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Damage to HDPE geomembrane from interface shear over gravelly compacted clay liner

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Author
Thielmann, Stuart

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Damage to HDPE Geomembrane from Interface
Shear over Gravelly Compacted Clay Liner

A Thesis submitted in partial satisfaction of the requirements
for the degree Master of Science

in

Structural Engineering

by

Stuart Thielmann

Committee in Charge:

Professor Patrick Fox, Chair
Professor Ahmed-Wael Elgamal
Professor Tara Hutchinson

2012
The Thesis of Stuart Thielmann is approved and it is acceptable in quality and form for publication on microfilm and electronically:

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Chair

University of California, San Diego

2012
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LIST OF ABBREVIATIONS

CCL compacted clay liner
GCL geosynthetic clay liner
GM geomembrane
GMS smooth geomembrane
GMX textured geomembrane
GT geotextile
HDPE high-density polyethylene
LCRS leachate collection and removal system
LDS leak detection system
LLDPE linear low-density polyethylene
MSW municipal solid waste
NP needle-punched
NW nonwoven
OMC optimum water content
PP polypropylene
W woven
\( \nu \) water content
\( \sigma, \sigma_n \) normal stress
\( \tau_p \) peak shear strength
\( \tau_{ld} \) large-displacement shear strength
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ABSTRACT OF THE THESIS

Damage to HDPE Geomembrane from Interface Shear over Gravelly Compacted Clay Liner

by

Stuart Thielmann

Master of Science in Structural Engineering
University of California, San Diego 2012
Professor Patrick Fox, Chair

Liner systems are used in a wide array of applications to prevent the spread of contaminants into the environment. These liner systems have two main components: a geomembrane (GM) and a compacted clay liner (CCL). GMs are susceptible to damage both during installation and the service life of the liner system. The most common source of damage is puncture due to granular material (e.g., gravel or stones) in direct contact with the GM resulting in the formation of a hole in the liner. While multiple studies have investigated puncture due to a granular layer above the GM,
puncture due to granular material in the subgrade below has not received comparable attention. A series of tests were performed using a large-scale direct shear machine to investigate possible damage to a high density polyethylene (HDPE) GM due to interface shearing over a gravelly CCL for a wide range of normal stress levels. Results indicate that shear displacement of a HDPE GM over a gravelly CCL can significantly increase GM damage compared to static pressure alone. Damage increased with increasing normal stress. Essentially no damage was observed at the lowest normal stress (72 kPa), while severe damage was observed at high normal stress (1658 kPa). Placement of a hydrated nonwoven/nonwoven (NW/NW) needle-punched (NP) geosynthetic clay liner (GCL) between a GM and gravelly CCL was shown to dramatically reduce GM damage, including one test conducted at very high normal stress (4145 kPa).
1 INTRODUCTION

Enacted in 1976, the Resource Conservation and Recovery Act (RCRA) was the first federal statute to address solid waste within the United States. RCRA defined standards for the treatment, storage, and disposal of solid and hazardous wastes. These regulations were placed under the jurisdiction of the Environmental Protection Agency (EPA). One of the main focuses of the RCRA was the protection of human health and the environment from the potential hazards of waste disposal. These hazards include liquid and gas contaminants which may migrate through a waste containment site and pose a threat to groundwater and nearby surface-water quality. To prevent this spread of contaminants, liner systems are placed between the waste and the surrounding environment. These liner systems have two main components: a geomembrane (GM) and a compacted clay liner (CCL). A geosynthetic clay liner (GCL) may also be included in the liner system in some situations. Liner systems are now used in a wide array of applications including: landfills, industrial ponds, waste piles, surface impoundments, oil and gas tank containment systems, and heap leach pads.

A geomembrane is a manufactured polymer sheet typically 1-2 millimeters thick that is commonly used as a barrier layer. For North American landfills, the most common GM material is high-density polyethylene (HDPE). GMs are almost impermeable with only very low possible rates of liquid and gas permeation. However, due to their low thickness they are susceptible to damage such as tears, punctures, and holes which compromise their effectiveness as a barrier layer. As such, GMs are used in combination with a CCL or GCL placed below the GM to create a
composite liner system. In this way, leakage through a hole in the GM is hindered by the presence of a low permeability layer underneath.

Current EPA regulations for municipal solid waste (MSW) landfill liner systems require a single composite liner system that meets the following requirements: leachate collection and removal system (LCRS) limiting the head of leachate on the liner system to 0.3 m or less, 1.5 mm thick GM if made of high-density polyethylene (HDPE), 0.6 m thick CCL with a maximum hydraulic conductivity of $1 \times 10^{-7}$ cm/sec. Figure 1.1 shows an example liner system that meets these requirements. Hazardous waste landfills are required to have two independent liners. A leak detection system (LDS) is placed between the liners to allow monitoring of the primary liner and provide a way to remove leachate between the primary and secondary liners. An example is shown in Figure 1.2

Damage to GMs can arise from multiple sources both during installation and the service life of the liner system. Heavy equipment or workers may directly damage the GM during installation. Defects may occur when the edges of the GM are seamed together on-site. The most common source of damage though is puncture due to granular material (e.g., gravel or stones) in direct contact with the GM, either above or below the liner (Nosko and Touze-Foltz 2000, Giroud and Touze-Foltz 2003). It has been shown in the laboratory that the permeability of a CCL may not be affected for gravel contents as high as 50-60% (Shakoor and Cook 1990, Shelly and Daniel 1993) and CCLs have been constructed in the field with gravel content as large as 22% (Benson et al 1994; Bonaparte et al. 2002). Typically the maximum allowable particle
size for a CCL is 25 to 50 mm, with the final surface rolled smooth and free of protruding stones, and the GM is placed directly on top of this surface. Gravel may also be in contact with a GM from above. Drainage layers, especially for use in mining applications, often consist of coarse soils layers, sometimes containing large-diameter angular rock, placed directly over the liner system.

In cases where the possibility of GM puncture exists, a protection layer may be required between the granular material and GM to act as a cushion and prevent damage to the GM. Numerous studies have been published investigating the use of protective layers, with the majority of this work focused on damage that occurs due to static overburden pressure. However, downdrag forces (e.g., construction operations, waste settlement, seepage forces, seismic loading) along the side slopes of a bottom liner can lead to global and/or local shear failure of the liner system. The potential for GM damage due to interface shear has not received comparable attention. Only one study has been performed to explore shear induced damage for GMs placed in direct contact with soils containing gravel (Fox et al. 2011).

Once the liner system is in operation and material is placed on top, detection of a failure and any resulting damage is not easily detected. Repair is often difficult and cost-prohibitive, especially for bottom liners. New facilities are increasingly located in close proximity to populated and environmentally sensitive areas. Given that leakage through the GM in many of these applications may pose significant risk to both the environment and to human health, the need to adequately protect GMs from puncture is essential.
Figure 1.1. Example of a single-composite liner system for MSW landfills (Bonaparte et al. 2002).

Figure 1.2. Example of a double-composite liner system for hazardous waste landfills. (Bonaparte et al. 2002).
This thesis reports a unique testing program designed to investigate possible damage to a GM due to shearing over a gravelly CCL. Chapter 2 presents a review of literature that has been published relating to GM puncture damage. Chapter 3 outlines the test materials and procedures used for the current study. Results and analysis are presented in Chapter 4, and Chapter 5 provides conclusions and recommendations for future research.
2 LITERATURE REVIEW

2.1 Gravel in Compacted Clay Liners

2.1.1 Shakoor and Cook (1990)

A laboratory study on the compaction characteristics of clay-stone mixtures was performed by Shakoor and Cook (1990) using a silty clay and limestone aggregate. Maximum dry density, permeability, and unconfined compressive strength were tested at stone contents varying from 10-80% by dry weight. The effect of angular versus rounded stone shape was investigated as well. Testing results showed that permeability was generally not affected for stone contents up to 50%. Between 50-70% stone content, permeability increased 4-5 orders of magnitude, as seen in Figure 2.1 (a). Stone shape had little effect on permeability until 60% stone content was reached, at which point angular stones yielded permeability values five times higher than rounded stones. Results from these tests are shown in Figure 2.1 (b).

![Figure 2.1. Results of stone content on CCL permeability for (a) stone size (b) stone shape (Shakoor and Cook 1990).](image)
For construction projects where low permeability is the most important property (e.g. a CCL), the study concludes that a stone content of 0-50% of any shape will not significantly affect permeability. As a compromise between the three properties studied, it is recommended that stone content be limited to 20% small stones between 2-4.75 mm in size.

2.1.2 Shelly and Daniel (1993)

Another study on the properties of compacted clay liners with gravel was published by Shelly and Daniel (1993). Two types of clayey soils, kaolinite and mine spoil, were mixed at gravel contents ranging from 0-80% by dry weight and compacted in a laboratory setting at 2-4% wet of optimum. Permeability tests were then performed on each specimen. It was determined that hydraulic conductivities less than $1 \times 10^{-7}$ cm/s could be achieved for gravel contents up to 60%. For the two clayey soils, results showed that the hydraulic conductivity of the soil mixtures increased 10-fold and 50-fold as the gravel content increased from 60% to 80%. Figure 2.2 shows the relationship between gravel content and hydraulic conductivity for both soils. For mixtures with kaolinite clay, gravel contents less than 60% had slightly lower hydraulic conductivities than pure clay specimens.
Additionally, the authors found that gravel contents less than 60% could achieve minimum hydraulic conductivity over a broadened range of molding water contents. Maximum dry unit weight for both clay-gravel mixtures peaked at 70-80% gravel, shown in Figure 2.3. These results led the authors to believe that the addition of gravel may actually facilitate a better-quality compacted clay liner. One drawback of large gravel contents is the possibility of isolated pockets of segregated gravel particles forming in the compacted clay liner. In these segregated pockets voids between gravel particles are not completely filled with clay material, increasing the hydraulic conductivity of the liner. Design and construction of a liner must therefore carefully consider and prevent gravel segregation. In conclusion, the authors recommend that the gravel content of a compacted clay liner not exceed approximately 50%.
2.1.3 Benson et al. (1994)

A database of compacted soil liners from 67 landfills across North America was created by Benson et al. (1994). The database indicated that 46 liners (69%) contained gravel, 12 liners (18%) had gravel content between 5-10%, and one liner had a gravel content of 22.2%. Laboratory measurements of hydraulic conductivity were taken for each liner and are plotted versus gravel content in Figure 2.4. The figure shows that hydraulic conductivities below $1 \times 10^{-7}$ cm/sec are attainable in the field for CCLs with gravel content up to 10%. It is not known whether the CCLs with poor hydraulic conductivities above this value were caused by gravel content or an unrelated reason such as poor compaction procedure.
2.1.4 Nosko and Touze-Foltz (2000)

Using electrical leak detection and location methods, Nosko and Touze-Foltz (2000) collected and analyzed data from the previous ten years on GM integrity. Over 300 sites were included in the study, totaling over four thousand GM failures from 16 countries. The main goal of the study was to determine the cause, size, and location of damage to GM liners. Table 2.1 shows the size and cause of damage within the liner system. The data indicates that 71% of damage was caused by stones, though it is not specified whether stones were above or below the liner. The authors found that damage caused by stones often occurred as an area with several small single holes grouped together. Therefore, holes grouped together within a 5 cm diameter area were defined as a single hole caused by stones.
Table 2.1. Cause and size of damage to GM liners (Nosko and Touze-Foltz 2000).

