2nd Edition

WASTE CONTAINMENT FACILITIES

Guidance for Construction Quality Assurance and Construction Quality Control of Liner and Cover Systems

David E. Daniel, Ph.D., P.E.
Robert M. Koerner, Ph.D., P.E.

ASCE PRESS

Waste Containment Facilities

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Preface

The important role of construction quality assurance (CQA) and construction quality control (CQC) in the development of environmentally safe waste-disposal facilities is widely acknowledged by all who are involved in the design, construction, permitting, and operation of such facilities. The best design will not necessarily lead to successful containment of wastes unless the facility is properly constructed. The CQC/CQA process, as part of a total effort aimed at ensuring quality in the constructed project, is crucial.

Proper CQC/CQA for waste containment facilities is a complex process. The materials used in constructing landfills, waste impoundments, and similar facilities include natural soil materials, processed earthen materials, and a wide range of geosynthetic materials. A large number of types of tests and observations are essential elements to good CQC/CQA. The purpose of this book is to describe those elements in detail and offer recommendations for types of tests and observations, frequency of tests and observations, and steps that are necessary to integrate the pieces into a successful CQC/CQA program.

This book originates from a 1990–1993 project conducted by the authors for the U.S. Environmental Protection Agency (EPA) under Cooperative Agreement No. CR-815546. The project had the express purpose of producing a Technical Guidance Document (TGD) on quality assurance and quality control for waste containment systems. The project was completed with the issuance of EPA Technical Guidance Document EPA/600/R-93/182, "Quality Assurance and Quality Control for Waste Containment Facilities," in September 1993. The EPA project officer was David A. Carson. The technical guidance document that was generated was reviewed in detail by Mr. Carson and Mr. Robert E. Landreth of the U.S. EPA, who worked closely with the authors in finalizing the document. Clearly, Mr. Carson and Mr. Landreth should be part of the authorship because of the insight, time, and energy that they invested in the project. Their respective positions with U.S. EPA, however, prevent them from this visibility. Their efforts are hereby acknowledged with sincere thanks. In addition to agency personnel, the TGD was reviewed by a number of industry and academic experts. They are acknowledged as a group and are listed in the preface of the TGD.

After the EPA guidance document was published, the authors felt that the document would experience wider dissemination through publication in book form and, with ASCE Press as the publisher, would reach a broad-based, consulting and design engineer audience in the United States as well as numerous other countries. As such, an introductory chapter was added as an explanation of liner systems, together with a brief background of the various natural and geosynthetic materials involved. The first edition of the book was published by ASCE in 1995. Since that time, however, the following activities have occurred:

- Many test methods have been developed or modified (by ASTM, GRI, and others) to reflect current practice in containment facilities.
- Generic specifications (by ASTM, GRI, PGI, and others) have been developed, particularly for geosynthetics.
- Field practice has been upgraded. In some instances, completely new practices have been adopted (e.g., bioreactor landfills and the electrical leak location method).
- In a few cases, practices have been modified in favor of more modern and effective methods.

Thus, this second edition of the book should prove useful. Please note that the structure of this second edition remains like that of the first, yet the material contained herein has been upgraded considerably.

The authors gratefully acknowledge the many individuals, too numerous to name here, who over the years have shared their experiences and recommendations concerning quality assurance and quality control with the authors. The member organizations of the Geosynthetic Institute are particularly thanked for support of this effort. Our sincere appreciation is extended to all involved in helping us to develop this book and very much to the anonymous reviewers of this second edition, who provided many insightful comments and issues. . . Thank you.

David E. Daniel
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Waste Containment Facilities

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Introduction to Waste Containment Systems

1.1 Introduction

Construction quality assurance (CQA) and construction quality control (CQC) are widely recognized as critically important factors in overall quality management for waste containment facilities. The best of designs and regulatory requirements will not necessarily translate to waste containment facilities that are protective of human health and the environment unless the waste containment and closure facilities are properly constructed. Additionally, for geosynthetic materials, manufacturing quality assurance (MQA) and manufacturing quality control (MQC) are equally important. Geosynthetics refer to fabricated polymeric materials such as geomembranes, geotextiles, geonets, geogrids, and geosynthetic clay liners.

The purpose of this book is to provide detailed guidance for proper MQA and CQA procedures for waste containment facilities. The book is also applicable to MQC and CQC programs on the part of the geosynthetic manufacturer, installer, and contractor. Although all waste containment facility designs are different, MQA and CQA procedures are similar. In this document, no distinction is made concerning the type of waste to be contained (e.g., hazardous or nonhazardous waste) because the MQA and CQA procedures needed to ensure quality lining systems, fluid collection and removal systems, and final cover systems are the same regardless of the waste type. This book has been written to apply to all types of waste disposal facilities, including hazardous-waste landfills and impoundments, municipal solid-waste landfills, various types of liquid impoundments, and final covers for new facilities and site remediation projects.

This book is also intended to aid those who are preparing MQA/CQA plans, reviewing MQA/CQA plans, performing MQA/CQA observations and tests, and reviewing field MQC/CQC and MQA/CQA procedures. Permitting agencies may use this book as a technical resource to aid in the review of site-specific MQA/CQA plans and to help identify any deficiencies in the MQA/CQA plan. Owner/operators and their MQA/CQA consultants may use this book for guidance on the plan, the process, and the final certification report. Field inspectors may use this book and the references herein as a guide to field MQA/CQA procedures. Geosynthetic manufacturers may use the book to help establish appropriate MQC procedures and as a technical resource to explain the reasoning behind MQA procedures.

Construction personnel may use this book to help establish appropriate CQC procedures and as a technical resource to explain the reasoning behind CQA procedures. Individuals seeking certification may use this book as a textbook. Individuals working on nonwaste-disposal facilities (e.g., liners for agriculture-related liners and covers, waste piles, and liquid-retention reservoirs) may use this book as guidance for MQA and CQA. The scope of this book includes all natural soil and geosynthetic components that might normally be used in waste containment facilities (e.g., in liner systems, fluid collection and removal systems, and cover systems).

This book draws heavily on information presented in several U.S. EPA Technical Guidance Documents: "Design, Construction, and Evaluation of Clay Liners for Waste Management Facilities" (1988), "Lining of Waste Containment and Other Impoundment Facilities" (1989), and "Inspection Techniques for the Fabrication of Geomembrane Field Seams" (1991). Both editions of this book are similar to the U.S. EPA document "Technical Guidance Document: Quality Assurance and Quality Control for Waste Containment Facilities" (Daniel and Koerner 1993), but they contain additional information and recommendations. In addition, technical information concerning many of the principles involved in construction of liner and cover systems for waste containment facilities is provided in three additional U.S. EPA documents: "Requirements for Hazardous Waste Landfill Design, Construction, and Closure" (1989), "Design and Construction of RCRA/CERCLA Final Covers" (1991), and "Assessments and Recommendations for Improving the Performance of Waste Containment Facilities" (2002). Additionally, numerous books and technical papers in the literature form a large database from which information is drawn in the appropriate sections.

This initial chapter introduces the general concepts of liner systems and customary components of a waste containment system as constructed in the United States. It should be recognized that this is a generalized approach and that there are many possible alternative strategies for waste containment. Furthermore, other countries have different strategies for the disposal of their wastes. Even within the United States, individual states have different requirements. The U.S. Environmental Protection Agency (U.S. EPA) promulgates rules and establishes minimum technology guidance, but individual states may go beyond these minimum requirements.

Neither this initial chapter nor the book itself covers design. The assumption is that the design has been completed and that the site-specific plans and specifications are in existence. This book picks up at that point where the necessary quality assurance (QA) plan and supporting documents are developed and implemented accordingly.

1.2 Waste Generation

The amount of solid waste generated in the United States is enormous and continues to grow despite aggressive recycling efforts. Figure 1-1 gives data collected by the U.S. EPA for municipal solid waste (MSW). Note that the data in this figure

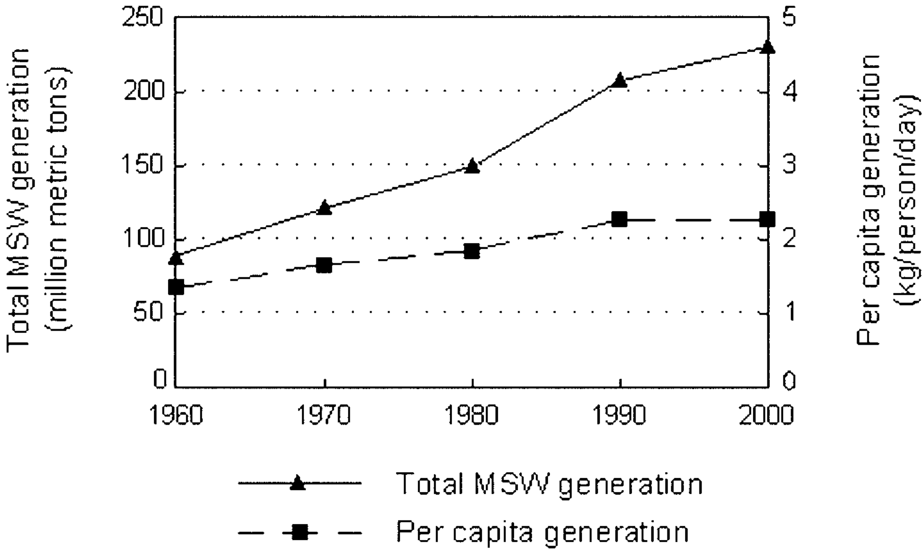


Figure 1-1. Municipal Solid-Waste Generation in the United States from 1960 to 2000.

Source: Adapted from U.S. EPA documents.

do not include construction demolition debris, incinerator ash, stabilized sludges, and nonhazardous industrial waste. These materials are often codisposed with MSW and approximately double the quantities shown. The problem of waste disposal, however, is worldwide, and all countries are confronted with a solid-waste disposal dilemma to various degrees. Table 1-1 gives some insight into the global situation in terms of all types and strategies of waste disposal. Note that Canada,

Table 1-1. Statistics for Municipal Solid-Waste Disposal Methods as Percent of Total (mid-1990s)

Country	*Landfill*	*Incineration*	*Recycling and Composting*
Canada	75	6	19
Denmark	22	54	24
France	59	32	9
Greece	93	0	7
Japan	27	69	4
Mexico	99	0	1
South Korea	72	4	24
Sweden	39	42	19
Switzerland	14	46	40
United Kingdom	84	9	7
United States	57	16	27

Source: Data from the Organization for Economic Cooperation and Development.

France, Greece, Mexico, South Korea, the United Kingdom, and the United States all use landfilling as their major waste-disposal method.

Additionally, there are other factors to consider. For example, the U.S. Supreme Court decided in 1994 that the ash from municipal solid-waste incinerators and trash-to-steam incinerators may be hazardous and must be evaluated accordingly. If found to be hazardous (see our later discussion of the definition of hazardous waste), the ash must be contained, as with other hazardous waste (i.e., in a hazardous-waste landfill with a double liner system).

The following classes of materials, listed in descending order of approximate degree of hazard, constitute the majority of solid-waste materials (modified from EPA 1992):

- radioactive waste,
- hazardous waste,
- hospital and research waste,
- municipal solid waste,
- sewage treatment sludge,
- contaminated dredge soil,
- incinerator ash,
- heap leach residual waste,
- electric power-station ash,
- mine spoil, and
- construction demolition waste.

The critical issue pertaining to waste containment facilities (i.e., landfills) is usually groundwater pollution. The use of some type of liner on the bottom and sides of landfills that contain solid wastes has been considered necessary in many countries since the late 1970s. This necessity is created by the liquids in the landfilled materials, augmented by rainfall and snowmelt, interacting with the waste and forming a liquid called “leachate.” The leachate flows downward by gravity and, if not for a liner, continues its migration, eventually causing groundwater and/or surface-water pollution. Both the quantity and quality of leachate are of concern. In addition, volatile organics in the waste or leachate, as well as gases of decomposition such as methane, contribute to landfill gas, which also requires containment and which, if not contained, poses a threat to the surrounding environment.

1.3 Regulations

In the United States, solid waste is regulated under the Resource Conservation and Recovery Act (RCRA) and the Hazardous and Solid Waste Amendments (HSWA) to RCRA. The term *hazardous waste* has a specific, legal definition. Waste is hazardous if the following conditions are met:

1. It is listed as a hazardous waste (hundreds of wastes are specifically identified in Appendix VIII of Title 40, Code of Federal Regulations, Part 251).

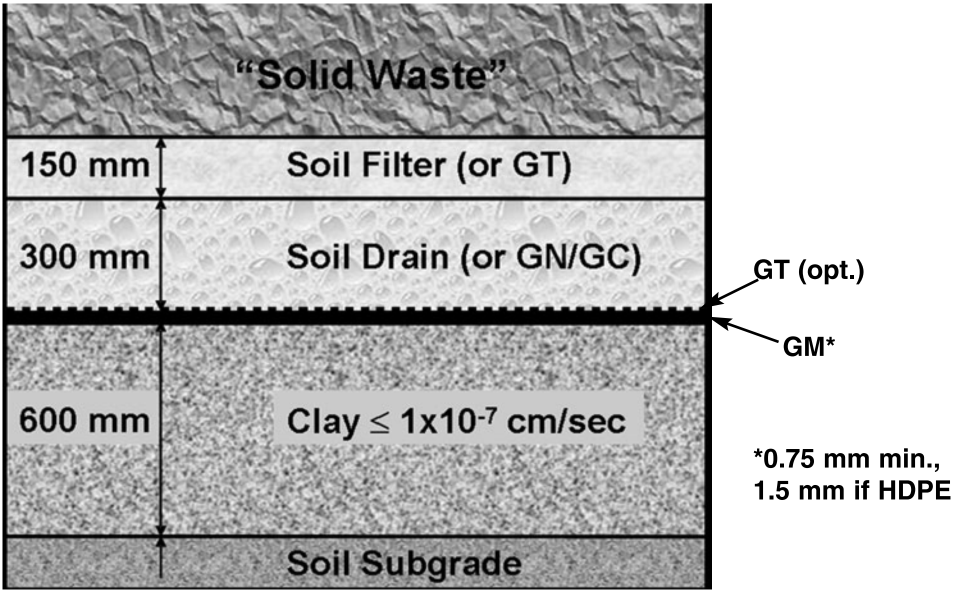
2. It is mixed with or derived from a hazardous waste.
3. It is not excluded (some wastes, such as municipal solid waste, are specifically identified and excluded as hazardous waste).
4. It possesses any one of four characteristics: (1) ignitability (flash point 60 °C); (2) corrosivity (pH between 2 and 12); (3) reactivity (reacts violently with water or is capable of detonation); or (4) toxicity as determined by the Toxicity Characteristic Leaching Procedure (TCLP) test.

For those waste materials considered nonhazardous, the applicable legislation is contained in Subtitle D of RCRA. Specific U.S. EPA regulations are published in Parts 257 and 258, Title 40, Code of Federal Regulations (CFR). Here the liner must be a composite liner made up of a geomembrane (GM) in "intimate contact" with an underlying compacted clay liner (CCL), i.e., a GM/CCL. Above this composite liner is a leachate collection and removal system (LCRS) consisting of a drainage material, within which is often located a perforated pipe removal system. A cross section of such a composite liner and leachate collection system is shown in Figure 1-2(a). A pipe network is generally contained within the drainage soil and usually drains into a sump at the low elevation of the landfill or cell. From here, leachate is removed by a submersible pump. The pump is lowered in vertical manholes that extend up through the waste mass or in large pipe risers extending up the sideslope of the facility. Leachate flow can also be gravitational to beyond the limits of the cell or landfill. Generally, the leachate must be removed and appropriately treated for the active life of the landfill plus a 30-year postclosure period. However, the 30-year period has yet to be reached for any landfill constructed under the current regulations; longer periods of leachate removal and treatment are expected for at least some sites. Alternative designs may be approved by individual states, but the alternative design must be shown to limit the concentration of contaminants in groundwater to acceptably low values at the critical point of compliance.

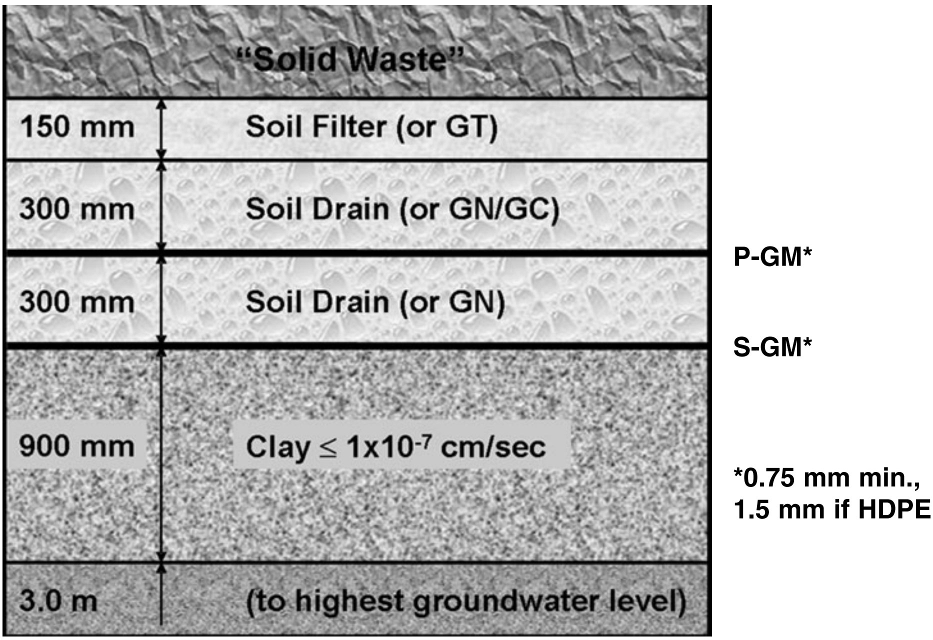
For waste materials that are considered hazardous, as described previously, the applicable legislation is contained in Subtitle C of RCRA. These U.S. EPA regulations are contained in 40 CFR 264.221. Here the strategy is to have two liner systems with a leak detection layer between them and a leachate collection layer above them. The purpose of the leak detection layer is to determine if (and to what extent) leakage is occurring through the upper or primary liner and to provide a mechanism for removing liquids that enter this layer. The double liner system with an intermediate leak detection system is the hallmark of hazardous waste landfills in the United States. Guidelines are contained in 40 CFR 260, 265, 270, and 271. The individual components, as they appear in Subtitle C regulations, are shown in Figure 1-2(b).

It should be emphasized that both Figs. 1-2(a) and (b) represent minimum requirements that individual states must follow or exceed by their specific regulations. Many states exceed the federally mandated minima.

Fahim and Koerner (1993) have compiled state regulations for Subtitle D liner systems for municipal solid-waste (often considered nonhazardous) landfills, as



(a) Municipal Solid Waste Landfill



(b) Hazardous Waste Landfill

Figure 1-2. Illustrations of Cross Sections of Minimum Liner Systems beneath Solid Waste for (a) Municipal Solid-Waste Landfill and (b) Hazardous-Waste Landfill.

Note: GT, geotextile; GM, geomembrane; GN, geonet; GC, geocomposite; HDPE, high-density polyethylene; P, primary; and S, secondary.

they existed in 1993 just before the Subtitle D rules took effect. Approximately 20 states required composite liners, and 19 states continued to place sole reliance on compacted clay liners (CCLs). At the two extremes, 8 states used only geomembranes, and 14 states used only natural soil. Some states had alternate strategies, so the total was greater than 50. The situation was mixed and was rapidly changing at the time of the survey. Regarding double MSW liner systems with leak detection capability, 12 states had adopted this type of strategy (as of 1993) for their MSW material or used it as an alternate strategy. No two states, however, appear to have had the same recommended cross sections. The general tendency appeared to be a single geomembrane primary liner with a composite secondary liner, as in the hazardous waste landfill liner shown in Figure 1-2(b). Regulations have shifted over the past decade; the largest changes include a uniform use of composite geomembrane and CCL (i.e., a GM/CCL liner) or the use of geosynthetic clay liners (GCLs) to replace the CCL (i.e., a GM/GCL liner) or to augment it (i.e., a GM/GCL/CCL liner). The state regulatory requirements for MSW landfill liners continue to undergo adjustments.

In addition to the liner system beneath and on the sideslopes of the waste, a final cover (or closure) must eventually be placed over the completed solid-waste mass. Requirements for landfill covers are also included in federal regulations. For liner systems of the type shown in Figs. 1-2(a) and (b), a possible cover above the waste is illustrated in Figure 1-3. For hazardous waste, the strategy for a barrier against water infiltration through the cover is a composite GM/CCL liner. For nonhazardous MSW, the regulations simply require a barrier to infiltration. The regulations are confusing because they require that the barrier layer be no more permeable than the bottom liner, but they do not specifically require a GM/CCL liner (or the equivalent) that has similar performance characteristics to a GM/CCL bottom liner. Furthermore, the required hydraulic conductivity of the CCL has been raised to 1×10^{-5} cm/s (Austin 1992). If methane is anticipated, a gas transmission layer may be necessary beneath the liner. Also, a drainage layer above the liner may be necessary to drain water coming through the cover soil as well as to maintain stability of the cover soil. The cover soil may be thick in northern states, where frost penetration is deep. This protection is required to prevent frost degradation of the CCL component of the barrier system. The vegetative layer is important for erosion control. In areas where vegetation cannot be grown or maintained (e.g., arid areas) the use of cobbles or stone riprap may be required.

Regarding cover systems for MSW, the heavy reliance on a single CCL barrier by the states was noticeable in 1993; it was the strategy of 36 states (Fahim and Koerner 1993). Equally noticeable was the lack of a requirement for a composite liner strategy by the states (required by only 6 states). Between these two extremes, 17 states had adopted a single geomembrane as the barrier system in the cover.

This is a changing situation because many states are rapidly coming into compliance with the federal minimum technology guidance (MTG) regulations. As with liner systems, the largest change in the past decade is the introduction of GCLs into final cover systems. GCLs have been used to replace the clay component.

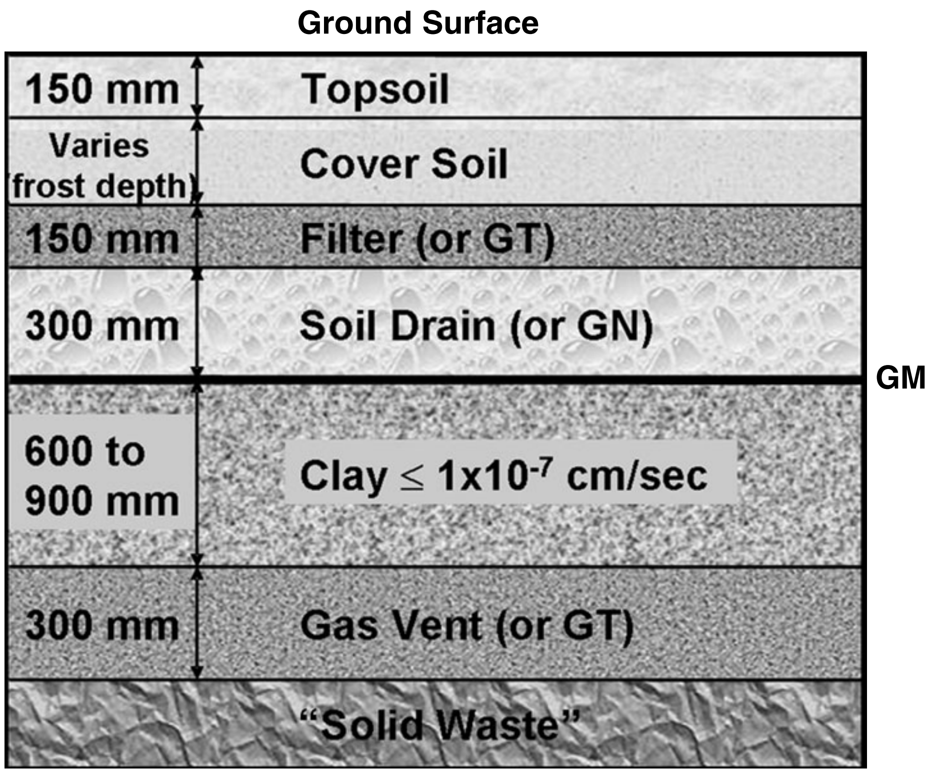


Figure 1-3. Typical Cover System Recommended by the U.S. EPA and U.S. Army Corps of Engineers for Landfills with Liner Systems as Shown in Figure 1-2.

Note: GT, geotextile; GN, geonet; and GM, geomembrane.

Landfill covers are an integral part of waste containment systems. A level of concern should be shown in both design and construction of final covers so that they are equal to the liners beneath the waste.

