage pipe junction (July 18, 2006). Lower inlet (slotted cap) drains water from the aggregate underdrain.
In order to produce a firm and level base and an underdrain for the wrap-face wall, two-inch clean gravel was ordered and placed with the skid loader to a height that was even with the top of concrete wall footing (1 foot thick), as shown in Figures 6.5 and 6.7.

Nonwoven, spun-bonded (NW-SB) geotextile was placed beneath the gravel to act as a separator between the subgrade and aggregate. The geotextile keeps fines from the subgrade from intruding into the aggregate and ensures good drainage through the underdrain. A drainage blanket was then placed on top of the NW-SB geotextile (Figure 6.8).
The bottom drainage blanket consists of a geonet in between two layers of NW-NP geotextile. This blanket was then rolled up, so that the skid-loader was not tracking over the underdrain system (Figure 6.9).

The second layer of the wrap-face wall consisted of geotextile reinforcement (Woven, Propex 2044) with clean gravel backfill. This layer was placed and compacted using the skid-loader bucket. The geotextile reinforcement wrapped around the existing perforated pipe drain and rested on the concrete footing.
After installation of the second layer it was apparent that the rolled up drainage blanket was slowing progress, and difficult to work around. As a result, the blanket was cut and thereafter overlapped under each lift separately. Before installation of the third lift was possible, the timber face forms were constructed and placed on top of the second lift.

Geotextile reinforcement was cut to the correct length, placed for the third lift, tucked behind the compaction timber, and backfilling began using native soil (CL) (Figure 6.10). The method for backfilling included filling up to the lift height at the wall face, and sloping the backfill away from the wall face. The geotextile reinforcement was then wrapped around on top of the backfill and tensioned. Backfilling continued until a level lift was achieved, and compaction by tracking back and forth with the skid-loader was performed.
The clean gravel backfill material used along the far west side of the excavation to facilitate drainage along the wall face.

The fourth through eighth lifts were constructed in this manner with design or slightly modified geotextile reinforcement lengths and lift thicknesses. The construction process is documented in photographs shown in Figures 6.11 through 6.33.
Figure 6.11  Installation of 4th lift – 0.75 ft thick (July 19, 2006).  2nd of native soil (CL) lifts.

Figure 6.12  Placement of backfill on geotextile, 5th lift – 0.75 ft thick (July 19, 2006).  3rd of native soil (CL) lifts.
As the lift number increased, so did the lift thickness. The increased lift thickness, from 0.75 feet to 1.25 feet and 1.5 feet resulted in an increased lateral force on the timber face forms. This added force made removing the forms too difficult to do by hand. Eye hooks were installed in the forms, enabling the skid-loader to lift the forms using a chain hooked to the bucket (Figures 6.13 and 6.14).

Figure 6.13 Preparing to remove timber face form using a chain hooked to the skid-loader bucket (July 19, 2006)
The density of soil lifts #7 and #8 after compaction were monitored using a nuclear density gauge (Figures 6.15 and 6.16). To perform the density measurement, a rod is driven into the compacted soil to create a hole for the gauge. The gauge is placed over the hole and a rod is extended from the gauge into the hole and a measurement of the moisture content and density of the backfill soil is performed (ASTM D2922).

Completion of lift #8 included placing NW-SB geotextile along the existing four foot wall and repairing the original drainage system resting on the footing. This area was then backfilled with gravel and covered with native backfill soil (Figure 6.17).
Figure 6.15  Driving rod to create hole for nuclear density gauge (July 20, 2006)

Figure 6.16  Initiating nuclear density test on backfill soil (July 20, 2006)
In the morning on Friday, July 21, 2006 the ninth and final lift was ready to be placed. Shortly after work began, precipitation followed. The precipitation event lasted approximately one hour, but saturated the site so that work could not continue. The excavation was not covered and no measures were established to divert runoff away from the reinforced soil mass. The roof on the adjacent house had no guttering system, and roof runoff water was diverted directly into the excavation. After the rainfall ceased, a temporary soil dam was built around the excavation so that a majority of the runoff would bypass the excavation and flow down gradient away from the wall. Figures 6.18 through 6.21 show conditions before, during, and after the rainfall event are shown.
Figure 6.18  Approaching storm at project site before 9th lift was finished (8:30 am, July 21, 2006)

Figure 6.19  Ponded water in excavation due to storm event (9:30 am, July 21, 2006). Note native soil (CL) shows evidence of poor drainage.
Figure 6.20 Construction of temporary dam to divert surface water from excavation (July 21, 2006)

Figure 6.21 Ponded water in drainage pipe excavation underneath existing footing - back side of wall (July 21, 2006). Note drainage pipe was not completed at this time.
On Friday, July 21, 2006, rainwater infiltrated the reinforced soil mass, and traveled through the drainage system (Figure 6.22). Although the precipitation was undesired and halted construction, the event showed that the drainage system constructed throughout the wrap-face wall was functioning as intended.

Figure 6.22 Profile view of drainage system beneath concrete wall footing

Construction of the final lift, #9, was completed on Monday, July 24, 2006. The addition of the lift brought the geotextile wrap-face wall height even with the top of the existing concrete retaining wall. Upon anticipated settlement, the final height of the wrap-face will likely be below the top of the concrete retaining wall.
Figure 6.23  Construction of final soil lift #9 (July 24, 2006)

Figure 6.24  Installation of deep geotextile overlaps for lift #9 (July 24, 2006)
Figure 6.25 Deep geotextile wrap being covered for final lift #9 (July 24, 2006)

Figure 6.26 Compaction of final lift #9 using skid-loader (July 24, 2006)
Figure 6.27  Grading of finished geotextile wrap-face wall (July 24, 2006)

Figure 6.28  Finished grading of final lift #9 – Wrap-face wall construction complete (July 24, 2006)
The final lift of the geotextile wrap-face wall was graded and compacted so that precipitation will flow away from the reinforced soil mass, not through it (Figure 6.26). This area will be sodded so that infiltration into the reinforced soil mass will be further reduced. Also, from Figure 6.28 it can be seen that the timber face forms are placed over the gap between the existing concrete wall and wrap-face wall to prevent anything from lodging in that area and mitigate geotextile UV exposure. A more permanent cover will be constructed and placed at a later date.

6.3 As-Built Conditions vs. Design Specifications

Several differences between the geotextile wall design specification and the as-built condition (Figure 6.29) are present due to the project conditions on-site. The total number of soil/gravel layers was increased from seven to nine. The layer thicknesses are thinner than originally specified. The bottom layer was utilized as a leveling pad and constructed using two-inch clean gravel, which was not specified in the original design specifications. The second layer, which was the first layer including geotextile reinforcement, was also constructed using clean gravel. While resting on the concrete wall footing, the geotextile reinforcement for the second layer wrapped around the four-inch diameter perforated pipe.
Geotextile overlap lengths of three feet were used for layers two, three, and four while five-foot overlap lengths were used for the remaining layers. The overlaps were increased midway through the wall construction to facilitate efficient wall construction. Lateral overlaps were also increased from one foot, to approximately two feet to maintain the desired overlap lengths even after shifting of the geotextile during installation.

A majority of the differences between the design and as-built conditions were found in the drainage system. The original drainage system consisted of perforated drainage pipe resting on the concrete wall footing and exiting on the west side. The under drain would tie into a proposed sand layer that would allow infiltrated water to enter the drainage pipe and flow out of the reinforced soil mass.
The as-built drainage system also consists of a perforated pipe running on top of the concrete wall footing, but there is a one-foot thick underdrain located below this level and the pipe exit is at a lower elevation to drain the underdrain, enabling infiltrated water to drain down slope (Figure 6.29 and 6.31).

All of these changes should effectively increase the stability of the geotextile wrap-face wall. The increased number of geotextile layers decreases the tension force in each geotextile layer. Increased geotextile overlaps decrease the potential of failure of the wrapped geotextile face due to unraveling. The drainage systems modifications reduce horizontal seepage forces on the reinforced fill, and help to maintain the shear resistance of the reinforced soil.

Figure 6.30  As-built geotextile wrap-face wall drainage system
The bottom two feet of the geotextile wall are constructed of two-inch clean gravel that permit infiltrated water to enter the drainage pipe and exit the reinforced soil mass. NW-SB geotextile was also placed on the east and west sides of the geotextile wall to transmit infiltrated water out of the system. The west side of the concrete wall was also backfilled with gravel to further increase the flow to the drainage pipe, and decrease pore water pressure buildup behind the wall. Positive pore water pressures produce a horizontal seepage force on the reinforced fill that decreases the stability, and reduces the shear resistance of the reinforced soil fill. Studies show that the total force against a retaining wall can be twice that of a properly drained reinforced fill soil (Christopher and Stulgis, 2005). Without drainage, the calculated horizontal effective stress per foot of wall in the long term condition increased by approximately 40 percent in the geotextile wrap-face wall.