<table>
<thead>
<tr>
<th>Size of damage (cm$^2$)</th>
<th>Stone</th>
<th>%</th>
<th>Heavy equip.</th>
<th>%</th>
<th>Welds</th>
<th>%</th>
<th>Cuts</th>
<th>%</th>
<th>Worker directly</th>
<th>%</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 0.5</td>
<td>332</td>
<td>11.1</td>
<td>-</td>
<td>-</td>
<td>115</td>
<td>43.4</td>
<td>5</td>
<td>8.5</td>
<td>-</td>
<td>-</td>
<td>452</td>
</tr>
<tr>
<td>0.5 – 2.0</td>
<td>1720</td>
<td>57.6</td>
<td>41</td>
<td>6.3</td>
<td>105</td>
<td>39.6</td>
<td>36</td>
<td>61.0</td>
<td>195</td>
<td>84.4</td>
<td>2097</td>
</tr>
<tr>
<td>2.0 – 10</td>
<td>843</td>
<td>28.2</td>
<td>117</td>
<td>17.9</td>
<td>30</td>
<td>11.3</td>
<td>18</td>
<td>30.5</td>
<td>36</td>
<td>15.6</td>
<td>1044</td>
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<tr>
<td>&gt;10</td>
<td>90</td>
<td>3.0</td>
<td>496</td>
<td>75.8</td>
<td>15</td>
<td>5.7</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>601</td>
</tr>
<tr>
<td>Amount</td>
<td>2985</td>
<td>654</td>
<td>265</td>
<td>59</td>
<td>59</td>
<td>231</td>
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<td>6.32</td>
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<td></td>
<td></td>
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</tr>
</tbody>
</table>

Calculations were also performed to estimate the rate of liquid flow through a composite liner due to a hole in the geomembrane. Two values of hydraulic conductivity were used for the transmissive layer between the GM and CCL, to account for contact conditions between GM and CCL. One value, $1.6 \times 10^{-8}$ cm/sec, corresponded to good contact conditions while a larger value of $1 \times 10^{-7}$ cm/sec corresponded to poor contact conditions, representative of cases where a geotextile is placed between the GM and CCL or a wrinkle is present in the GM. For a constant hydraulic head of 0.3 m on top of the liner, the rate of flow for both hydraulic transmissivity values is shown in Figure 2.5 as a function of the size of a circular hole in the GM.

Surprisingly, hole size was not a determining factor in the rate of flow. The transmissive layer had a much greater influence on flow, as increasing the hydraulic conductivity of this layer by a factor of five increased the rate of flow through the liner by approximately the same factor. This suggests that the placement of a geotextile between GM and CCL will significantly increase flow through a liner system and may not be a desirable practice. The authors suggest that it is much more important to
know the location and condition of GM holes rather than their size in order to estimate the rate of flow through the liner.

![Diagram](image.png)

**Figure 2.5.** Rate of liquid flow through a composite liner due to circular hole in GM of varying size (Nosko and Touze-Foltz 2000).

### 2.1.5 Bonaparte et al. (2002)

An assessment of waste containment sites within North America was published by Bonaparte et al. (2002) in a joint report with the EPA. Typical MSW regulations require a CCL with a saturated hydraulic conductivity of not more than $1 \times 10^{-7}$ cm/s. For such a liner the current recommended design practice limits the maximum gravel content to 20-50%. One objective of the report was to collect information on the field performance of CCLs. Data was collected from 89 sites which used natural clay material as a component of the liner system. Of the 89 total CCLs it was found that 27 (30%) contained gravel and 10 liners (11%) had gravel content between 5-10%. The largest gravel content recorded was 10%, and was found at 3 sites.
2.2  *Geomembrane Puncture Testing*

2.2.1  *Narejo (1995)*

While most studies on geomembrane puncture make use of smooth GMs, Narejo (1995) compared the puncture resistance between smooth and textured GMs. Texturing increases the interface friction angle and is commonly used where increased shear resistance is required, such as slopes. The texturing process has its drawbacks, as elongation of the GM at break can be reduced by as much as 50%. 1.5 mm smooth and textured high-density polyethylene (HDPE) GMs were tested under static loading at pressures up to 700 kPa. Three types of each GM (smooth and textured) were used. In general textured GMs had lower puncture resistance for the case of an unprotected GM (Figure 2.6) as well as when protected by a nonwoven (NW) needle-punched (NP) GT (Figure 2.7). Puncture resistance was dependent on the type of texturing; decrease in failure pressures was variable and as large as 50% for an unprotected GM and 80% for a protected GM. Types of textured GMs tested were not specified.

Due to the reduced puncture resistance of textured GMs, protection criteria developed for smooth GMs may not be applicable to textured GMs of the same material and thickness. In designing for cases involving textured GMs it may be necessary to: a) limit the stone size in contact with the GM, and b) increase the mass per unit area of the protection material as compared to cases with smooth GMs.
Figure 2.6. Puncture failure pressures at a cone height of 25 mm for unprotected smooth and textured HDPE GMs (Narejo 1995).

Figure 2.7. Puncture failure pressures at a cone height of 25 mm for smooth and textured HDPE GMs with a 540 g/m² NW-NP GT protection layer (Narejo 1995).
2.2.2 Narejo et al. (2007)

GM protection from protrusions in the subgrade using GT and GCL materials was evaluated through laboratory testing by Narejo et al. (2007). The authors note that the use of a GCL as a protective layer affects its use as a hydraulic barrier. The effects of the protrusions on hydraulic conductivity of the GCL must be considered by the design engineer, as the GCL may not be able to meet the necessary hydraulic conductivity requirements of a barrier layer if it is also used as a protective layer.

Puncture was evaluated using a hydrostatic truncated cone apparatus, as shown in Figure 2.8. Height of the protrusions was varied by adjusting the amount of bedding sand used in the bottom pressure vessel. All GM and protective layers were sandwiched between the flanges of the pressure vessel and top lid. Normal stress is applied by introducing water to the top of the GM through an inlet valve. Normal stresses up to 700 kPa were achievable for the setup used. All tests were performed using a 1.5 mm smooth HDPE GM.

As would be expected, data shows that as protrusion height is decreased, normal stress required to puncture the GM increased. Results for an unprotected GM are shown in Figure 2.9 along with previous research data. The critical cone height is the maximum size of protrusion that should be allowed in the subgrade, identified where the curve becomes asymptotic to the y-axis. A prudent design engineer should use a factor of safety in combination with this value for calculations. In the case of the unprotected GM, critical cone height was 10 mm. GT cushions commonly used in practice have mass per unit areas ranging from 540 to 680 g/m²; geotextiles used for
this study were chosen in this range. Protection behavior using NW-NP geotextiles is shown in Figure 2.10. Critical cone height increased to 20 mm with the use of a NW-NP GT, double that of an unprotected GM.

Figure 2.8. Truncated cone apparatus used to evaluate GM puncture and protective layers (Narejo et al. 2007).
Figure 2.9. Puncture behavior of an unprotected 1.5 mm smooth HDPE GM (Narejo et al. 2007).

Figure 2.10. Puncture behavior using NW-NP geotextile protection layer (Narejo et al. 2007).
Seven types of GCLs were used in the study as protective layers. All GCLs were reinforced products with polypropylene (PP) nonwoven needle-punched (NW-NP) cover and carrier geotextiles, each with a unit weight of 200 g/m². The effect of the bentonite layer within the GCL was studied by varying the amount of bentonite loading within each sample used. Figure 2.11 presents GM puncture behavior with the use of a GCL protection layer. Critical cone height using a GCL protective layer was 26 mm. No GM puncture occurred with the use of any GCL at a cone height of 25 mm for normal stresses up to 700 kPa. For tests with a 38 mm cone height no clear advantage was observed for GCLs with increased bentonite content; comparable protection of the GM was observed for GCLs with total mass per unit areas of 2541 and 5130 g/m².

Figure 2.11. Puncture behavior using geotextile-based GCL protection layers of varying bentonite content (Narejo et al. 2007).
The data shows that the use of a GCL as a protective layer results in the greatest improvement in GM puncture protection. The amount of bentonite mass per unit area of the GCL was not a significant factor. This can be attributed to increased bentonite migration and thinning as the bentonite amount is increased, negatively affecting GCL cushioning. NW-NP geotextiles performed only marginally worse than the GCLs tested. However, the use of a GT as a protective layer may not be desirable due to the increased potential leakage if GM puncture does occur. This increased leakage was also discussed in Nosko and Touze-Foltz (2000). Therefore, GCLs present a more effective option when their hydraulic performance is taken into account. Improved leakage performance of the liner system and lower construction quality assurance costs can be used to partially offset the additional cost of using a GCL protective layer.

2.2.3 Athanassopoulos et al. (2009)

Athanassopoulos et al. (2009) performed a study of composite liners under high normal loads in order to better understand their use in mining applications such as tailing impoundments and heap leach pads. Such applications are increasingly utilizing geosynthetic liners and can involve normal stresses up to 3450 kPa for heap leach heights of 180 meters. Heap leach pads typically place a layer of large size, angular drainage rock directly on top of the liner system. Due to the vulnerability of lining systems to damage and associated leakage, multiple liners and backup systems
are routinely included in construction. Multiple geosynthetic liner products were tested under large compressive forces and are listed in Table 2.2.

Table 2.2. Geosynthetic liner materials tested (Athanassopoulos et al. 2009).

<table>
<thead>
<tr>
<th>Material Designation</th>
<th>Description</th>
<th>Manufacturer</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Geomembranes</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60-mil LLDPE-S</td>
<td>1.5-mm smooth LLDPE</td>
<td>Polytex</td>
</tr>
<tr>
<td>80-mil LLDPE-S</td>
<td>2.0-mm smooth LLDPE</td>
<td>Polytex</td>
</tr>
<tr>
<td>60-mil HDPE-S</td>
<td>1.5-mm smooth LLDPE</td>
<td>Polytex</td>
</tr>
<tr>
<td>60-mil LLDPE-T1</td>
<td>1.5-mm textured (co-extruded) LLDPE</td>
<td>Polytex</td>
</tr>
<tr>
<td>60-mil LLDPE-T2</td>
<td>1.5-mm textured (embossed) LLDPE</td>
<td>Agru America</td>
</tr>
<tr>
<td><strong>Geosynthetic Clay Liners</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bentomat ST</td>
<td>3.6 kg/m² bentonite, needlepunched between woven and nonwoven geotextiles</td>
<td>CETCO</td>
</tr>
<tr>
<td>Bentomat STM</td>
<td>2.4 kg/m² bentonite, needlepunched between woven and nonwoven geotextiles</td>
<td>CETCO</td>
</tr>
<tr>
<td>Bentomat DN</td>
<td>3.6 kg/m² bentonite, needlepunched between two nonwoven geotextiles</td>
<td>CETCO</td>
</tr>
<tr>
<td><strong>Geotextiles</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GEOTEX 1701</td>
<td>540 g/m² nonwoven geotextile</td>
<td>Propex</td>
</tr>
</tbody>
</table>

GM puncture tests were conducted to a maximum pressure of 5172 kPa using a custom fabricated cylinder with an inside diameter of 344 mm. Mortar sand was used as a bedding layer below the GM with a 230 mm thick layer of 50 mm minus crushed stone placed above the GM. GCLs or GTs were placed below or above the GM, respectively, for specimens evaluating puncture protection. Test duration was 48 hours, except one test which was kept under load for 2 weeks. Significant yielding of the GM was observed for nearly all specimens above 2586 kPa. Lower average strain values were recorded for tests with a GCL under the GM than for tests with a GM alone, seen in Figure 2.12. Peak strains were much more variable. Two specimens at 5172 kPa normal stress suffered two holes each. The authors suggest that peak strains likely depend on the random orientation of particles in direct contact with the GM.
rather than layers below the GM. Due to the small contact areas and large stresses created by the crushed stones, GCL and GT protective layers could not fully mitigate damage to the GM. The authors go on to conclude that protective layers may have limited benefit against rocks randomly aligned with a sharp edge or point perpendicular to the point of contact with the GM. Additional results from this program will be discussed in Section 2.4.1.