1.4 A Generalized Waste Containment System

In all federal legislation and (to our knowledge) state legislation as well, a permit applicant can suggest that an alternative be used in place of the standardized design. This option is embodied under the concept of "technical equivalency." The concept creates the possibility of using various geosynthetic materials because the regulations are primarily based on natural soil materials. The following substitutions might be, and frequently are, considered:

- Geonet (GN) drains may be considered to replace or augment soil drainage layers.
- Geotextile (GT) filters may be used to replace soil filter layers.

- Geosynthetic clay liner (GCL) barriers may be used to replace or augment compacted clay liners (CCLs).
- Geotextile gas drainage layers may be used to replace soil drainage layers beneath the barrier layer in a landfill cover.
- Geogrid (GG) reinforcement layers may be used to stabilize soil slopes, or to cover soils, or to build berms for lateral containment of the waste.
- Geotextile protection layers may be incorporated in the design to prevent puncture of the geomembrane.
- Geosynthetic erosion control (GEC) materials may be used to stabilize topsoil and vegetation in the cover system.

With these alternatives in mind, we illustrate a possible cross section in Figure 1-4. It illustrates a double liner system consisting of GM/GCL as the primary liner and GM/CCL as the secondary liner. The leak detection system is a GT/GN composite. The leachate collection layer on the bottom of the landfill is gravel with a perforated pipe network contained therein. A geotextile filter covers the entire footprint of the landfill and prevents clogging of the leachate collection and removal system. A geotextile cushion beneath the gravel protects the primary geomembrane from puncture by stones in the overlying gravel. On the sideslopes, the leachate collection system is a GT/GN composite merging into the gravel on the base. As noted on Figure 1-4, the steep side soil slopes beneath the liner system may require the use of geogrid reinforcement layers.

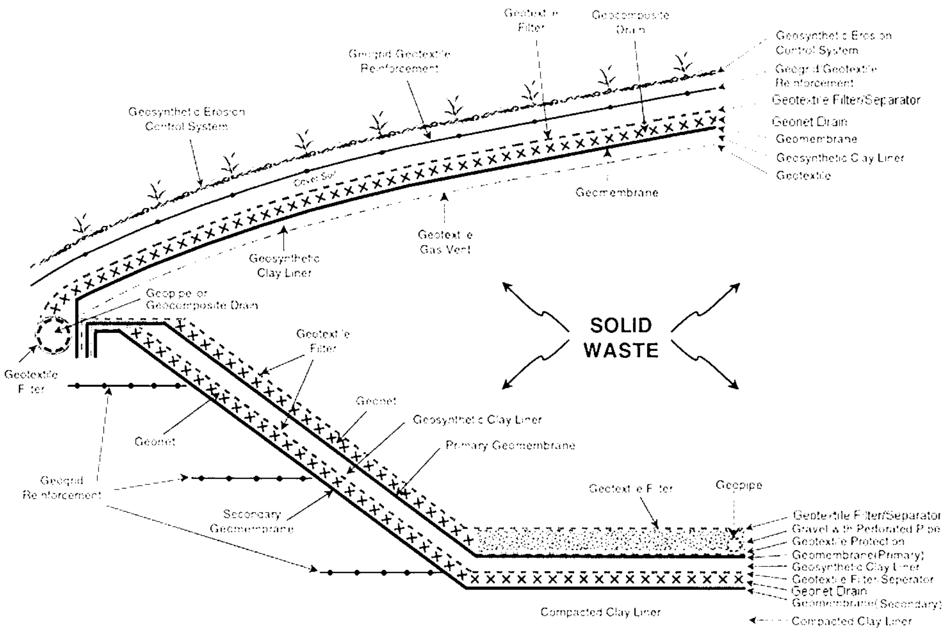


Figure 1-4. Solid-Waste Containment System with High Geosynthetic Usage.

The cover system in Figure 1-4 contains a composite GM/GCL as the barrier layer. A geotextile gas vent is beneath it, and a composite GT/GN/GT (or other type of geosynthetic composite) is above it. The cover soil contains geogrids or high-strength geotextiles as veneer reinforcement for stability. A geocomposite erosion control system is used on the upper portion of the topsoil. Both temporary and permanent erosion control materials are used, depending on site-specific conditions.

An abbreviated discussion on each of the natural soil components and geosynthetic components is provided in the next sections.

1.5 Natural Soil Components

1.5.1 *Compacted Clay Liners*

Low-permeability, compacted soil liners, also referred to as compacted clay liners (CCLs), are the historic engineered component used in landfills. Clay-rich soil is placed in layers and compacted with heavy equipment to form a barrier to movement of liquids and gases. The soil liner is typically designed to have a hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s. The origin of this design criterion is unclear; 1×10^{-7} cm/s was evidently selected on the assumption that this was an achievable value that would result in negligibly small seepage through the liner. Experience has shown that 1×10^{-7} cm/s is often difficult to achieve and requires great care in construction and careful CQC/CQA. CCLs are constructed either from natural soil materials that contain sufficient clay to attain the required low hydraulic conductivity or, if suitable soils are not available near the site, from a blend of commercially processed clay (almost always bentonite) and native soils obtained on or near a site. Compacted clay liners are usually 600 or 900 mm (2 or 3 ft) thick, but are sometimes 1.2 to 1.5 m (4–5 ft) thick and occasionally are as much as 3 m (10 ft) thick.

For CCLs, CQA focuses on three crucial components: ensuring that proper materials are used in constructing the liner; ensuring that materials are placed and compacted properly; and confirming that the liner is adequately protected from damage. Details are provided in Chapter 3.

1.5.2 *Soil Drainage Layers*

Soil materials such as sand, gravel, and processed stone are commonly used as drainage materials in liner and cover systems. The materials are usually required to have a hydraulic conductivity from 0.01 to 1 cm/s. A drainage layer is typically 300 to 600 mm (1–2 ft) thick. Important CQA issues are confirmation that suitable materials are used, verification of proper placement, and confirmation that the underlying materials (e.g., a geomembrane) have not been damaged. Chapter 6 presents specific guidance.

1.5.3 Soil Filtration Layers

Soils are sometimes used as filter layers, but designers use geotextiles for this purpose much more commonly than soil. Soil filters generally consist of sands with a thickness of approximately 150 to 300 mm (0.5–1.0 ft). The important CQA issue is verification of proper grain-size distribution, placement, and protection. Soil filters for drainage materials are covered in Chapter 6.

1.5.4 Alternative Final Cover Materials

Alternative final covers refers to final cover systems that are designed to manage water infiltration into and percolation through the cover system using natural storage and seepage processes, without the benefit of low-permeability layers such as geomembranes, geosynthetic clay liners, or compacted clay liners. For example, a very thick evapotranspirative layer of soil or a fine-textured soil may overlay a coarse-textured soil that acts as a "capillary break," tending to cause moisture to be retained in the overlying fine-textured soil.

Alternative final covers were originally developed for arid sites, where soil materials function as water storage layers or capillary breaks and effectively function as low-permeability layers in unsaturated soils. Increasingly, alternative barriers are being designed at semiarid and arid sites.

Usually the important construction issues for alternative final cover systems are grain-size distribution and separation of coarse and fine layers. For example, a very thick soil layer can store water in an arid site such that the underlying waste does not receive water and evapotranspiration occurs at a commensurate rate. Alternatively, relatively clean coarse sand or gravel can make a capillary break material. However, "contamination" with excessive fines can render the material hydraulically conductive and thus ineffective in a relatively dry, unsaturated state. A "capillary barrier" concept is intended to serve as a capillary break, but it can only do that if it lacks fine-grained material that would enable capillary water to exist in a relatively dry state.

1.5.5 Vertical Cutoff Barriers

Vertical cutoff walls are sometimes used for new landfills and are commonly used in conjunction with pump-and-treat operations for site remediation projects. Because construction of the vertical cutoff wall is often part of the contract for construction of other components of waste containment facilities, CQA of vertical cutoff barriers is covered in Chapter 8.

1.6 Geosynthetic Components

Geosynthetics are polymeric materials consisting of various formulations. Geosynthetics are manufactured and are packaged as large rolls or cartons. The rolls or cartons are transported to the site where they are placed, overlapped onto adjacent sheets, and seamed or joined for the final use. Following are the individual types of products within the geosynthetics family:

- geomembranes (GMs),
- geotextiles (GTs),
- geonets (GNs),
- geogrids (GGs),
- geosynthetic clay liners (GCLs),
- plastic pipes, also known as “geopipes” (GPs),
- geocomposites (GCs), and
- geosynthetic erosion control (GEC) materials.

In the context of this book, geomembranes, geotextiles, geonet/geotextile composites (geocomposites), and geogrids/geotextile reinforcement layers (reinforcement geosynthetics) are the most significant. They will be described briefly in the next subsections. For more details, see Koerner (2005).

1.6.1 Geomembranes

Geomembranes are essentially impermeable sheets of polymeric formulations used as barriers to liquids and vapors. Geomembranes are required by both federal and state regulations to be used on the bottom, sides, and generally in the covers of waste containment facilities. Geomembranes are usually placed directly over a compacted clay liner (CCL) or geosynthetic clay liner (GCL). The exception is the primary geomembrane of a Subtitle C facility, in which the geomembrane can act alone. The cover of a landfill also requires a geomembrane if the bottom liner contains one. RCRA Subtitle D regulations clearly state that the cover must be as impermeable as the liner beneath the waste to prevent long-term buildup of liquids in the landfill.

The most common types of geomembranes are high-density polyethylene (HDPE), linear low-density polyethylene (LLDPE), polyvinyl chloride (PVC), flexible polypropylene (fPP), reinforced chlorosulfonated polyethylene (CSPE-R), and nonreinforced or reinforced ethylene propylene diene monomer (EPDM or EPDM-R), although other types of geomembranes are also available.

Geomembranes are usually 0.75 to 2.5 mm (30–100 mils, where 1 mil = 0.001 in.) thick and 4 to 15 m (13–50 ft) wide. It is necessary to prepare and approve the subgrade or substrate and then to place the geomembrane accordingly. Placement is followed by seaming, inspection, approving, and backfilling with soil or the superstratum material in as short a time as possible. A properly designed geomembrane has the potential of hundreds of years of service lifetime, but its installation must be accomplished according to the best possible quality management principles. Geomembrane manufacture, specification, installation, seaming, backfilling, and inspection are described in detail in Chapter 4.

1.6.2 Geotextiles

Geotextiles are permeable textiles made from polymeric fibers. Polypropylene is the most common polymer (approximately 95% of the total); however, a small amount of polyester is still used. Geotextiles are manufactured into the following major types, based on the type of fiber used and the manufacturing method:

- woven monofilament,
- woven slit film,
- nonwoven needle punched, and
- nonwoven heat bonded.

Geotextiles in waste containment applications function as follows:

- filtration: above leachate collection sand, gravel, or geonet in the base and sideslopes of a landfill;
- separation: beneath CCLs or GCLs and above leak detection geonets or granular soils;
- protection: beneath leachate collection or leak detection gravel and above geomembranes;
- drainage: above the waste to collect and transmit gases that are generated by decomposing waste materials; and
- reinforcement: in sideslopes, berms, and cover soils.

The joining of the deployed rolls of geotextiles can be accomplished by overlapping or sewing. The design must be specific on the amount of overlap or strength of sewn seam. The geotextile is covered by either soil or by an overlying geosynthetic material. Geotextiles must be covered in a timely manner because geotextiles are the most susceptible category of geosynthetics to UV light degradation. This susceptibility is due mainly to the high surface area of the individual fibers that make up the geotextile. Protection from UV degradation is most important for nonwoven geotextiles for sites with a high intensity of UV light. The design plans and specifications must include specific criteria about the allowable period of exposure.

The manufacture, specification, shipping, placement, seaming, and various aspects of inspection of geosynthetics used for filtration, separation, and protection are presented in Chapter 7 and for reinforcement in Chapter 9.

1.6.3 Geocomposites

Geocomposites represent a subset of geosynthetics where two or more individual materials are combined together. They are often laminated and/or bonded to one another in the manufacturing facility and shipped to the site as a completed unit. The most common geocomposite used in waste containment is a geotextile bonded to a geonet, or some other type of drainage core. The principal applications in landfills are illustrated in Figure 1-4, which shows a geotextile/geonet composite used in a leak detection application and on the sideslopes in a leachate collection application. Figure 1-4 also illustrates a geotextile/geonet composite used as an infiltrating water drain in the cover.

In geocomposites used for such applications, the geotextile serves as both a separator and a filter, and the geonet or built-up core serves as the associated drain. The design plans and specifications must be specific as to the type of geotextile and drain, as well as the method of bonding. It should be noted that there may be geotextiles on both the top and bottom of the drainage core and that they may be different from one another. For example, the lower geotextile may be a thick needle-punched nonwoven used as protection material for the underlying geomembrane, whereas the top geotextile may be a thinner nonwoven heat bonded or woven product. Geocomposite drains are described in Chapter 7.

1.6.4 Reinforcement Geosynthetics

Geogrids and high-strength geotextiles can be used in waste containment systems in various applications requiring soil or solid-waste reinforcement. Geogrids are stiff unitized polyethylene or polypropylene products; flexible textilelike polyester fibers coated with bitumen, latex, or polyvinyl chloride; or stiff polyester or polypropylene straps or rods. High-strength geotextiles are usually woven polyester fabrics, but they can be polypropylene as well. As shown in Figure 1-4, these materials can be used to reinforce slopes beneath the waste as well as for veneer reinforcement of the cover soils above the geomembrane. A growing area for geosynthetic reinforcement materials is in vertical and horizontal expansions of landfills. Here the geogrids or high-strength geotextiles are used as support systems for geomembranes in resisting differential settlement of the underlying waste, or in high berms. In all of these applications, the design plans and specifications must be specific as to type of product, placement, seaming, and backfilling.

Manufacture, specification, shipping, installation, seaming, backfilling, and inspection of geogrids and high-strength reinforcement materials are covered in Chapter 9.

1.7 Geosynthetic Clay Liners

Geosynthetic clay liners (GCLs) represent a composite material consisting of bentonite and geosynthetics. The bentonite used for GCLs in North America is sodium bentonite, and the geosynthetics are either two geotextiles or a single geomembrane. In the first format, the bentonite is contained by geotextiles on upper and lower surfaces via an adhesive, needle punching, or stitch bonding. For the single geomembrane, the bond is achieved by using an adhesive. Numerous styles of each type of product are currently available, with variations that include a thin film and polymer coating.

GCLs are typically 5 to 10 mm (0.25–0.375 in.) thick and have approximately 5 kg/m^2 (1 lb/ft^2) of bentonite. The rolls are 4 to 5 m (13–17 ft) wide, 30 to 60 m

(100–200 ft) long, weigh up to 1,800 kg (4,000 lb) each, and are wrapped in the factory to prevent premature hydration. It is important to keep the rolls wrapped and protected until they are ready for field deployment due to the high moisture absorption of bentonite. In the field, they are unrolled in their final position and overlapped. Some GCLs require that additional bentonite be placed in the overlap area. The project plans and specifications should be clear on all of these details.

GCLs are commonly used as the lower component of a GM/GCL composite in primary liner systems of double-lined waste containment facilities. GCLs are also used as a GM/GCL composite in landfill closure systems and sometimes as replacements for GM/CCL liner systems beneath the facility. It is important to recognize that GCLs can be used to augment GM/CCL composite liners in many possible formats (e.g., a GM/GCL/CCL composite).

Chapter 5 describes the manufacture, specification, shipment, handling, placement, backfilling, and inspection of GCLs.

1.8 Other Components of Waste Containment Systems

A properly functioning landfill, surface impoundment, or waste pile may have numerous other components that are relevant to a properly functioning system. Selected details are presented in Chapter 9.

1.8.1 Leachate Removal Systems

As leachate gravitationally flows through the waste and into the leachate collection system, it eventually enters a sump, where it must be removed. Access to the sump for leachate removal is by a vertical manhole rising through the waste, a sloped riser pipe following the sideslope, or a penetration through the liner system to a sump external to the landfill cell. The first two alternatives protrude through the cover materials. The leachate is actually withdrawn by a submersible pump, which is lowered into the manhole or riser and operates when leachate accumulates. The leachate that is removed and collected is held in storage tanks or surface impoundments until it can be transported to a treatment facility. Alternatively, some treatment facilities are on-site and still others have piping leading directly to an off-site treatment system. Numerous strategies are possible, and all are site-specific design issues.

1.8.2 Bioreactor Landfills (Also Known as Wet Landfills)

As of 2004, the director of an approved state solid waste program can issue research, demonstration, and development permits for introduction of liquids into MSW for the dual purposes of accelerating biodegradation and enhancing removal of harmful constituents. So-called “leachate recycling” is practiced at many landfills. Furthermore, bioreactor landfills have additional liquid to optimize the degradation of the organics leading to an anaerobic bioreactor, or even (with the introduction of air) an aerobic bioreactor (Reinhart and Townsend 1998).

The liquids (e.g., leachate, biosolids, waste water, and local precipitation) are reintroduced into the waste via injection wells or by means of a perforated pipe network placed beneath a temporary cover. Attention to such details as the pipe delivery system, holds or slots in the pipe, filter materials (sand or geotextiles), pipe couplings, waste subsidence, and landfill gas capture is important. Wet landfills are a major change in the manner of design, operation, and performance of landfilling, which will probably see widespread use and acceptance in the near future.

1.8.3 Gas Extraction Systems

Municipal solid-waste landfills generate a number of gases that tend to rise up through the waste to the bottom of the cover, where they must be collected and removed. In some passive systems, the gases (mainly methane) are merely flared or they are collected in a manifold system above the cover. When the latter is the case, the gases are collected and used. Active systems are sometimes considered, in which a vacuum is drawn on the extraction wells or manifold system to maximize the output and use the gas accordingly. Owner/operators of large landfills may collect landfill gases, remove (scrub) the liquid and contaminants, and use the methane for power generation. The power is usually used on site, but if quantities are large, it can be sold to a local industry or to the local electric power company.

1.8.4 Alternative Daily Cover Materials

All federal and state regulations call for each lift of waste to be covered, usually with 150 mm (6 in.) of soil. The purposes of this soil cover are the following:

- to control blowing litter,
- to control vectors,
- to limit odors,
- for fire protection, and
- for reasonable aesthetics.

The soil used for daily cover is usually locally available materials from a borrow pit. For excavated, below-grade landfill cells, the excavated soil is the logical choice for daily cover. The soil cover material is often a clayey soil of low hydraulic conductivity. Such soil layers often become de facto hydraulic barriers and tend to isolate each day's placement of new waste. Downward-moving leachate cannot easily penetrate a layer of low-permeability daily cover and is forced to travel horizontally, sometimes seeping through the cover and running down the exterior sideslopes. Such a situation defeats the purpose of leachate collection systems, makes management of liquids difficult, and may make leachate recycling impossible.

As a possible replacement to daily soil cover, numerous alternative daily cover materials (ADCMs) have been developed (Pohland and Graven 1993). These fall under the categories of

- foams,
- spray-on products,
- indigenous materials, and
- reusable geosynthetics.

The decision to use an ADCM in place of soil cover is a site-specific decision and should be covered in the design plans and specifications. The products and some of their details are described in Chapter 9.

1.8.5 Erosion Control Materials

The potential for erosion of cover soils after completion of the facility and during its postclosure care period must be considered in the design plans and specifications. The use of both temporary and permanent erosion control materials (as illustrated in Figure 1-4) is becoming more common.

The particular material selected is a site-specific decision and must be clearly stated in the design plans and specifications. The various types of geosynthetic erosion control materials are described in Chapter 9.

1.9 Importance of CQC/CQA

Proper construction quality control and quality assurance for waste containment facilities is neither easy nor inexpensive. There are several motivations for insisting on comprehensive CQC/CQA: better performance of the facility, avoidance of expensive repairs later, and avoidance of minimization of claims and subsequent litigation.

Almost everyone who is experienced in construction can cite examples of major construction errors that led to problems and sometimes catastrophe. Although good CQA does not guarantee to eliminate all construction problems, it is widely believed that it will catch most problems. Good CQA is expected to add value through better performance to almost all waste containment facilities and to virtually eliminate major construction errors in which the contractor fails to follow plans and specifications.

Leakage rates from double-lined facilities demonstrate the importance of CQC/CQA. The measurement of actual leakage rates from double-lined landfills indicates the value of CQA. Bonaparte and Gross (1990) found that leakage rates of 50 to 500 L/ha-day (5–50 gal/acre-day) are achievable with the presence of CQA. This early data set also indicated that CQA significantly reduces the leakage rate through liner systems.

The fact that CQA has been practiced regularly since the 1990 study is evidenced in a 2002 study of leakage from 289 double-lined landfill cells in the

United States (Bonaparte et al. 2002). Figure 1-5 presents average leakage rates from these landfill cells at different stages. Stage 1 is during construction and initial waste placement, Stage 2 is after considerable waste has been placed, and Stage 3 is after final cover is placed. Each point represents the average of the num-

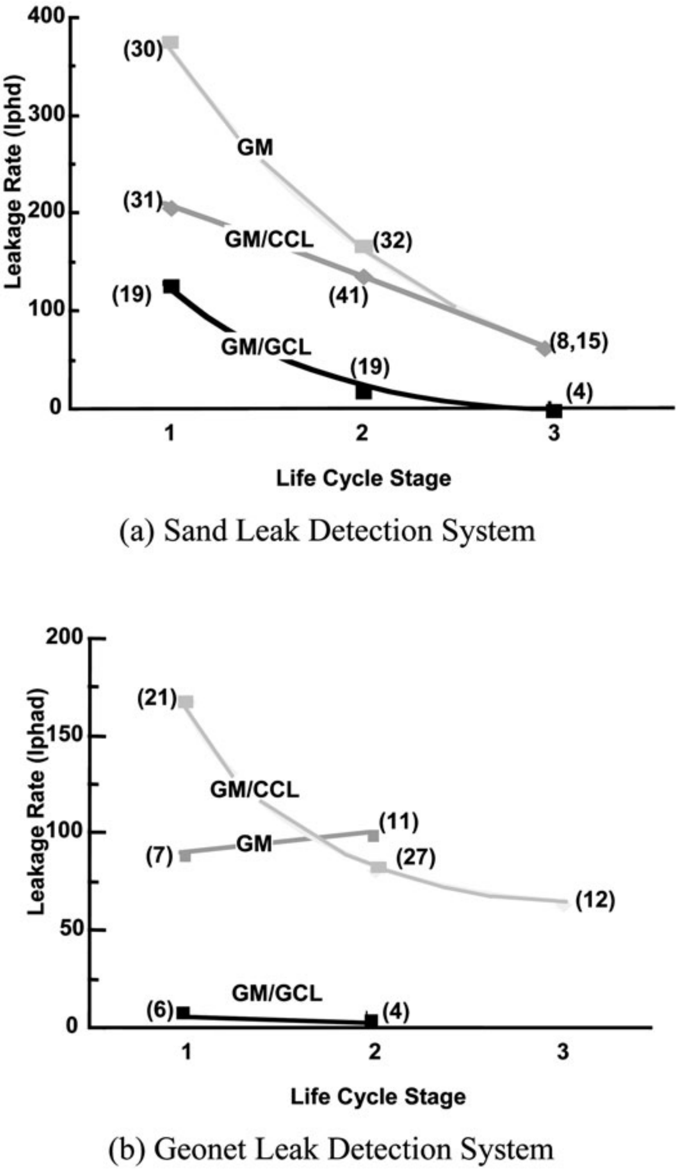


Figure 1-5. Leakage Rates from 289 Double-Lined Landfill Cells in the United States with Different Types of Primary Liners: (a) Sand Leak Detection System and (b) Geonet Leak Detection System.

Note: GM, geomembrane; CCL, compacted clay liner; GCL, geosynthetic clay liner and 1.0 L per hectare-day ≃ 0.1 gal/acre-day.

ber of landfill cells indicated in parentheses. The data indicate that a geomembrane (by itself) as primary liner allows for the highest leakage. A geomembrane over compacted clay liner (GM/CCL) composite results in almost as much leakage, the true amount of leakage being masked by expelled consolidation water from the CCL. A geomembrane over geosynthetic clay liner (GM/GCL) composite is clearly the preferred system for a primary liner system, resulting in extremely low leakage rates approaching negligible after Stage 2 is reached. Field data (such as those in Figure 1-5) are powerful in helping us understand the behavior of liner systems and set values for action leakage rates (ALRs). The situation can become quite contentious if an ALR has been set for the site and it is exceeded because of lack of, or poor, CQC and CQA. This same study highlighted a number of critical issues that must be incorporated into all CQA plans and documents. They are the following:

- soil and geosynthetic material conformance with the project specifications;
- proper preconditioning and placement of CCL lifts;
- proper compaction moisture content and density of CCLs;
- protection of CCLs from desiccation and freezing;
- placement of GMs without excessive waves and backfilling the GMs in a manner that minimizes the trapping of waves (the goal of these measures is intimate contact between the GM and the underlying CCL or GCL);
- prevention of premature GCL hydration;
- inspection of GM seams, including nondestructive and destructive testing; and
- protection of GMs from puncture by backfilling materials or equipment.