### 6.3a. Change in Stability Resulting from As-Built Conditions

An increase in the safety factor, compared to the original geotextile wall design, was incorporated in the construction of the geotextile wrap-face wall as a result of the construction differences. The change in stability was calculated using Equation 6.1.

\[
\Delta \text{Stability} = \left( \frac{(As - built) - (As - designed)}{(As - designed)} \right) \times 100 
\]

(Equation 6.1)

A negative value correlates to a decrease in stability while a positive value correlates to an increase in wall stability compared to design. The changes in stability from the designed condition to the as-built condition are shown in Table 6.2, 6.3, and 6.4.
### Table 6.2 Change in stability of geotextile wall resulting from as-built geotextile vertical spacing

<table>
<thead>
<tr>
<th>Depth, $z$ (ft)</th>
<th>Design, $S_v$ (ft)</th>
<th>As-built, $S_v$ (ft)</th>
<th>Change in Stability</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>2.0</td>
<td>2.0</td>
<td>1.5</td>
<td>+25%</td>
</tr>
<tr>
<td>3.5</td>
<td>1.5</td>
<td>1.25</td>
<td>+17%</td>
</tr>
<tr>
<td>5.0</td>
<td>1.5</td>
<td>1.25</td>
<td>+17%</td>
</tr>
<tr>
<td>6.0</td>
<td>1.0</td>
<td>0.75</td>
<td>+25%</td>
</tr>
<tr>
<td>7.0</td>
<td>1.0</td>
<td>0.75</td>
<td>+25%</td>
</tr>
<tr>
<td>8.0</td>
<td>1.0</td>
<td>0.75</td>
<td>+25%</td>
</tr>
<tr>
<td>9.0</td>
<td>1.0</td>
<td>1.0</td>
<td>0%</td>
</tr>
</tbody>
</table>

+ = more stable, - = less stable

### Table 6.3 Change in stability of geotextile wall resulting from as-built due geotextile runout lengths

<table>
<thead>
<tr>
<th>Depth, $z$ (ft)</th>
<th>Design $L_e+L_r$ (ft)</th>
<th>As-built $L_e+L_r$ (ft)</th>
<th>Change in Stability</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>2.0</td>
<td>10</td>
<td>10</td>
<td>0%</td>
</tr>
<tr>
<td>3.5</td>
<td>9</td>
<td>10</td>
<td>+11%</td>
</tr>
<tr>
<td>5.0</td>
<td>7</td>
<td>9</td>
<td>+29%</td>
</tr>
<tr>
<td>6.0</td>
<td>6</td>
<td>7</td>
<td>+17%</td>
</tr>
<tr>
<td>7.0</td>
<td>5</td>
<td>5</td>
<td>0%</td>
</tr>
<tr>
<td>8.0</td>
<td>4</td>
<td>4</td>
<td>0%</td>
</tr>
<tr>
<td>9.0</td>
<td>4</td>
<td>--</td>
<td>--</td>
</tr>
</tbody>
</table>

+ = more stable, - = less stable
Table 6.4 Change in stability of geotextile wall resulting from as-built geotextile overlap lengths

<table>
<thead>
<tr>
<th>Depth, z (ft)</th>
<th>Design Lo (ft)</th>
<th>As-built Lo (ft)</th>
<th>Change in Stability</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>2.0</td>
<td>3</td>
<td>5</td>
<td>+67%</td>
</tr>
<tr>
<td>3.5</td>
<td>3</td>
<td>5</td>
<td>+67%</td>
</tr>
<tr>
<td>5.0</td>
<td>3</td>
<td>5</td>
<td>+67%</td>
</tr>
<tr>
<td>6.0</td>
<td>3</td>
<td>5</td>
<td>+67%</td>
</tr>
<tr>
<td>7.0</td>
<td>3</td>
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<td>+67%</td>
</tr>
<tr>
<td>8.0</td>
<td>3</td>
<td>3</td>
<td>0%</td>
</tr>
<tr>
<td>9.0</td>
<td>3</td>
<td>3</td>
<td>0%</td>
</tr>
</tbody>
</table>

+ = more stable, - = less stable

Due to a decrease in the geotextile vertical spacing, a 17 to 25 percent increase in the geotextile wall stability was calculated. Extending the geotextile runout lengths also increased the stability of the geotextile wall, with values ranging from 11 to 29 percent in layers three through five. Other geotextile layers were constructed with design specified runout lengths. Increased geotextile overlap lengths in all but the bottom two layers increase the geotextile wall stability by 67 percent in this area of construction. The combination of increased stability of each aspect of the wall increases the internal stability of the geotextile wrap-face wall.

The external stability of the as-built geotextile wrap-face wall also increased due to the changes in design (calculations shown in Appendix J).
Table 6.5  Increase in geotextile wall external safety factors due to designed and as-built conditions

<table>
<thead>
<tr>
<th>Stability Condition</th>
<th>Designed Safety Factor</th>
<th>As-built Safety Factor</th>
<th>Percent Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overturning</td>
<td>5.0</td>
<td>6.6</td>
<td>32%</td>
</tr>
<tr>
<td>Sliding</td>
<td>1.1</td>
<td>1.8</td>
<td>64%</td>
</tr>
</tbody>
</table>

The geotextile wall factor of safety against overturning increased from 5.0 to 6.6 due to the changes made during construction. The factor of safety against sliding of the geotextile wall also increased from 1.1 to 1.8. The factors of safety against shallow bearing capacity failure between the as-built and design do not significantly differ.

6.3b  External Stability of Concrete Wall With Geotextile Wall Contact

During geotextile wall construction, unraveling occurred of some soil layers. Due to the partial unraveling, the two walls came in contact with one another in various sections on the wall face. To examine changes in the external stability of the concrete wall due to geotextile wall loading, the concrete wall was re-analyzed with the geotextile wall situated behind it using two different scenarios (calculations shown in Appendix K). The scenarios included: each layer of the geotextile wall in contact with the concrete wall and half of geotextile layers in contact with the concrete wall. The lateral forces on the concrete wall due to geotextile layers were calculated assuming a failure surface for each layer acting at an angle of 45 degrees from the horizontal at each layer height (Figure 6.31). Safety factors against overturning and sliding for each scenario are presented in Table 6.6.
Figure 6.31  Horizontal forces on concrete wall due to contact with geosynthetic wall

Table 6. 6  Overturning and sliding safety factors of concrete wall with partial contact of geotextile wall

<table>
<thead>
<tr>
<th>Stability Condition</th>
<th>Overturning w/ all GT layers in contact w/ concrete wall</th>
<th>Overturning w/ half GT layers in contact w/ concrete wall</th>
<th>Sliding w/ all GT layers in contact w/ concrete wall</th>
<th>Sliding w/ half GT layers in contact w/ concrete wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor of Safety</td>
<td>0.61</td>
<td>1.22</td>
<td>1.89</td>
<td>3.78</td>
</tr>
</tbody>
</table>
When all of the geotextile layers are in contact with the concrete wall, overturning failure of the concrete wall was calculated (FS = 0.61). When only half of the geotextile layers are in contact with the concrete wall the overturning safety factor doubles to approximately 1.2. This approximate condition was verified in the field (Figure 6.32).

Table 6.32 Gap between geotextile wall and concrete wall (July 24, 2006)

Throughout the geotextile wall face, several areas were in contact with the concrete wall, but not all. Approximately half, or less, of the geotextile wall face area was found to be in contact with the concrete wall.

The sliding potential of the concrete is not as critical as the overturning potential. When all layers of geotextile wall are in contact with the concrete wall, a safety factor of 1.89 was calculated, thus the potential for sliding failure of the concrete wall is minimal compared to overturning.
6.4 Summary of Geotextile Wrap-Face Wall As-Built Conditions

The construction of the geotextile wrap-face wall using marginal backfill was completed July 17 – 24, 2006 in Boone County, Missouri. The as-built conditions differ from design. Differences include: geotextile layers, overlaps, and drainage features. In general, the differences increase the stability by 32 and 64 percent, against overturning and sliding failure, respectively. Although the concrete wall is calculated to be stable, several areas on the geotextile wall face are in contact with the existing concrete wall. The concrete wall was calculated to be stable (from overturning failure) if approximately half or less of the geotextile wall layers are in contact with the concrete wall face.
Chapter 7: Field Performance Monitoring System

7.1 Introduction

A system was developed to monitor the performance of the geotextile wrap-face wall using marginal backfill. The monitoring system utilizes the existing concrete wall as an indicator as to how the geotextile wall is performing. The design included a gap between the two walls; however, immediately after construction the geotextile and concrete walls were in contact. As the geotextile wall deforms, the gap width decreases and additional contact between the walls is increased. In areas on the wall face where the geotextile wall deformation is greater than the gap width, lateral loading of the concrete wall occurs. Lateral loading of the concrete wall is indicative of new cracks in concrete wall, existing crack width, inclination and translation of the concrete wall. Other geotextile wall performance techniques include: monitoring the drainage system for clogs and standing water, performing a water balance of infiltrated water, and an areal elevation survey of the geotextile wrap-face wall surface.