![Figure 2.12. Average GM strain values for puncture testing for the unprotected case and with a GCL layer below the GM (Athanassopoulos et al. 2009).](image)

2.2.4 *Brachman and Sabir (2010)*

One of the few studies that has investigated geomembrane puncture from a gravelly compacted clay liner is by Brachman and Sabir (2010). Tensile strains of a smooth HDPE geomembrane over compacted clay were measured under static normal pressure. Five gravel particles were intentionally placed in the clay layer, each 35 mm
in size and labeled Stones A-E. The clay had an optimum water content (OMC) of 12%. Geotextiles and GCLs were also investigated for their effectiveness as protective layers. The effect of water content was also considered as it was thought the gravel particles may locally attract normal forces and press down into the clay, reducing the height of gravel protrusion and decreasing tensile strains in the GM. Tests used either a poorly-graded sand or coarse gravel layer on top of the GM. For comparison purposes with the current study, only specimens with sand over the GM will be considered.

For specimens with gravel particles initially flush with the clay surface, maximum tensile strain in the GM as a function of applied vertical pressure and clay water content is shown in Figure 2.13. No geomembrane puncture occurred for these tests. At OMC \( w = 12\% \), the clay has low compressibility and no strains are measured up to 1000 kPa. As the clay water content is increased, the compressibility of the clay also increases. At larger normal stresses the clay begins to settle around the gravel particles creating protrusions in the CCL surface which form indentations and tensile strains in the GM. Maximum tensile strain increased to 10% at 1000 kPa and 16% water content. A 540 g/m\(^2\) NW-NP GT and hydrated GCL were then alternately placed between the CCL and GM. Figure 2.14 shows the resulting maximum tensile strains for a normal stress of 1000 kPa and initial water content of 16%. The GT reduced maximum tensile strain to 4% while the GCL essentially eliminated all tensile strains. Placing a GT between the CCL and GM is seen as undesirable by the authors
since potential leakage through any GM holes would be increased due to the larger hydraulic conductivity, again as was discussed in Nosko and Touze-Foltz (2000).

Figure 2.13. Relationship between initial clay water content, applied pressure, and maximum tensile strain in the geomembrane. For stones initially flush with the CCL surface (Brachman and Sabir 2010).

Tensile strains decreased with increasing burial depth of the gravel particles. The worst case scenario was gravel particles initially placed on top of the CCL. This may represent the case where a loose stone is not picked up prior to placement of the GM, or a stone is dislodged during the installation of the GM. In this situation, tensile strains are reduced for initial water contents higher than OMC as shown in Figure 2.15. As the water content and compressibility of the clay increases the stones are able to press down into the clay, reducing their effect on the GM.
Figure 2.14. Effectiveness of protection layers in preventing GM tensile strains from gravel particles initially flush with CCL surface. Initial clay water content of 16% and normal stress of 1000 kPa (Brachman and Sabir 2010).

Figure 2.15. Influence of CCL water content on GM tensile strain for stones initially on top of clay surface at 200 kPa normal stress (Brachman and Sabir 2010).
2.3 Theoretical Puncture Analysis and Protection Design

2.3.1 Giroud et al. (1995)

A theoretical analysis of geomembrane puncture was carried out by Giroud et al. (1995). The method is limited to geomembranes that rupture or yield at less than 57% strain. HDPE geomembranes are a suitable material as they typically yield at strains on the order of 10-15%. Using a simple model, relationships are presented to predict GM puncture for a design based on a set of known puncture parameters. In these relationships, puncture data from a laboratory test or field performance evaluation are used to predict puncture for a GM design case with comparable conditions and materials.

GM puncture resistance is shown to depend on the size and angularity of stones as well as the thickness of the geomembrane. Puncture resistance can be increased by reducing the particle size, reducing particle angularity, and increasing the GM thickness. A drawback of the analysis is the assumption of a circular contact area between the GM and stone. For field conditions with stones of irregular, plate-like shape the presented relationships may be of limited value.

2.3.2 Narejo et al. (1996)

Despite the known risk of GM puncture and the need for protection materials, puncture design in years past was commonly considered in an arbitrary manner due to the lack of a rational design method. To address this issue a comprehensive three part study was carried out and a puncture design method proposed by Narejo et al. (1996).
The puncture resistance of HDPE GMs was evaluated through truncated cone testing up to 1100 kPa in the laboratory, and a set of design equations were then developed based on the results. The testing program accounted for multiple variables, including: GM thickness, use of a GT protection layer, short and long term puncture strength, particle angularity, and protrusion height.

For an unprotected GM, vertical pressures at GM puncture are listed in Table 2.3 for different GM thicknesses and cone heights (CH). For comparison, 100 kPa corresponds to approximately 8 m of municipal solid waste. As would be expected, increasing GM thickness and decreasing cone height result in increasing pressures at failure. From these results, the authors suggest that the use of a protection material is necessary for protrusions greater than 12 mm.

Table 2.3. HDPE GM puncture failure pressures under static loading as determined by truncated cone test (Narejo et al. 1996).

<table>
<thead>
<tr>
<th>Geomembrane thickness (mm)</th>
<th>Failure pressure (kPa)</th>
<th>Cone height (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>50</td>
</tr>
<tr>
<td>1.0</td>
<td>21</td>
<td>28</td>
</tr>
<tr>
<td>1.5</td>
<td>34</td>
<td>55</td>
</tr>
<tr>
<td>2.0</td>
<td>48</td>
<td>62</td>
</tr>
</tbody>
</table>

NW-NP GT protection layers with mass per unit areas of 130-1350 g/m² were then tested. The results shown in Figure 2.16 display a linear relationship between the mass per unit area and failure pressure for each cone height. Testing was also performed to determine the effects of protrusion height and particle angularity, shown
in Figure 2.17. For each stone shape, pressure to puncture the GM decreased with increasing protrusion height. Angular stones represented the worst case, with the lowest failure pressure for each protrusion height. Subrounded stones had geomembrane pressures two times greater and rounded stones four times greater than an angular stone of the same size. Failure pressures for packed stones were found to be much higher than for an isolated stone, with an effective protrusion height for packed stones determined to be half the maximum stone size.

![Graph](image)

Figure 2.16. Failure pressure at GM puncture versus mass per unit area of geotextile protective layer at various cone heights (Narejo et al. 1996).
Based on this experimental data, a design formula is determined to predict the allowable pressure for HDPE GMs both with and without GT protection. From the short term puncture behavior, Equation (2.1) can be used to determine the maximum allowable pressure \( p_{allow} \) on a GM with a GT protective layer of mass per unit area \( M_A \) and effective protrusion height \( H \). Units for \( M_A \) are g/m\(^2\) and \( H \) is in mm. A pressure of 50 kPa is used conservatively as the failure pressure for a GM with no protection material.

\[
p_{allow} = 450 \frac{M_A}{H^2} \geq 50 \text{ kPa} \tag{2.1}
\]

A series of modification factors are then applied to correlate the experimental data to actual field conditions. Modification factors for protrusion shape \( MF_S \), packing density \( MF_{PD} \) and soil arching \( MF_A \) have values less than or equal to 1.0.
since experiments were conducted for worst-case conditions. Safety factors for creep ($FS_{CR}$) and chemical/biological degradation ($FS_{CBD}$) are also added and have values of 1.0 or greater since long periods of time are required for their effect. The modified allowable pressure ($p'_{allow}$), representative of field conditions, can be found using Equation (2.2).

$$p'_{allow} = p_{allow} \left( \frac{1}{MF_S \times MF_{PD} \times MF_A} \right) \left( \frac{1}{FS_{CR} \times FS_{CBD}} \right)$$

(2.2)

A global factor of safety should be used in conjunction with Equation (2.2) to account for uncertainties relating to site specific conditions. The authors recommended a value of 3.0 or greater.

In the design of a liner system, the mass per unit area ($M_A$) of the geotextile is unknown and must be determined by the engineer. The maximum pressure on the GM can be determined from the depth and unit weight of material on top of the GM. Multiplying this result by the global factor of safety will yield the modified allowable pressure on the left-hand side of Equation (2.2). Using the appropriate factors for site conditions, the equation can be back-solved to determine the necessary geotextile to be used as a protective layer. The above equations are referenced by Bonaparte et al. (2002) for use as a geotextile design method.

While the above equations are useful for GMs in direct contact with isolated protrusions or gravel layers, no mention is made concerning applicability to a gravelly CCL. For a smooth rolled CCL with gravel particles initially flush with the CCL
surface effective protrusion height \( (H) \) in Equation (2.1) would be zero, suggesting that no protection layer is needed.

### 2.4 Geosynthetic Shear Testing

#### 2.4.1 Athanassopoulos et al. (2009)

In addition to static puncture testing, Athanassopoulos et al. (2009) also tested composite liners subjected shear. Instead of a typical 300 × 300 mm shear box, a reduced 150 × 150 mm box was used to increase maximum normal stress \( (\sigma) \) to 2758 kPa. Testing was performed using Bentomat DN and textured LLDPE geomembranes to match heap leach pad recommendations. Two types of geomembrane texturing were tested; embossed and co-extruded. Peak \( (\tau_p) \) and large-displacement \( (\tau_{ld}) \) shear envelopes for both GM types are shown in Figure 2.18. All specimens failed at the GM/GCL interface except the co-extruded texturing test at 2758 kPa, which experienced partial internal failure of the GCL. A peak secant friction angle of 20 degrees and a large-displacement secant friction angle of 7 degrees were found for LLDPE geomembranes with embossed texturing. For geomembranes with co-extruded texturing, peak secant friction angle was 19 degrees and large-displacement secant friction angle was 6 degrees. These values are much higher than expected, due to the fact that sliding occurred at the GM/GCL interface and internal reinforcement of the GCL remained intact after shearing.
Figure 2.18. Peak and large displacement shear strength envelopes for GCL/GM interface with (a) co-extruded GM texturing (b) embossed GM texturing (Athanassopoulos et al. 2009).

2.4.2 Fox et al. (2011)

Geomembrane damage due to static pressure, cyclic loading, and large-displacement static shear over compacted gravelly sand was investigated by Fox et al. (2011). The study was conducted to evaluate geomembrane integrity for a mineral reclamation facility under specific operational conditions. Specimens consisted of compacted subgrade soil with 25% gravel, a geomembrane, and an overlying potash salt. Both HDPE and LLDPE smooth geomembranes were used in the study. All tests were performed at a normal stress of 958 kPa and consisted of a static pressure stage and a shearing stage.

Results from the static pressure stage showed no holes and only minor to moderate dimples were created in the geomembrane after the application of normal
stress for 24 h. Figure 2.19 shows the LLDPE geomembrane after the static pressure stage and highlights dimpling created by the gravelly subgrade. Specimens were also subjected to static shear to a final displacement of 130 mm. Visual inspection after shearing showed that significant damage occurred as a result of interface shear between the geomembrane and subgrade soil. Scratches and gouges in the geomembrane approximately 130 mm in length were evident post-shear. However, post-shearing inspection revealed no holes were created in the LLDPE specimen, as seen in Figure 2.20. Of note was the fact that each damage mark was created by a gravel particle which protruded from the surface of the subgrade. The HDPE geomembrane displayed similar results, with scratching and gouging but no holes after shearing.
Figure 2.19. LLDPE after static pressure stage (Fox et al. 2011).