1.10 Cost of CQA

Numerous aspects are involved in the costs associated with construction quality assurance and control (CQA/CQC) of field installations, as well as manufacturing quality assurance and control (MQA/MQC) of manufactured geosynthetics. Because both CQC and MQC are actions taken on the part of contractors, installers, and manufacturers of their respective materials, it is expected that expenditures are more than offset by reduced failure rates of samples and the improved quality of the final installation. Indeed, this improvement is the hallmark of total quality management, which is the keyword of current industrial practice.

More controversial are the costs associated with CQA and MQA and the benefits derived therefrom. Shepherd et al. (1993) have summarized these CQA expenses from the perspective of a major owner/operator. As seen in Table 1-2, leakage rates in double-lined systems appear to be significantly reduced by CQA. Admittedly, the data are sparse, but this is the trend that one would anticipate. Shepherd and others have found that CQA costs for a single composite liner range from approximately \$31,000 to \$74,000 per hectare (\$12,500–\$30,000 per acre). The CQA costs for double composite liner systems range from \$53,000 to \$121,000 per hectare (\$21,000–\$49,000 per acre). Understandably, there is a major difference

Table 1-2. Comparative Costs of CQA to Liner Components

Item	*Units*	*Typical Range of Costs*	*Units*	*Typical Range of Costs*
Third-party CQA	ha	\$31,000–\$74,000	acre	\$12,500–\$30,000
1.5-mm (60-mil) HPDE liner	ha	\$42,000–\$62,000	acre	\$17,000–\$25,000
Geosynthetic clay liner	ha	\$52,000–\$74,000	acre	\$21,000–\$30,000
Extra sump liners	each	\$1,000–\$5,000	each	\$1,000–\$5,000
Detection system, sumps	each	\$15,000–\$30,000	each	\$15,000–\$30,000
Extra liners under pipes	ha	\$25,000–\$49,000	acre	\$10,000–\$20,000
Extra 300 mm (1 ft) of compacted clay	ha	\$12,000–\$62,000	acre	\$5,000–\$25,000

Source: Shepherd et al. 1993, with permission from Geosynthetic Information Institute.

in CQA costs between single composite liners and double composite liners. If the costs cited included the MQA costs of the geosynthetics, the totals would be marginally higher.

Shepherd et al. (1993) also itemized comparative costs of CQA of single liner systems versus costs of other components of liner systems (Table 1-2). The cost of CQA is approximately equal to the cost of an additional liner. A rule of thumb is that CQA, at a reasonable level of effort, adds an additional 5% to 15% to the cost of construction and installation.

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CHAPTER 2

Manufacturing Quality Assurance (MQA) and Construction Quality Assurance (CQA) Concepts and Overview

2.1 Introduction

As a prelude to description of the detailed components of a waste containment facility, some introductory comments are necessary. These comments are meant to clearly define the role of the various parties associated with the manufacture, installation, and inspection of the components of a total liner or closure system for landfills, surface impoundments, and waste piles.

2.1.1 Scope

Construction quality assurance (CQA) and construction quality control (CQC) are widely recognized as critically important factors in overall quality management for waste containment facilities. Additionally, for geosynthetic materials, manufacturing quality assurance (MQA) and manufacturing quality control (MQC) of the final product are equally important.

The purpose of this book is to provide detailed guidance for proper MQA and CQA procedures for waste containment facilities. The book also is applicable to MQC and CQC programs on the part of the manufacturer and contractor, respectively. Although facility designs are different, MQA and CQA procedures are the same. In this book, no distinction is made concerning the type of waste to be contained (e.g., hazardous or nonhazardous waste) because the MQA and CQA procedures needed to inspect quality lining systems, fluid collection and removal systems, and final cover systems are the same regardless of the waste type. This book has been written to apply to all types of waste-disposal facilities, including new hazardous-waste landfills and impoundments, new municipal solid-waste landfills, nonhazardous waste liquid impoundments, and final covers for new facilities, as well as site remediation projects.

This book is intended to aid those who are preparing MQA/CQA plans, reviewing MQA/CQA plans, performing MQA/CQA observations and tests, and reviewing field MQA/CQA and MQC/CQC procedures. Permitting agencies may use the book as a technical resource to aid in the review of site-specific MQA/CQA plans and to help in the identification of any deficiencies in MQA/CQA plans. Owner/operators and their MQA/CQA consultants may consult the book for guidance on the plan, the process, and the final certification report. Field inspectors (also called field monitors) may use the book and the references herein as a guide to field MQA/CQA procedures. Geosynthetics manufacturers may use the book to help in establishing appropriate MQC procedures and as a technical resource to explain the reasoning behind MQA procedures. Construction personnel may use the book to help in establishing appropriate CQC procedures and as a technical resource to explain the reasoning behind CQA procedures.

This book draws heavily on technical information presented in three U.S. EPA Technical Guidance Documents: "Design, Construction, and Evaluation of Clay Liners for Waste Management Facilities" (U.S. EPA 1988a), "Lining of Waste Containment and Other Impoundment Facilities" (1988b), and "Inspection Techniques for the Fabrication of Geomembrane Field Seams" (1991a). In addition, general technical backup information concerning many of the principles involved in construction of liner and cover systems for waste containment facilities is provided in three additional U.S. EPA documents: "Requirements for Hazardous Waste Landfill Design, Construction, and Closure" (U.S. EPA 1989), "Design and Construction of RCRA/CERCLA Final Covers" (1991b), and "Assessment and Recommendations for Improving the Performance of Waste Containment Systems" (2002). Additionally, numerous books and technical papers in the open literature form a large database from which information and reference will be drawn in the appropriate sections. This is the second edition of the original 1993 publication (U.S. EPA 1993; Daniel and Koerner 1995); it maintains the same structure but updates and extends test method practices, guides, and generic specifications accordingly.

2.1.2 Definitions

It is important to define and understand the differences between MQC and MQA and between CQC and CQA and to show where the different activities contrast and complement one another. The following definitions are appropriate in this regard.

- *Manufacturing Quality Control (MQC):* A planned system of inspections that is used directly to monitor and control the manufacture of a material that is factory originated (U.S. EPA 1993). MQC is normally performed by the manufacturer of geosynthetic materials and is necessary to ensure minimum (or maximum) specified values in the manufactured product. MQC refers to measures taken by the manufacturer to determine compliance with the requirements for materials and workmanship as stated in certification documents and contract specifications.
- *Manufacturing Quality Assurance (MQA):* A planned system of activities that provides assurance that the materials were constructed as specified in the certification documents and contract plans (U.S. EPA 1993). MQA includes manufacturing facility inspections, verifications, audits, and evaluation of the raw materials and geosynthetic products to assess the quality of the manufactured

materials. MQA refers to measures taken by the MQA organization to determine if the manufacturer is in compliance with the product certification and contract specifications for a project.

- *Construction Quality Control (CQC):* A planned system of inspections that is used directly to monitor and control the quality of a construction project (U.S. EPA 1986, 1993). Construction quality control is normally performed by the geosynthetics installer or, for natural soil materials, by the earthwork contractor and is necessary to achieve quality in the constructed or installed system. CQC refers to measures taken by the installer or contractor to determine compliance with the requirements for materials and workmanship as stated in the plans and specifications for the project.
- *Construction Quality Assurance (CQA):* A planned system of activities that provides the owner and permitting agency assurance that the facility was constructed as specified in the design (U.S. EPA 1986, 1993). Construction quality assurance includes inspections, verifications, audits, and evaluations of materials and workmanship necessary to determine and document the quality of the constructed facility. CQA refers to measures taken by the CQA organization to assess if the installer or contractor is in compliance with the plans and specifications for a project.

MQA and CQA are performed independently from MQC and CQC. Although MQA/CQA and MQC/CQC are separate activities, they have similar objectives and, in a smoothly running project, the processes will complement one another. Conversely, an effective MQA/CQA program can lead to identification of deficiencies in the MQC/CQC process, but an MQA/CQA program by itself (in complete absence of an MQC/CQC program) is unlikely to lead to acceptable quality management. Quality is best ensured with effective MQC/CQC *and* MQA/CQA programs. See Figure 2-1 for the usual interaction of the various elements in a total program. Note that the concepts embodied in Figure 2-1 should also pertain to ancillary operations such as test pads, leak location surveys, and related critical field activities.

2.2 Responsibility and Authority

Many individuals are involved directly or indirectly in MQC/CQC and MQA/CQA activities. The individuals, their affiliations, and their responsibilities and authority are discussed below.

The principal organizations and individuals involved in designing, permitting, constructing, and inspecting waste containment facilities are the following:

- *Permitting Agency*. The permitting agency is usually a state regulatory agency but may include local or regional agencies and the federal U.S. Environmental Protection Agency (U.S. EPA). Other federal agencies, such as the U.S. Army Corps of Engineers, the U.S. Bureau of Reclamation, the U.S. Bureau

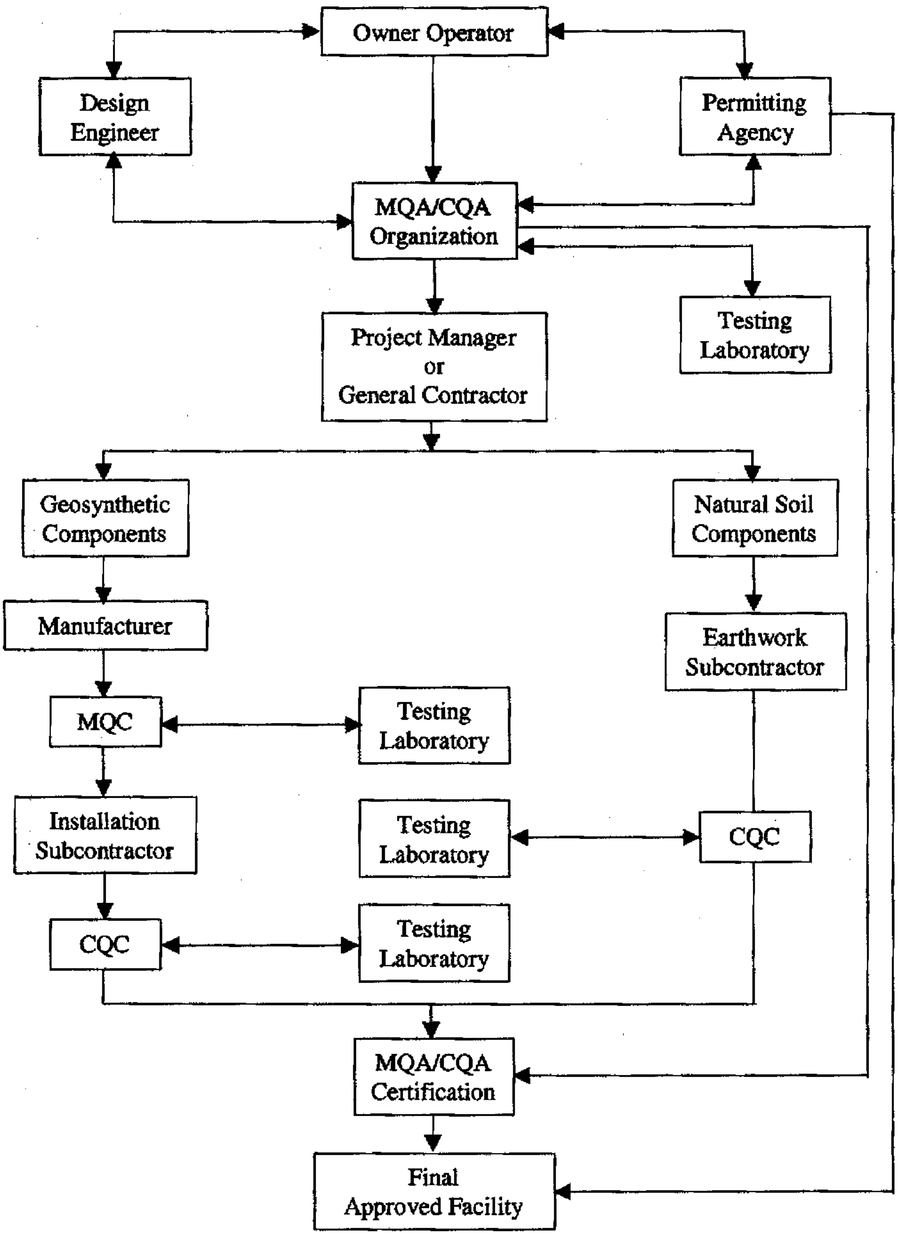

Figure 2-1. Organizational Structure of Manufacturing and Construction Quality Assurance (MQA/CQA) Inspection Activities.

of Mines, or their regional or state affiliates are sometimes also involved. The permitting agency reviews the owner/operator's permit application, including plans, specifications, and the site-specific MQA/CQA document, for compliance with the agency's regulations and to make a decision to issue or deny a permit based on this review. The permitting agency also has the responsibility to review all MQA/CQA documentation during or after construction of a facility, possibly including visits to the manufacturing facility and construction site to observe the MQC/CQC and MQA/CQA practices and to confirm that the approved MQA/CQA plan was followed and that the facility was constructed as specified in the design.

- *Owner/Operator*. This organization (private or public) will own and operate the disposal unit. The owner/operator is responsible for the design, construction, and operation of the waste disposal unit. This responsibility includes complying with the requirements of the permitting agency, submitting MQA/CQA documentation, and assuring the permitting agency that the facility was constructed as specified in the construction plans and specifications and as approved by the permitting agency. The owner/operator has the authority to select and dismiss organizations charged with design, construction, and MQA/CQA. If the owner and operator of a facility are different organizations, the owner is ultimately responsible for these activities. Often the owner/operator or owner will be a municipality rather than a private corporation. The interaction of a state regulatory office with another state or local owner/operator organization should have absolutely no effect on procedures, intensity of effort, and ultimate decisions of the MQA/CQA or MQC/CQC process as described herein.
- *Owner's Representative*. The owner/operator has an official representative who is responsible for coordinating schedules, meetings, and field activities. This responsibility includes coordination among all parties involved, that is, the owner's representative, the permitting agency, material suppliers, the general contractor, specialty subcontractors or installers, and the MQA/CQA engineer.
- *Design Engineer*. The design engineer's primary responsibility is to design a waste containment facility that fulfills the operational requirements of the owner/operator, complies with accepted design practices for waste containment facilities, and meets or exceeds the minimum requirements of the permitting agency. The design engineer may be an employee of the owner/operator or a design consultant hired by the owner/operator. The design engineer may be requested to change some aspects of the design if unexpected conditions are encountered during construction (e.g., a change in site conditions, unanticipated logistical problems during construction, or lack of availability of certain materials). Because design changes during construction are not uncommon, the design engineer is often involved in the MQA/CQA process. The plans and specifications referred to in this manual will generally be the product of the design engineer. The design engineer is a major and essential part of the permit application process and the subsequently constructed facility.
- *Manufacturer*. Many components of a waste containment facility, including all geosynthetics, are manufactured materials. The manufacturer is responsible for the manufacture of its materials and for quality control during manufacture (i.e., MQC). The minimum or maximum (when appropriate) characteristics of acceptable materials should be specified in the permit application. The manufacturer is responsible for certifying that its materials conform to those spec-

ifications and any more stringent requirements or specifications included in the contract of sale to the owner/operator or its agent. The quality control steps taken by a manufacturer are critical to overall quality management in construction of waste containment facilities. Such activities often take the form of process quality control, computer-aided quality control, and the like. All efforts at producing better quality materials are highly encouraged. If requested, the manufacturer should provide information to the owner/operator, permitting agency, design engineer, fabricator, installer, or MQA engineer that describes the quality control (MQC) steps that are taken during the manufacturing of the product. Many manufacturers have quality control programs following ISO 9000 guidelines. Such programs are to be encouraged. In addition, the manufacturer should be willing to allow the owner/operator, permitting agency, design engineer, fabricator, installer, and MQA engineer to observe the manufacturing process and quality control procedures if they so desire. Such visits should be able to be made on an announced or unannounced basis. However, such visits might be coordinated with the manufacturer to ensure that the appropriate people are present to conduct the tour and that manufacture of the proper geosynthetic is scheduled for that date to obtain the most information from the visit. The manufacturer should have a designated individual who is in charge of the MQC program and to whom questions can be directed and through whom visits can be arranged. Random samples of materials should be available for subsequent analysis and archiving. However, the manufacturer should retain the right to insist that any proprietary information concerning the manufacturing of a product be held confidential. Signed agreements of confidentiality are at the discretion of the manufacturer. The owner/operator, permitting agency, design engineer, fabricator, installer, or MQA engineer may request that they be allowed to observe the manufacture and quality control of some or all of the raw materials and final products to be used on a particular job; the manufacturer should be willing to accommodate such requests. Note that these same comments apply to sales organizations that represent a manufactured product made by others.

- *Fabricator*. Some geosynthetic materials are fabricated from individual manufactured components. For example, certain geomembranes are fabricated by seaming together smaller, manufactured geomembrane sheets at the fabricator's facility. The minimum characteristics of acceptable fabricated materials are specified in the permit application. The fabricator is responsible for certifying that its materials conform to those specifications and any more stringent requirements or specifications included in the fabrication contract with the owner/operator or its agent. The quality control steps taken by a fabricator are critical to overall quality in construction of waste containment facilities. If requested, the fabricator should provide information to the owner/operator, permitting agency, design engineer, installer, or MQA engineer that describes the quality control steps that are taken during fabrication of the product. In addition, the fabricator should be willing to allow the owner/operator, permitting agency, design engineer, installer, or MQA engineer to observe the fabrication process and quality control procedures. Such visits may be made on an announced or unannounced basis. However, such visits might be coordinated with the fabricator to ensure that the appropriate people are present to conduct the tour and that the proper geosynthetic is scheduled for fabrication for that date to obtain the most information from the visit. Random samples of materials should be available for subsequent analysis and archiving. However, the fabricator should retain the right to insist that any proprietary information concerning the fabrication of a product be held confidential. Signed agreements of confidentiality are at the discretion of the fabricator. The owner/operator, permitting agency, design engineer, or MQA engineer may request that they be allowed to observe the fabrication process and quality control of some or all fabricated materials to be used on a particular job; the fabricator should be willing to accommodate such a request.

- *Project Manager.* For large waste containment facilities, a project manager may be hired by the owner/operator to control and monitor the construction activities. One of the main tasks in this regard is the decision as to whether to contract with a general contractor or to hire individual subcontractors (e.g., separate contractors for installation of geosynthetics or earthwork placement). Furthermore, the project manager may decide to take on some of the activities typically done by contractors (e.g., procurement of materials). These decisions are made by the owner/operator working with the identified project manager. Also, the project manager must carefully coordinate the activities described below for the general, installation, and earthwork contractors.
- *General Contractor*. The general contractor has overall responsibility for construction of a waste containment facility and for CQC during construction. The general contractor arranges for purchase of materials that meet the plans and specifications, enters into a contract with one or more fabricators (if fabricated materials are needed) to supply those materials, contracts with one or more installers (if separate from the general contractor's organization), and has overall control over the construction operations, including scheduling and CQC. The general contractor has the primary responsibility for ensuring that a facility is constructed in accordance with the plans and specifications that have been developed by the design engineer and approved by the permitting agency. The general contractor is also responsible for informing the owner/operator and the MQA/CQA engineer of the scheduling and occurrence of all construction activities. As mentioned previously, a waste containment facility may be constructed without a general contractor. For example, an owner/operator or project manager may arrange for all the necessary material, fabrication, and installation contracts. In such cases, the owner/operator's representative or project manager will serve the same function as the general contractor.
- *Installation Contractor*. Manufactured products (such as geosynthetics) are placed and installed in the field by an installation contractor who is a general contractor, a subcontractor to the general contractor, or a specialty contractor hired

directly by the owner/operator. The installer's personnel may be employees of the owner/operator, manufacturer, or fabricator, or they may work for an independent installation company hired by the general contractor, the owner/operator, or the project manager. The installer is responsible for handling, storage, placement, and installation of manufactured and fabricated materials. The installer should have a CQC plan that details the proper manner in which materials are to be handled, stored, placed, and installed. The installer is also responsible for informing the owner/operator and the MQA/CQA engineer of the scheduling and occurrence of all geosynthetic construction activities.

- *Earthwork Contractor*. The earthwork contractor is responsible for grading the site to elevations and grades shown on the plans and for constructing earthen components of the waste containment facility (e.g., compacted clay liners and granular drainage layers) according to the specifications. The earthwork contractor may be hired by the general contractor or, if the owner/operator serves as the general contractor, by the owner/operator directly. In some cases, the general contractor's personnel may serve as the earthwork contractors. The earthwork contractor is responsible not only for grading the site to proper elevations but also for obtaining suitable earthen materials, transport and storage of those materials, preprocessing of materials (if necessary), placement and compaction of materials, and protection of materials during and (in some cases) after placement. If a test pad is required, the earthwork contractor is usually responsible for construction of the test pad. It is highly suggested that the same earthwork contractor that constructs the test fill also constructs the waste containment facility's compacted clay liner so that the experience gained from the test fill process will not be lost. Earthwork functions must be carried out in accord with plans and specifications approved by the permitting agency. The earthwork contractor should have a CQC plan (or agree to one written by others) and is responsible for CQC operations aimed at controlling materials and placement of those materials to conform with project specifications. The earthwork contractor is also responsible for informing the owner/operator and the CQA engineer of the scheduling and occurrence of all earthwork construction activities.
- *CQC Personnel*. Construction quality control personnel are individuals who work for the general contractor, installation contractor, or earthwork contractor and whose job is to ensure that construction is taking place in accord with the plans and specifications approved by the permitting agency. In some cases, CQC personnel, perhaps even a separate company, may also be part of the installation or construction crews. In other cases, supervisory personnel provide CQC or, for large projects, separate CQC personnel, perhaps even a separate company, may be used. It is recommended that a certain portion of the CQC staff should be certified. Such a program is available through the International Association of Geosynthetic Installers (IAGI).
- *MQA/CQA Engineer*. The MQA/CQA engineer has overall responsibility for manufacturing quality assurance and construction quality assurance. The engineer is usually an individual experienced in a variety of activities, although

particular specialists in soil placement, polymeric materials, and geosynthetic placement will invariably be involved in a project. The MQA/CQA engineer is responsible for reviewing the MQA/CQA plan, as well as general plans and specifications for the project so that the MQA/CQA plan can be implemented with no contradictions or unresolved discrepancies. Other responsibilities of the MQA/CQA engineer include educating inspection personnel on MQA/CQA requirements and procedures and special steps that are needed on a particular project, scheduling and coordinating MQA/CQA inspection activities, ensuring that proper procedures are followed, ensuring that testing laboratories conform to MQA/CQA requirements and procedures, ensuring that sample custody procedures are followed, confirming that test data are accurately reported and that test data are maintained for later reporting, and preparing periodic reports. The most important duty of the MQA/CQA engineer is overall responsibility for confirming that the facility was constructed in accord with plans and specifications approved by the permitting agency. In the event of nonconformance with the project specifications or CQA plan, the MQA/CQA engineer should notify the owner/operator about the details and, if appropriate, recommend work stoppage and possibly remedial actions. The MQA/CQA engineer is usually hired by the owner/operator and functions separately and independently. The MQA/CQA engineer must be a registered professional engineer who has shown competence and experience in similar projects and is considered qualified by the permitting agency. It is recommended that the person's resume and record on similar facilities be submitted in writing and accordingly accepted by the permitting agency before activities commence. The permitting agency may request additional information from the prospective MQA/CQA engineer and his or her associated organization, including experience record, education, registry, and ownership details. The permitting agency may accept or deny the MQA/CQA engineer's qualifications based on such data and revelations. If the permitting agency requests additional information or denies the MQA/CQA engineer's qualifications, it should be done before construction so that information can be supplied or another engineer can be found in time so that the process will not negatively affect the progress of the work. The MQA/CQA engineer is usually required to be at the construction site during all major construction operations to oversee MQA/CQA personnel. The MQA/CQA engineer is usually the MQA/CQA certification engineer who certifies the completed project.

- *MQA/CQA Personnel.* Manufacturing quality assurance and construction quality assurance personnel are responsible for making observations and performing field tests to ensure that a facility is constructed in accord with the plans and specifications approved by the permitting agency. MQA/CQA personnel are usually employed by the same firm as the MQA/CQA engineer or by a firm hired by the firm employing the MQA/CQA engineer. Construction MQA/CQA personnel report to the MQA/CQA engineer. A relatively large proportion (if not the entire group) of the MQA/CQA staff should be trained specifically for MQA/CQA purposes. In this regard, professional courses are available, many offering continuing education units (CEUs). Certification of CQA personnel for both geosynthetic materials and compacted clay liners is available from the Geosynthetic Certification Institute's Inspectors Certification Program (GCI-ICP).