7.2 Concrete Wall Crack Monitoring

Before construction of the geotextile wall was initiated a survey was made of the existing cracks in the concrete wall. The crack locations were labeled and the crack widths were measured (Figure 7.1). Tell-tale® crack gauges were installed on six existing cracks. A close-up view of a gauge is shown in Figure 7.3. The gauges are installed by placing the gauge across an existing crack, and quick setting epoxy was used to adhere the gauge to the concrete wall. The gauge is made up of two plastic parts that are taped together during installation.
When installation of the gauges is complete, the tape is cut and each piece of the gauge is allowed to move freely with concrete wall in which it is attached.

A grid on the gauge shows the movement of the crack in the horizontal and vertical direction within the plane of the wall face. Figures 7.2 through 7.8 show the location and placement of each crack gauge on the concrete wall. Further deformation of the geotextile wall can increase the lateral load on the concrete wall, which will impact the orientation or width of the existing concrete wall cracks. Periodic monitoring of the crack gauges is used to assess the performance of the geotextile wrap-face wall.
Figure 7.2 Crack gauge locations on concrete wall

Figure 7.3 Crack gauge #1 on west face of concrete wall (July 18, 2006)
Figure 7.4  Crack gauge #2 on west face of concrete wall (July 18, 2006)

Figure 7.5  Crack gauge #3 on southwest corner of concrete wall (July 18, 2006)
Figure 7.6 Crack gauge #4 on south face southwest corner of concrete wall (July 18, 2006)

Figure 7.7 Crack gauage #6 on south face of concrete wall (July 18, 2006)
7.3 Concrete Wall Inclination Monitoring

The inclination of the concrete wall at twelve locations on the concrete wall face was performed prior to geotextile wall construction (Figure 7.9). Lateral loading of the concrete wall can change the inclination of the concrete wall face. A four foot level was used to measure the distance off of the vertical plane at each crack gauge location (Figure 7.10). The level was adjusted by moving either end toward or away from the concrete wall face until a vertical plane was found. The distance from the level end away from the wall face on the vertical plane to the concrete wall face was measured and recorded. Periodic measuring of this distance enables detection of increased lateral loading of the concrete wall from the geotextile wall.
Figure 7.9 Inclination survey points on concrete wall face

Figure 7.10 Concrete wall inclination monitoring at crack gauge location #1 (July 18, 2006)
7.4 Translation Monitoring of Concrete Wall

Monitoring the translation of the concrete wall provides information on deformation of the concrete wall caused by lateral loading due to the geotextile wall. The translation monitoring system uses known locations near the concrete wall to document wall movement. The locations are situated so that movement of the wall does not affect the point locations. Multiple points with a known relation are established so that a point can be re-located if destroyed by construction activities in the area. A vertical line over the ground point is established using a tripod and plumb bob (Figure 7.11).

Figure 7.11  Establishing a vertical line over control point using tripod and plumb bob
(July 25, 2006)
Points on the concrete wall face at known angles were established using a survey level. A tape measure was then used to measure the distance between the point location and point on the wall at the same elevation. Three points on the concrete wall face were measured so that movement along the wall face can be monitored, and are labeled lines one through three (Figure 7.12). Periodic measurements of these distances show concrete wall translation with time.
7.5 Visual Inspection of Drainage System

The drainage system constructed throughout the geotextile wall has experienced several flow events. After a rainfall event, the drainage system is visually inspected to see if infiltrated water is exiting, or has exited, the system. Flow out of the drainage system ensures that some drainage is occurring within the geotextile wall, and pore water pressures are not building in the reinforced soil mass.

7.6 Visual Settlement Survey

The surface above the geotextile wall is anticipated to settle with time. Settlement occurs due to a decrease in void ratio of the reinforced soil mass. Settlement in relation to the existing concrete wall was visually inspected during each monitoring site visit.

Photographs taken immediately after construction are compared to photos of the site during periodic monitoring visits. The ground contours and total height of soil surface are indicators to how much, if any, the reinforced mass has settled.

It is desired to establish a five foot by five foot monitoring grid above the reinforced soil mass using a site level and rod. Topographical contours of the surface above the reinforced soil mass would be defined using this method. The north west corner of the porch slab on the north side of the house was used as a benchmark. The porch is assumed to be relatively stable, with no significant vertical movements occurring throughout the monitoring period. A survey of the grid taken periodically would provide data to assess the total and local settlement of the reinforced soil mass. Due to changes in the soil surface, such as periodic grading and sodding, the procedure above could not be practically implemented.
7.7 Summary of Field Monitoring System

A monitoring system was established to record the movements of the concrete and geotextile wall. Several monitoring techniques use the performance of the concrete wall as a performance indicator for the geotextile wrap-face wall. Existing cracks in the concrete wall are measured, photographed, and instrumented with Tell-tale® crack gauges to monitor the movement of the cracks. The inclination of the concrete wall was defined at various points along the concrete wall to monitor the verticality of the concrete wall face. A system was implemented to monitor the translation of the concrete wall face in relation to a known points away from the wall. The drainage system is visually inspected after rainfall events to monitor flow through the system. A visual settlement survey examines the amount of settlement occurring in the reinforced soil mass. These methods provide deformation data of the geotextile wrap-face wall that are used to monitor the performance of a geotextile wrap-face wall using marginal backfill.
Chapter 8: Geotextile Wrap-Face Wall Field Performance

8.1 Introduction

Periodic monitoring of the geotextile wrap-face wall is an integral component in assessing the performance of the structure. The instrumentation and visual performance surveys on the project site allow the movements of the combination wall structure to be followed. Each monitoring tool provides a different type of information related to the deformation of the concrete and geotextile walls. An analysis of data obtained from each monitoring tool is presented in the following section.

8.2 Climate and its Affect on Geotextile Wall Performance

The climate in which a structure is located has a large impact on the performance over time. Factors such as temperature and precipitation influence the interaction between the geotextile wall and its environment. Low temperatures can cause freezing of water within the geotextile wall, which can clog the drainage system. A clogged drainage system can lead to the buildup of pore water pressures within the reinforced soil mass behind the wall face. Precipitation can also negatively impact the performance of geotextile wall. Infiltration into the reinforced soil mass causes restructuring of the soil particles, leading to settlement. This settlement increases the tension within the geotextile reinforcement and can cause deformation of the wall face. Deformation of the geotextile wall face can load the concrete wall and reduce its stability. Precipitation can also lead to the buildup of excess water pressures if the drainage system is not properly functioning.
Temperature and precipitation data has been obtained for the area going back 16 months for Columbia, Missouri at the Sanborn Field Station (Figures 8.1 and 8.2) (http://agebb.missouri.edu). The climatic data show a trend in average precipitation. Greater volumes of precipitation occur in the early Fall and Spring. This time period is within a couple months following the geotextile wall construction. As a result, the wall drainage system will be tested early in the life of the structure. High average temperatures were experienced during geotextile wall construction. The temperature trend shows that freezing temperatures are probable during the first six months after construction. This climatic trend could also negatively affect the wrap-face wall performance if the drainage system is not functioning properly. Actual daily precipitation amounts before, during, and after geotextile wall construction are shown in Figure 8.3.
Figure 8.1 Monthly cumulative precipitation for Columbia, MO (Sanborn Field)

Figure 8.2 Monthly average high and low temperature for Columbia, MO (Sanborn Field)
The first significant flow experienced by the wall drainage system occurred on July 21, 2006, before wall construction was complete. Other significant flow events occurred within approximately one month post construction, and were as high as two inches per day. Monitoring of the drainage system following these events proved that the system is transmitting a significant amount of infiltrated precipitation through drainage network, and downgradient of the reinforced soil mass. Quantifying the inflow volume as compared to the outflow volume would aid in analyzing drainage system performance, but complete installation of the drainage exit is not yet complete, at the time of submittal.