Figure 2.20. LLDPE after static shearing stage (Fox et al. 2011).
2.4.3 Fox and Ross (2011)

Fox and Ross (2011) sought to clarify the relationship between GCL internal and GM/GCL interface shear strengths at peak and large displacement conditions. The same shear testing equipment and similar procedures were used as in the current study. Bentomat DN, a NW-NP GCL, and a 1.5 mm HDPE GM were used in the study. GMs had single-sided micro spike texturing. GCL specimens were hydrated and allowed free access to water during testing. All tests used the large scale direct shear machine designed by the author. The data from this study is unique for its large specimen size (305 × 1067 mm), large maximum normal stress (2071 kPa) and large shear displacement (200 mm). Internal GCL shear testing was performed by placing specimens between rigid metal plates, each with an aggressive gripping surface, forcing the specimen to fail internally. Interface testing required setting the GCL on top of one of these metal plates while the smooth side of the GM was glued to a metal pullout plate. This allowed specimens to fail either internally within the GCL or at the GM/GCL interface. Specimens were sheared at the rate of 0.1 mm/min, corresponding to the maximum recommended value for internal shear testing of a hydrated GCL.

Shear stress-displacement curves for both internal and interface shear are shown in Figure 2.21. Both failure modes display significant post-peak strength reduction over the range of normal stresses tested. For GM/GCL specimens, failure mode transitioned from interface failure at low normal stresses to internal GCL failure at high normal stress. This transition occurred between 692 and 2071 kPa normal stress. For each normal stress tested GM/GCL specimens had lower peak strength than
the GCL specimens, while at low normal stress levels GM/GCL specimens had higher large displacement strength. With the exception of large displacement GCL internal shear, which reached residual conditions, failure envelopes were nonlinear and are shown in Figure 2.22. This finding is significant, as linear extrapolation of shear strength data for design purposes may significantly overestimate the available strength of these materials.

Figure 2.21. Shear stress-displacement relationships for (a) internal GCL shear, (b) GM/GCL interface shear (Fox and Ross 2011).
2.5 Landfill Liner Systems

2.5.1 Giroud and Touze-Foltz (2003)

A special discussion on geomembranes in landfills was held at the 7th International Conference on Geosynthetics in cooperation with Geosynthetics International. The edited transcript of this discussion was published by Giroud and Touze-Foltz (2003) and is significant for its inclusion of both practitioners and researchers. One topic of note in this discussion was how holes are created in geomembrane liners. Approximately 66-80% of holes are created from the placement of cover soils and waste layers within a landfill; estimates vary between panel members. This results in 10-25 holes per hectare of GM area. The main source of damage was the placement of inappropriate material such as coarse stones directly on top of the geomembrane. Damage of this nature can easily be avoided by proper design and selection of materials.

Another topic of discussion was long term damage (10-20 years) to geomembranes. It is speculated that locations in a GM with large tensile strains will
eventually turn into holes, even if no hole is present at the end of construction. The need for proper amount of GM protection is stressed. More rigorous GM protection standards are used in Germany (including the use of sand cushions) than compared to North America, which testing has shown to significantly reduce tensile strains created in the GM. The importance of hole location within the GM was also stressed, as creation of a hole at the same location as a GM wrinkle will substantially increase the amount of leakage through the hole than if the location had good contact conditions between the GM and underlying soil.

One last item of note was the widespread use of GM liners in industrial ponds. In the United States there are approximately 4000 landfills, however, there are over 200,000 industrial ponds. This would suggest that pond liners may currently pose a larger environmental concern than landfills.

2.6 Summary

Lining systems with geosynthetic products are mandated for a variety of projects including landfills, hazardous waste sites, heap leach pads, and industrial ponds. Of particular concern for these liners is damage to the geomembrane, resulting in puncture of the GM and the formation of a hole in the liner. For waste containment sites, a hole in the lining system results in harmful leachate being introduced to the surrounding environment. In mining applications leakage also translates to lower metal recovery rates, at potentially significant monetary value. The most common
source of GM puncture in the field is granular material, present either in the CCL or as a drainage layer above the GM.

Due to the risk of GM puncture, numerous studies have investigated GMs in contact with granular material. Most of this research is focused on granular material in the drainage layer above the GM. In this situation a GT or sand layer may be placed between the GM and granular material to act as a cushion and protect the GM from puncture. Review of the literature reveals that it is a common occurrence for a natural CCL to contain gravel. It has also been shown that the interactions between gravel and a liner system are complex. Compaction and permeability of the liner may actually improve with the addition of gravel for contents up to approximately 50%. In practice, CCL design recommendations typically limit gravel content to 20% and maximum particle size to 25-50 mm. Use of a GT or GCL protective layer between a gravelly subgrade and GM has been shown to reduce GM strains. While a GT protective layer may reduce the possibility of damage, the high hydraulic conductivity of this layer will significantly increase flow rate through the liner if GM puncture does occur. GCLs present a more attractive option as a protection layer below the GM due to their extremely low hydraulic conductivity properties.

2.7 Need for Current Research

Previous research on GM puncture has primarily focused on the drainage layer above the GM. Puncture due to granular material in the subgrade below has not received comparable attention. The majority of this work has investigated GM damage
due to static pressure. However, liners can be subjected to large shear forces (e.g. waste settlement, seismic loading, slopes) which can lead to global and/or local shear failure of the liner system. Only one study (Fox et al. 2011) has investigated effects due to both static pressure and shearing. The current work is a continuation of this study, and is a broader investigation of GM sheared over a gravelly CCL subgrade.
3  EQUIPMENT AND TESTING PROCEDURE

3.1 Large Dynamic Direct Shear Machine

All tests were performed using the large dynamic direct shear machine described in detail by Fox et al. (2006). This section highlights the unique capabilities of the machine and details modifications made for the current testing program. The machine is shown in Figure 3.1. Plan and profile drawings of the machine are shown in Figure 3.2 and a detail of the test chamber is in Figure 3.3. Dimensions of the test chamber are 305 × 1067 mm in plan, with the long dimension in the direction of shearing. The floor of the test chamber is fitted with modified metal connector plates (i.e., truss plates) that are designed for use in wood truss construction. These truss plates have teeth protruding from one side, and provide sufficient a gripping surface such that end clamping of specimens is not needed. Specimens are placed on the floor of the test chamber and a steel pullout plate is placed on top. The pullout plate is of the same width but extends beyond the length of the chamber, so that a constant shearing surface area is maintained during testing. A layer of free-rolling stainless steel balls is added on top of the pullout plate to reduce machine friction during shearing. The balls also extend beyond the length of the test chamber so that distribution of normal force on the pullout plate remains constant during testing. Thin stainless steel bearing plates are placed above and below the layer of balls to prevent indentation and damage to components made of softer metals at high normal stresses. With this system in place, the shear stress due to machine friction has been measured at 0.27% of the applied normal stress.
Figure 3.1. Large dynamic direct shear machine (Fox et al. 2006).

Figure 3.2. Large dynamic direct shear machine (a) plan view (b) profile view (Fox et al. 2006).
Normal stress is applied by means of two specially designed rubber air bags. An aluminum load plate is placed on top of the upper steel bearing plate, to ensure force from the airbags is distributed evenly across the specimen. Air bags are contained by metal top plates and spacer blocks above. A series of reaction beams spans the width of the machine at the top of the test chamber. These beams are bolted into the machine and provide a reaction force for the air bags. The air bags are capable of pressures up to 2069 kPa. Face plates are bolted across the front and back of the test chamber so the load plate and airbags do not displace during shear.

Specimen shearing is achieved by attaching the end of the pullout plate to a hydraulic actuator. The actuator has a capacity of 245 kN and is capable of both static and dynamic shear loading; static displacement rates as low as 0.01 mm/min and as high as 30,000 mm/min can be achieved. Maximum displacement of the actuator piston is 254 mm, in most cases large enough to measure residual or near-residual shear strengths. The actuator is bolted to the machine frame through slotted holes, allowing 250 mm of vertical travel to accommodate specimens of varying thickness.
3.2 Machine Modifications

As described in the previous section, normal stress is applied via two air bags located above the pullout plate in the shear machine. These airbags are capable of producing normal stresses up to 2069 kPa. One goal of this study was to investigate GM puncture damage for higher normal stresses for which no data currently exists. For example, heap leach pads can impose normal stresses up to 3450 kPa on a liner system (Athanassopoulos et al. 2009). As originally designed, the shear machine was incapable of creating the necessary conditions. To increase the maximum normal stress, a new pullout plate was designed for use in the current study. The top of this pullout plate measures 305 mm in width, allowing the normal stress to be applied using the same components as before. However, the bottom of the pullout plate in contact with the specimen was reduced to 152 mm in width. The result is a shearing area with dimensions 152 \times 1067 mm in plan. This narrower shearing surface area concentrates the applied load from the air bags, producing normal stresses up to 4137 kPa.

Final normal stress values for each specimen must also include the weight of all machine components below the air bags, resulting in slightly larger values of applied normal stress than those specified above.
3.3 **Testing Materials**

3.3.1 **Geomembrane**

Geomembranes were cut from a sample roll of material manufactured and provided by GSE Lining Technology, LLC of Houston, Texas. GMs were made out of high-density polyethylene (HDPE) with a thickness of 1.5 mm (60 mil). Coextruded texturing was present on one side of the GM, with an average asperity height of 0.51 mm. Material properties as provided by the manufacturer for the GM are listed in Table 3.1. MD denotes machine direction and TD transverse direction.

<table>
<thead>
<tr>
<th>Average / Minimum Thickness (mm)</th>
<th>Density (g/cc)</th>
<th>MD Strength at Yield / Break (kN/m)</th>
<th>TD Strength at Yield / Break (kN/m)</th>
<th>MD Elongation at Yield / Break (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.47 / 1.42</td>
<td>0.945</td>
<td>28.0 / 35.6</td>
<td>27.1 / 30.1</td>
<td>16 / 586</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TD Elongation at Yield / Break (%)</th>
<th>MD / TD Tear Resistance (N)</th>
<th>Puncture Resistance (N)</th>
<th>Asperity Height (textured side) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>17 / 475</td>
<td>227 / 218</td>
<td>654</td>
<td>0.508</td>
</tr>
</tbody>
</table>

3.3.2 **Geosynthetic Clay Liner**

A test roll of geosynthetic clay liner (GCL) material was obtained from CETCO of Hoffman Estates, Illinois. The product is Bentomat DN, a nonwoven/nonwoven (NW/NW) needle-punched (NP) GCL with no thermal bonding. The GCLs contained granular bentonite at a dry mass per area of 4.9 kg/m². The bentonite is held between two NW polypropylene geotextiles, each with a mass per
unit area of 200 g/m². To provide reinforcement, fibers from the cover geotextile are needle-punched through both the bentonite and carrier geotextile. The manufacturer reported an average peel strength of 2231 N/m, as determined from five wide-width tests with a coefficient of variation (standard deviation/mean) equal to 8.5%. GCL specimens were cut to size prior to hydration with the long dimension of the specimen parallel to the factory roll direction.