- *Testing Laboratories.* Commercial laboratories perform many MQC/CQC and MQA/CQA tests. The testing laboratories should have their own internal quality control (QC) plan to ensure that laboratory procedures conform to the appropriate American Society for Testing and Materials (ASTM) standards or other applicable testing standards. The testing laboratories are responsible for ensuring that tests are performed in accordance with applicable methods and standards, following internal QC procedures, maintaining sample chain-of-custody records, and reporting data. The testing laboratory should be accredited. For geosynthetic materials, such an accreditation is available through the Geosynthetic Accreditation Institute Laboratory Accreditation Program (GAI-LAP). The testing laboratory must be willing to allow the owner/operator, permitting agency, design engineer, installer, or MQA/CQA engineer to observe the sample preparation and testing procedures or the record-keeping procedures. The owner/operator, permitting agency, design engineer, or MQA/CQA engineer may request that they be allowed to observe some or all tests on a particular job at any time, either announced or unannounced. The testing laboratory personnel must be willing to accommodate such a request, but the observer should not interfere with the testing or slow the testing process.
- *MQA/CQA Certifying Engineer.* The MQA/CQA certifying engineer is responsible for certifying to the owner/operator and permitting agency that the facility has been constructed in accordance with plans and specifications and that the MQA/CQA document has been approved by the permitting agency. The certification statement is usually accompanied by a final MQA/CQA report that contains all the appropriate documentation, including daily observation reports, sampling locations, test results, drawings of record or sketches, and other relevant data. The MQA/CQA certifying engineer may be the MQA/CQA engineer or someone else in the MQA/CQA engineer's organization who is a registered professional engineer with experience and competency in certifying such installations.

2.3 Personnel Qualifications

The key individuals involved in MQA/CQA and their minimum recommended qualifications are listed in Table 2-1.

2.4 Written MQA/CQA Plan

Quality assurance begins with a plan that eventually becomes the QA document. The words "plan" and "document" are used interchangeably. The final work

Table 2-1. Recommended Personnel Qualifications

Individual	*Minimum Recommended Qualifications*
Design engineer	Registered professional engineer with design experience in similar waste containment facilities.
Project manager	The organization or individual designated by the owner with knowledge of the project, its plans, specifications, and QC/QA documents. Often omitted for small projects.
Owner's representative	The individual designated by the owner with knowledge of the project, its plans, specifications, and QC/QA documents.
Manufacturer/fabricator	Experience in properly manufacturing, or fabricating, at least 1 million m^2 (10 million ft^2) of similar geosynthetic materials. Registry via ISO 9000 is encouraged.
MQC personnel	Manufacturer- or fabricator-trained personnel in charge of quality control of the geosynthetic materials to be used in the specific waste containment facility.
MQC officer	The individual designated by a manufacturer or fabricator, in charge of geosynthetic material quality control.
Geosynthetic installer's representative	Experience in properly installing at least 1 million m^2 (10 million ft^2) of similar geosynthetic materials.
CQC personnel	Employed by the general contractor, installation contractor, or earthwork contractor involved in waste containment facilities; certification via IAGI for geosynthetics or equivalent is recommended.
CQA personnel	Employed by an organization that operates separately from the contractor and the owner/operator; experience via professional courses or certification is recommended.
Testing laboratory personnel	Experience in testing similar natural soils or geosynthetics involved in waste containment facilities. Laboratories testing geosynthetics should be accredited by GAI-LAP or its equivalent.
MQA/CQA engineer	Employed by an organization that operates separately from the contractor and owner/operator, a registered professional engineer approved by the permitting agency.
MQA/CQA certifying engineer	Employed by an organization that operates separately from the contractor and owner/operator, a registered professional engineer in the state in which the waste containment facility is constructed and approved by the appropriate permitting agency.
MQA/CQA personnel	Employed by the MQA/CQA engineer or certifying engineer. Certification via GCI-ICP for geosynthetic materials and compacted clay liners is recommended.

product includes both MQA and CQA. These activities are never ad hoc processes that are developed while they are being implemented. A written MQA/CQA plan or document must precede any field construction activities.

The MQA/CQA plan is the owner/operator's written document for MQA/CQA activities. The MQA/CQA document should include a detailed description of all MQA/CQA activities that will be used during materials manufacturing and construction to manage the installed quality of the facility. The MQA/CQA document should be tailored to the specific facility to be constructed and be completely integrated into the project plans and specifications. Differences should be settled before any construction work commences.

Most state and federal regulatory agencies require that the MQA/CQA document be submitted by the owner/operator and be approved by that agency before construction. The MQA/CQA document is usually part of the permit application.

A copy of the site-specific plans and specifications, MQA/CQA plan, and MQA/CQA documentation reports should be retained at the facility by the owner/operator or the MQA/CQA engineer. The plans, specifications, and MQA/CQA documents may be reviewed during a site inspection by the permitting agency and will be the chief means for the facility owner/operator to demonstrate to the permitting agency that the MQA/CQA objectives for a project are being met.

Written MQA/CQA documents vary greatly from project to project. No general outline or suggested list of topics is applicable to all projects or all regulatory agencies. The elements covered in this document provide guidance on topics that should be addressed in the written MQA/CQA plan.

2.5 Documentation

A major purpose of the MQA/CQA process is to provide documentation for those individuals who were unable to observe the entire construction process (e.g., representatives of the permitting agency) so that those individuals can make informed judgments about the quality of construction for the project. MQA/CQA procedures and results must be thoroughly documented.

2.5.1 Daily Inspection Reports

Routine daily reporting and documentation procedures should be required. Inspectors should prepare daily written inspection reports that may ultimately be included in the final MQA/CQA document. Copies of these reports should be available from the MQA/CQA engineer. The daily reports should include information about work that was accomplished, tests and observations that were made, and descriptions of the adequacy of the work that was performed.

2.5.2 Daily Summary Reports

A daily written summary report should be prepared by the MQA/CQA engineer. This report provides a chronological framework for identifying and recording all other reports and aids in tracking what was done and by whom. At a minimum, the daily summary reports should contain the following (modified from Spigolon and Kelly 1984; U.S. EPA 1986, 1993):

- date, project name, location, waste containment unit under construction, personnel involved in major activities, and other relevant identification information;
- description of weather conditions, including temperature, cloud cover, and precipitation;
- summaries of any meetings held and actions recommended or taken;
- specific work units and locations of construction underway during that particular day;
- equipment and personnel being used in each work task, including subcontractors;
- identification of areas or units of work being inspected;
- testing conducted and test methods that are used;
- unique identifying sheet number of geomembranes for cross-referencing and document control;
- description of off-site materials received, including any quality control data provided by suppliers;
- calibrations or recalibrations of test equipment, including actions taken as a result of recalibration;
- decisions made regarding approval of units of material or of work, and corrective actions to be taken in instances of substandard or suspect quality;
- unique identifying sheet numbers of inspection data sheets and problem reporting and corrective measures used to substantiate any MQA/CQA decisions described in the previous item; and
- the signature of the MQA/CQA engineer.

2.5.3 Inspection and Testing Reports

All observations, results of field tests, and results of laboratory tests performed on site or off site should be recorded on a suitable data sheet. Recorded observations may take the form of notes, charts, sketches, photographs, or any combination of these. Where possible, a checklist may be useful to ensure that pertinent factors are not overlooked.

At a minimum, the inspection data sheets should include the following information (modified from Spigolon and Kelly 1984; U.S. EPA 1986, 1993):

- description or title of the inspection activity;
- location of the inspection activity or location from which the sample was obtained;
- type of inspection activity and procedure used (referenced to a standard method when appropriate or the specific method described in the MQA/CQA plan);
- the unique identifying geomembrane sheet number for cross-referencing and document control;

- recorded observations or test data;
- results of the inspection activity (e.g., pass/fail); comparison with specification requirements;
- personnel involved in the inspection besides the individual preparing the data sheet; and
- the signature of the MQA/CQA inspector and review signature of the MQA/CQA engineer.

2.5.4 Problem Identification and Corrective Measures Reports

A problem is defined as material or workmanship that does not meet the requirements of the plans, specifications, or MQA/CQA document for a project or any obvious defect in material or workmanship, even if there is conformance with plans, specifications, and the MQA/CQA documents. At a minimum, problem identification and corrective measures reports should contain the following information (modified from U.S. EPA 1986, 1993):

- location of the problem;
- description of the problem (in sufficient detail and with supporting sketches or photographic information where appropriate);
- unique identifying geomembrane sheet number for cross-referencing and document control;
- probable cause;
- how and when the problem was located (reference to inspection data sheet or daily summary report by inspector);
- where relevant, estimation of how long the problem has existed;
- any disagreement noted by the inspector between the inspector and contractor about whether or not a problem exists or the cause of the problem;
- suggested corrective measures;
- documentation of the correction if corrective action was taken and completed before finalization of the problem and corrective measures report (reference to inspection data sheet, where applicable);
- where applicable, suggested methods to prevent similar problems; and
- the signature of the MQA/CQA inspector and review signature of the MQA/CQA engineer.

2.5.5 Drawings of Record

Drawings of record (also called "as-built" drawings) should be prepared to document the actual lines, grades, and conditions of each component of the liner system. For soil components, the record drawings should include survey data that show bottom and top elevations of a particular component, the plan dimensions of the component, and locations of all destructive test samples. For geosynthetic components, the drawings of record should show the dimensions of all geomembrane field panels, the location of each panel, identification of all seams and panels with appropriate identification numbering or lettering, the location of all patches and repairs, and the location of all destructive test samples. Separate drawings are often needed to show cross sections and special features such as sump areas and penetrations.

2.5.6 Final Documentation and Certification

At the completion of a project, or as a component of a large project, the owner/operator should submit a final report to the permitting agency. This report may include all of the daily inspection reports, the daily MQA/CQA engineer's summary reports, inspection data sheets (including tests conducted and test methods used), problem identification and corrective measures reports, and other documentation, such as quality control data provided by manufacturers or fabricators, laboratory test results, photographs, as-built drawings, internal MQA/CQA memoranda or reports with data interpretation or analyses, and design changes made by the design engineer during construction. The document should be certified by the MQA/CQA certifying engineer.

The final documentation should emphasize that areas of responsibility and lines of authority were clearly defined, understood, and accepted by all parties involved in the project (assuming that this was the case). Signatures of the owner/operator's representative, design engineer, MQA/CQA engineer, general contractor's representative, specialty subcontractor's representative, and MQA/CQA certifying engineer may be included as confirmation that each party understood and accepted the areas of responsibility and lines of authority outlined in the MQA/CQA plan.

2.5.7 Document Control

The MQA/CQA documents that have been agreed on should be maintained under a document control procedure. Any portion of the documents that is modified must be communicated to and agreed on by all parties involved. An indexing procedure should be developed for convenient replacement of pages in the MQA/CQA plan, should modifications become necessary, with revision status indicated on appropriate pages.

A control scheme should be implemented to organize and index all MQA/CQA documents. This scheme should be designed to allow easy access to all MQA/CQA documents and should enable a reviewer to identify and retrieve original inspection reports or data sheets for any completed work element.

2.5.8 Storage of Records

During construction, the MQA/CQA engineer should be responsible for all MQA/CQA documents. This includes a copy of the design criteria, plans, specifications, MQA/CQA plan, and originals of all data sheets and reports. Duplicate records should be kept at another location to avoid loss of this valuable information if the originals are destroyed.

Once construction is complete, the document originals should be stored by the owner/operator in a manner that will allow for easy access while still protecting them from damage or loss. An additional copy should be kept at the facility if this is in a different location from the owner/operator's main files. A final copy should be kept by the permitting agency. All documentation should be maintained through the operating and postclosure monitoring periods of the facility by the owner/operator and the permitting agency in an agreed-on format (e.g., paper hard copy, microfiche, or electronic medium).

2.6 Meetings

Communication is extremely important to quality management. Quality construction is easiest to achieve when all parties involved understand clearly their responsibility and authority. Meetings can be helpful to make sure that responsibility and authority of each organization is clearly understood. During construction, meetings can help to resolve problems or misunderstandings and to find solutions to unanticipated problems that have developed.

2.6.1 Pre-Bid Meeting

The first meeting is held to discuss the project plans and specifications along with the MQA/CQA plan and to resolve differences of opinion before the project is released for bidding. The pre-bid meeting is held after the permitting agency has issued a permit for a waste containment facility and before a construction contract has been awarded. The pre-bid meeting is held before construction bids are prepared so that the companies bidding on the construction will better understand details of the project and the level of MQA/CQA to be used on the project. Also, if the bidders identify problems with the MQA/CQA plan, this affords the owner/operator an opportunity to rectify those problems early in the process.

2.6.2 Resolution Meeting

The objectives of the resolution meeting are to establish lines of communication, review construction plans and specifications, emphasize the critical aspects of a project necessary to ensure proper quality, begin planning and coordination of tasks, and anticipate any problems that might cause difficulties or delays in construction. The meeting should be attended by the owner/operator's representative, design engineer, project manager, representatives of the general contractor and major subcontractors, the MQA/CQA engineer, and the MQA/CQA certifying engineer.

The resolution meeting usually involves the following activities:

- An individual is assigned to take minutes (usually a representative of the owner/operator or of the MQA/CQA engineer's organization).

- Individuals are introduced to one another, and their responsibilities (or potential responsibilities) are identified.
- Copies of the project plans and specifications are made available for discussion.
- The MQA/CQA plan is distributed.
- Copies of any special permit restrictions that are relevant to construction or MQA/CQA are distributed.
- The plans and specifications are described; any unique design features are discussed (so that the contractors will understand the rationale behind the general design); any potential construction problems are identified and discussed; and questions from any of the parties concerning the construction are discussed.
- The MQA/CQA plan is reviewed and discussed, with the MQA/CQA engineer and MQA/CQA certifying engineer describing their expectations and identifying the most critical components.
- Procedures for MQC/CQC proposed by manufacturers, installers, and contractors are reviewed and discussed.
- Corrective actions to resolve potential construction problems are discussed.
- Procedures for documentation and distribution of documents are discussed.
- Each organization's responsibility, authority, and lines of communication are discussed.
- Suggested modifications to the MQA/CQA plan that would improve quality management on the project are solicited.
- Construction variables (e.g., precipitation, wind, and temperature) and schedule are discussed.

It is important that the procedures for inspection and testing be known to all, that the criteria for pass/fail decisions be clearly defined (including the resolution of test data outliers), that all parties understand the key concerns that the MQA/CQA personnel will be particularly careful to identify, that each individual's responsibilities and authority be understood, and that procedures regarding resolution of problems be understood. The resolution meeting may be a separate meeting or may be held in conjunction with either the pre-bid meeting (rarely) or the preconstruction meeting (often).

2.6.3 Preconstruction Meeting

The preconstruction meeting is held after a general construction contract has been awarded and the major subcontractors and material suppliers are established. It distinguishes itself from the resolution meeting in that the specific individuals who will actually perform the work are now in attendance. It is the first time in a particular project that actual individuals are known for each element of the work activity. This meeting is usually held concurrent with the initiation of construction. The purpose of this meeting is to review the details of the MQA/CQA plan, to make sure that the responsibility and authority of each individual is clearly understood, to agree on procedures to resolve construction problems, and to establish a foundation of cooperation in quality management. The preconstruction meeting should be attended by the owner/operator's representative, the design engineer, the project manager, representatives of the general contractor and major subcontractors, the MQA/CQA engineer, the MQA/CQA certifying engineer, and a representative from the permitting agency, if that agency expects to visit the site during construction or independently observe MQA/CQA procedures.

The preconstruction meeting should include the following activities.

- Assign an individual (usually a representative of the MQA/CQA engineer) to take minutes.
- Introduce parties and identify their responsibility and authority.
- Distribute the MQA/CQA plan, identify any revisions made after the resolution meeting, and answer any questions about the MQA/CQA plan, procedures, or documentation.
- Discuss responsibilities and lines of communication.
- Discuss reporting procedures, distribution of documents, the schedule for any regular meetings, and resolution of construction problems.
- Review site requirements and logistics, including safety procedures.
- Review the design, discuss the most critical aspects of the construction, and discuss scheduling and sequencing issues.
- Discuss MQC procedures that the geosynthetics manufacturers will use.
- Discuss CQC procedures that the installer or contractor will use; for example, establish and agree on geomembrane repair procedures.
- Make a list of action items that require resolution and assign responsibilities for these items.

2.6.4 Progress Meeting

Weekly progress meetings should be held. Weekly meetings can be helpful in maintaining lines of communication, resolving problems, identifying action items, and improving overall quality management. When numerous critical work elements are being performed, the frequency of these meetings can be increased to biweekly, or even daily. People who should attend these meetings are those involved in the specific issues being discussed. At all times, the MQA/CQA engineer, or a designated representative, should be present.

2.7 Sample Custody

All samples shall be identified as described in the MQA/CQA plan. Whenever a sample is taken, a chain of custody record should be made for that sample. If the sample is transferred to another individual or laboratory, records shall be kept of the transfer so that chain of custody can be traced. The purpose of keeping a record of sample custody is to assist in tracing the cause of anomalous test results or other testing problems and to help prevent accidental loss of test samples.

Soil samples are usually discarded after testing. Destructive testing samples of geosynthetic materials are often taken in triplicate, with one sample tested by CQC personnel, one tested by CQA personnel, and the third retained in storage as prescribed in the CQA plan.

2.8 Weather

Weather can play a critical role in the construction of waste containment facilities. Installation of all geosynthetic materials (including geosynthetic clay liners) and natural clay liners is particularly sensitive to weather conditions, including temperature, wind, humidity, and precipitation. The contractor or installer is responsible for complying with the contract plans and specifications (along with the MQC/CQC plans for the various components of the system). Included in this information should be details for restrictive weather conditions in which certain activities can take place. It is the responsibility of the contractor or installer to make sure that these weather restrictions are observed during construction.

2.9 Work Stoppages

Unexpected work stoppages can occur because of a variety of causes, including labor strikes, contract disputes, weather, or QC or QA problems. The MQA/CQA engineer should be particularly careful during such stoppages to determine

- whether in-place materials are covered and protected from damage (e.g., lifting of a geomembrane by wind, premature hydration of geosynthetic clay liners, or freezing of compacted clay liners);
- whether partially covered materials are protected from damage (e.g., desiccation of compacted clay liners); and
- whether manufactured materials are properly stored and properly or adequately protected (e.g., geotextiles should be protected from UV exposure).

The cessation of construction should not indicate the cessation of MQA/CQA inspection and documentation.

2.10 References

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CHAPTER 3

Compacted Clay Liners

3.1 Introduction and Background

3.1.1 Types of Compacted Clay Liners

Compacted clay liners (CCLs) are widely used as hydraulic barriers for water retention and waste containment facilities. Compacted clay liners are sometimes used by themselves, but more frequently are used in combination with a geomembrane to form a *composite liner,* which usually consists of a geomembrane placed directly on the surface of a CCL. Some liner and cover systems contain a single compacted clay liner, but others may contain two or more CCLs. Examples of CCLs used in liner and cover systems are shown in Figure 3-1. The thickness of a CCL is usually between 600 and 900 mm, but occasionally the thickness may reach 1.2 to 3.0 m.

3.1.1.1 Natural Mineral Materials

The most common type of compacted clay liner is one that is constructed from naturally occurring soils that contain a significant quantity of clay (i.e., soils that are classified as CL, CH, or SC in the Unified Soil Classification System (USCS) outlined in ASTM D2487). Clay liner materials are excavated from *borrow areas*. Sources of clay liner materials include lacustrine deposits, glacial tills, aeolian materials, deltaic deposits, residual soils, and other types of soil deposits. Weakly cemented or highly weathered rocks (e.g., mudstones and shales) can also be used for soil liner materials, provided they are processed properly.

3.1.1.2 Soil–Bentonite Blends

If the soils available in the vicinity of a waste disposal facility are not sufficiently clayey for direct use as a soil liner material, a common practice is to blend natural soils with *bentonite*. Bentonite is a commercially processed material that is composed primarily of the mineral group smectite (the specific mineral is usually montmorillonite). Bentonite may be supplied in granular or pulverized form. The dominant adsorbed cation of commercial bentonite is usually sodium or calcium, although the sodium form is much more commonly used for soil sealing applications in North America. Calcium bentonite is more commonly used in parts of

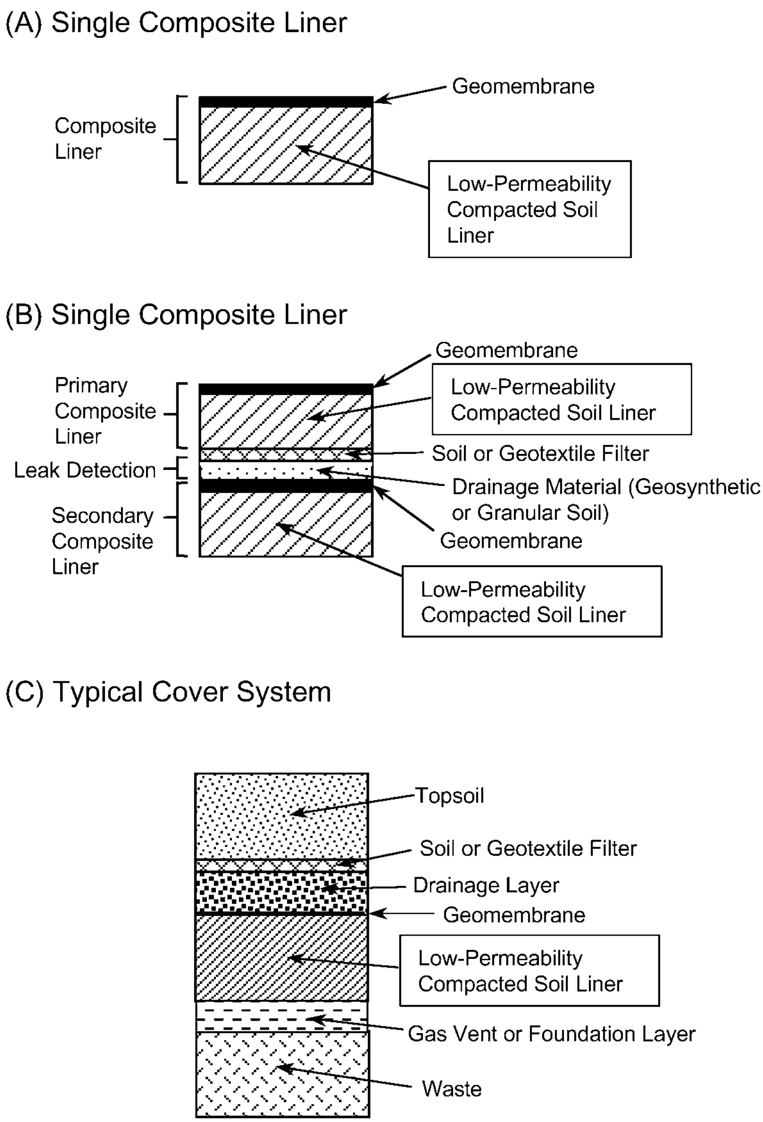

Figure 3-1. Examples of Compacted Soil Liners in Liner and Cover Systems.

Europe. Bentonite is mixed with native soils either in place or in a *pugmill*, which is a large machine for mixing bulk ingredients such as concrete.

3.1.1.3 Other Materials

Other materials have occasionally been used for compacted clay liners. For example, bentonite may be blended with fly ash to form a liner under certain circumstances. Modified soil minerals and commercial additives (e.g., polymers) have sometimes been used. Alternative materials such as paper industry sludge have occasionally been employed (Zimmie and Moo-Young 1995; Kraus et al. 1997b).

Asphaltic concrete has occasionally been used as a substitute for a CCL (Wing and Gee 1994; Bowders et al. 2003).

3.1.2 Critical CQC and CQA Issues

The CQC and CQA processes for CCLs are intended to accomplish three objectives: (1) to ensure that liner materials are suitable; (2) to ensure that liner materials are properly placed and compacted; and (3) to ensure that the completed liner is properly protected.

Some of these issues, such as protection of the liner from desiccation after completion, simply require application of common-sense procedures. Other issues, such as preprocessing of materials, are potentially more complicated because, depending on the material, many construction steps may be involved. Furthermore, tests alone will not adequately address many of the critical CQC and CQA issues. Visual observations by qualified personnel, supplemented by intelligently selected tests, provide the best approach to ensure quality in the constructed liner.

The objective of CQA is to ensure that the final product meets specifications. A detailed program of tests and observations is necessary to accomplish this objective. The objective of CQC is to control the manufacturing or construction process to meet project specifications. With geosynthetics, the distinction between CQC and CQA is obvious: The geosynthetics installer performs CQC, and a separate organization conducts CQA. However, CQC and CQA activities for soils are more closely linked than in geosynthetics installation. For example, on many earthwork projects, the CQA personnel will typically determine the water content of the soil and report the value to the contractor; in effect, the CQA organization is also providing CQC input to the contractor. On some projects, the contractor is required to perform extensive tests as part of the CQC process, and the CQA organization performs tests to check or confirm the results of CQC tests.