**8.3 Performance Assessment – Crack Monitoring**

Deformations of the geotextile wall face exceed the gap width between the geotextile wall and the concrete wall exert a lateral load onto the concrete wall, causing slight movements of the concrete wall that can be detected by monitoring.
Existing cracks in the concrete wall are an indicator of geotextile wall performance. As previously mentioned, five crack gauges were placed on the concrete wall face, and one gauge was placed on the east side concrete wall top. Monitoring of the crack gauges located on the concrete wall face indicate that little movement within the cracks has occurred (Figures 8.4 and 8.5). Maximum vertical displacement of the wall face of up to 0.5 millimeters was detected in all crack gauges. No horizontal movement in crack gauges three, four, and five located on the concrete wall face was detected. Horizontal movements of 0.5 and 0.25 millimeters was detected in crack gauges one and two, respectively. Different gauges with equal movements are indistinguishable in Figures 8.4 and 8.5, due to being stacking of the data points.

Figure 8.4 Vertical crack gauge movement on wall face
Crack gauge number six, located on the east side of the concrete wall top, experienced greater movements than the other five gauge locations (Figure 8.6).
Movement of crack gauge number six is defined as parallel, movement along the length of the concrete wall, and perpendicular, movement into the plane of the concrete wall face. Maximum crack displacement in the parallel direction detected by the crack gauge at this location was 0.5 millimeters. The greatest displacement detected by all gauges was at gauge number six in the perpendicular direction, with a value of 1.75 millimeters. Figure 8.7 shows crack gauge number six and the associated crack enlargement due to the wall movement.

The crack gauges detect slight movements of the concrete wall cracks. These movements may indicate that deformation of the geotextile wall have transferred some lateral load onto the concrete wall, but could also be caused from settlement of the structure or movements generated from temperature effects.
Although movements were detected, the movements thus far are slight, expected, and appear to be non-critical in the overall stability of the concrete wall. The development of additional concrete wall cracks has not been observed post construction throughout the monitoring period.

![Figure 8.7 Crack gauge #6 and associated crack enlargement (September 20, 2006)](image)

**8.4 Performance Assessment – Concrete Wall Inclination Survey**

The inclination of the concrete wall face immediately following construction was measured on July 24, 2006. Periodic monitoring of the wall indicates changes in inclination in the months following geotextile wall construction (Figures 8.8 and 8.9). Maximum changes in concrete wall inclination occurred at test points number four and eight, with a change in inclination of 0.36 and 0.24 degrees, respectively, from the initial measurement recorded.
Figure 8.8 Inclination change with time of concrete wall face at test points 1 – 6

Figure 8.9 Inclination change with time of concrete wall face at test points 7 - 12
In the case of test point number four, the wall is leaning further out, away from the geotextile wall, while the wall is leaning further in, toward the geotextile wall, at test point number eight.

Changes in concrete wall inclination indicate that deformation of the geotextile wall is occurring, and the concrete wall is carrying a lateral load due to the geotextile wall as a result. Monitoring will continue to determine further changes in concrete wall inclination.

8.5 Performance Assessment – Concrete Wall Translation Survey

Translation is movement of the concrete wall perpendicular to the wall face. Using the control point, three lines along the concrete wall face are measured periodically (Figure 8.10).

![Figure 8.10](image-url) Translation of concrete wall measured during each monitoring date
No significant translational movement of the concrete wall has occurred during the monitoring period. Slight changes in line measurements are due to the precision in the translation measuring procedure.

8.6 Performance Assessment – Visual Observation of Drainage System

The drainage system in the geotextile wall has experienced 13 rainfall events thus far (7/06 – 10/06). Two of these events were larger than 1.5 inches per day. Following these events, the drainage outlets were transmitting water downgradient from the reinforced soil mass. These observations indicate that the geotextile wall drainage system does perform properly, and pore water pressures are less likely to develop within the reinforced soil mass.

8.7 Performance Assessment – Visual Settlement Survey

A visual inspection of the reinforced soil mass surface is performed on each monitoring visit. When the reinforced soil mass surface, immediately after construction (Figure 8.11), is compared to the surface two months post construction (Figures 8.12 and 8.13), minimal differences in surface elevation and topography are noticeable. Although vegetation has taken root on the surface, which conceals the view of minor elevation differences, one difference between the two photographs is apparent. Erosion on the north, center portion of the backfill area has caused a slight depression in the area. This area is away from the reinforced soil mass, and does not appear to be caused by any settlement of the reinforced backfill. Overall, slight settlement of the reinforced soil mass appears to have occurred. Minimal to no differential settlement over the reinforced soil mass surface is apparent.
Figure 8.11 Reinforced soil mass surface once construction was complete (July 24, 2006)

Figure 8.12 View looking southwest of reinforced soil mass surface with vegetation two months post construction (September 20, 2006)
8.8 Summary of Geotextile Wall Performance Assessment

The monitoring procedures and instrumentation implemented all indicate excellent performance of the geotextile wrap-face wall. The concrete wall crack survey showed slight movements in the existing concrete wall cracks, but no development of new cracks post construction of the geotextile wall. Monitoring the inclination of concrete wall face shows some inclination changes post geotextile wall construction, but is considered satisfactory in correlation with the concrete wall performance. The translation survey indicated no translational movement of the concrete wall. Multiple rainfall events proved drainage capability of the geotextile wall drainage network by collecting infiltrated water and transporting it downstream from the reinforced soil mass. Visual monitoring shows minimal settlement of the reinforced soil mass surface after
three months. Some monitoring data indicate lateral loading of the concrete wall, but overall, performance of the geotextile wrap-face wall is considered satisfactory.
Chapter 9: Project Conclusions and Recommendations

9.1 Project Summary

An existing concrete retaining wall was constructed at a residence in Rocheport, Missouri and exhibited unsatisfactory performance. A geotextile wrap-face wall, located behind the existing wall, was designed and constructed to carry the lateral load from the backfill, while the concrete wall remained in place. The wrap-face wall was designed using on-site cohesive, or marginal, backfill. Laboratory testing was performed to establish the design parameters of the geotextile reinforcement and backfill soil. The wall was constructed using nine layers of geotextile reinforcement. The as-built geotextile wall differed slightly from the original design. These differences increased the stability of the structure. Procedures and instrumentation to monitor the performance of the wrap-face wall were implemented. Monitoring has continued approximately three months post-construction, indicating satisfactory performance of the geotextile wall using marginal backfill.

9.2 Project Conclusions

Considering the project site conditions, and the importance of the structure, a geotextile wrap-face wall using marginal backfill was a logical site remediation. The construction process of a wrap-face wall was conducive to the existing geometry dictated by the existing concrete wall. Incorporating marginal material as the wall backfill, resulted in an approximate 55 percent savings (Appendix L), when compared to conventional granular backfill.
Laboratory testing of the construction materials established the design parameters for the wrap-face wall. Index tests indicate that the on-site backfill is a low-plasticity clay, with a maximum dry unit weight and optimum moisture content of 105 pounds per cubic foot, and 16 percent, respectively. Interface friction tests between the backfill soil and geotextile reinforcement indicated ranges for the shear strength parameters, cohesion and angle of interface friction, of 1.0 – 2.5 pounds per square inch, and 30 – 50 degrees, respectively.

The initial wrap-face wall design had seven layers of geotextile reinforcement with geotextile lengths ranging from four to ten feet and extending from the geotextile wall face. The drainage system also had one exit that was located on the west side of the concrete wall. Field construction changes of the wall resulted in nine layers of geotextile reinforcement with geotextile lengths ranging from four to eleven feet and extending from the concrete wall face. A new drainage system was constructed and utilized the narrow concrete wall base to install a second drainage system exit on the south side of the excavation. Due to the differences between initial design and field construction, internal and external stability of the geotextile wall was increased.

A monitoring plan was established to assess the performance of the geotextile wrap-face wall post construction. Many of these procedures use the existing concrete wall as a geotextile wall movement indicator. The monitoring data show that little movement of the concrete wall has occurred since construction of the geotextile wall. Performance of the geotextile wall is satisfactory thus far, approximately three months after construction. The concrete wall movements are minimal, indicating that the
majority of the lateral load induced by the backfill soil is being carried by the geotextile wrap-face wall.

Marginal soil utilized as MSE wall backfill can provide satisfactory performance if properly designed. Proper design using marginal backfill must incorporate aggressive drainage on all sides of the wall. Increased pore pressures, due to low permeability, drastically increase the lateral load on the wall. Instrumentation and monitoring of the wall and drainage system should be implemented, and used as indicators for loading of the structure. A properly designed MSE wall using marginal backfill, as compared to high quality backfill, is more economical, while matching the same level of satisfactory performance.