3.3.3 Compacted Clay Liner

Raw source material for CCL specimens was obtained from the Tajiguas Landfill outside Santa Barbara, CA. The material was transported to the laboratory and processed to produce three soil types. For Soil #1, raw material was passed through a 12.5 mm sieve. The resulting soil had 7% gravel and 63% fines contents. Soil #2 removed all gravel particles by passing the soil through a 4.75 mm sieve. Soil #3 was prepared by first passing the material through a 4.75 mm sieve and then adding a gravel mixture with a maximum particle size of 19 mm. Gravel content for Soil #3 was 20% with a fines content of 54%. Particle size distribution curves for all three soils are in Figure 3.4. Since only the gravel content was altered between the soil types, the three soils have the same Atterberg Limit values. Liquid limit was equal to 56 and plastic limit was 29.
Standard and modified Proctor compaction tests were performed on Soils #1 and #3. Compaction curves are shown in Figure 3.5. The addition of 20% gravel content to the Tajiguas clay resulted in an increase in dry unit weight and a decrease in optimum water content (OMC) for both compaction methods. These trends are consistent with previous research as discussed in Section 2.1. Specific gravity for the zero-air-voids (ZAV) curve was estimated to be 2.75. A summary of the three soil types is in Table 3.2.
Figure 3.5. Standard proctor (SP) and modified proctor (MP) compaction curves for Soils #1 and #3.

<table>
<thead>
<tr>
<th>Classification</th>
<th>Soil #1</th>
<th>Soil #2</th>
<th>Soil #3</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Gravel</td>
<td>7</td>
<td>0</td>
<td>20</td>
</tr>
<tr>
<td>% Fines</td>
<td>63</td>
<td>69</td>
<td>54</td>
</tr>
<tr>
<td>OMC, Standard Proctor (%)</td>
<td>20</td>
<td>-</td>
<td>19</td>
</tr>
<tr>
<td>Maximum Dry Density, Standard Proctor (kN/m$^3$)</td>
<td>15.1</td>
<td>-</td>
<td>16.2</td>
</tr>
</tbody>
</table>

* Note: Proctor tests were not performed on Soil #2
3.3.4 Gravel

To prepare Soil #3, existing gravel particles in the raw Tajiguas material were removed by passing through a 4.75 mm sieve. Gravel particles were then added to bring the final soil mixture to 20% gravel content by dry weight. The added gravel was a combination of crushed and decomposed granite, both obtained locally. Maximum particle size was 19 mm. A particle size distribution plot for the gravel is shown in Figure 3.6. To represent worst case conditions rounded gravel particles were removed by hand prior to adding to the Tajiguas clay, leaving only angular and subangular particles. A photo of typical crushed granite particles is shown in Figure 3.7.

![Particle size distribution curve for gravel used to produce Soil #3.](image_url)
3.4 Procedures

Geomembrane damage effects due to gravel in the underlying compacted clay liner were investigated using multi-interface specimens. Two configurations of test specimens were used. GM/CCL tests used an unprotected GM in direct contact with the CCL subgrade. GM/GCL/CCL tests placed a hydrated GCL protection layer between the GM and CCL. A profile view of each configuration is illustrated in Figure 3.8. Test preparation and procedures were identical for the two configurations; the only difference being the addition of the GCL. A 25 mm thick layer of sand was placed above the GM for all tests. This allowed the GM to deform and puncture as necessary, and is representative of liners with an adequate protection layer from damage due to material above the GM.
Figure 3.8. Profile drawings of multi-interface specimens for (a) GM/CCL (b) GM/GCL/CCL.

3.4.1 CCL Preparation and Compaction

The CCL subgrade was composed of either Soil #1, #2 or #3 depending on the test. The soil was sieved and gravel added as needed according to the soil type. Tap water was added to bring the soil from the as-received condition to the target water content, which was 22% for the majority of tests. Soil mixtures were kept in sealed containers for 24 hours to allow even hydration throughout the sample prior to placement in the test chamber. All CCL subgrades were compacted with dimensions 305 × 1067 mm in plan with a 75 mm thickness. Compaction was performed by hand in two equal lifts using the end of a 4 × 6 wood tamper, seen in Figure 3.9. A
A carpenter’s level was used to keep the CCL surface level throughout the compaction process. Four passes were made on the first lift, while only three passes were made on the second lift to ensure a slightly rougher testing surface. A typical CCL subgrade after the compaction process using Soil #3 is shown in Figure 3.10. Gravel particles are lighter in color than the clay and are flush with the subgrade surface with no outward protrusions. After compaction 100 cc of tap water was sprayed on top of the CCL surface. This wetted the shearing surface and prevented excessive drying during testing. CCL subgrades were removed from the test chamber after shearing and replaced with new material for each test.

Figure 3.9. Wooden 4 x 6 inch hand tamper.
Figure 3.10. Typical CCL subgrade with Soil #3 after compaction and before normal stress was applied.

3.4.2 GCL Hydration

GCLs were cut from a sample roll of Bentomat DN, with the long dimension parallel to the roll direction. GCL specimens were cut longer than 1067 mm in length to allow material to be drawn into the machine during shear, if needed. Needle-punched reinforcement was removed from the GCL for areas outside the initial 1067 mm specimen length to maintain proper shear surface area. To simulate field conditions, GCLs were tested in a hydrated state. Previous GCL shear testing by Fox et al. (1998) has shown that a two-stage hydration procedure can be used in order to accelerate the hydration procedure. This process was adapted for the current program. Once the GCL specimen had been cut to its proper dimensions, it was placed in a shallow pan. The appropriate amount of tap water was added to increase the GCL to
its expected final water content. The expected GCL final water content was obtained from previous testing using similar GCL products at similar normal stress levels, (e.g., Fox and Ross 2011). For normal stresses where no data was available, final water content was estimated by extrapolation. The pan was then covered to prevent evaporation and the GCL was subjected to a normal stress of 1 kPa. Specimens were allowed to cure for a minimum 24 hours under these conditions before being placed in the shear machine. Once placed in the test chamber, no additional water was added to the GCL specimens.

3.4.3 Specimen Assembly

Assembly of test specimens began with the preparation, compaction, and wetting of the CCL subgrade (Section 3.4.1). Hydrated GCL (if required) and GM were then placed on top of the subgrade. Both GCL and GM specimens were cut longer than the 1067 mm length of the test chamber to allow material to be drawn in during shear and maintain a constant shearing surface area. GM specimens were placed smooth side down in contact with either the CCL or GCL. This was done to ensure shear failure occurred at the GM/CCL or GM/GCL interface, in line with the primary purpose of the investigation.

A layer of clean medium sand 25 mm thick was placed on top of the GM specimen. This layer is representative of a protection layer while still allowing GM deformation and damage due to the CCL subgrade. Sand was lightly compacted using the wooden hand tamper. For specimens with a reduced 152 × 1067 mm shearing
surface area, a specially constructed aluminum confinement box was used to make sure the sand was at a constant width of 152 mm throughout the shearing process (Figure 3.11). Sand was added on top of the GM and within the confinement area. Threaded bolts across the width of the confinement kept the width uniform along the length of the specimen, even under high normal stresses. These bolts also provided vertical support for the confinement box, allowing a small gap between the GM and confinement box so that no additional frictional forces were created. Cross members were sized to allow the narrow pullout plate to fit inside the confinement box and be in direct contact with the sand.

Figure 3.11. Sand confinement box used on narrow specimens to maintain a reduced width of 152 mm.
Pullout plates were placed directly on top of the sand layer. A GM was glued smooth side against the pullout plate using a high-strength epoxy. The textured side of the GM against the sand provided sufficient gripping to transmit shear force from the pullout plate to the specimen. The position of the glued GM was carefully marked on the pullout plate and any tests with interface displacement at this surface were rejected. A new GM was attached for each test. Remaining machine components were then positioned on top of the pullout place as outlined in Section 3.1. Air pressure was applied at a rate of approximately 170 kPa every 10 minutes until the required normal stress level was reached. Air pressure was kept constant with the use of a high pressure digital gauge.

3.4.4 Static Pressure Stage

After 24 hours under static pressure, normal stress was released and machine components were disassembled and removed from above the specimen. Position of the GM inside the test chamber was marked, and the GM was removed from the machine. An assessment of damage was performed and any dimples, indentations, scratches, gouges, wrinkles or other deformations in the GM were noted and photographed. Holes in the GM were identified using a bright light test. For the bright light test, GM specimens are placed in between a halogen lamp and an observer inside a room with no other light source. The lamp is moved along length of the GM, with the observer visually looking for any locations of light passing through a hole. Any holes present were counted and two perpendicular measurements of the width and length of the hole.
were taken with calipers and averaged to obtain the diameter of the hole. CCL subgrades were also visually inspected and photographed at this stage. After visual inspection of specimens was completed they were placed back into the test chamber in their original location and again subjected to normal stress.

3.4.5 Shearing Stage

Specimens were kept under pressure for an additional 24 hours after the static pressure stage. The entire multi-interface specimen was then sheared under constant normal stress. Shearing was performed at the rate of 1.0 mm/min to a final large displacement of 150 mm. Upon completion of shearing, the specimen was immediately removed from the machine and was again visually inspected for signs of damage. Three samples of the CCL subgrade at the shearing surface were taken with a sharp edge. For specimens with a hydrated GCL protection layer, three 102 × 102 mm samples were cut out and removed. Each of these samples was dried 24 hours in an oven to obtain final water content values.

Two full depth samples of the CCL were also taken to calculate dry unit weight of the CCL layer. Due to the gravel content of the CCL, sample tubes could not be used as the tubes were likely to disturb gravel particles during the driving process. Therefore, a wax coating and water immersion method was used. A sample of the CCL subgrade was removed from the test chamber using a chisel and hammer. Samples were irregular in shape but contained material from the full depth of the subgrade. The wet mass of the sample, $M_{wet}$, was obtained using a digital scale.
Samples were then dipped into a liquid mixture of paraffin and petroleum jelly with a specific gravity, $S_G$, equal to 0.9. Care was taken to guarantee the entire surface of the sample was coated with the paraffin mixture. The sample was again weighed to obtain $M_{coated}$. Subtracting $M_{wet}$ from $M_{coated}$ yielded the mass of paraffin coating the sample, $M_{paraffin}$. The sample was then placed into a graduated cylinder with a known volume of water. With the sample completely covered in paraffin, no water would be absorbed into the soil. With the sample fully submerged, the change in volume of water inside the graduated cylinder was equal to the volume of the coated sample, $V_{coated}$.

With the mass and specific gravity of the paraffin mixture coated on the sample known, the volume of paraffin was calculated using Equation (3.1). The density of water, $\rho_w$, is equal to 1.0 g/mL.

$$V_{paraffin} = \frac{M_{paraffin}}{S_G \rho_w} \tag{3.1}$$

Volume of the original wet sample, $V_{wet}$, was found by subtracting $V_{coated}$ and $V_{paraffin}$. Using the water content, $w_{wet}$, determined from an additional sample of CCL taken near the location of the first sample the dry unit weight, $\gamma_{dry}$, of the subgrade was calculated with Equation (3.2).

$$\gamma_{dry} = \frac{M_{wet}}{V_{wet}(1 + w_{wet})} = \frac{M_{wet}}{(V_{coated} - V_{paraffin})(1 + w_{wet})} \tag{3.2}$$
3.5 Testing Program

The complete experimental program is presented in Table 3.3. Tests 1A-5A investigated the effect of normal stress on GM damage for a CCL with 20% gravel content compacted wet of optimum \(w = 22\%\). Test 5P is a repeat of Test 5A, except that the shearing process was halted intermittently to investigate the effect of shear displacement on GM damage. Shearing was stopped at displacements of 11, 27, 52, 104 and 150 mm for this test. The GM specimen was removed from the machine and visually inspected for damage after each shearing interval before being placed back in the machine and shearing continued. Additional tests investigated the effect of CCL gravel content (Tests 3C, 3D, 5C, 5D) and CCL water content (5L1, 5L2, 5H1, 5H2). The use of a hydrated NP GCL protection layer to prevent GM damage was investigated in Tests 2B-7B.