The lack of clearly separate roles for CQC and CQA in the earthwork industry is a result of historic practices and procedures. This chapter focuses on CQA procedures for clay liners, but CQA and CQC practices are often closely linked in earthwork. In any event, the QA plan should clearly establish QA procedures and should consider whether there will be QC tests and observations to complement the QA process.

3.1.3 Liner Requirements

As stated in Section 3.1.2, proper construction of clay liners requires the use of suitable materials, proper placement and compaction of the materials, and adequate protection of the completed liner. The steps required to fulfill these requirements may be summarized as follows:

1. The subgrade on which the CCL will be placed should be properly prepared.
2. The materials used in constructing the CCL should be suitable and should conform to the plans and specifications for the project.

3. The liner material should be preprocessed, if necessary, to adjust the water content, to remove oversized particles, to break down clods of soil, or to add amendments, such as bentonite.
4. The soil should be placed in lifts of appropriate thickness and then properly remolded and compacted.
5. The completed CCL should be protected from damage caused by desiccation or freezing temperatures.
6. The final surface of the CCL should be properly prepared to support the next layer that will be placed on top of the soil liner.

These six steps are described in more detail in the succeeding subsections. Detailed requirements are discussed later.

3.1.3.1 Subgrade Preparation

The subgrade on which a clay liner is placed should be properly prepared, that is, it should have adequate support for compaction and be free from mass movements. The CCL may be placed on a natural or geosynthetic material, depending on the particular design and the individual component in the liner or cover system. If the CCL is the lowest component of the liner system, native soil or rock forms the subgrade. In such cases, the subgrade is usually compacted to eliminate soft spots. Water should be added or removed as necessary to produce a suitably firm subgrade per specification requirements. In other instances, the CCL may be placed on top of geosynthetic components of the liner system (e.g., a geotextile). In such cases, the main concerns are the smoothness of the geosynthetic on which soil is placed, conformity of the geosynthetic to the underlying material (e.g., no bridging over ruts left by vehicle traffic), and protection of the geosynthetic component from damage during placement of the first lift of the soil liner.

Sometimes it is necessary to "tie in" a new section of soil liner to an old one (e.g., when a landfill is being expanded laterally). In such cases, a lateral excavation should be made about 2 to 5 m (5–15 ft) into the existing section of CCL, and the existing CCL should be stair-stepped, as shown in Figure 3-2 to tie the new

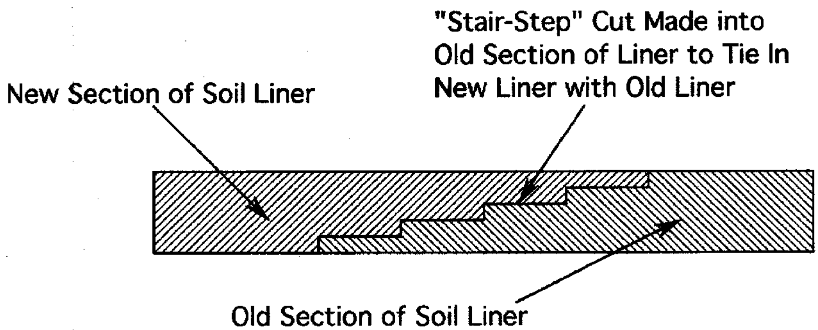

Figure 3-2. Tie-In of New Soil Liner to Existing Soil Liner.

liner into the old one. The surface of each of the steps in the old liner should be scarified (roughened) to maximize bonding between the new and old sections.

3.1.3.2 Material Selection

Clay liner materials are selected so that a low hydraulic conductivity will be produced after the soil is remolded and compacted. Although the performance specification is usually hydraulic conductivity, CQA considerations dictate that restrictions be placed on certain properties of the soil used to build a liner. For example, limitations may be placed on the liquid limit, plastic limit, plasticity index, percent fines, and percent gravel allowed in the soil liner material.

The process of selecting construction materials and verifying the suitability of the materials varies from project to project. In general, the process is as follows:

1. A potential borrow source is located and explored to determine the vertical and lateral extent of the source and to obtain representative samples, which are tested for properties such as liquid limit, plastic limit, and percent fines. The borrow source should also be checked for the presence of deleterious material such as roots, organic matter, and debris.
2. Once construction begins, additional CQC and CQA observations and tests may be performed in the borrow pit to confirm the suitability of materials being removed.
3. After a lift of soil has been placed, additional CQA tests should be performed for final verification of the suitability of the materials.

On some projects, the process may be somewhat different. For example, a materials company may offer to sell clay liner materials from a commercial pit, in which case the first step listed above (location of borrow source) is not relevant.

A variety of tests is performed at various stages of the construction process to ensure that the soil liner material conforms to specifications. However, tests alone will not necessarily ensure an adequate material; observations by qualified CQA personnel are essential to confirm that deleterious materials (such as roots, wood, organic matter, rocks, stones, bricks, construction or demolition debris, or other unacceptable materials not allowed in the specifications) are not present in the soil liner material.

3.1.3.3 Preprocessing

Some soil liner materials require processing before use. Preprocessing steps that may be required include drying of soil that is too wet, wetting of soil that is too dry, removal of oversized particles, pulverization of clods of soil, homogenization of soil, and addition of bentonite.

The degree of processing can affect the performance of the CCL. For example, Benson et al. (1997) studied four test pads built from the same material and constructed to the same specifications, but by different contractors. The principal differences among the pads related to the degree of soil preprocessing. The more extensive the soil processing, and the longer the period of time allowed for the wetted soil to hydrate, the lower the hydraulic conductivity.

Tests are performed by CQA personnel to confirm proper preprocessing, but visual observations by CQC and CQA personnel are needed to confirm that proper procedures have been followed and that the liner material has been properly preprocessed.

3.1.3.4 Placement, Remolding, and Compaction

The soil liner material should first be placed in a loose lift of appropriate thickness. If a loose lift is too thick, adequate compactive energy may not be delivered to the bottom of a lift. The specifications should state the maximum thickness of a loose lift, compacted lift, or both.

The type and weight of compaction equipment can have an important influence on the hydraulic conductivity of the constructed liner. The CQC/CQA program should be designed to ensure that the soil liner material will be properly placed, remolded, and compacted as described in the plans and specifications for the project.

3.1.3.5 Protection

The completed CCL should be protected from damage caused by desiccation or freezing temperatures. Each completed lift of the soil liner, as well as the completed liner, should be protected.

3.1.3.6 Final Surface Preparation

The surface of the liner should be properly compacted and smoothed to serve as a foundation for an overlying geomembrane liner or other component of a liner or cover system. Verification of final surface preparation is an important part of the CQA process.

3.1.4 Compaction Requirements

3.1.4.1 Compaction Curve

A compaction curve is developed by preparing several batches of soil at different water contents and then compacting material from each of the batches into molds of known volume with a specified compaction procedure. The total unit weight (γ) of each compacted specimen is determined by weighing the compacted specimen and dividing the total weight by the total volume.

The term "unit weight" means weight per unit volume. The term "mass density" means mass per unit volume. The term "density" is ambiguous: To some it means mass density, and to others it means unit weight. In compaction work, "density" and "unit weight" are often used interchangeably. The total unit weight (γ) is sometimes called "wet density" or "bulk density."

The water content (w) of each compacted specimen, which is defined as mass of water in a specimen divided by oven-dry mass of solids, is determined by oven-drying the specimen. The dry unit weight (γ_d) is calculated as follows:

$$\gamma_d = \gamma/(1 + w) \tag{3-1}$$

To prepare a compaction curve, the (w, γ_d) points for the samples prepared over a range of water contents are plotted, and a smooth curve is drawn between the points (Figure 3-3). Judgment, rather than an analytic algorithm, is usually used to draw the compaction curve through the measured points. A slight error may result from an improperly drawn curve, but a more serious problem is reliance on too few data points for constructing the compaction curve.

The *maximum dry unit weight* ($\gamma_{d,max}$) occurs at a water content that is called the *optimum water content*, w_{opt} (Figure 3-3). The main reason for developing a compaction curve is to determine the optimum water content and maximum dry unit weight for a given soil and compaction procedure.

The *zero air voids curve* (Figure 3-3), also referred to as the *100% saturation curve,* is a curve that relates dry unit weight to water content for a saturated soil that contains no air. The equation for the zero air voids curve is

$$\gamma_d = \gamma_w/[w + (1/G_s)] \tag{3-2}$$

where G_s is the specific gravity of solids (typically 2.6–2.8) and γ_w is the unit weight of water. If the soil's specific gravity of solids changes, the zero air voids curve will also change. Theoretically, no points can lie above the zero air voids curve, but in practice some points usually do as a result of soil variability and inherent limitations in the accuracy of measurements of water content and unit weight (Schmertmann 1989).

Benson and Boutwell (1992) summarized the maximum dry unit weights and optimum water content measured on soil liner materials from 26 soil liner projects and found that the degree of saturation at the point of (w_{opt}, $\gamma_{d,max}$) ranged from 71% to 98%, based on an assumed G_s value of 2.75. The average degree of saturation at the optimum point was 85%.

3.1.4.2 Compaction Tests

Several methods of laboratory compaction are commonly used. The two procedures that are most commonly used are standard and modified compaction (ASTM D698 and ASTM D1557, respectively). Both techniques usually involve compacting the soil into a mold having a volume of 0.00094 m^3 (1/30 ft^3). The number of lifts, weight of hammer, and height of fall are listed in Table 3-1. The compaction tests are sometimes called *Proctor* tests after Proctor (1933), who developed the tests and described the procedures in several 1933 issues of *Engineering News Record.* Thus, the compaction curves are sometimes called *Proctor curves*, and the maximum dry unit weight is sometimes called the *Proctor density*.

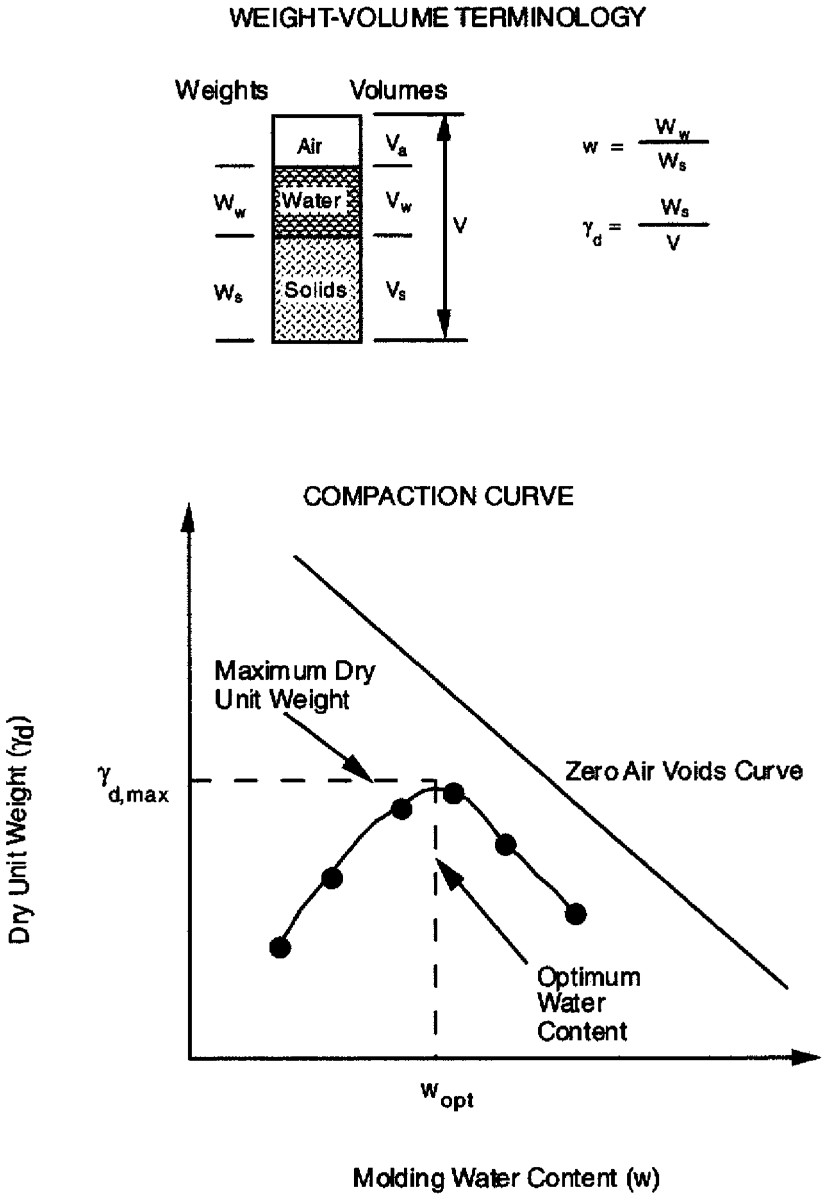

Figure 3-3. Compaction Curve.

Proctor's original test, now frequently called the *standard Proctor compaction test*, was developed to control compaction of soil bases for highways and airfields. The maximum dry unit weights obtained from the standard Proctor compaction test were approximately equal to unit weights observed in the field on well-built fills using compaction equipment available in the 1920s and 1930s. During World

Table 3-1. Compaction Test Details

Compaction Procedure	*Number of Lifts*	*Weight of Hammer*	*Height of Fall*	*Compactive Energy*
Standard	3	24.5 N	305 mm	594 kN-m/m^3
		5.5 lb	12 in.	12,375 ft-lb/ft^3
Modified	5	44.5 N	457 mm	2,693 kN-m/m^3
		10 lb	18 in.	56,250 ft-lb/ft^3

War II, much heavier compaction equipment was developed, and the unit weights attained from field compaction sometimes exceeded the laboratory values. Proctor's original procedure was modified by increasing compactive energy. By today's standards:

- Standard compaction (ASTM D698 and the American Association of State Highway and Transportation Officials' standard AASHTO T-99) produces maximum dry unit weights approximately equal to field dry unit weights for soils that are well compacted using modest-sized compaction equipment (i.e., compactors with a mass of 9–20 Mg [weight of 10–22 tons]).
- Modified compaction (ASTM D1557 and AASHTO T-180) produces maximum dry unit weights approximately equal to field dry unit weights for soils that are well compacted using the heaviest compaction equipment available (i.e., compactors with a mass of 32–36 Mg [weight of 35–40 tons]).

3.1.4.3 Percent Compaction

The compaction test is used to help CQA personnel determine (1) whether the soil is at the proper water content for compaction and (2) whether the soil has received adequate compactive effort. Field CQA personnel will typically measure the water content (w) of the field-compacted soil and compare that value with the optimum water content (w_{opt}) from a laboratory compaction test. The construction specifications may limit the value of w relative to w_{opt} (e.g., specifications may require w to be between 0 and +4 percentage points of w_{opt}). Field CQC personnel should measure the water content of the soil before remolding and compaction to ensure that the material is at the proper water content before the soil is compacted. However, experienced earthwork personnel can usually tell if the soil is at the proper water content from the look and feel of the soil. Field CQA personnel should measure the water content and unit weight after compaction to verify that the water content and dry unit weight meet specifications. Field CQA personnel often compute the percent compaction, P, which is defined as follows:

$$P = \gamma_d / \gamma_{d,max} \times 100\% \tag{3-3}$$

where γ_d is the dry unit weight of the field-compacted soil. Construction specifications often require a minimum acceptable value of P.

In summary, the purpose of the laboratory compaction test as applied to CQC and CQA is to provide water content (w_{opt}) and dry unit weight ($\gamma_{d,max}$) reference points. The actual water content of the field-compacted soil liner may be compared to the optimum value determined from a specified laboratory compaction test. If the water content is not in the proper range, the engineering properties of the soil are not likely to be in the range desired. For example, if the soil is too wet, the shear strength of the soil may be too low. Similarly, the dry unit weight of the field-compacted soil may be compared to the maximum dry unit weight determined from a specified laboratory compaction test. If the percent compaction is too low, the soil has probably not been adequately compacted in the field. Compaction criteria may also be formulated in ways that do not involve percent compaction, as discussed later, but one way or another, the laboratory compaction test provides the reference against which field results are compared.

3.1.4.4 Estimating Optimum Water Content and Maximum Dry Unit Weight

Many CQA plans require that the water content and dry unit weight of the field-compacted soil be compared to values determined from laboratory compaction tests. Laboratory compaction tests are a routine part of almost all CQA programs. However, there are two practical problems with compaction tests: (1) they often take two to four days to complete—for economic reasons, and to keep the job progressing, field personnel usually cannot wait for the completion of a laboratory compaction test to make "pass/fail" decisions; and (2) the soil will inevitably be variable—as a result, the optimum water content and maximum dry unit weight will vary from one location to another. The values of w_{opt} and $\gamma_{d,max}$ determined for one location may not be applicable to another location. This has been termed a "mismatch" problem (Noorany 1990).

Because dozens (sometimes hundreds) of field water content and density tests are performed, it is impractical to perform a laboratory compaction test each time a field measurement of water content and density is obtained. Thus, simple techniques for estimating the maximum dry unit weight and optimum water content are almost always used for rapid field CQA assessments. Four techniques are used: subjective assessment, one-point compaction test, three-point compaction test, and statistical correlations.

3.1.4.4.1 Subjective Assessment

Relatively homogeneous fill materials produce similar results when repeated compaction tests are performed on the soil. A common approach is to estimate optimum water content and maximum dry unit weight based on the results of previous compaction tests. The results of at least two to three laboratory compaction tests should be available from tests on borrow soils before actual compaction of any soil liner material for a project. With subjective assessment, CQA personnel estimate the optimum water content and maximum dry unit weight based on the results of the previously completed compaction tests and their evaluation of the soil at a particular location in the field. Slight variations in the composition of fill materials will cause only slight variations in w_{opt} and $\gamma_{d,max}$. As an approximate guide, a relatively homogeneous borrow soil would be considered a material in which w_{opt} does not vary by more than ±3 percentage points and $\gamma_{d,max}$ does not vary by more than ±0.8 kN/ft^3 (5 pcf). The optimum water content and maximum dry unit weight should not be estimated in this manner if the soil is heterogeneous; too much guess work and opportunity for error would exist.

3.1.4.4.2 One-Point Compaction Test

The idea behind a one-point compaction test is shown in Figure 3-4. A sample of soil is taken from the field and dried to a water content that appears to be just dry of optimum. Experienced field personnel can usually tell without much difficulty when the water content is just dry of optimum. The sample of soil is compacted into a mold of known volume according to the compaction procedure relevant to a particular project (ASTM D698 or ATM D1557). The weight of the compacted specimen is measured, and the total unit weight is computed. The sample is dried using one of the rapid methods of measurement discussed later to determine water content. Dry unit weight is computed using Eq. 3-2. The water content–dry unit weight point from the one-point compaction test is plotted as shown in Figure 3-4 and used in conjunction with available compaction curves to estimate w_{opt} and $\gamma_{d,max}$. The single compaction point must lie on the more general compaction curve; the idea is to estimate the shape of the curve based on other com-

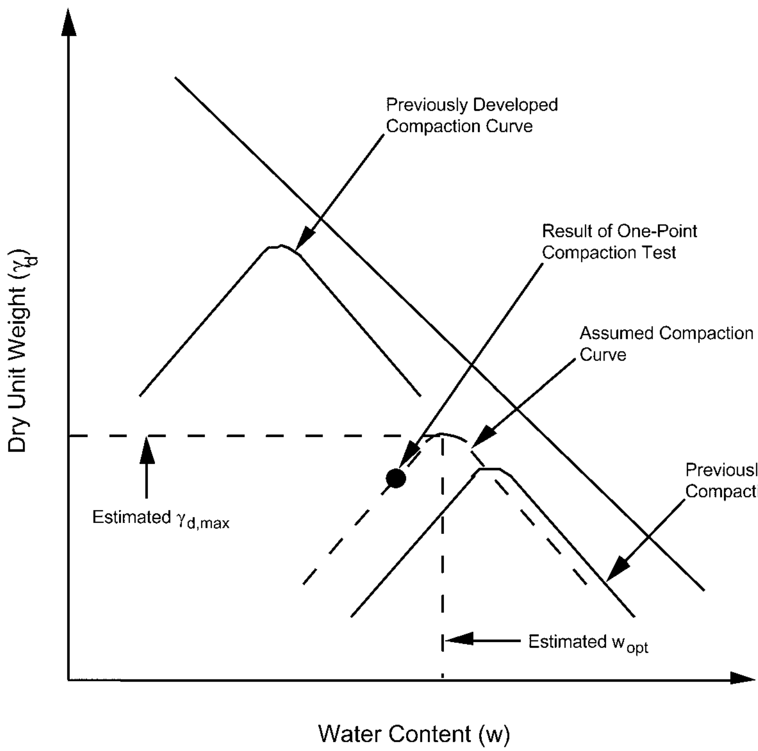

Figure 3-4. One-Point Compaction Test.

paction curves developed for the borrow soil. One assumes that the shapes of the compaction curves are similar to the previously developed curves. The dashed curve in Figure 3-4 is the estimated compaction curve.

The one-point compaction test is commonly used for variable soils. In extreme cases, a one-point compaction test may be required for almost all field water content and density measurements to compute percent compaction. However, if the material is so variable as to require a one-point compaction test for almost all field density measurements, the material may be too variable to be suitable for use in a CCL. The best use of the one-point compaction test is to assist with estimation of the optimum water content and maximum dry unit weight for questionable materials and to fill in data gaps when results of complete compaction tests are not available quickly enough. More one-point compaction tests should be used in the initial stages of construction (compared to later stages), when experience with the borrow soil is limited.

3.1.4.4.3 Three-Point Compaction Test

A more reliable technique than the one-point compaction test for estimating the optimum water content and maximum dry unit weight is to use a minimum of three compaction points to define a curve per ASTM D5080 rather than relying on a single compaction point. A representative sample of soil is obtained from the field at the same location where the in-place water content and dry unit weight have been measured. The first sample of soil is compacted at the field water content. A second sample is prepared at a water content two percentage points wetter than the first sample. However, for extremely wet soils that are more than 2% wet of optimum (which is often the case for soil liner materials), the second sample should be dried 2% below the natural water content. The sample is then compacted and the density determined. Depending on the outcome of this compaction test, a third sample is prepared at a water content either two percentage points dry of the first sample or two percentage points wet of the second sample (or, for wet soil liners, 2 percentage points dry of the second sample). A parabola is fit to the three compaction data points, and the optimum water content and maximum dry unit weight are determined from the equation of the best-fit parabola. This technique is significantly more time-consuming than the one-point compaction test but offers a standard ASTM procedure and greater reliability and repeatability in estimated w_{opt} and $\gamma_{d,max}$.

3.1.4.4.4 Statistical Correlations

Another technique for estimating w_{opt} and $\gamma_{d,max}$ is to make use of statistical correlations, such as those presented by Blotz et al. (1998). The methodology predicts the optimum water content at a particular compaction energy ($w_{opt,E}$, expressed as a percentage) and dry unit weight ($\gamma_{d,max,E}$, in units of kN/m^3) as follows for a compactive energy of E (kilojoules per cubic meter):

$$w_{opt,E} = [12.39 - 12.21 \log(LL)] \log(E) + 0.67\ LL + 9.21 \tag{3-4}$$

$$\gamma_{d,max,E} = [2.27 \log(LL) - 0.94] \log(E) - 0.16\ LL + 17.0 \tag{3-5}$$

This technique is more valuable for general engineering purposes than CQA because the energy applied in the field (E) is generally unknown, and liquid limit tests require a similar amount of time and effort compared to a one-point compaction test.

3.1.5 Test Pads

If laboratory compaction and hydraulic conductivity tests could accurately predict the performance of all CCLs, there would be no need for test pads. Experience has shown that for well-constructed clay liners, laboratory tests do serve as excellent predictors of field performance (Benson et al. 1999). However, experience has also shown that if construction procedures are not adequate, large-scale field hydraulic conductivity can be substantially larger than small-scale laboratory hydraulic tests would indicate. In the extreme, laboratory hydraulic conductivity tests can under predict the large-scale field hydraulic conductivity by a factor of up to 100,000 (Elsbury et al. 1990).

If the factors that cause field hydraulic conductivity to substantially exceed laboratory values on some projects were fully understood, changing materials or construction procedures would eliminate the problem. Unfortunately, quantitative information relating field performance to construction practices is not sufficient at this time to allow one to determine in advance whether the target hydraulic conductivity is achieved. Demonstration of low hydraulic conductivity from small-scale samples taken from the field is a necessary condition for a well-constructed liner but is not sufficient by itself to demonstrate that the large-scale hydraulic conductivity meets the target value.

Test pads are constructed to verify that the materials and methods of construction and CQC/CQA procedures proposed for a project will produce the desired low hydraulic conductivity at field scale. The test pad should not be constructed to better standards than the actual liner. By the same token, if the test pad has an acceptably low field-scale hydraulic conductivity, and the actual liner is built to standards that equal or exceed those used in building the test pad, then the actual liner should be considered acceptable.