9.3 Recommendations when Using Marginal Backfill in a Wrap-Face Wall

As in any retaining structure, the buildup of water pressures can significantly increase the stresses exerted on the structure face. As a result, drainage of the backfill is a key factor affecting the performance of the structure. It is recommended to incorporate aggressive drainage within, and surrounding the backfill soil. Freely draining, or granular, backfill materials do not require the degree of drainage as compared to marginal backfill. Although much effort was incorporated into installing an adequate drainage system on the above project, increased efforts would yield a more efficient drainage system. These efforts include installing a trench drain upgradient of the reinforced mass that does not allow infiltrated water to enter the system. The underdrain should also extend out away from the wall face as far as possible, bringing it closer to the ground surface.
Measures such as these would decrease the likelihood of infiltrated water increasing the positive pore water pressure within the reinforced soil mass, causing increased deformation of the wrap-face wall.

The underdrain is installed prior to placing backfill. As a result a non-stable working surface is formed, making it difficult for the movement of machinery. To improve the working surface and decrease damage to the underdrain it is recommended to install the underdrain in segments, as backfilling of the excavation progresses. This allows for increased construction efficiency and a safer working environment. It should be noted that care should be taken when placing underdrain segments to install sufficient overlaps to create a continuous drainage path.

Prior to backfill placement, controls should be constructed to divert run-off away from the excavation, especially during construction when soil layers are exposed. Roof water from any nearby structures, if any, should be controlled. The project site should be prepared for rain so that surrounding area run-off does not drain directly into the excavation.

Deformation of the geotextile wall face is a performance indicator that can be drastically improved with proper construction techniques. After placing backfill on any geotextile layer, the wrap-around length should be tensioned, pulled tight as possible, to reduce post construction deformations at the wall face. Practices aiding in this task include thinner placement of soil layers, placing the wrap-around tail deep, or farther away from the wall face, as possible, and use finer, containing no large clods, material in this location.
Compaction of the backfill is another key construction component that influences wall performance. The best possible compaction equipment should be used for the project. Multiple sets of compaction face forms should be constructed and utilized to aid in compaction of the wall face. Multiple form sets prevent layers below the current layer from unraveling during compaction.

When deformation of the structure is critical, monitoring instrumentation should be installed. Settlement of the ground surface above the reinforced soil mass should be monitored, as well as deformation of the wall face. Deformation monitoring of the wall face could be accomplished by installing extensometers on the wall face (Figure 9.1), or performing a survey of the wall face using a total station. This information is valuable when assessing wall performance.

Figure 9.1 Example extensometer device to measure geotextile wall deformation
References


Missouri Agricultural Experimental Station – Sanborn Field. MU College of Agriculture, Food, and Natural Resources. 
http://agebb.missouri.edu/weather/stations/boone/index.htm


Appendix A: Preliminary Concrete Wall Stability Analysis

Preliminary Stability Calculations of Concrete Wall

Assuming $\gamma = 100 \text{ psf} (< L. s. o.)$

$\gamma_n = \gamma / 2 = 50 \text{ psf}$

Weight, Stem:

$W_t = H \times W \times 1 \times \gamma_{concrete} = 8' \times 0.83 \times 1' \times 150 \text{ psf} = 996 \text{ lb/linear ft}$

Weight, Footing:

$W_f = B \times D_f \times 1' \times \gamma_{concrete} = 2' \times 1' \times 1' \times 150 \text{ psf} = 300 \text{ lb/linear ft}$

Total Weight of Wall = $W_{total} = 996 + 300 = 1296 \text{ lb/linear ft.}$
\[
P_r = \int_0^z z\,dz = \frac{z^2}{2}
\]

\[
= \frac{1}{2} \cdot 2 \int_0^z z\,dz = \frac{z^2}{2}
\]

\[
= \frac{1}{2} \cdot 2 \cdot 2 = 2 \cdot 2 = 4
\]

\[
W_c = 1250 \text{ lb/ft}
\]

\[
W_d = 100 \text{ psf} \cdot 3^2 = 900 \text{ psf}
\]

**Assuming Fixed Soil:**

- C: 2 psf
- q: 0 ft
- LL: 42°
- D2: 116°

**FS_{st}**

\[
FS_{st} = \frac{\sum \text{Masonry}}{W_{Masonry}} = \frac{467 \left( \frac{20.5}{16} \right) + 1256(2) \cdot 50}{3200 (5.67)} = 0.18
\]

**FS_{sl}**

\[
FS_{sl} = \frac{\sum \text{Reinforcing}}{W_{Reinforcing}} = \frac{256 (2 \times 24) + (324 \times 144)(\text{sec} 24)}{3200 (8.12)} = 0.40
\]

**FS_{bc}**

\[
FS_{bc} = \frac{q_{ut}}{q_{ut}} \Rightarrow q_{ut} = C \cdot N_c + q \cdot N_e + \frac{1}{2} \cdot Y \cdot N_g
\]

Assuming \( C = 1000 \text{ psf} \) (undetermined)

\[
q_{ut} = 1000 \text{ psf} \cdot (5) = 5000 \text{ psf}
\]

\[
FS_{bc} = \frac{5000}{882} = 5.7
\]

**FS_{sw}**

Assuming stable (but will check later)
Appendix B: Atterberg Limit Test Results

### Liquid Limit

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<td>42.0%</td>
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### Plastic Limit

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<tr>
<td>Wt. dish</td>
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<td>11.06</td>
<td>10.96</td>
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Plastic Limit = 25.6

**Liquid Limit = 41.8**

**Plastic Limit = 25.6**

**LL = 41.8**

**PL = 25.6**

**PI = 16.2**
## Appendix C: Compaction Test Results

### Compaction Test Data Sheet

**Wrap-face Retaining Wall**
- **Energy Level:** 50% Standard Proctor

#### Density

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<td>3451.2</td>
<td>3638.3</td>
<td>3566.8</td>
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#### Moisture Content

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<tr>
<td>Wt. Wet Soil + Cont. (g)</td>
<td>133.8</td>
<td>110</td>
<td>122.4</td>
<td>102.8</td>
</tr>
<tr>
<td>Wt. dry soil + Cont. (g)</td>
<td>125.2</td>
<td>100.66</td>
<td>109.59</td>
<td>88.69</td>
</tr>
<tr>
<td>Wt. Water (g)</td>
<td>8.51</td>
<td>9.34</td>
<td>12.81</td>
<td>14.11</td>
</tr>
<tr>
<td>Wt. Dry Soil (g)</td>
<td>94.19</td>
<td>70.36</td>
<td>78.09</td>
<td>57.79</td>
</tr>
<tr>
<td>Moisture Content (%)</td>
<td>9</td>
<td>13.27</td>
<td>16.4</td>
<td>24.4</td>
</tr>
</tbody>
</table>

---

**Wrap-face Retaining Wall**
- **Energy Level:** 100% Standard Proctor

#### Density

<table>
<thead>
<tr>
<th>Test #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wt. Mold + Wet Soil (lb)</td>
<td>11.6</td>
<td>12.1</td>
<td>11.6</td>
<td>12.1</td>
</tr>
<tr>
<td>Wt. Mold (lb)</td>
<td>8</td>
<td>8</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>Wt. Wet Soil (lb)</td>
<td>3.6</td>
<td>4.1</td>
<td>3.6</td>
<td>4.1</td>
</tr>
<tr>
<td>Total Unit Wt. (lb/ft^3)</td>
<td>108</td>
<td>123</td>
<td>108</td>
<td>123</td>
</tr>
<tr>
<td>Moisture Content (%)</td>
<td>0.1</td>
<td>0.219</td>
<td>0.116</td>
<td>0.169</td>
</tr>
<tr>
<td>Dry Unit Wt. (lb/ft^3)</td>
<td>98</td>
<td>101</td>
<td>97</td>
<td>105</td>
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</tbody>
</table>

#### Moisture Content

<table>
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<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wt. Cont. (g)</td>
<td>31</td>
<td>31</td>
<td>30.8</td>
<td>31</td>
</tr>
<tr>
<td>Wt. Wet Soil + Cont. (g)</td>
<td>143.9</td>
<td>155.8</td>
<td>143.74</td>
<td>163.64</td>
</tr>
<tr>
<td>Wt. dry soil + Cont. (g)</td>
<td>133.62</td>
<td>133.74</td>
<td>131.82</td>
<td>145.05</td>
</tr>
<tr>
<td>Wt. Water (g)</td>
<td>10.28</td>
<td>22.06</td>
<td>11.35</td>
<td>18.59</td>
</tr>
<tr>
<td>Wt. Dry Soil (g)</td>
<td>102.62</td>
<td>102.74</td>
<td>101.74</td>
<td>114.05</td>
</tr>
<tr>
<td>Moisture Content (%)</td>
<td>10.0</td>
<td>21.5</td>
<td>11.2</td>
<td>16.3</td>
</tr>
</tbody>
</table>
### Appendix D: Tilt Table Test Results