This testing program represents an advance over previous testing of GM damage from two standpoints. First, specimens were inspected for damage after both static pressure and after 150 mm of shearing displacement. Damage due to shearing has been neglected by previous studies. Second, the tests were conducted over a very wide range of normal stresses. GM/CCL specimens were tested at normal stresses ranging from \(\sigma_n = 72\) to 1658 kPa while GM/GCL/CCL specimens were tested from \(\sigma_n = 348\) to 4145 kPa.
Table 3.3. Overview of experimental test program.

<table>
<thead>
<tr>
<th>Test</th>
<th>Normal Stress $\sigma_n$ (kPa)</th>
<th>CCL</th>
<th>Specimen Configuration</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Soil Type (gravel content %)</td>
<td>Target w (%)</td>
</tr>
<tr>
<td>1A</td>
<td>72</td>
<td>Soil #3 (20)</td>
<td>22</td>
</tr>
<tr>
<td>2A</td>
<td>348</td>
<td>Soil #3 (20)</td>
<td>22</td>
</tr>
<tr>
<td>3A</td>
<td>693</td>
<td>Soil #3 (20)</td>
<td>22</td>
</tr>
<tr>
<td>4A</td>
<td>1176</td>
<td>Soil #3 (20)</td>
<td>22</td>
</tr>
<tr>
<td>5A</td>
<td>1658</td>
<td>Soil #3 (20)</td>
<td>22</td>
</tr>
<tr>
<td>5P</td>
<td>1658</td>
<td>Soil #3 (20)</td>
<td>22</td>
</tr>
<tr>
<td>3C</td>
<td>693</td>
<td>Soil #1 (7)</td>
<td>22</td>
</tr>
<tr>
<td>5C</td>
<td>1658</td>
<td>Soil #1 (7)</td>
<td>22</td>
</tr>
<tr>
<td>3D</td>
<td>693</td>
<td>Soil #2 (0)</td>
<td>22</td>
</tr>
<tr>
<td>5D</td>
<td>1658</td>
<td>Soil #2 (0)</td>
<td>22</td>
</tr>
<tr>
<td>5L1</td>
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<td>Soil #3 (20)</td>
<td>18</td>
</tr>
<tr>
<td>5L2</td>
<td>1658</td>
<td>Soil #3 (20)</td>
<td>18</td>
</tr>
<tr>
<td>5H1</td>
<td>1658</td>
<td>Soil #3 (20)</td>
<td>26</td>
</tr>
<tr>
<td>5H2</td>
<td>1658</td>
<td>Soil #3 (20)</td>
<td>26</td>
</tr>
<tr>
<td>2B</td>
<td>348</td>
<td>Soil #3 (20)</td>
<td>22</td>
</tr>
<tr>
<td>3B</td>
<td>693</td>
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<td>4B</td>
<td>1176</td>
<td>Soil #3 (20)</td>
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<td>5B</td>
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<tr>
<td>6B</td>
<td>2146</td>
<td>Soil #3 (20)</td>
<td>22</td>
</tr>
<tr>
<td>7B</td>
<td>4145</td>
<td>Soil #3 (20)</td>
<td>22</td>
</tr>
</tbody>
</table>
4 RESULTS

4.1 GM/GCL: Unprotected Geomembrane

4.1.1 Static Pressure

Inspection of unprotected geomembrane specimens for Tests 1A-5A after the 24 hour static pressure stage revealed minimal damage and no puncturing. For the lowest normal stress \( \sigma_n = 72 \text{ kPa} \) virtually no deformations were evident upon visual inspection. With increasing normal stress, deformations of the GM increased. Moderate indentations (“dimples”) were visible in the GM as the clay compressed under pressure and the gravel particles protruded outward from the CCL surface. The most significant deformations occurred for specimens at the highest normal stress levels. Photographs of test 4A \( \sigma_n = 1176 \text{ kPa} \) are shown in Figure 4.1 and Figure 4.2. The CCL subgrade at this stage exhibited a rougher surface than after compaction (Figure 3.10) with gravel particles now slightly protruding above the clay. A photograph for the highest normal stress, Test 5A \( \sigma_n = 1658 \text{ kPa} \), is shown in Figure 4.3. Bright light tests determined that no GM holes were present for any specimen after the static pressure stage.
Figure 4.1. CCL subgrade for Test 4A after static pressure stage.

Figure 4.2. Textured side of GM specimen for Test 4A after static pressure stage.
4.1.2 Shearing

Key data for the shear stage of the GM/CCL tests is provided in Table 4.1. Visual inspection of GMs after the shearing stage revealed damage levels much higher than that due to static pressure. The GM specimen for Test 1A exhibited shallow scratches 150 mm long in the direction of shear but was otherwise free from damage. Photographs of GM specimens for tests 2A through 5A are shown in Figure 4.4 through Figure 4.8. Increasing normal stress resulted in greater levels of GM damage. Test 2A produced significant scratching and gouging in the GM, but the bright light test indicated no holes were created. Deep gouges and wrinkles were present in Test 3A, and a total of 13 holes with a maximum size of 16.8 mm were measured. Hole locations were often in combination with wrinkling of the GM. The highest level of
damage was experienced by Test 5A, in which severe wrinkling and gouging of the GM occurred. This specimen contained a total of 55 holes and a maximum hole size of 23.4 mm. A photograph of the CCL subgrade after shearing is shown in Figure 4.9. Localized shear displacements and large gravel protrusions that were responsible for damage to the GM can be observed. GM damage is summarized in Figure 4.10, clearly showing a trend of increasing hole count and maximum hole size with increasing normal stress. Maximum hole size was actually larger than the maximum particle size (19 mm) for Tests 4A and 5A.

<table>
<thead>
<tr>
<th>Test</th>
<th>Normal Stress $\sigma_n$ (kPa)</th>
<th>Peak Shear Strength $\tau_p$ (kPa)</th>
<th>Large Displ. Shear Strength $\tau_{ld}$ (kPa)</th>
<th># GM Holes $n$</th>
<th>Max. GM Hole Size $s$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>72</td>
<td>31.4</td>
<td>31.1</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>2A</td>
<td>348</td>
<td>123</td>
<td>107</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>3A</td>
<td>693</td>
<td>271</td>
<td>271</td>
<td>13</td>
<td>16.8</td>
</tr>
<tr>
<td>4A</td>
<td>1176</td>
<td>372</td>
<td>318</td>
<td>13</td>
<td>20.6</td>
</tr>
<tr>
<td>5A</td>
<td>1658</td>
<td>722</td>
<td>627</td>
<td>55</td>
<td>23.4</td>
</tr>
<tr>
<td>5P</td>
<td>1658</td>
<td>610</td>
<td>578</td>
<td>32</td>
<td>17.0</td>
</tr>
</tbody>
</table>
Figure 4.4. Smooth side of GM for Test 2A after shearing.

Figure 4.5. Smooth side of GM for Test 3A after shearing.
Figure 4.6. Smooth side of GM for Test 4A after shearing.

Figure 4.7. Smooth side of GM for Test 5A after shearing.
Figure 4.8. Textured side of GM for Test 5A after shearing.

Figure 4.9. CCL surface for Test 5A after shearing.
4.1.3 Effect of Shear Displacement

Shearing was stopped several times during Test 5P to evaluate the progress of GM damage and the effects of shear displacement. Although it is likely that the discontinuous shearing process altered the results, shear strengths and final damage of the specimen are comparable to those for Test 5A as seen in Table 4.1. Stress-displacement relationships and damage measurements for each shearing interval are listed in Table 4.2 and shown in Figure 4.11. Inspection at 11 and 27 mm displacement indicated heavy dimpling but no holes in the GM. Peak shear strength occurred at a displacement of 52 mm. At peak strength one hole was present in the GM, though the hole was not large enough to be measured with calipers. After peak
strength GM damage continued to increase to 32 total holes with a maximum size of 17.0 mm at a displacement of 150 mm.

Table 4.2. Shear and damage data as a function of displacement in Test 5P.

<table>
<thead>
<tr>
<th>Shearing Increment</th>
<th>Displacement Δ (mm)</th>
<th>Shear Strength τ (kPa)</th>
<th># GM Holes n</th>
<th>Max. GM Hole Size s (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5P1</td>
<td>11</td>
<td>566</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5P2</td>
<td>27</td>
<td>597</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5P3</td>
<td>52</td>
<td>610</td>
<td>1</td>
<td>0.01</td>
</tr>
<tr>
<td>5P4</td>
<td>104</td>
<td>584</td>
<td>16</td>
<td>7.0</td>
</tr>
<tr>
<td>5P5</td>
<td>150</td>
<td>578</td>
<td>32</td>
<td>17.0</td>
</tr>
</tbody>
</table>

Figure 4.11. Effect of shear displacement on shear strength and GM damage for test 5P.
Although only one hole was created at the peak strength condition, damage at this point was still significant. Deep indentation and gouging was visible as shown in Figure 4.12, which would severely affect the overall long-term performance of the GM. Final damage of the GM after 150 mm displacement is shown in Figure 4.13. As with previous GM/CCL tests, holes were often accompanied by severe wrinkling of the GM.

![Image](image126x271to522x535_199x690_151x690_69)

Figure 4.12. Test 5P at peak strength condition, 52 mm displacement.
4.1.4 Effect of Gravel Content

Tests at 693 and 1658 kPa were repeated for an unprotected GM using Soil #1 (Tests 3C and 5C) and Soil #2 (Tests 3D and 5D) to investigate the effect of CCL gravel content. Results are summarized in Table 4.3. Much less damage was observed for these tests conducted with lower CCL gravel contents. Recall that Soil #1 has 7%
gravel content and a maximum particle size of 12.5 mm. The GM for Test 5C exhibited deep scratches on the bottom surface in contact with the CCL while the top textured side showed little evidence of damage as shown in Figure 4.14. No holes were created in the GM. This is in stark contrast to damage for Test 5A (Figure 4.7 and Figure 4.8) over Soil #3 where a total of 55 holes were created in the GM. Even at this reduced damage level, final condition of the GM for Test 5C would likely be considered marginal with regard to long-term performance. Test 3C was conducted at a lower normal stress and displayed less damage and similar trends as Test 5C.

Table 4.3. Summary of shearing and GM damage for GM/CCL tests and the effect of CCL gravel content.