The specific objectives of a test pad should be as follows:

1. To verify that the materials and methods of construction will produce a compacted clay liner that meets the hydraulic conductivity objectives defined for a project. Hydraulic conductivity should be measured on the test pad with techniques that will characterize the large-scale hydraulic conductivity and identify any construction defects that cannot be observed with small-scale laboratory hydraulic conductivity tests.
2. To verify that the proposed CQC and CQA procedures will result in a high-quality CCL that will meet performance objectives.
3. To provide a basis of comparison for full-scale CQA: If the test pad meets the performance objectives for the CCL (as verified by appropriate hydraulic conductivity tests) and the full-scale liner is constructed to standards that equal or

exceed those used in building the test pad, then assurance is provided that the full-scale liner will also meet performance objectives.

4. If appropriate, a test pad provides an opportunity for the facility owner to demonstrate that unconventional materials or construction techniques will lead to a soil liner that meets performance objectives.

In terms of CQA, the test pad can provide a powerful tool to ensure that performance objectives are met. The authors strongly recommend a test pad for any project in which failure of the CCL to meet performance objectives would have a significant, negative environmental impact. A test pad is recommended regardless of how much laboratory testing is performed. Laboratory testing is critical (see our later discussion) but does not address questions concerning applicability of small-scale laboratory tests to full-scale construction.

A test pad need not be constructed if results are already available for the soil and construction methodology proposed for a project. By the same token, if the materials or methods of construction used on a project change, an additional test pad is recommended to test the new materials or construction procedures. Specific CQA tests and observations that are recommended for the test pad are described later, in Section 3.11.

3.2 Critical Construction Variables That Affect Clay Liners

Proper construction of compacted clay liners requires careful attention to construction details. In this section, basic principles are reviewed to set the stage for discussion of specific CQC and CQA procedures.

3.2.1 Properties of the Liner Material

The construction specifications should restrict the materials that can be used in constructing a CCL. Some of the restrictions are more important than others, and it is helpful for CQC and CQA personnel to understand how material properties can influence the performance of a soil liner.

3.2.1.1 Plasticity Characteristics

The plasticity of a soil refers to the capability of a material to behave as a plastic, moldable material. Soils are classified as either plastic or nonplastic. Soils that contain clay are usually plastic, whereas those that do not contain clay are usually nonplastic. If the soil is nonplastic, the soil is almost always considered unsuitable for a soil liner unless additives such as bentonite are introduced.

The plasticity characteristics of a soil are quantified by three parameters: liquid limit, plastic limit, and plasticity index. These terms are defined as follows:

- Liquid limit (LL): The water content corresponding to the arbitrary limit between the liquid and plastic states of consistency of a soil.

- Plastic limit (PL): The water content corresponding to the arbitrary limit between the plastic and solid states of consistency of a soil.
- Plasticity index (PI): The numerical difference between liquid and plastic limits (i.e., LL − PL).

The liquid limit and plastic limit are measured using ASTM D4318.

If the soil has a very low plasticity, the soil possesses insufficient clay to develop low hydraulic conductivity when the soil is compacted. Also, soils that have very low PIs tend to grade into nonplastic soils in some locations. The question of how low the PI can be before the soil is not sufficiently plastic is impossible to answer universally. Daniel (1990) recommends that the soil have a PI ≥ 10% but notes that some soils with PIs as low as 7% have been used successfully to build soil liners with extremely low in situ hydraulic conductivity (Albrecht and Cartwright 1989). Benson et al. (1999) and Bonaparte et al. (2002) compiled a database from CQA documents and related the hydraulic conductivity measured in the field on large-scale samples of field-compacted soil to various soil characteristics. The observed relationship between hydraulic conductivity and plasticity index is shown in Figure 3-5. The database reflects a broad range of construction conditions, soil

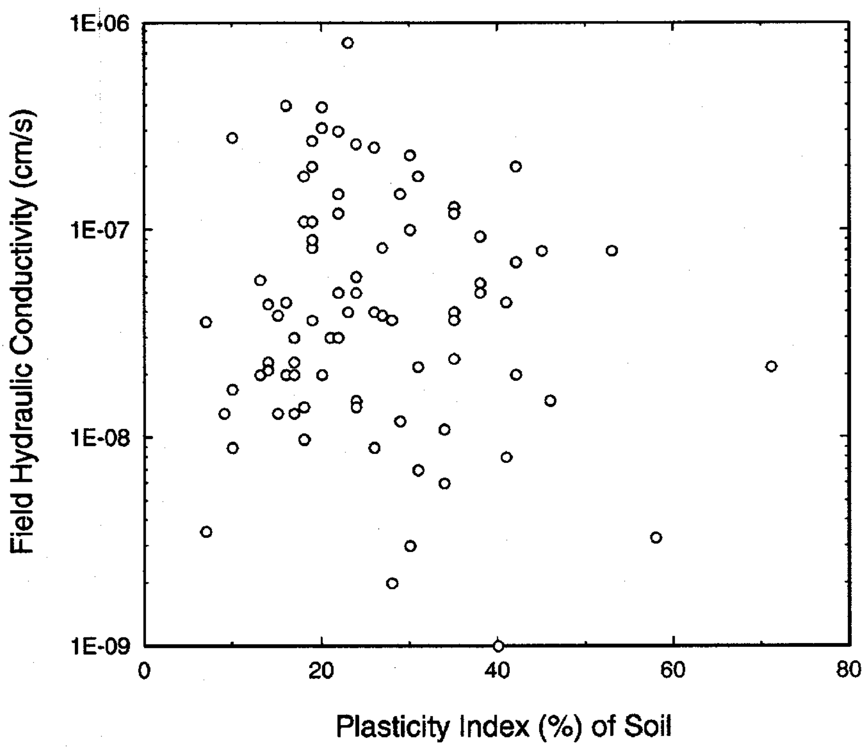

Figure 3-5. Relationship between Hydraulic Conductivity and Plasticity Index.

Source: Bonaparte et al. 2002.

materials, and CQA procedures. It is clear from the database that soils with PIs as low as approximately 10% can be compacted to achieve a hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s.

Soils with high plasticity index (>30%) tend to form hard clods when dried and sticky clods when wet. Highly plastic soils also tend to shrink and swell when wetted or dried. With highly plastic soils, CQC and CQA personnel should be particularly watchful for proper processing of clods, effective remolding of clods during compaction, and protection from desiccation.

3.2.1.2 Percentage of Fines

Some earthwork specifications place a minimum requirement on the percentage of fines in the soil liner material. *Fines* are defined as the fraction of soil on a dry-weight basis that pass through the openings of the No. 200 sieve (opening size = 0.075 mm). The test method is ASTM D1140. Soils with inadequate fines typically have too little silt- and clay-sized material to produce suitably low hydraulic conductivity. Daniel (1990) recommends that the soil liner materials contain at least 30% fines. Many state regulatory agencies require at least 50% fines. Data from Bonaparte et al. (2002) show that no relationship exists between percentage fines and large-scale field hydraulic conductivity (Figure 3-6). Nevertheless, field per-

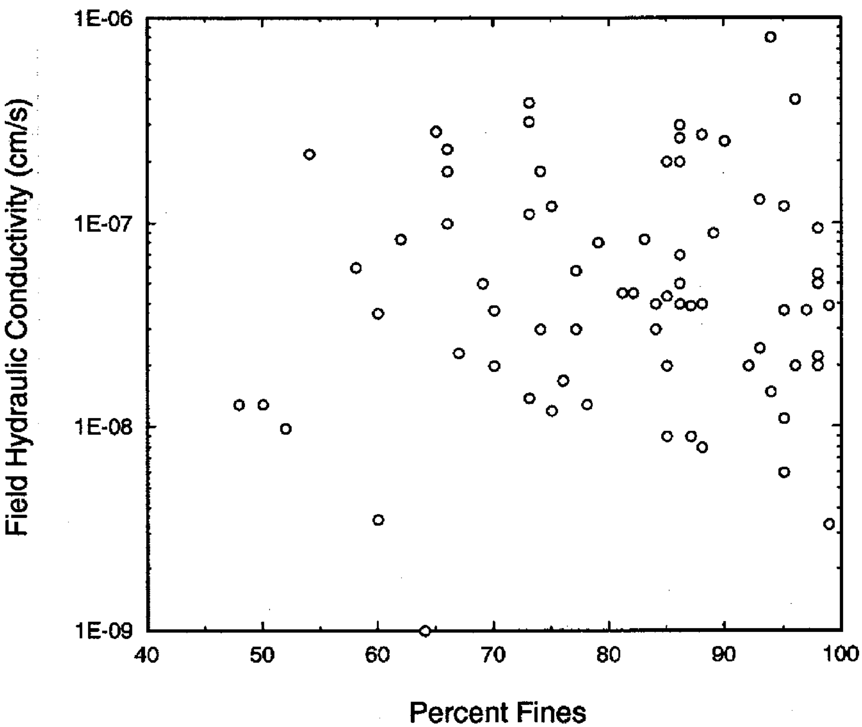

Figure 3-6. Relationship between Hydraulic Conductivity and Percent Fines.

Source: Bonaparte et al. 2002.

sonnel should check the soil to make sure that the percentage of fines meets or exceeds the minimum stated in the construction specifications and should be particularly watchful for soils with less than 30% to 50% fines.

3.2.1.3 Percentage of Gravel

Gravel is herein defined as particles that will not pass through the openings of a No. 4 sieve (opening size = 4.76 mm). Deposits of clean gravel have a high hydraulic conductivity (often greater than 1 cm/s). However, gravel can be uniformly mixed with a clayey soil liner material up to a gravel content of 50% to 60% (dry-weight basis) without significantly increasing the hydraulic conductivity of the material (Figure 3-7). The hydraulic conductivity of mixtures of gravel and clayey soil is low because the clayey soil fills the voids between the gravel particles. Gravel can be beneficial when present in a clayey soil because the gravel strengthens the material and minimizes the tendency of wet clay to "pump" when compacted (Day 1996). The critical problem for CQA personnel to watch for is possible segregation of gravel into pockets that do not contain sufficient soil to plug the voids between the gravel particles. If there are insufficient fines, not only will the hydraulic conductivity be too high, but the open pores between gravel particles may provide an avenue for migration and erosion of soil particles (a phenomenon called "piping").

It is important for field CQC and CQA personnel to understand that (1) more than 50% by weight of gravel is unacceptable, (2) the uniformity with which gravel is mixed with clayey soil is important, and (3) a little bit of gravel is actually beneficial in terms of maximizing strength and bearing capacity of the soil liner. Gravel also may possess the capability of puncturing geosynthetic materials; the maximum

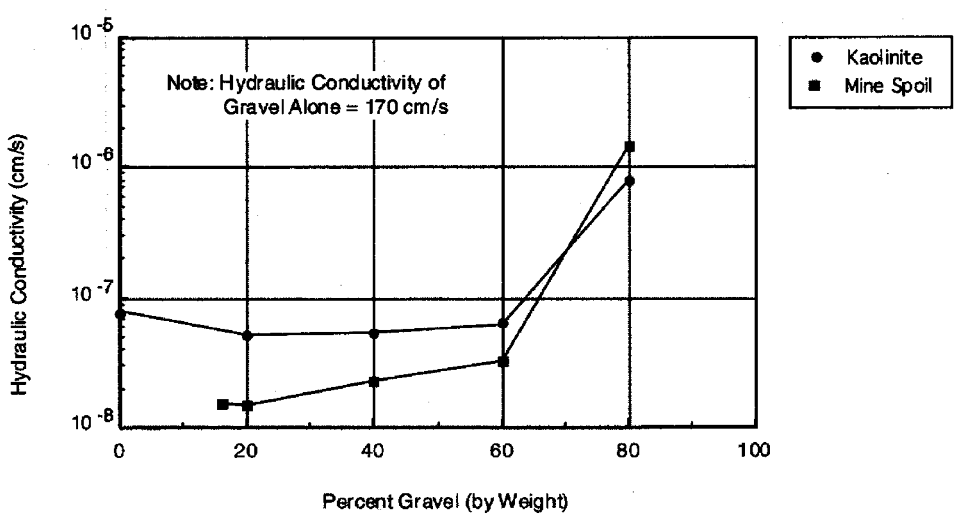

Figure 3-7. Relationship between Hydraulic Conductivity and Percentage Gravel Added to Two Clayey Soils.

Source: After Shelley and Daniel 1993, ASCE.

size and the angularity of the gravel are very important for the layer of soil that will serve as a foundation layer for a geomembrane.

3.2.1.4 Maximum Particle Size

The maximum particle size is important because (1) cobbles or large stones can interfere with compaction and (2) if a geomembrane is placed on top of the compacted soil liner, oversized particles can damage the geomembrane. Construction specifications should restrict the maximum allowable particle size, which is usually between 25 and 50 mm (1–2 in.) for compaction considerations but which may be much less (e.g., ≤9–12 mm, or 3/8–1/2 in.) for protection against puncture of an adjacent geomembrane. If a geomembrane is to be placed on the soil liner, only the upper lift of the soil liner is relevant in terms of protection against puncture. Construction specifications may place more stringent requirements on the upper lift to protect the geomembrane from puncture. Sieve analyses on small samples will not usually lead to detection of an occasional piece of oversized material. Observations by attentive CQC and CQA personnel are the most effective way to ensure that oversized materials have been removed.

3.2.1.5 Clay Content and Activity

The clay content of the soil may be defined in several ways, but it is usually considered the percentage of soil that has an equivalent particle diameter, smaller than 0.005 or 0.002 mm; 0.002 mm is the more common definition. The clay content is measured by sedimentation analysis using a hydrometer (ASTM D422), although other techniques such as the buoyancy method (Bardet and Young 1997) are sometimes used. Some construction specifications require a minimum clay content, but many do not.

Because hydrometer analysis uses Stoke's law, which applies to sedimentation of spherical particles, and because clay particles are not spherical, the actual maximum particle size may be substantially larger than suggested by hydrometer analysis (Lu et al. 2000). Hydrometer analyses applied to clays should be viewed as semiempirical; the theory upon which the methodology is based is sound, but plate-shaped clay particles bear little resemblance to spheres. Furthermore, clays can aggregate despite attempts to disperse them, which can lead to underestimation of the clay content (Nettleship et al. 1997).

A parameter that is sometimes useful is the activity, A, of the soil, which is defined as the plasticity index (expressed as a percentage) divided by the percentage of clay (defined as percentage of soil, on a dry-weight basis, finer than 0.002 mm) in the soil. A high activity (>1) indicates that expandable clay minerals such as montmorillonite are present. Lambe and Whitman (1969) report that the activities of kaolinite, illite, and montmorillonite (three common clay minerals) are 0.38, 0.9, and 7.2, respectively. Activities for naturally occurring clay liner materials, which contain a mix of minerals, are frequently between 0.5 and 1.

The relationship between clay content (defined as particles <0.002 mm) and large-scale field hydraulic conductivity from data compiled by Bonaparte et al. (2002) is shown in Figure 3-8. The data show no particular relationship between

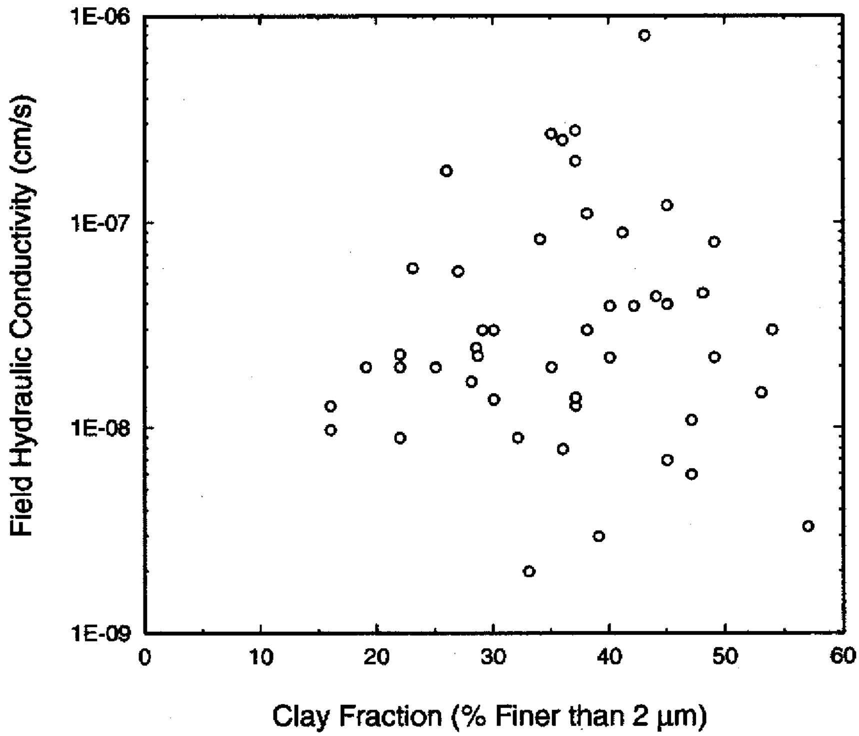

Figure 3-8. Relationship between Hydraulic Conductivity and Clay Content.

Source: Bonaparte et al. 2002.

large-scale field hydraulic conductivity and clay content. Clay content correlates closely with plasticity index (Figure 3-9). Soils with PI >10% will generally contain at least 10% to 20% clay.

Measurement of clay content (ASTM D422) is difficult, time-consuming (about two days), and subject to operator error during high-speed agitation of a soil–water mixture. Liquid limit, plastic limit, and percentage of fines are much easier to measure. It is recommended that construction specifications and regulations indirectly account for clay content by requiring the soil to have an adequate percentage of fines and a suitably large plasticity index. If the soil has an adequate amount of fines and adequate plasticity, the soil will have an adequate amount of clay.

3.2.1.6 Clod Size

The term *clod* refers to chunks of cohesive soil. The maximum size of clods may be specified in the construction specifications. Clod size is very important for dry, hard, clay-rich soils (Benson and Daniel 1990). These materials generally must be broken down into small clods to be properly hydrated, remolded, and compacted. Clod size is less important for wet soils. Soft, wet clods can usually be remolded into a homogeneous, low-hydraulic-conductivity mass with a reasonable compactive

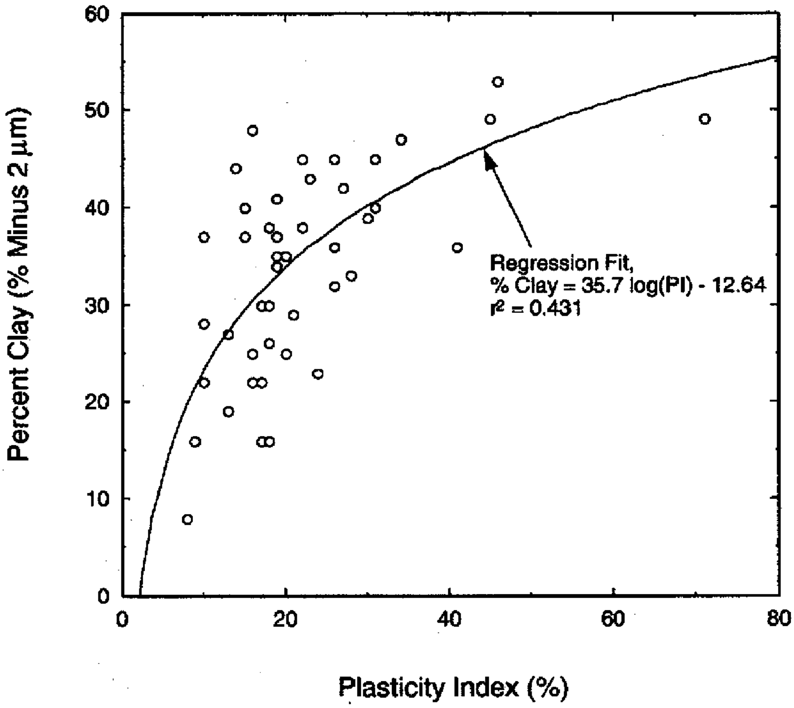

Figure 3-9. Relationship between Clay Content and Plasticity Index.

Source: data from Bonaparte et al. 2002.

effort; no limit on clod size is necessary if clods are sufficiently soft that they are easily remolded by the compaction equipment.

No standard method is available to determine clod size. Inspectors should observe the soil liner material and occasionally determine the dimensions of clods by direct measurement with a ruler to verify conformance with construction specifications.

3.2.1.7 Bentonite

Bentonite may be added to clay-deficient soils to fill the voids between the soil particles and to produce a material that, when compacted, has a low hydraulic conductivity. The effect of the addition of bentonite upon hydraulic conductivity is shown in Figure 3-10 for a silty sand. For this particular soil, addition of 4% sodium bentonite was sufficient to lower the hydraulic conductivity to less than 1×10^{-7} cm/s. The grain-size distribution of the soil to which bentonite is added can have a significant influence on the hydraulic conductivity of the mixture (Sivapullaiah et al. 2000). However, all other factors being equal, the soil with the lowest void ratio will have the lowest hydraulic conductivity. Thus, a well-graded soil with a variety of particle sizes will tend to have a relatively small void ratio and, hence, small hydraulic conductivity. Generally, the more uniform the soil's particle size, the larger the amount of bentonite that must be added. Well-graded soils

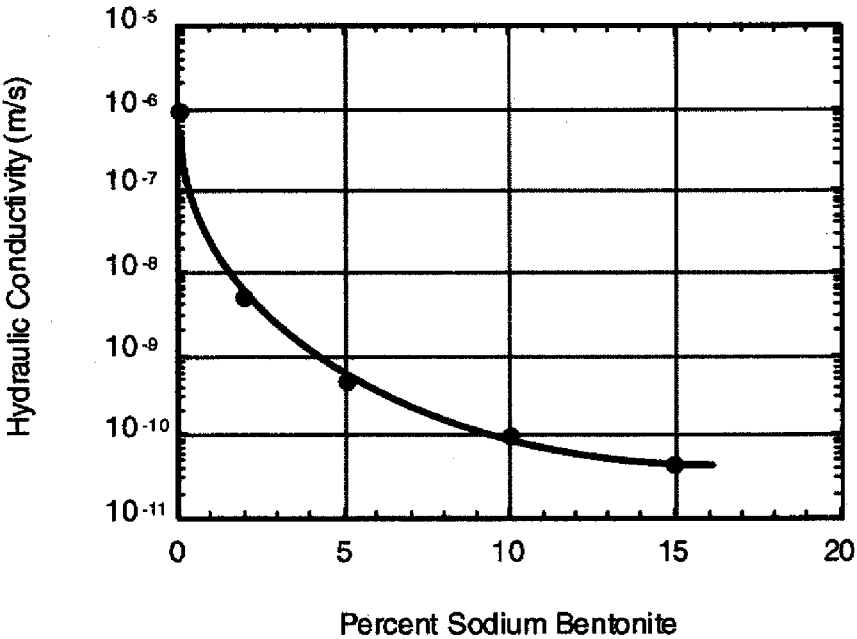

Figure 3-10. Effect of Addition of Bentonite to Hydraulic Conductivity of Compacted Silty Sand.

with a wide range of particle sizes may only require a small amount of bentonite to meet the design requirements for hydraulic conductivity.

Bentonite is a highly expansive mineral that can undergo increases in hydraulic conductivity as a result of interactions with some leachates, but this issue should be addressed in design and is usually not a matter of concern for CQA personnel other than to verify that the properties of the bentonite conform to specifications.

The critical CQC and CQA parameters are the type of bentonite, the grade of bentonite, the grain-size distribution of the processed bentonite, the amount of bentonite added to the soil, and the uniformity of mixing of the bentonite with the soil. Two types of bentonite are primary commercial materials: sodium bentonite and calcium bentonite. Sodium bentonite has much greater water absorbency and swelling potential, but calcium bentonite may be more stable when exposed to certain chemicals. Sodium bentonite is used far more frequently than calcium bentonite as a soil amendment for lining applications.

Any given type of bentonite may be available in several grades. The grade is a function of impurities in the bentonite, processing procedures, and additives. Calcium bentonites can be treated with sodium to modify the bentonite to a sodium form. Some companies add polymers or other compounds to the bentonite to make the bentonite more absorbent of water or more resistant to alteration by certain chemicals. Careful attention should be paid to the specification requirements for bentonite because they sometimes specify a natural sodium bentonite (rather than a calcium bentonite that has been treated and converted to a sodium bentonite).

Another variable is the gradation of the bentonite. A factor often overlooked by CQC and CQA personnel is the grain-size distribution of the processed bentonite. Bentonite can be sorted or ground during processing to different degrees. A fine, powdered bentonite added to a soil may produce a different hydraulic conductivity than the same amount of coarse, granular bentonite; if the bentonite was supposed to be finely ground but too coarse a gradation was delivered, the bentonite may be unsuitable in the mixture amounts specified. The compaction characteristics of soil–bentonite mixtures are affected by gradation of bentonite, whether water is added before or after the bentonite is added, and the hydration period (Howell et al. 1997). In general, the laboratory materials and procedures should match the field as closely as possible. Because bentonite is available in variable degrees of pulverization, sieve analysis (ASTM D422) of the processed dry bentonite is recommended to determine the grain-size distribution of the material.