#### Specimen Data

<table>
<thead>
<tr>
<th>Specimen</th>
<th>weight (lb)</th>
<th>area (in^2)</th>
<th>height (in)</th>
<th>volume (ft^3)</th>
<th>γ/wet (pcf)</th>
<th>γ/d (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>19.4</td>
<td>95.03</td>
<td>3</td>
<td>0.165</td>
<td>117.6</td>
<td>97.2</td>
</tr>
</tbody>
</table>

#### Compaction Data

<table>
<thead>
<tr>
<th></th>
<th>Target</th>
<th>Actual</th>
</tr>
</thead>
<tbody>
<tr>
<td>γ/d =</td>
<td>94 pcf</td>
<td>97 pcf</td>
</tr>
<tr>
<td>w =</td>
<td>23%</td>
<td>21%</td>
</tr>
</tbody>
</table>

*50% Standard Proctor Compaction Energy

#### Tilt Table Data

<table>
<thead>
<tr>
<th>Weight (lb)</th>
<th>δ (degrees)</th>
<th>Normal Force (lb)</th>
<th>σn (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>45</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>45</td>
<td>46</td>
<td>31.3</td>
<td>47.4</td>
</tr>
<tr>
<td>90</td>
<td>45</td>
<td>63.6</td>
<td>96.4</td>
</tr>
</tbody>
</table>

![Estimated Interface Friction Angle vs Normal Stress](image)

Estimated Interface Friction Angle vs Normal Stress
Appendix E: Shear Rate Calculation

\[
\text{Shear Rate} = R = \frac{d_f \times t_{50} \times O}{50}
\]

*ASTMD5321*)

- **R** = rate of horizontal displacement (mm/min)
- **d_f** = estimated horizontal displacement at peak shear stress (mm)
- **t_{50}** = time needed to reach 50% consolidation (min)
- **O** = factor to account for drainage conditions on the shear plane
  - 1 (geosynthetic with low permeability or soil with low k)
  - 0.002 (geosynthetic with high permeability and a pervious soil)

\[
d_f = 0.7'' = 17.78 \text{mm}
\]

\[
t_{50} = 20 \text{min (estimated)}
\]

\[
O = 1 \text{ (soil with low k)}
\]

\[
R = \frac{17.78 \times 20 \times 1}{50} = 7.1 \text{ mm/min} = 0.28 \text{ in/min}
\]

Set shear rate at 0.5" per hour

\[
0.5'' = 0.00833 \text{ in} = 0.211 \text{ mm/min} < 7.1 \text{ mm/min}
\]

\[
R = 0.00833 \text{ in/min}
\]
Appendix F: Interface Friction Tests by Direct Shear

Soil/Geotextile Interface Friction Test by Direct Shear - Cross Machine Direction

\[ \phi = 32 \text{ degrees} \]
Soil/Geotextile Interface Friction Test by Direct Shear - Machine Direction

\[ \phi = 50 \text{ degrees} \]
Soil/Geotextile Interface Friction Test by Direct Shear - (Submerged) Cross Machine Direction

\[ \phi = 30 \text{ degrees} \]
Appendix G: Design of Geotextile Wrap-face Wall –
Internal Stability

*Following example (Kearns, 1998) p. 187

**Assumptions:** In situ, low plasticity clay backfill

- γ = 115 psf
- φ = 0
- γ\text{geotextile} = 40 kN/m \text{^2}
- C = 2 psi = 288 psf

**Lab Testing:**
- LL = 42
- PL = 26
- PI = 16
- classifies as CL

**Geotextile:** Procop 2044 (High Performance/Reinforcement)

- Thickness (waterbar) = 400 lb/lin ft (70 kN/m)

- 8 rolls (current) = 9' x 150' (delivered 3/1/06) by Andy

(A) **Horizontal Pressure = f(z):**

\[ K_0 = \tan^2 (45 - \frac{\phi}{2}) \]
\[ = \tan^2 (45 - 0) \]
\[ = 1.0 \]

\[ E_h = 8 \times 2 \times K_0 \text{ (no surcharge load) } \]
\[ = 160 \text{ (2) psf} \]

**Allowable Geotextile Strength:**

\[ T_{allow} = \frac{1}{T_{ut}} \times \frac{1}{RF_{ta} \times RF_{cr} \times RF_{c} \times RF_{bb}} \times \frac{400 \text{ psf}}{1.2 \times 2.5 \times 1.1 \times 1.0} \]

\[ T_{allow} = 121.21 \approx 120 \text{ kN/m} \]

**Geotextile Reduction Factors:** (Kearns, 1998) p. 190

\[ RF_{ta} \Rightarrow \text{installation damage} = 1.2 \]
\[ RF_{cr} \Rightarrow \text{creep} = 2.5 \]
\[ RF_{c} \Rightarrow \text{chemical damage} = 1.1 \]
\[ RF_{bb} \Rightarrow \text{biological damage} = 1.0 \]
Determine Geotextile Layer Spacing

\[ S_v = \frac{T_{aw}}{\Delta u(\text{FS})} \]

- \( S_v \): vertical spacing
- \( \Delta u \): horizontal stress at depth \( z \)
- \( \text{FS} \): factor of safety against failure of GT breakage

\[ S_v = \frac{100 \times 12\%}{115 \times 2 \times 1.4} = 0.99 \text{ in} \]

<table>
<thead>
<tr>
<th>( z (\text{in}) )</th>
<th>Cold ( S_v (\text{in}) )</th>
<th>Design ( S_v (\text{in}) )</th>
<th>Layer #</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>8.9</td>
<td>2.0</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>4.5</td>
<td>1.5</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>3.0</td>
<td>1.0</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>2.2</td>
<td>1.0</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>1.8</td>
<td>-</td>
<td>5</td>
</tr>
<tr>
<td>6</td>
<td>1.5</td>
<td>-</td>
<td>6</td>
</tr>
<tr>
<td>7</td>
<td>1.3</td>
<td>-</td>
<td>7</td>
</tr>
<tr>
<td>8</td>
<td>1.1</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>0.99</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

(b) Determine Length of Geotextile

assuming \( c = 288 \text{ psf} \)

\[ L_e = \frac{S_v}{2}\left(\frac{115 \times 12\%}{2(8+2\times1.4)}\right) \]

\[ L_e = \frac{S_v (115 \times 12\%)}{2(288 + 115 \tan 15)} = \frac{S_v (161) \times 1.4}{576 + 61.6} \]

\[ L_e = \frac{S_v (161)^2}{576 + 61.6} \]
\[ L_R = (H - 2) \tan (45 - 2^\circ) \]
\[ = (9 - 2) \tan 45^\circ \]
\[ L_R = 9 - 2 \]

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (ft)</th>
<th>Spacing (ft)</th>
<th>La (ft)</th>
<th>Le (ft)</th>
<th>Lr (ft)</th>
<th>Lined (ft)</th>
<th>Lspec (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.0</td>
<td>7.0</td>
<td>7.3</td>
<td>7.0</td>
<td>10</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>3.5</td>
<td>1.5</td>
<td>1.1</td>
<td>3.0</td>
<td>5.5</td>
<td>8.5</td>
<td>9</td>
</tr>
<tr>
<td>3</td>
<td>5.0</td>
<td>1.5</td>
<td>1.4</td>
<td>3.0</td>
<td>4.0</td>
<td>7.0</td>
<td>7</td>
</tr>
<tr>
<td>4</td>
<td>6.0</td>
<td>1.0</td>
<td>1.0</td>
<td>3.0</td>
<td>3.0</td>
<td>6.0</td>
<td>6</td>
</tr>
<tr>
<td>5</td>
<td>7.0</td>
<td>1.0</td>
<td>1.1</td>
<td>3.0</td>
<td>2.0</td>
<td>5.0</td>
<td>5</td>
</tr>
<tr>
<td>6</td>
<td>8.0</td>
<td>1.0</td>
<td>1.2</td>
<td>3.0</td>
<td>1.0</td>
<td>4.0</td>
<td>4</td>
</tr>
<tr>
<td>7</td>
<td>9.0</td>
<td>1.0</td>
<td>1.3</td>
<td>3.0</td>
<td>0.0</td>
<td>3.0</td>
<td>4</td>
</tr>
</tbody>
</table>

*Although La is ≤ LDA value, Lr = 2 ft is mandated.