<table>
<thead>
<tr>
<th>Test</th>
<th>Normal Stress $\sigma_n$ (kPa)</th>
<th>Peak Shear Strength $\tau_p$ (kPa)</th>
<th>Large Displ. Shear Strength $\tau_{ld}$ (kPa)</th>
<th># GM Holes $n$</th>
<th>Max. GM Hole Size $s$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3C</td>
<td>693</td>
<td>199</td>
<td>111</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>3D</td>
<td>693</td>
<td>219</td>
<td>128</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>5C</td>
<td>1658</td>
<td>437</td>
<td>218</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>5D</td>
<td>1658</td>
<td>454</td>
<td>234</td>
<td>0</td>
<td>-</td>
</tr>
</tbody>
</table>
Soil #2 (no gravel) was used as a CCL subgrade for Test 5D. While some light scratching of the GM occurred, these would not be expected to seriously affect the long-term performance of the GM. A photograph is shown in Figure 4.15. Results for Test 3D were similar, with light scratching of the GM and no holes.
4.1.5  Effect of CCL Water Content

The effect of CCL water content on GM damage was evaluated using specimens compacted at ± 4% from the reference Test 5A (w = 22%). Results of these tests are provided in Table 4.4 and combined with damage results from Tests 5A and 5P in Figure 4.16. Significant scatter is present in the results. Test 5L1 at w = 18% had 12 holes with a maximum size of 7.1 mm while Test 5L2, under identical conditions, had 1 hole with a size of 14.2 mm. Tests 5H1 and 5H2 at w = 26% displayed slightly less scatter, with 12 and 6 holes measured for the two tests. Despite the variation in results, the general trend indicates less GM damage on either side of w = 22%. Recall that Test 5A had 55 holes after shearing. Photographs of GM specimens for Tests 5L1 and 5H2 are in Figure 4.17.
Table 4.4. Summary of shearing and GM damage for GM/CCL tests and the effect of CCL water content.

<table>
<thead>
<tr>
<th>Test</th>
<th>Normal Stress $\sigma_n$ (kPa)</th>
<th>Peak Shear Strength $\tau_p$ (kPa)</th>
<th>Large Displ. Shear Strength $\tau_{ld}$ (kPa)</th>
<th># GM Holes $n$</th>
<th>Max. GM Hole Size $s$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5L1</td>
<td>1658</td>
<td>480</td>
<td>351</td>
<td>12</td>
<td>7.1</td>
</tr>
<tr>
<td>5L2</td>
<td>1658</td>
<td>478</td>
<td>305</td>
<td>1</td>
<td>14.2</td>
</tr>
<tr>
<td>5H1</td>
<td>1658</td>
<td>379</td>
<td>332</td>
<td>12</td>
<td>14.0</td>
</tr>
<tr>
<td>5H2</td>
<td>1658</td>
<td>472</td>
<td>314</td>
<td>6</td>
<td>6.8</td>
</tr>
</tbody>
</table>

Figure 4.16. GM hole count and maximum hole size for varying CCL water content.
Figure 4.17. Smooth side of GM specimens after shear (a) Test 5L1 ($w = 18\%$) (b) Test 5H2 ($w = 26\%$).
Changes in CCL water content affected the compressibility of the clay matrix. Increasing the water content for Tests 5H decreased stiffness of the clay, which allowed gravel particles to push into the clay layer under pressure. This yielded reduced gravel protrusions from the CCL surface. Interestingly, this result is contrary to results published in Brachman and Sabir (2010). This study found that maximum GM strain increased as water content was increased from OMC for a gravelly CCL. One likely explanation is that GM damage will tend to increase with increasing water content from OMC. At some critical level, however, the clay will not be able to provide enough force to the gravel particles to puncture the GM and less damage will be observed. For Tests 5L with a lower CCL water content compressibility of the clay matrix was reduced, which reduced settlement of the clay around the gravel particles. These reduced gravel protrusions consequently yielded lower levels of GM damage.

4.2 **GM/GCL/CCL: GCL Protection Layer**

4.2.1 *Static Pressure*

Placement of a hydrated NP GCL at the GM/CCL interface greatly reduced the level of damage experienced due to static pressure. It was observed that the GCL was much thinner in locations where gravel protruded from the CCL and essentially eliminated most of the dimpling of the GM previously observed. Photographs for Test 5B are shown in Figure 4.18 through Figure 4.20. The CCL subgrade and a large gravel protrusion are highlighted in Figure 4.18. The underside of the GCL which was in contact with the CCL is shown in Figure 4.19. Areas in which the geotextile were in
contact with clay stained and are darker in color, while areas in contact with gravel particles remain white. The indentation shown in Figure 4.20 is caused by the gravel protrusion from the center of Figure 4.18. Other than this single indentation, the GM showed essentially no other signs of damage after static pressure. Similar results were found for normal stresses up to 4145 kPa.

Figure 4.18. CCL subgrade for Test 5B after static pressure stage.
Figure 4.19. Bottom of GCL for Test 5B after static pressure stage.

Figure 4.20. Smooth side of GM for Test 5B after static pressure stage.
4.2.2 Shearing

Results from the shearing stage of Tests 2B-7B are provided in Table 4.5. Relative to GM/CCL tests conducted under the same normal stress, GM specimens from Tests 2B-5B experienced far less damage due to shearing. Of these specimens, Test 5B experienced the most damage and is shown in Figure 4.21. Several shallow indentations equal to the maximum displacement length (150 mm) were observed but a bright light test revealed no holes in the GM. This is in stark contrast to Test 5A, in which 55 holes were created due to shear displacement. Additional tests were conducted at even higher normal stress levels, with the worst damage occurring for Test 7B ($\sigma_n = 4145$ kPa). As shown in Figure 4.22, general condition of the GM was very good with only light gouging and no holes present in the GM. Damage levels for Test 7B were similar to Test 5B, even though normal stress was 2.5 times larger. For all GM/GCL/CCL tests conducted, no holes were observed in any GM specimen after shearing. GCL specimens experienced reduced thickness at gravel particle locations, but no puncture of GCL specimens occurred for even the highest normal stress.
Table 4.5. Summary of shearing and GM damage for GM/GCL/CCL tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>Normal Stress $\sigma_n$ (kPa)</th>
<th>Peak Shear Strength $\tau_p$ (kPa)</th>
<th>Large Displ. Shear Strength $\tau_{ld}$ (kPa)</th>
<th># GM Holes $n$</th>
<th>Max. GM Hole Size $s$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2B</td>
<td>348</td>
<td>80</td>
<td>69</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>3B</td>
<td>693</td>
<td>157</td>
<td>109</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>4B</td>
<td>1176</td>
<td>271</td>
<td>208</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>5B</td>
<td>1658</td>
<td>367</td>
<td>259</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>6B</td>
<td>2146</td>
<td>430</td>
<td>295</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>7B</td>
<td>4145</td>
<td>643</td>
<td>487</td>
<td>0</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 4.21. Smooth side of GM for Test 5B after shear.
4.3 **CCL and GCL Water Contents**

Final water contents and dry unit weights are listed in Table 4.6 for GM/CCL tests and Table 4.7 for GM/GCL/CCL tests. Final CCL water contents are determined from a soil chunk representing the entire subgrade depth, while shearing surface values are shallow depth samples from the top of the CCL. Water contents are shown as a function of normal stress for test series A and B in Figure 4.23. Excluding tests at different water contents, 5L and 5H, final CCL water contents for GM/CCL tests vary from 19.4% to 21.6%. This is lower than the initial 22% water content; the length of time specimens are in the machine allows the CCL to slightly dry out. Except for Test 5L2, tests at different water contents displayed a similar trend with final CCL water content values lower than the initial target value. Final water contents for
GM/GCL/CCL tests were higher than for corresponding GM/CCL tests. Placement of the hydrated GCL on top of the CCL provided a source of water and was likely the cause of this difference. A general trend of decreasing final CCL water content with increasing normal stress is observed. Shearing surface values are generally lower than corresponding “whole” CCL water contents. This points to the formation of a thin hard top layer at the CCL surface due to large shear strains at this interface. GCL water contents for GM/GCL/CCL tests decrease in a non-linear fashion from 80.3% at \( \sigma_n = 348 \) kPa to 36.5% at \( \sigma_n = 4145 \) kPa. These values are in good agreement with previous studies for similar materials and testing conditions (e.g., Fox and Ross 2011).
Table 4.6. Final CCL properties after shear for GM/CCL tests.

<table>
<thead>
<tr>
<th>Test #</th>
<th>Normal Stress (\sigma_n) (kPa)</th>
<th>Soil Type</th>
<th>Target (w) (%)</th>
<th>Final (w) (%)</th>
<th>Shearing Surface (w) (%)</th>
<th>Final Dry Unit Weight (kN/m(^3))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>72</td>
<td>Soil #3</td>
<td>22</td>
<td>-</td>
<td>16.9</td>
<td>-</td>
</tr>
<tr>
<td>2A</td>
<td>348</td>
<td>Soil #3</td>
<td>22</td>
<td>21.0</td>
<td>17.7</td>
<td>16.1</td>
</tr>
<tr>
<td>3A</td>
<td>693</td>
<td>Soil #3</td>
<td>22</td>
<td>21.5</td>
<td>17.6</td>
<td>15.6</td>
</tr>
<tr>
<td>3C</td>
<td>693</td>
<td>Soil #1</td>
<td>22</td>
<td>21.6</td>
<td>22.1</td>
<td>15.4</td>
</tr>
<tr>
<td>3D</td>
<td>693</td>
<td>Soil #2</td>
<td>22</td>
<td>21.3</td>
<td>21.9</td>
<td>15.7</td>
</tr>
<tr>
<td>4A</td>
<td>1176</td>
<td>Soil #3</td>
<td>22</td>
<td>20.2</td>
<td>19.1</td>
<td>16.0</td>
</tr>
<tr>
<td>5A</td>
<td>1658</td>
<td>Soil #3</td>
<td>22</td>
<td>19.2</td>
<td>17.2</td>
<td>16.7</td>
</tr>
<tr>
<td>5C</td>
<td>1658</td>
<td>Soil #1</td>
<td>22</td>
<td>20.9</td>
<td>20.9</td>
<td>15.9</td>
</tr>
<tr>
<td>5D</td>
<td>1658</td>
<td>Soil #2</td>
<td>22</td>
<td>21.1</td>
<td>21.1</td>
<td>16.0</td>
</tr>
<tr>
<td>5P</td>
<td>1658</td>
<td>Soil #3</td>
<td>22</td>
<td>20.4</td>
<td>-</td>
<td>16.8</td>
</tr>
<tr>
<td>5L1</td>
<td>1658</td>
<td>Soil #3</td>
<td>18</td>
<td>17.4</td>
<td>20.5</td>
<td>16.8</td>
</tr>
<tr>
<td>5L2</td>
<td>1658</td>
<td>Soil #3</td>
<td>18</td>
<td>19.4</td>
<td>20.2</td>
<td>16.6</td>
</tr>
<tr>
<td>5H1</td>
<td>1658</td>
<td>Soil #3</td>
<td>26</td>
<td>25.0</td>
<td>22.8</td>
<td>15.7</td>
</tr>
<tr>
<td>5H2</td>
<td>1658</td>
<td>Soil #3</td>
<td>26</td>
<td>23.5</td>
<td>18.6</td>
<td>15.8</td>
</tr>
</tbody>
</table>
Table 4.7. Final CCL and GCL properties after shear for GM/GCL/CCL tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>Normal Stress $\sigma_n$ (kPa)</th>
<th>Soil Type</th>
<th>Target $w$ (%)</th>
<th>Final $w$ (%)</th>
<th>Shearing Surface $w$ (%)</th>
<th>Final Dry Unit Weight (kN/m$^3$)</th>
<th>Final $w$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2B</td>
<td>348</td>
<td>Soil #3</td>
<td>22</td>
<td>24.6</td>
<td>21.1</td>
<td>15.9</td>
<td>71.6</td>
</tr>
<tr>
<td>3B</td>
<td>693</td>
<td>Soil #3</td>
<td>22</td>
<td>24.4</td>
<td>-</td>
<td>15.5</td>
<td>46.7</td>
</tr>
<tr>
<td>4B</td>
<td>1176</td>
<td>Soil #3</td>
<td>22</td>
<td>23.7</td>
<td>22.0</td>
<td>16.3</td>
<td>52.9</td>
</tr>
<tr>
<td>5B</td>
<td>1658</td>
<td>Soil #3</td>
<td>22</td>
<td>23.1</td>
<td>19.3</td>
<td>16.6</td>
<td>48.2</td>
</tr>
<tr>
<td>6B</td>
<td>2146</td>
<td>Soil #3</td>
<td>22</td>
<td>21.9</td>
<td>19.6</td>
<td>16.7</td>
<td>36.1</td>
</tr>
<tr>
<td>7B</td>
<td>4145</td>
<td>Soil #3</td>
<td>22</td>
<td>21.8</td>
<td>21.2</td>
<td>16.8</td>
<td>36.9</td>
</tr>
</tbody>
</table>