The most difficult parameters to control are sometimes the amount of bentonite added to the soil and the thoroughness of mixing. Field CQC and CQA personnel should observe operational practices carefully.

3.2.2 Molding Water Content

For natural soils, the water content of the clay liner material at the time of compaction is perhaps the single most important variable that controls the engineering properties of the compacted material. The typical relationship between hydraulic conductivity and molding water content is shown in Figure 3-11. Soils compacted at water contents less than optimum (*dry of optimum*) tend to have a relatively high hydraulic conductivity; soils compacted at water contents greater than optimum (*wet of optimum*) tend to have a low hydraulic conductivity and low strength. For some soils, the water content relative to the plastic limit (which is the water content of the soil when the soil is at the boundary between being a solid and plastic material) may indicate the degree to which the soil can be compacted to yield low hydraulic conductivity. In general, if the water content is greater than the plastic limit, the soil is in a plastic state and should be capable of being remolded into a low-hydraulic-conductivity material. Soils with water contents dry of the plastic limit will exhibit little plasticity and may be difficult to compact into a low-hydraulic-conductivity mass without delivering enormous compactive energy to the soil. With soil–bentonite mixes, molding water content is usually not as critical as it is for natural soils, but compaction wet of optimum tends to produce the lowest hydraulic conductivity.

The water content of highly plastic soils is particularly critical. A photograph of a highly plastic soil (PI = 41%) compacted 1% dry of the optimum water content of 17% is shown in Figure 3-12. Large interclod voids are visible; the clods of clay were too dry and hard to be effectively remolded with the compactive effort used. A photograph of a compacted specimen of the same soil moistened to 3% wet of optimum and then compacted is shown in Figure 3-13. At this water content, the soft soil can be remolded into a homogenous, low-hydraulic-conductivity mass.

It is usually preferable to compact the soil wet of optimum to minimize hydraulic conductivity. However, the soil must not be placed at too high a water con-

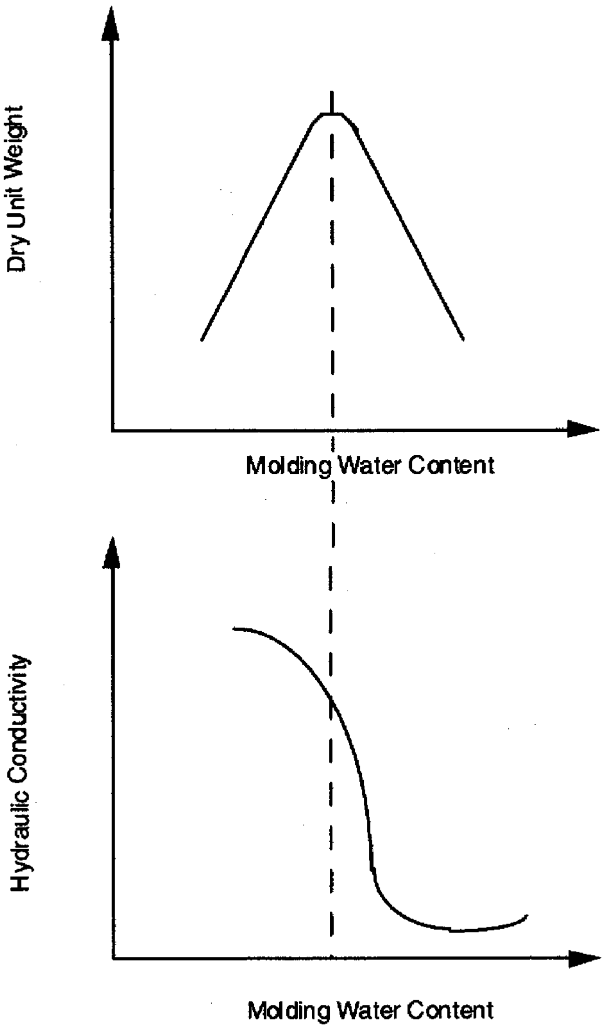

Figure 3-11. Effect of Molding Water Content on Hydraulic Conductivity.

tent. Otherwise, the shear strength may be too low, there may be great risk of desiccation cracks forming if the soil dries, and ruts may form when construction vehicles pass over the liner. It is critical that CQC and CQA personnel verify that the water content of the soil is within the range specified in the construction documents.

3.2.3 Type of Compaction

In the laboratory, soil can be compacted in four ways:

1. Impact compaction: A ram is repeatedly raised and dropped to compact a lift soil into a mold (Figure 3-14(a)) (e.g., standard and modified Proctor).

Figure 3-12. Photograph of Highly Plastic Clay Compacted with Standard Proctor Effort at a Water Content of 16% (1% Dry of Optimum).

Figure 3-13. Photograph of Highly Plastic Clay Compacted with Standard Proctor Effort at a Water Content of 20% (3% Wet of Optimum).

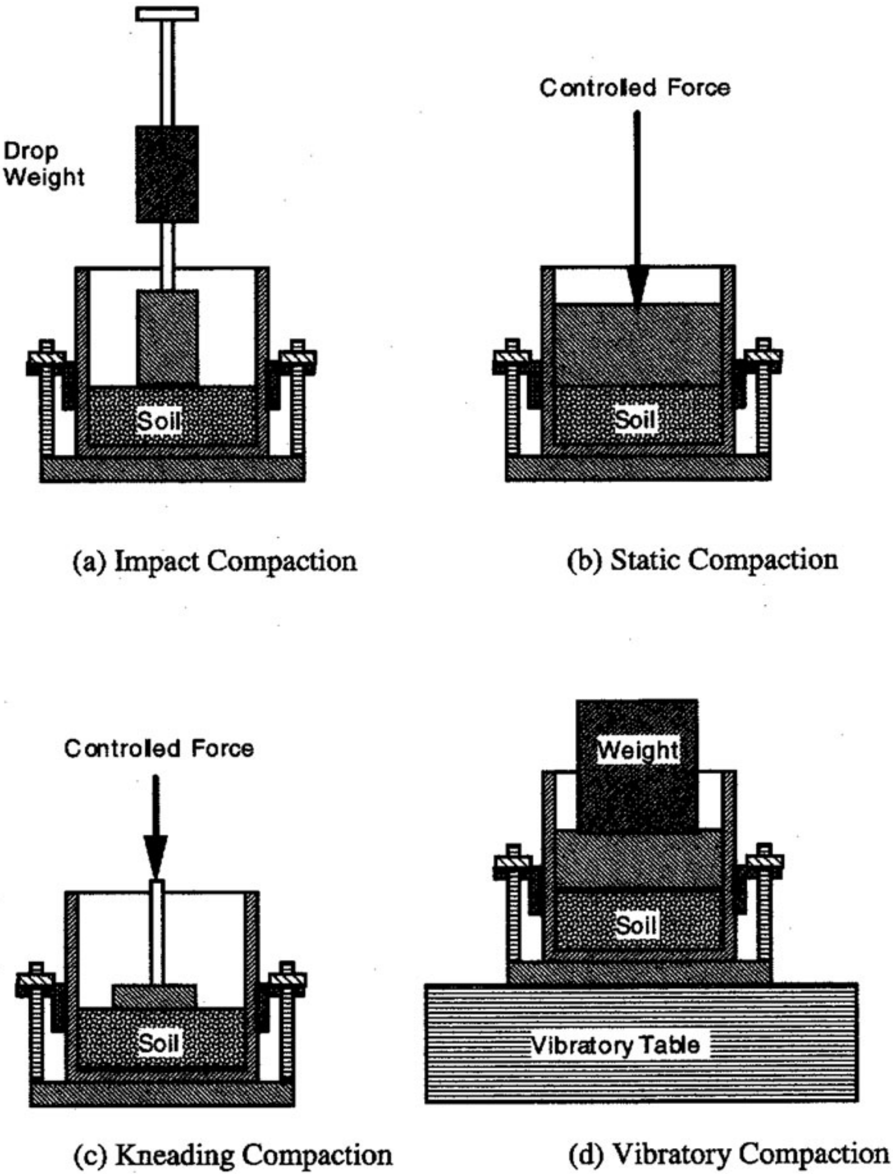

Figure 3-14. Four Types of Laboratory Compaction Tests: (a) Impact Compaction, (b) Static Compaction, (c) Kneading Compaction, and (d) Vibratory Compaction.

2. Static compaction: A piston compacts a lift of soil with a constant stress (Figure 3-14(b)).
3. Kneading compaction: A "foot" kneads the soil (Figure 3-14(c)).
4. Vibratory compaction: The soil is vibrated to densify the material (Figure 3-14(d)).

Experience from the laboratory has shown that the type of compaction can affect hydraulic conductivity (as shown in Figure 3-15). Kneading breaks down clods and remolds the soil into a homogenous mass that is free of voids or large pores. Kneading of the soil is particularly beneficial for highly plastic soils. For bentonite–soil blends that do not form clods, kneading is usually not necessary.

Most soil liners are constructed with "footed" rollers. The "feet" on the roller penetrate into a loose lift of soil and knead the soil with repeated passages of the roller. The dimensions of the feet on rollers vary considerably. Footed rollers with short feet (~100 mm or 4 in.) are called "pad foot" rollers; the feet are said to be "partly penetrating" because the foot is too short to penetrate fully a typical loose lift of soil. Footed rollers with long feet (~200 mm or 8 in.) are often called "sheepsfoot" rollers; the feet fully penetrate a typical loose lift. Figure 3-16 contrasts rollers with partly and fully penetrating feet. Kouassi et al. (2000) discuss laboratory compaction methods designed to simulate modern footed rollers.

Some construction specifications place limitations on the type of roller that can be used to compact a soil liner. Personnel performing CQC and CQA should be watchful of the type of roller to make sure that it conforms to construction specifications. It is particularly important to use a roller with fully penetrating feet if such a roller is required; use of a nonfooted roller or pad foot roller would result in less kneading of the soil.

3.2.4 Energy of Compaction

The energy used to compact soil can have an important influence on hydraulic conductivity. Figure 3-17 shows that increasing the compactive effort produces

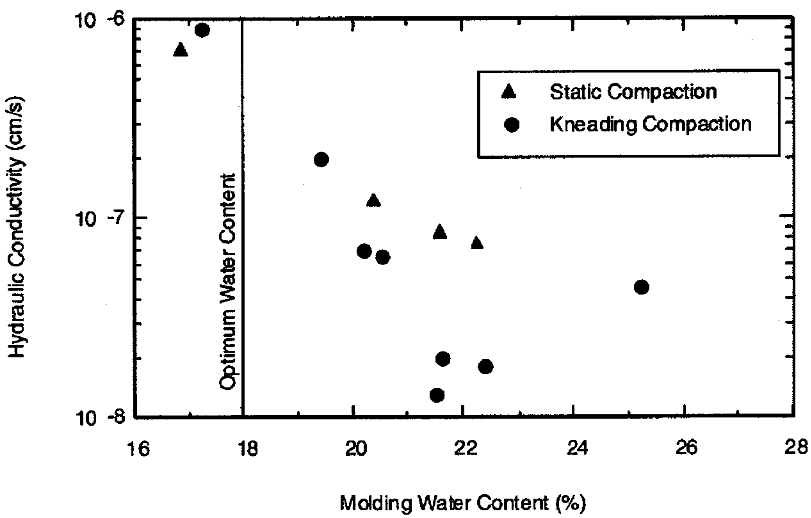

Figure 3-15. Effect of Type of Compaction on Hydraulic Conductivity.

Source: Mitchell et al. 1965, ASCE.

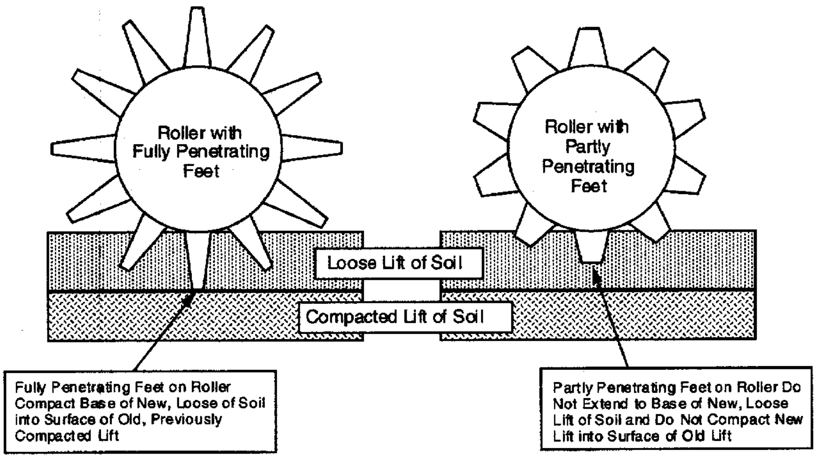

Figure 3-16. Footed Rollers with Partly and Fully Penetrating Feet.

soil that has a greater dry unit weight and lower hydraulic conductivity. The soil must be compacted with adequate energy to achieve low hydraulic conductivity.

In the field, compactive energy is controlled by three things:

1. The weight of the roller and the way the weight is distributed (greater weight produces more compactive energy).
2. The thickness of a loose lift (thicker lifts produce less compactive energy per unit volume of soil).
3. The number of passes of the compactor (more passes produce more compactive energy).

Many engineers and technicians assume that percent compaction is a good measure of compactive energy. For soils near optimum water content or dry of optimum, percent compaction is indeed a good indicator of compactive energy; if the percent compaction is low, then the compactive energy was almost certainly low. However, for soil compacted wet of optimum, percent compaction is not a particularly good indicator of compactive energy. This is illustrated by the curves in Figure 3-18. It is assumed that the same soil is compacted with compactive Energy A and Energy B (Energy B > Energy A) to develop the compaction and hydraulic conductivity curves shown in Figure 3-18. Next, two specimens are compacted to the same water content ($w_A = w_B$). The dry unit weights are practically identical ($\gamma_{d,A} \sim \gamma_{d,B}$) despite the fact that the energies of compaction were different. The hydraulic conductivity (k) of the specimen compacted with the larger energy (Energy B) has a lower hydraulic conductivity than the specimen compacted with Energy A, despite the fact that $\gamma_{d,A} \sim \gamma_{d,B}$. The percent compaction for the two compacted specimens is computed as follows:

$$P_A = \gamma_{d,A}/[\gamma_{d,max}]_A \times 100\% \tag{3-6}$$

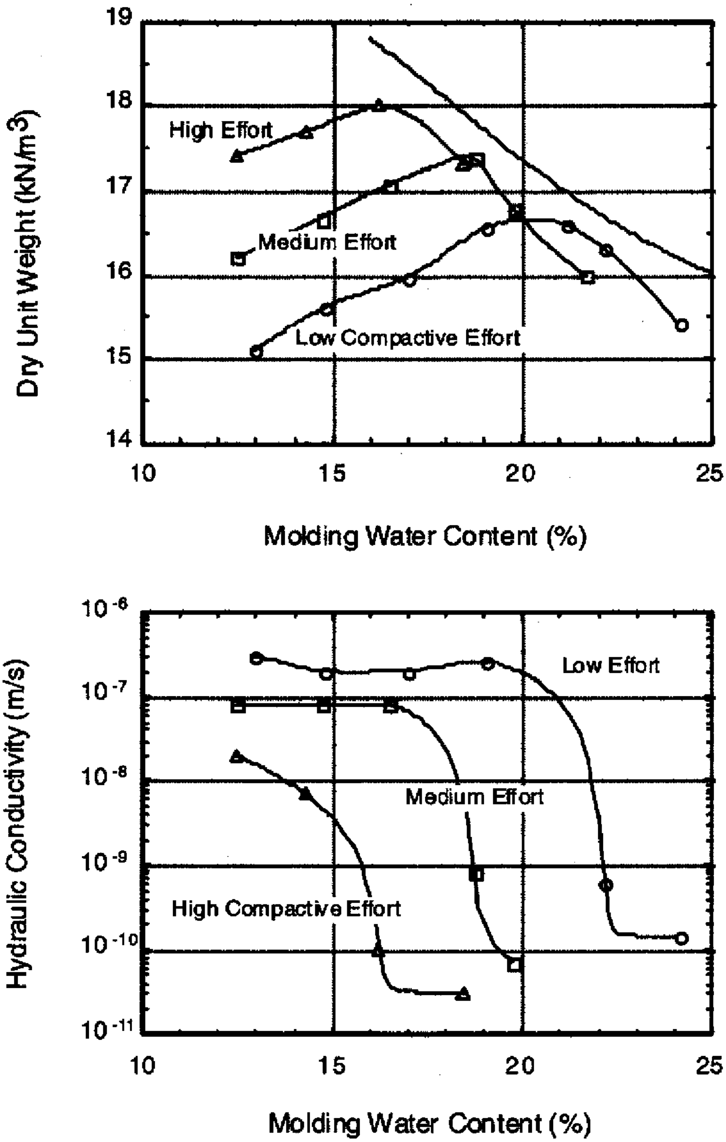

Figure 3-17. Effect of Compactive Energy on Hydraulic Conductivity.

Source: Mitchell et al. 1965, ASCE.

$$P_B = \gamma_{d,B}/[\gamma_{d,max}]_B \times 100\% \quad (3\text{-}7)$$

Because $\gamma_{d,A} = \gamma_{d,B}$ but $[\gamma_{d,max}]_B > [\gamma_{d,max}]_A$, then $P_A > P_B$. Thus, based on percent compaction, because $P_A > P_B$, one might assume that Soil A was compacted with greater compactive energy than Soil B. In fact, just the opposite is true. CQC and CQA personnel are strongly encouraged to monitor equipment weight, lift thickness, and number of passes (in addition to dry unit weight) to ensure that appro-

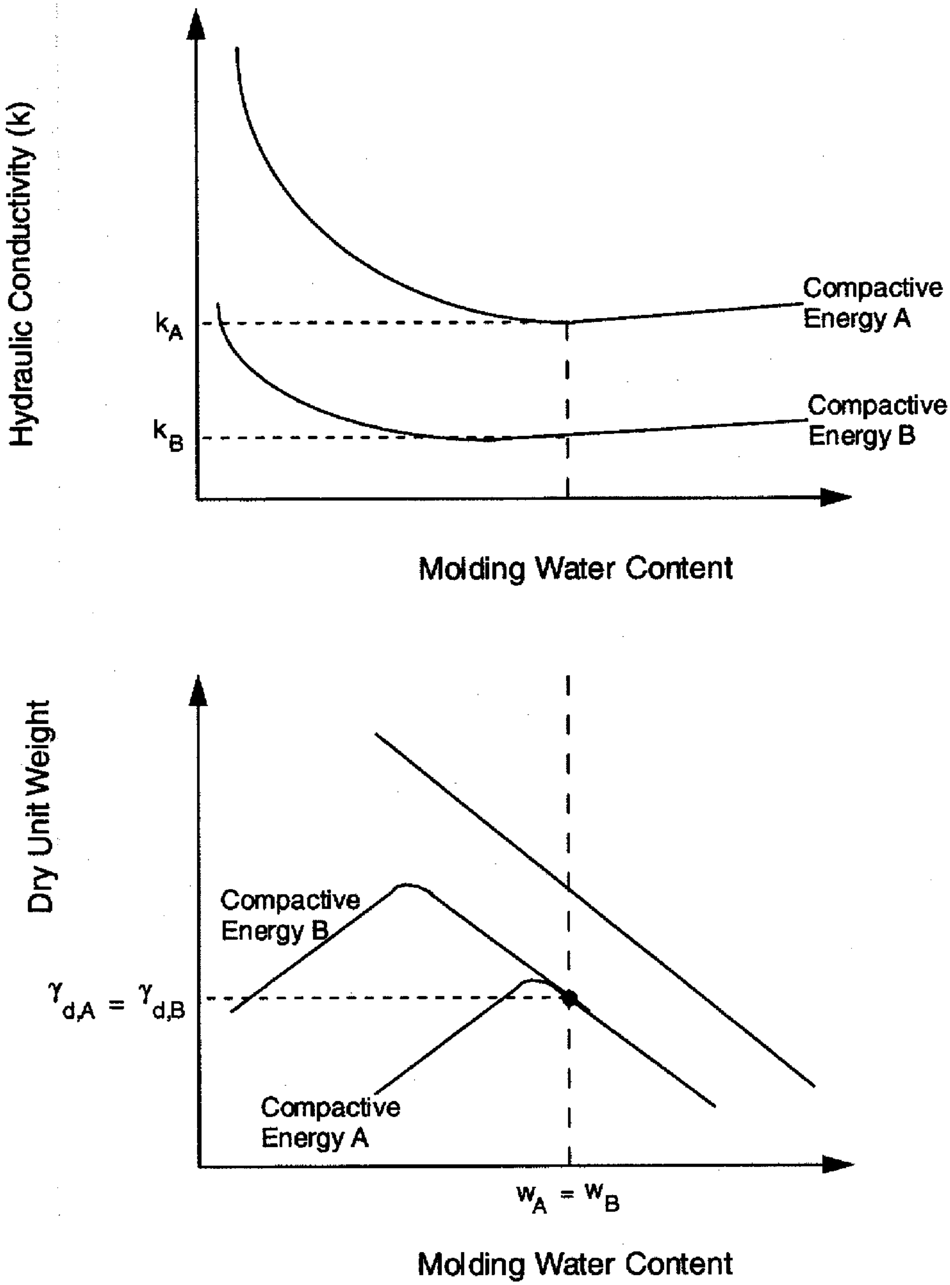

Figure 3-18. Illustration of Why Dry Unit Weight Is a Poor Indicator of Hydraulic Conductivity for Soil Compacted Wet of Optimum.

priate compactive energy is delivered to the soil. Some CQC and CQA inspectors have failed to realize that footed rollers towed by a dozer must be filled with liquid to have the intended large weight. Rollers that are filled with liquid should be regularly checked.

Experience has shown that effective CQC and CQA for soil liners can be accomplished using the "line of optimums" as a reference. The *line of optimums* is the locus of (w_{opt}, $\gamma_{d,max}$) points for compaction curves developed on the same soil with different compactive energies (Figure 3-19). The greater the percentage of actual (w, γ_d) points that lie above the line of optimums, the better the overall quality of construction (Benson and Boutwell 1992; Benson et al. 1999; Bonaparte et al. 2002). The percentage of moisture–density points that lie on or above (i.e., wet of) the line of optimums is defined as P_o, as noted in Figure 3-20. Inspectors are encouraged to monitor the percentage of field-measured (w,γ_d) points that lie on or above the line of optimums. Experience shows that if the percentage is high, the liner is likely to have been well compacted (Bonaparte et al. 2002; Figure 3-21). If the percentage is less than about 80%, CQC and CQA personnel should carefully consider whether adequate compactive energy is being delivered to the soil.

3.2.5 Bonding of Lifts

If lifts of soil are poorly bonded, a zone of high hydraulic conductivity will develop at interfaces between lifts. Poorly bonded lift interfaces provide hydraulic connection between more permeable zones in adjacent lifts (Figure 3-22). It is important to bond lifts together to the greatest extent possible.

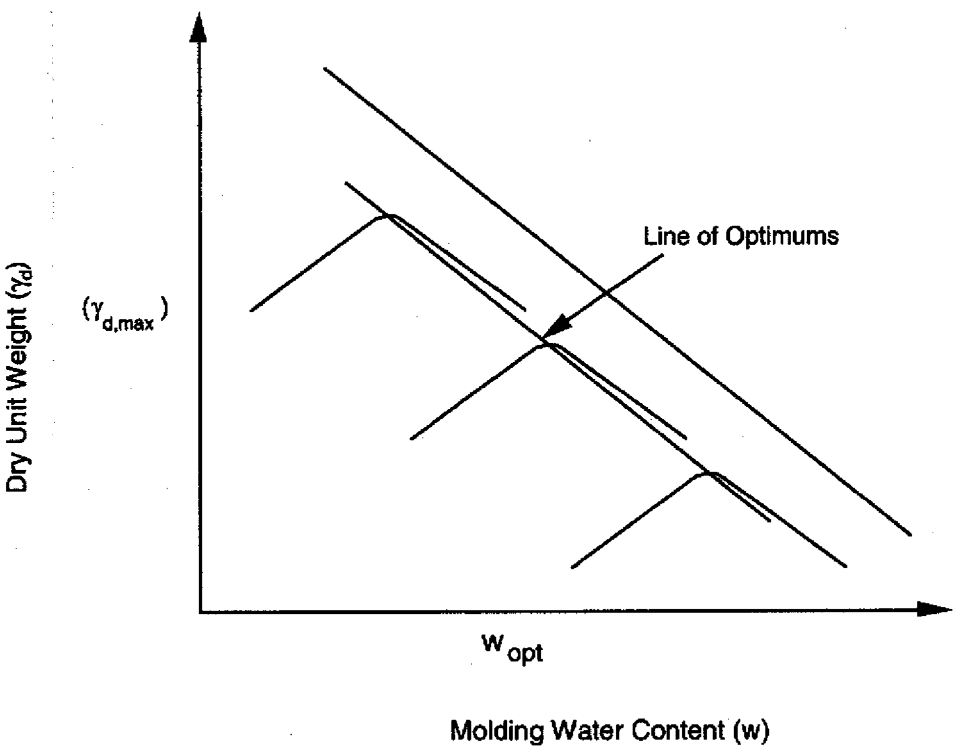

Figure 3-19. Line of Optimums.