(C) Check concrete length (Le). Le must be ≥ 1.6 \( LF \) than the 
AFI recommendation:

\[ L_e = \frac{S_v \times 0.6 \times F_2}{4(L + \frac{6}{8} \tan 8)} = \frac{S_v (L + 2.0) \times 1.4}{4(2.88 + 1.52 \tan 15)} = \frac{S_v (L + 2.0)}{11.52 + 12.32} \]

\( (L \text{ is max @ upper layers, } z = 24) \)

\[ L_e = \frac{20 \times (16) \times 2.0}{11.52 + 12.32(2)} = \frac{640}{23.84} = 0.25 \text{ ft} \]

Once \( \alpha > 3 \times L_e \) \( \text{'will be sufficient} \)
Appendix H: External Stability Analysis of Geotextile Wall Design

Stability calculations, assuming reinforced wall has dimensions below:

\[ \text{FS}_{\text{ overturning}} \geq 2.0 \]
\[ \text{FS}_{\text{sliding}} \geq 2.0 \]
\[ \text{FS}_{\text{bearing capacity}} \geq 7.0 \]
\[ \text{FS}_{\text{slope failure}} \geq 1.5 \]

For stability calculations, assuming reinforced wall has dimensions below:

\[ K_{r} = 1 - \sin \phi \]
\[ K_{d} = 0.6 \text{ (high)} \]
\[ P_{d} = 0.4 \]

\[ P_{d} = \frac{1}{2} (115) q^{2} (0.4) \]
\[ = 186.2 \text{ lb/ft} \]

\[ P_{a} = 144.3 \text{ lb/ft} \]
\[ H/3 = 3' \]

\[ FS_{\text{o,t}} = \frac{\sum \text{Resisting Moments}}{\sum \text{Driving Moments}} \]

\[ = \frac{115 \rho \left[ (42 \times 2') + (17 \times 4.5') + (16 \times 5.5') + (15 \times 6.5') + (12 \times 7.5') + (1 \times 2 \times 9.5') \right]}{186.2 \times 64'' (18')} \]

\[ = \frac{115 \times 24.4}{558.9} = 5.0 > 2.0 \]
FS sliding at Base = \sum \frac{\text{Resisting Forces}}{\text{Driving Forces}}

FS_{sl} = \frac{C_a + \gamma D \tan \delta}{P_a} = \frac{230(4') + (4'(4)15) + (4'(4)15) + (4'(4)15) + (4'(4)15)}{186.5} + 15

= \frac{915 + 1910.5}{186.5} = 1.5

FS_{sl} (\gamma') = \frac{K_\theta (\gamma') \theta}{186.5} = 1.09

FS Bearing capacity ~ Shallow Foundation Bearing Capacity

q_{ult} = C'_{ref} + q_a \gamma = \frac{1}{3} \gamma B N_k

~ checking undrained failure ~ i.e. \phi = 0

\text{estimate} C'_{ref} = 1000 \text{ psf}

q_{ult} = 1000 (5) = 5000 \text{ psf}

\gamma_{actual} = 11.5 (4') = 1035 \text{ psf}

FS_{bc} = \frac{5000}{1035} = 4.8 > 2.0 \text{ safe}

~ checking drained case

\phi \text{ from soil } = 20°

q_{ult} = 0 + 0 + \frac{1}{2} (115) \gamma (5) = 1150

FS_{bc} = \frac{1150}{900} = 1.3

FS slope failure will be analyzed at a later date
Appendix I: Point Load Calculations on Concrete Wall During Backfill Compaction

\[ \begin{align*}
\text{Wt. Stem} &= 8 \times 0.83 \times 1 \times 150 = 996 \text{ lb/ft} \\
\text{Wt. Footing} &= 2 \times 1 \times 1 \times 150 = 300 \text{ lb/ft} \\
\text{Total Wt. of Wall} &= 996 + 300 = 1296 \text{ lb/ft}
\end{align*} \]

What \( P_c \) acting at 1' from wall top will cause overturning in concrete wall?

\[ FS_{OT} = \frac{\Sigma M_E}{\Sigma M_D} = \frac{W_{wall}(1) + P_c(1'\text{)} \times 1}{P_c(1')} \]

\[ P_c = \frac{1}{2} \left( \frac{W_{wall}(1)}{H} \right) \]

\[ H = 9' \]

For: \( k = 1 \)

\( n = 0.11 \)

\( X = 1' \)

\( m = 0.11 \)

\( P_h = 146 \text{ lb} \)

(Koerner, 1998)

\[ P_c = \frac{12A_b(1) + 50(1')}{P_c(1')} \implies P_c = 164 \text{ lb/ft} \]

\( P_c \) needed in horizontal direction for failure

\[ = 164 \text{ lb/ft} \]

\[ \frac{X}{H} = n = 0.11 \]

\[ m = 0.11 \implies \text{From Figure 2.46 (Koerner, 1998) } \]

\[ \frac{H}{g} = \frac{H}{g} = 0.5 \]

\[ C_p = \frac{5H(H^2)}{6} = 73 \text{ psf (9')} \times \frac{9}{0.5} = 1,826 \text{ lb} \]

(Vertical pt load ned to fail wall 1' from top)

acting within the top layer
Assumed Compactor Weight = 5400 lb (Paul Keauig)

\[
\begin{align*}
Z = 1' & \quad \sigma_H = 0.15 \left( \frac{5400}{1.5} \right) = 33.3 \text{ psf} \\
& \quad P_H = 33.3 (24^2) = 667.2 \text{ lb} < 1466 \text{ lb} \checkmark \text{ok}
\end{align*}
\]

\[
\begin{align*}
Z = 2.75' & \quad \sigma_H = 1.325 \left( \frac{667.2}{2.75} \right) = 88.33 \text{ psf} \\
& \quad P_H = 88.33 \times 1.5 \cdot 24^2 = 133 \text{ psf} \checkmark \text{ok}
\end{align*}
\]

\[
\begin{align*}
Z = 5.5' & \quad \sigma_H = 0.7 \left( \frac{667.2}{5.5} \right) = 46.7 \text{ psf} \\
& \quad P_H = 46.7 (1.5) = 46.7 \text{ lb} \checkmark \text{ok}
\end{align*}
\]

Layer #5  \quad \sigma_H = 1.0 \left( \frac{5400}{81} \right) = 67 \text{ psf} \\
\quad P_H = 67 (1.5) = 100.5 \text{ lb} \checkmark \text{ok}

Layer #3  \quad \sigma_H = 0.5 \left( \frac{5400}{81} \right) = 32.5 \text{ psf} \\
\quad P_H = 33 (24) = 33 \text{ lb} \checkmark \text{ok}

Layer #2  \quad \sigma_H = 0.3 \left( \frac{5400}{81} \right) = 20 \text{ psf} \\
\quad \frac{P}{H} = 0.83 \quad P_H = 1 (20) = 20 \text{ lb} \checkmark \text{ok}

Layer #1  \quad \sigma_H = 0.18 \left( \frac{5400}{81} \right) = 19 \text{ psf} \\
\quad \frac{P}{H} = 0.94 \quad P_H = 1 (19) = 19 \text{ lb} \checkmark \text{ok}
Assumed Compactor weight = 8500 lb (www.bobcat.com)

Top Layer (z=1) \(\Rightarrow\) Middle of layer \(\Rightarrow\) \(z=1\) ft

\[
\frac{z}{H} = \frac{1}{4} = 0.25, M \quad Q_p = 8500 \text{ lb} \\
x = 1' \\
\sigma_H = 0.5 \left( \frac{Q_p}{H^2} \right) = 0.5 \left( \frac{8500}{1^2} \right) = 52.5 \text{ psf}
\]

-if acting area of \(P_H\) is \(2' \times 1'\), then

\[
P_H = 52.5 \text{ psf (2.0')} = 105 \text{ lb} < 164 \text{ lb} \quad \checkmark
\]

Layer = 6 \(\Rightarrow\) Middle of Layer \(\Rightarrow\) \(z = 2.75\) ft

\[
\frac{z}{H} = \frac{2.75}{9} = 0.31, M = 0.2 \quad Q_p = 8500 \text{ lb} \\
x = 1.8 \text{ ft} \\
\sigma_H = 1.325 \left( \frac{Q_p}{H^2} \right) = 1.325 \left( \frac{8500}{1^2} \right) = 139 \text{ psf} \\
P_H = 139 (1.5) = 208.5 \text{ lb} < 210 \quad \checkmark
\]

Layer = 4 \(\Rightarrow\) Middle of Layer \(\Rightarrow\) \(z = 5.5\) ft

\[
\frac{z}{H} = \frac{5.5}{9} = 0.61, M = 0.2 \\
x = 1.8 \text{ ft} \\
Q_p = 8500 \text{ lb} \\
\sigma_H = 0.7 \left( \frac{Q_p}{H^2} \right) = 0.7 \left( \frac{8500}{1^2} \right) = 73.5 \text{ psf} \\
P_H = 73.5 (1) = 73.5 \text{ lb} < 875 \quad \checkmark
\]
Appendix J: As-built Geotextile Stability Analysis