Figure 4.23. CCL and GCL water contents after shearing for test series A and B.
4.4 CCL Dry Unit Weight

A plot of final water content versus dry unit weight of CCLs for test series A and B is shown in Figure 4.24 and includes the compaction curves previously presented for comparison. Final CCL conditions are typically wet of optimum. The data displays a general trend of increasing dry unit weight with increasing normal stress for tests under similar conditions. Tests at a reduced water content (5L) produced similar dry unit weight values as Tests 5A and 5P, while tests with an increased water content (5H) yielded lower values. GM/GCL/CCL tests have higher water contents, but similar dry unit weights as corresponding GM/CCL tests. Test 5B is seen to plot outside the ZAV-line, indicating that the estimated specific gravity of the soil \( G_s = 2.75 \) may be too low.

![Graph showing CCL final water content versus dry unit weight.](image-url)
4.5 *Stress-Displacement Relationships*

Shear stress was recorded continuously during the shearing process, and is presented as a function of shear displacement in Figure 4.25 for GM/CCL Tests 1A-5A and for GM/GCL/CCL Tests 2B-7B. Most tests displayed similar stress-displacement behavior. Peak strength was quickly achieved at displacements ranging from 3-12 mm, followed by gradual post-peak strength reduction. Most tests had not reached residual conditions at the conclusion of shearing (150 mm). Tests 1A and 3A exhibited a rapid rise followed by a gradual increase in shear stress during the shearing process, with peak and large displacement shear strengths roughly equal. Peak ($\tau_p$) and large displacement ($\tau_{ld}$) strengths have previously been presented in Table 4.1 and Table 4.5. Increasing normal stress resulted in higher peak and large displacement shear strengths for all tests. Post-peak strength ratios ($\tau_{ld} / \tau_p$) are shown in Figure 4.26. Ratios are consistently lower for GM/GCL/CCL tests than GM/CCL. This can be attributed to the additional shear resistance provided by the gravelly CCL as the GM deformed and interlocked with underlying gravel particles during shear.

Specimen failure occurred at either the GM/CCL or GM/GCL interface for all tests performed. This was by design, as single-side textured GMs were placed smooth side down against the CCL/GCL for each test. Failure mode would likely have transitioned to internal GCL failure for GM/GCL/CCL tests under increasing normal stress if a double-sided textured GM was used, as was found by Fox and Ross 2011 for similar testing materials.
Figure 4.25. Shear stress-displacement relationships for (a) GM/CCL tests (b) GM/GCL/CCL tests.
Figure 4.26. Post-peak strength ratios for Tests 1A-5A and 2B-7B.

4.6 Shear Strength

Failure envelopes for peak and large displacement shear strengths for Tests 1A-5A and 2B-7B are shown in Figure 4.27. Shear strengths for GM/GCL/CCL tests are lower than GM/CCL tests. Failure envelopes for GM/CCL tests had a highly irregular shape with a particularly sharp increase for Test 5A. Interestingly, this irregular shape roughly correlates to the GM hole count shown in Figure 4.10. This suggests that interlocking of gravel particles with a damaged GM contributes to an increased GM/CCL interface strength. Application of normal stress compressed the surrounding clay matrix, causing gravel particles to protrude from the CCL surface. Under pressure the GM closely conformed around these protrusions, with gravel particles even puncturing through the GM during shear at high levels of normal stress.
This produced greater frictional resistance and mechanical interlocking between the GM and underlying gravel particles.

GM/CCL envelopes indicate an increasing friction angle with increasing normal stress, while GM/GCL/CCL envelopes indicate the reverse. Both envelopes are nonlinear, indicating they should not be extrapolated to normal stresses outside the values tested. Secant friction angles for series A and B are presented in Table 4.8. Minimum values for the GM/CCL interface are 17.5 degrees for peak strength and 15.1 degrees for large displacement strength, both obtained from Test 4A. Minimum values for the GM/GCL interface are 8.8 degrees for peak and 6.7 degrees for large displacement, from Test 7B. Both of these GM/GCL values are higher than the expected value 4.6-4.9 degrees for residual internal shear of a hydrated GCL (Fox and Ross 2011).

![Figure 4.27. Peak and large displacement failure envelopes.](image)
Table 4.8. Peak and large displacement secant friction angles.

<table>
<thead>
<tr>
<th>Test</th>
<th>Normal Stress $\sigma_n$ (kPa)</th>
<th>Peak Secant Friction Angle $\phi_p$ (deg.)</th>
<th>Large Displ. Secant Friction Angle $\phi_{ld}$ (deg.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>72</td>
<td>23.0</td>
<td>23.0</td>
</tr>
<tr>
<td>2A</td>
<td>348</td>
<td>19.5</td>
<td>17.0</td>
</tr>
<tr>
<td>3A</td>
<td>693</td>
<td>20.3</td>
<td>21.3</td>
</tr>
<tr>
<td>4A</td>
<td>1176</td>
<td>17.5</td>
<td>15.1</td>
</tr>
<tr>
<td>5A</td>
<td>1658</td>
<td>23.5</td>
<td>20.9</td>
</tr>
<tr>
<td>2B</td>
<td>348</td>
<td>12.9</td>
<td>11.3</td>
</tr>
<tr>
<td>3B</td>
<td>693</td>
<td>12.6</td>
<td>8.9</td>
</tr>
<tr>
<td>4B</td>
<td>1176</td>
<td>13.0</td>
<td>10.1</td>
</tr>
<tr>
<td>5B</td>
<td>1658</td>
<td>12.5</td>
<td>8.9</td>
</tr>
<tr>
<td>6B</td>
<td>2146</td>
<td>11.3</td>
<td>7.8</td>
</tr>
<tr>
<td>7B</td>
<td>4145</td>
<td>8.8</td>
<td>6.7</td>
</tr>
</tbody>
</table>
5 CONCLUSIONS

5.1 Project Summary

The main objective of this research was to investigate the potential of puncture to a HDPE geomembrane from a gravelly CCL due to static pressure and large shear displacement. Results clearly show that shear displacement of a HDPE GM over a compacted gravelly CCL can significantly increase GM damage compared to static pressure alone. Severe damage is possible with wrinkling, tearing, and puncturing of GM specimens observed. For example, Test 5A at $\sigma_n = 1658$ kPa yielded a hole density of 169 holes/m$^2$ and maximum hole size of 23.4 mm after shearing although no holes were present after the static pressure stage. Current industry standards often place unprotected GMs directly in contact with CCLs with gravel content up to 20% and a maximum particle size of 25-50 mm, provided the final surface is smooth rolled and free from gravel protrusions. Under this practice, none of the GMs in this study would have required a protective layer.

Damage levels were directly related to CCL gravel content. Tests with no gravel content revealed essentially no damage to GM specimens after shearing for normal stresses up to $\sigma_n = 1658$ kPa. For CCL subgrades with gravel content, the amount of GM damage was largely dependent on the normal stress. For subgrades with 7% gravel content, damage consisted of scratching and gouging which reduced overall condition of the GM. Tests with 20% gravel content observed low to moderate levels of damage for $\sigma_n \leq 348$ kPa with no GM puncture occurring. At high level of normal stress ($\sigma_n \geq 693$ kPa) damage became severe and included wrinkles, gouges,
holes, and tears of the GM specimen. Changes in CCL water content reduced, but did not eliminate, GM damage levels. The majority of damage was observed to occur during post-peak shear, though considerable damage was present at the peak strength condition.

Placement of a hydrated NW/NW NP GCL between a GM and gravelly CCL was shown to dramatically improve performance with respect to GM damage. Even under very high normal stress ($\sigma_n = 4145$ kPa) GMs with this protective layer showed no holes, though scratches and indentations were observed. Additionally, if holes were to develop for a GM/GCL/CCL liner the GCL would be expected to limit flow rate through the liner (Nosko and Touze-Foltz 2000). Previous research also suggests that the GCL may seal around the gravel protrusion, also limiting flow rate (Fox et al. 2000).

5.2 Implications for Design

As a result of this study, GM puncture due to the subgrade below has three requirements: 1) the CCL must contain gravel, 2) normal stress must exceed low levels ($\sigma_n \geq 348$ kPa), and 3) shear displacement must occur at the GM/CCL interface. In practice, the GM/CCL interface is often considered critical for stability and some displacement would be assumed by a prudent design engineer. Therefore, the following recommendations can be made, subject to further review and verification. GM/CCL composite liners using CCLs with gravel content can be considered for low stress applications such as landfill cover systems, but should be used with caution for
bottom liner systems and other moderate to high stress applications. If a gravelly CCL is being considered for these conditions and there is reasonable expectation of GM/CCL interface shear displacement, project-specific direct shear tests should be conducted to investigate potential GM damage effects. Should these tests reveal the GM is susceptible to damage, several options may be available: 1) require top lift of the CCL to be free of gravel, 2) place a NP GCL between the GM/CCL interface, or 3) include an intentional slip surface (e.g., smooth GM/smooth GM) above the GM/CCL liner to limit shear displacements at the interface.

5.3 Future Testing

This study shows the significant effect gravel content can have on the GM/CCL interface. Data variability and testing limitations of the current study emphasize the need for further research on the topic of GM damage. For testing to be representative of field conditions, multi-interface specimens should be used that are representative of field materials and subjected to the range of normal stresses expected. Additionally, the GM should be sandwiched between actual field materials to allow out-of-plane deformations and damage to occur.

With regard to CCLs, further testing is needed to better understand the effects of gravel content and particle characteristics (e.g., size, angularity), compaction water content, and magnitude of shear displacement (pre-peak, peak, post-peak strength conditions). Numerous variables were not studied as part of the current program, including: geomembrane polymer type, geomembrane thickness, CCL soil type, CCLs
with natural gravel content, geomembrane temperature, and GCL type. A limited comparison of GM damage is provided by Fox et al. (2011) for LLDPE and HDPE geomembranes sheared over a compacted gravelly sand layer.

More broadly, the current study highlights the importance accounting for shear-induced damage in GM design. In additional to a gravelly CCL, GMs can be placed in contact with granular material via an overlying drainage layer. Heap leach pads represent a set of particularly aggressive conditions, placing a GM in direct contact with large-diameter material under very high normal stress conditions. Current design practice is based on damage tests results based solely on static pressure tests. Applicability of the static pressure puncture test for design of the GM and relevant protective layers may need to be reevaluated if interface shear displacements are expected. The development of new standard test procedures and geomembrane protection guidelines may be necessary in light of this new data.
REFERENCES