Stability Calculation for Constructed GT Wall

Following example (Koerner, 1998) p. 189

Assumptions:
\( Y_{soil} = 112 \text{ psf (avg. of field density tests)} \)
\( \phi = 20^\circ \)

*For stability calculations, assume reinforced wall has dimensions below.

\[ F_{so} = \sum \frac{G \text{ resisting moments}}{G \text{ driving moments}} \]

\[ = \frac{115 \text{ psf} \left[ \left(1\times 4\times 1.5\right) + \left(2\times 5\times 2.5\times 1.5\right) + \left(3\times 5\times 2.5\times 7.5\right) \right]}{1814 \times \frac{3}{2}} \]

\[ = \frac{35730}{542} = 6.6 \]

Added \( F_{so} \) compared to original design = 6.6 = 1.3
\[ F_{S_{\text{SIDING @ Base}}} = \Sigma F_{\text{Resisting Forces}} - \Sigma F_{\text{Driving Forces}} \]

Assuming \( C_0 = 0.8 C_u = 0.8 \text{GPa} \)

\[ \frac{F_{S_{UL}}}{P_a} = \frac{2300 + 4(6)(112) \tan 30}{1814} = 1.8 \]

Additional \( F_{S_{UL,\text{As designed}}} = \frac{1.8}{1.09} = 1.7 \)

\[ F_{S_{B,C}} \text{ does not change due to as-built conditions} \]

\[ F_{S_{\text{Overall}}} \text{ performed using SlopeW} \]
Appendix K: Concrete Wall Stability with Geotextile
Wall Contact

\[ W_1 = 0.83 \times 8.10 \times 1 \times 150 \text{pcf} = 99.6 \text{ kN/m} \]
\[ W_2 = 1 \times 2.0 \times 1 \times 150 \text{pcf} = 262.5 \text{ kN/m} \]

Total wall load = 1256.5 lb/linear ft

\[ P_{A1} = \frac{1}{2} \times 12 \times 7 \times 1 \times 150 \text{pcf} = 56.5 \text{ kN/m} \]
\[ P_{A2} = \frac{1}{2} \times 12 \times 7 \times 1 \times 150 \text{pcf} = 33.75 \text{ kN/m} \]
\[ P_{A3} = \frac{1}{2} \times 12 \times 7 \times 1 \times 150 \text{pcf} = 31.5 \text{ kN/m} \]
\[ P_{A4} = 31.5 \text{ kN/m} \]
\[ P_{A5} = \frac{1}{2} \times 12 \times 7 \times 1 \times 150 \text{pcf} = 87.5 \text{ kN/m} \]
\[ P_{A6} = \frac{1}{2} \times 12 \times 7 \times 1 \times 150 \text{pcf} = 87.5 \text{ kN/m} \]

\[ F_2 = \frac{1}{2} \times 12 \times 7 \times 1 \times 150 \text{pcf} = 504 \text{ kN/m} \]

\[ P_{\text{total}} = 538.25 \text{ kN/m} \]
All layers touching well

FS_Skipping = X_Foasing
FS_Fanning = P2 + (W/8)x1 + P1

= .504 + (.258)x120 + 50 = 1.89

FS_Skipping = X_Foasing
FS_Skipping = \frac{5(1.5) + 9(4) + 2(2.5)(\frac{12}{10}) - (1.5)(5)(12)(1.54)}{6(4) + 9(2)(6) + 2(2.5)(12) + 3(1)(2.5)(10) + 8.5(18)}

= \frac{1204.3}{2794.11} = 0.44

Half the layers touching - load is half previous load calculated

FS_\text{calc} = 3.78

FS_{set} = 1.72
Appendix L: Cost Analysis Comparison – Select vs. Marginal Backfill

### Alternative 1: Granular Backfill

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit Price</th>
<th>Units Needed</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel, assuming 1 ton = 0.5 yd^3</td>
<td>$240/15 tons</td>
<td>160 tons</td>
<td>$2,560</td>
</tr>
<tr>
<td>Geoxtextile Reinforcement</td>
<td>$0.70/yd^2</td>
<td>415 yd^2</td>
<td>$291</td>
</tr>
<tr>
<td>Labor, assuming 3 man crew, 6 hrs/day, 5 days</td>
<td>$12/hr</td>
<td>90 hrs</td>
<td>$1,080</td>
</tr>
<tr>
<td>Skid-Loader Rental</td>
<td>$100/day</td>
<td>5 days</td>
<td>$500</td>
</tr>
<tr>
<td><strong>Total Cost</strong></td>
<td></td>
<td></td>
<td><strong>$4,431</strong></td>
</tr>
</tbody>
</table>

### Alternative 2: On-Site Marginal Backfill

*all costs same as above except backfill

<table>
<thead>
<tr>
<th>Item</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>in-situ soil</td>
<td>~100 yd^3</td>
<td>$0</td>
<td>$0</td>
</tr>
<tr>
<td><strong>Total Cost</strong></td>
<td></td>
<td></td>
<td><strong>$1,871</strong></td>
</tr>
</tbody>
</table>

Total Cost = $4,431
Appendix M: Calculation of Geotextile Wall Settlement

Constant Rate of Strain (CRS) Test Results

- $w_0 = 17.4$
- $\gamma_d = 115$ pcf
- $e_0 = 0.719$
- $R_c = 0.104$
Settlement Analysis

Assumptions:
- Rigid Foundation (no settlement)
- $\gamma_t = 112$ psf
- $R_c = 0.104$ (CRS Test)
- Rock fill incompressible

$$S_i = R_c H_i \log \left( \frac{\bar{\sigma}_{vo} + \Delta \bar{\sigma}}{\sigma_{vo}} \right)$$

$$\Delta \bar{\sigma} = \frac{H_i \gamma_t}{12}$$

$$\bar{\sigma}_{vo} = \gamma_t \times \frac{H_i \gamma_t}{2}$$

Layer 3 $\Rightarrow \bar{\sigma}_{vo} = 112 \times \frac{0.75}{2} = 42$ psf

$$\Delta \bar{\sigma} = 6.5' \times 112$ psf = 728 psf

$$S_3 = (0.104)(0.75) \log \left( \frac{42 + 728}{42} \right) = 0.099 \Rightarrow = 1.2$ inches
Layer 4 ⇒
\[ \delta_0 = 112 \times \frac{0.75}{2} = 42 \text{ psf} \]
\[ \delta = 5.75' \times 112 \text{ psf} = 644 \text{ psf} \]
\[ S_4 = (0.104)(0.75) \log \left( \frac{42 + 644}{42} \right) = 0.095 \text{ in.} \]
\[ H = 1.1 \text{ in.} \]

Layer 5 ⇒
\[ \delta_0 = 112 \times \frac{0.75}{2} = 42 \text{ psf} \]
\[ \delta = 5' \times 112 \text{ psf} = 560 \text{ psf} \]
\[ S_5 = (0.104)(0.75) \log \left( \frac{42 + 560}{42} \right) = 0.90 \text{ in.} \]
\[ H = 1.1 \text{ in.} \]

Layer 6 ⇒
\[ \delta_0 = 112 \times \frac{1.25}{2} = 70 \text{ psf} \]
\[ \delta = 3.75' \times 112 \text{ psf} = 420 \text{ psf} \]
\[ S_6 = (0.104)(1.25) \log \left( \frac{70 + 420}{70} \right) = \]

Layer 7 ⇒
\[ \delta_0 = 112 \times \frac{1.25}{2} = 70 \text{ psf} \]
\[ \delta = 2.5' \times 112 \text{ psf} = 280 \text{ psf} \]
\[ S_7 = (0.104)(1.25) \log \left( \frac{70 + 280}{70} \right) = 0.091 \text{ in.} \]
\[ H = 1.1 \text{ in.} \]

Layer 8 ⇒
\[ \delta_0 = 112 \times \frac{1.25}{2} = 70 \text{ psf} \]
\[ \delta = 1.25' \times 112 \text{ psf} = 140 \text{ psf} \]
\[ S_8 = (0.104)(1.25) \log \left( \frac{70 + 140}{70} \right) = 0.062 \text{ in.} \]
\[ H = 0.7 \text{ in.} \]

Layer 9 ⇒
\[ \delta_0 = 112 \times \frac{1.25}{2} = 70 \text{ psf} \]
\[ \delta = 0 \text{ psf} \]
\[ S_9 = \text{no settlement (} \delta = 0 \text{)} \]

\[ \text{Total Settlement} = \sum S_i = 1.2 + 1.1 + 1.1 + 1.1 + 0.7 = 5.2 \text{ in.} \]