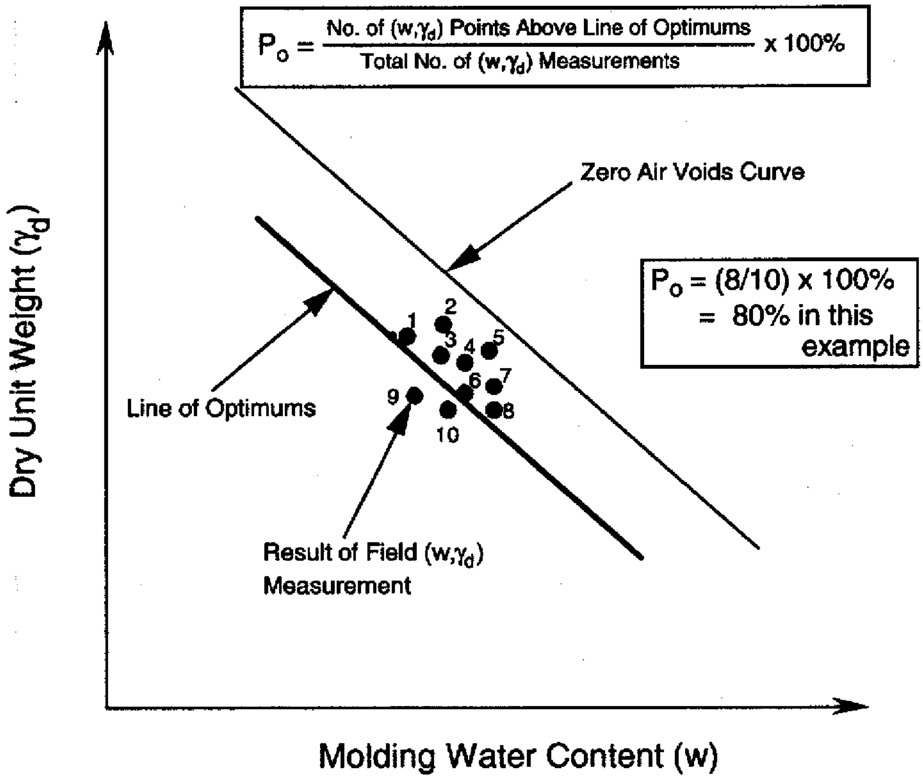

Figure 3-20. Definition of Percentage of Data Points on or above Line of Optimums (P_o), with an Example Given of 10 Data Points and P_o = 80%.

Bonding of lifts is enhanced by two methods:

1. The surface of a previously compacted lift must be rough before placing the new lift of soil (the previously compacted lift is often scarified with a disk before placement of a new lift), which promotes bonding and increased hydraulic tortuosity along the lift interface.
2. A fully penetrating footed roller should be used because the feet pack the base of the new lift into the surface of the previously compacted lift.

Particular attention should be given to requirements for scarification and the length of feet on rollers.

3.2.6 Protection against Desiccation and Freezing

Clay soils shrink when they are dried and, depending on the amount of shrinkage, may crack. Cracks that extend deeper than one lift can be disastrous. The vulnerability to shrinkage cracking is primarily a function of water content (the wetter the soil, the greater the potential for shrinkage cracking), but soil characteristics

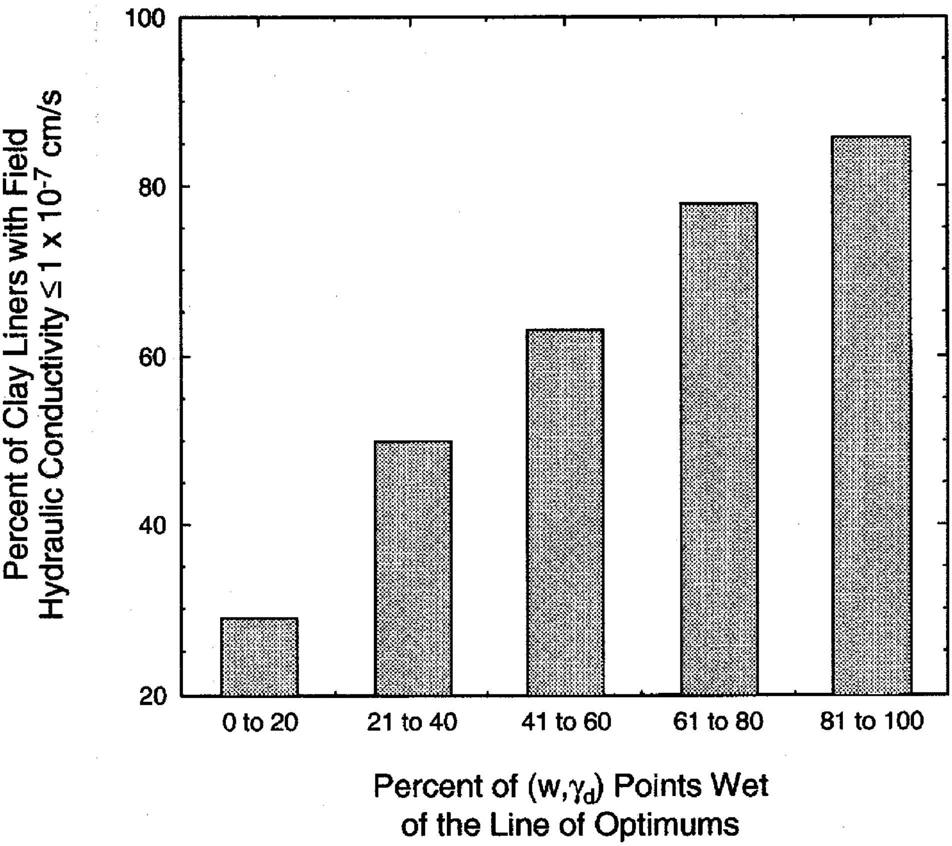

Figure 3-21. Correlation of Large-Scale Field Hydraulic Conductivity with Percentage of Water Content–Dry Density Points Lying on or above the Line of Optimums (P_o).

Note: Correlation shows a high rate of success when most of the moisture–density points lie on or above the line of optimums.

Source: Bonaparte et al. 2002.

and compaction conditions influence shrinkage vulnerability, as well (Kleppe and Olson 1985; Daniel and Wu 1993; and Albrecht and Benson 2001). Although hydration of the clay may cause swelling and partial self-sealing, field studies indicate that the cracks do not fully heal upon hydration (McBrayer et al. 1997). The application of significant overburden stress (>100 kPa) helps to close desiccation cracks during the hydration process and promotes greater self-healing (Boynton and Daniel 1985). CQA personnel should be careful to make sure that no significant desiccation occurs during or after construction. Water content should be measured if there are doubts.

Freezing of a soil liner may cause the hydraulic conductivity to increase (Kim and Daniel 1992; Benson and Othman 1993; Othman et al. 1994; and Benson et al. 1995), although soil–bentonite liners appear to be less vulnerable to damage

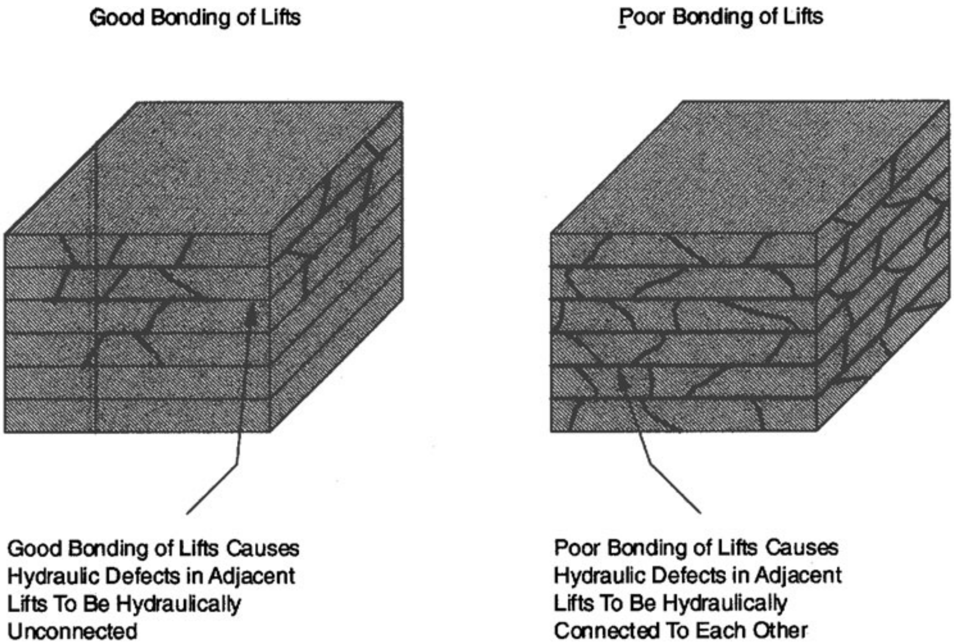

Figure 3-22. Flow Pathways Created by Poorly Bonded Lifts.

(Kraus et al. 1997a). Damage caused by superficial freezing to a shallow depth is easily repaired by rerolling the surface. Deeper freezing is not so easily repaired and requires detailed investigation. (This problem is discussed in Section 3.10.2.3.) CQC and CQA personnel should be watchful during periods when freezing temperatures are possible.

3.3 Field Measurement of Water Content and Dry Unit Weight

3.3.1 Water Content Measurement

3.3.1.1 Overnight Oven-Drying (ASTM D2216)

The standard method for determining the water content of a soil is to oven-dry the soil overnight in a forced-convection oven at 110 °C. This is the fundamental and most accurate method for determining the water content of a soil. All other methods of measurement are referenced to the value of water content determined with this method.

If water content test results are not needed quickly, ASTM D2216 is the primary method of water content measurement used for CQC and CQA. However, field personnel cannot wait overnight to make decisions about continuation with the construction process. Water content values are typically needed in a matter of a few minutes to keep the construction process moving ahead at a reasonable pace.

3.3.1.2 Microwave Oven-Drying (ASTM D4643)

Soil samples can be dried in a microwave oven to obtain water contents much more quickly than can be obtained with conventional overnight oven-drying. The main problem with microwave oven-drying is that if the soil dries for too long in the microwave oven, the temperature of the soil will rise significantly above 110 °C. If the soil is heated to a temperature greater than 110 °C, the water content will be greater than the value determined by drying at 110 °C. Overheating the soil drives water out of the crystal structure of some minerals and thereby leads to too much loss of water upon drying.

To guard against overdrying the soil, ASTM D4643 requires that the soil be dried for three minutes and then weighed. The soil is then dried for an additional minute and reweighed. The process of repeated drying for one minute and weighing the soil as necessary prevents overheating of the soil and forces the operator to cease the drying process once the weight of the soil has stabilized.

Under ideal conditions, microwave oven-drying can yield water contents that are almost indistinguishable from values measured with conventional overnight oven-drying. Problems can occur if the soil contains significant metal or explodes from expansion of gas in the interior of the sample. Because errors can occasionally arise with microwave oven-drying, the water content determined with microwave oven-drying should be periodically checked with the value determined by conventional overnight oven-drying (ASTM D2216).

3.3.1.3 Direct Heating (ASTM D4959)

Direct heating of the soil was common practice until about two decades ago. To dry a soil with direct heating, a mass of soil is placed in a metallic container (such as a cooking utensil) and then heated over a flame (e.g., a portable cooking unit) until the soil first appears dry. The mass of the soil plus container is then measured. Next, the soil is heated further and then reweighed. This process is repeated until the mass ceases to decrease significantly (i.e., to change by $<0.1\%$).

The main problem with direct heating is that if the soil is overheated during drying, the water content that is measured will be too large. Although ASTM D4959 does not eliminate this problem, the ASTM method does warn the user not to overheat the soil. Because errors can arise with direct heating, the water content determined with direct heating should be regularly checked with the value determined by conventional overnight oven-drying (ASTM D2216).

3.3.1.4 Calcium Carbide Gas Pressure Tester (ASTM D4944)

A known mass of moist soil is placed in a testing device, and calcium carbide is introduced. Mixing is accomplished by shaking and agitating the soil with the aid of steel balls and a shaking apparatus. A measurement is made of the gas pressure produced. Water content is determined from a calibration curve. Because errors can occasionally arise with gas pressure testing, the water content determined with gas pressure testing should be periodically checked with the value determined by conventional overnight oven-drying (ASTM D2216).

3.3.1.5 Nuclear Method (ASTM D3017)

The most widely used method of measuring the water content of compacted soil is the nuclear method. Measurement of water content with a nuclear device involves the moderation or thermalization of neutrons. The radioactive source of fast neutrons is embedded in the interior part of a nuclear water content–density device (Figure 3-23). As the fast neutrons move into the soil, they undergo a reduction in energy every time a hydrogen atom is encountered. A series of energy reductions takes place when a neutron sequentially encounters hydrogen atoms. Finally, after an average of 19 collisions with hydrogen atoms, a neutron ceases to lose further energy and is said to be a "thermal" neutron. A detector in the nuclear device senses the number of thermal neutrons that are encountered. The number of thermal neutrons that are encountered over a given period of time is a function of the number of fast neutrons that are emitted from the source and the density of hydrogen atoms in the soil located immediately below the nuclear device. Through appropriate calibration, and with the assumption that the only source of hydrogen in the soil is water, the nuclear device provides a measure of the water content of the soil over an average depth of about 200 mm (8 in.).

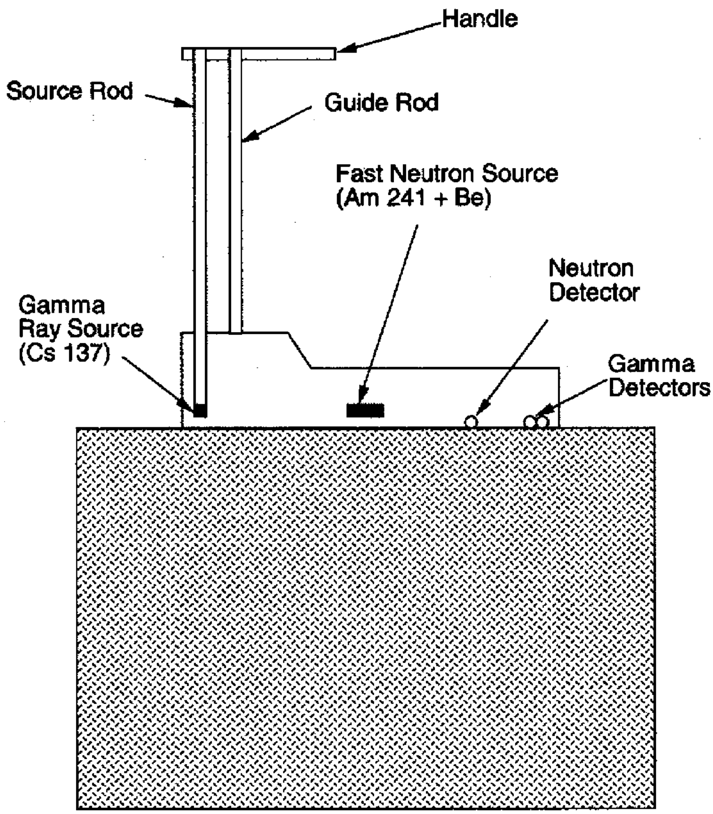

Figure 3-23. Schematic Diagram of Nuclear Water Content–Density Device.

There are a number of potential sources of error with the nuclear device. The most important potential source of error is extraneous hydrogen atoms not associated with water. Possible sources of hydrogen other than water include hydrocarbons, methane gas, hydrous minerals (e.g., gypsum), hydrogen-bearing minerals (e.g., kaolinite, illite, and montmorillonite), and organic matter in the soil. Under extremely unfavorable conditions, the nuclear device can yield water content measurements that are as much as 10 percentage points in error (almost always on the high side). Under favorable conditions, measurement error is less than 1%. The nuclear water content measurement is frequently too high by 1 to 3 percentage points. The nuclear device can be used if there is a bias in the water content, but CQC and CQA personnel must know what the bias is. The nuclear device should be calibrated for site-specific soils and changing conditions within a given site. Specific procedures are suggested in Section 3.9.3.3.

Another potential source of error is the presence of individuals, equipment, or trenches located within 1 m of the device; all of these things can cause errors. The device must be warmed up for an adequate period of time, or the readings may be incorrect. If the surface of the soil is improperly prepared and the device is not sealed properly against a smooth surface, erroneous measurements can result. If the standard count, which is a measure of the intensity of radiation from the source, has not been taken recently, an erroneous reading may result. Finally, many nuclear devices allow the user to input a moisture adjustment factor to correct the water content reading by a fixed amount. If the wrong moisture adjustment factor is stored in the device's computer, the reported water content will be in error.

The CQC and CQA personnel must be well versed in the proper use of nuclear water content measurement devices. There are many opportunities for error if personnel are not properly trained or do not use the equipment correctly. As indicated later, the nuclear device should be checked with other types of equipment to ensure that site-specific variables are not influencing test results. Nuclear equipment may be checked against other nuclear devices (particularly new devices or recently calibrated devices) to minimize potential for errors.

3.3.1.6 Time Domain Reflectometry (ASTM D6780)

Time domain reflectometry (TDR) is a relatively new technique that senses the dielectric properties of soil, which are related to moisture content. Because the method is relatively new and may not be suitable for highly plastic soils, its use for CCLs in the near term may be limited. However, if suitable comparative data can be provided for a particular soil to demonstrate the applicability of TDR, it provides a useful means for measurement of water content.

3.3.2 Unit Weight

3.3.2.1 Sand Cone (ASTM D1556)

The sand cone is a device for determining the volume of a hole that has been excavated into soil. The idea is to determine the weight of sand required to fill a hole of unknown volume. Through calibration, the volume of sand that fills the hole can be determined from the weight of sand needed to fill the hole. A schematic diagram of the sand cone is shown in Figure 3-24.

The sand cone is used as follows. First, a template is placed on the ground surface. A circle is scribed along the inside of the hole in the template. The template is removed, and soil is excavated from within the area marked by the scribed circle. The soil that is excavated is weighed to determine the total weight (W) of the soil excavated. A sample of the excavated soil is oven-dried (e.g., with a microwave oven) to determine the water content of the soil. The bottle in a sand cone device is filled with sand, and the bottle is weighed. The template is placed over the hole, and the sand cone device is placed on top of the template. A valve on the sand cone device is opened, which allows sand to rain down through the inverted funnel of the device and inside the excavated hole. When the hole and funnel are filled with sand, the valve is closed and the bottle containing sand is weighed. The difference in weight is calculated. Through calibration, the weight of sand needed to fill the funnel is subtracted, and the volume of the hole is computed from the weight of sand that filled the hole. The total unit weight is calculated by dividing the weight of soil excavated by the computed volume of the excavated hole. The dry unit weight is then calculated using Eq. 3-1.

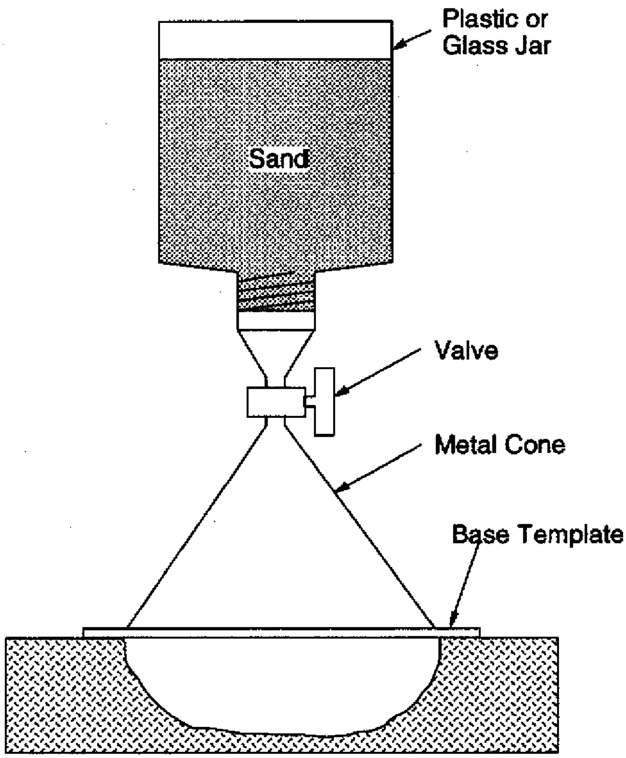

Figure 3-24. Sand Cone Device.

If used properly, the sand cone device provides a reliable technique for determining the dry unit weight of the soil. The primary sources of error are improper or infrequent calibration of the device, excavation of an uneven hole that has sharp edges or overhangs that can produce voids in the sand-filled hole, variations in the sand, contamination of the sand by soil particles if the sand is reused, and vibration from equipment operating close to the sand cone.

3.3.2.2 Rubber Balloon (ASTM D2167)

The rubber balloon is similar to the sand cone except that water is used to fill the excavated hole rather than sand. A rubber balloon device is sketched in Figure 3-25. As with the sand cone test, the rubber balloon test is performed with the device located on a template positioned at the desired location over the leveled soil. Then a hole is excavated into the soil and the density-measuring device is again placed on top of a template at the ground surface. Water inside the rubber balloon device is pressurized with air to force the water into the excavated hole. A thin mem-

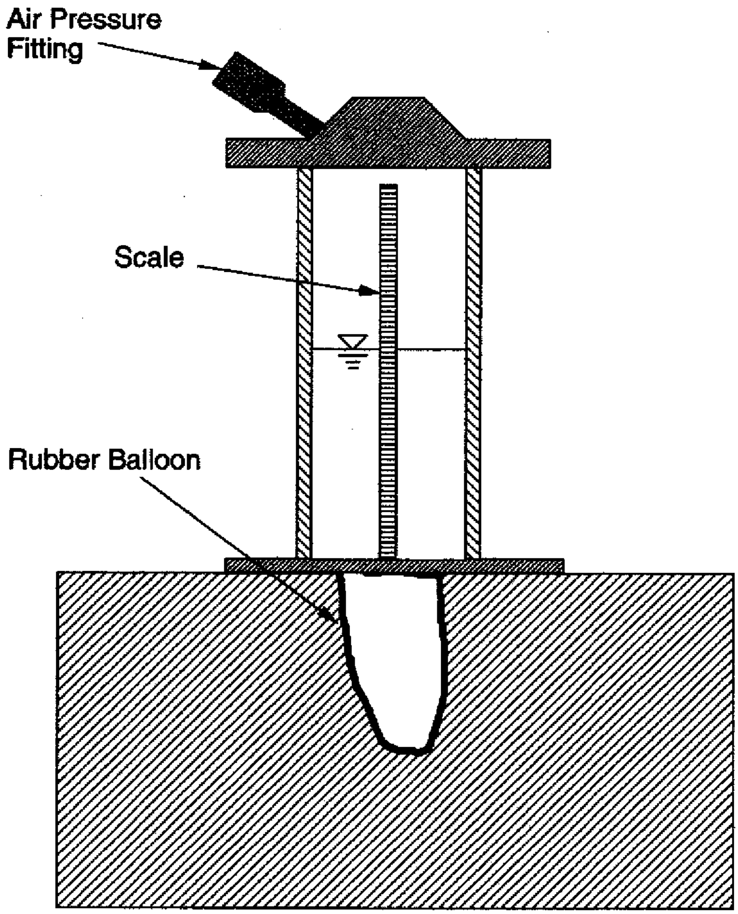

Figure 3-25. Schematic Diagram of Rubber Balloon Device.

brane (balloon) prevents the water from entering the soil. The pressure in the water forces the balloon to conform to the shape of the excavated hole. A graduated scale on the rubber balloon device enables one to determine the volume of water required to fill the hole. The total unit weight is calculated by dividing the known weight of soil excavated from the hole by the volume of water required to fill the hole with the rubber balloon device. The dry unit weight is computed from Eq. 3-1. Water content of the excavated soil is determined by a method such as microwave oven-drying.

The primary sources of error with the rubber balloon device are improper excavation of the hole (leaving small zones that cannot be filled by the pressurized balloon), excessive pressure that causes local deformation of the adjacent soil or lifting of the device, leakage from or rupture of the balloon, and carelessness in operating the device (e.g., not applying enough pressure to force the balloon to fill the hole completely).

3.3.2.3 Drive Cylinder (ASTM D2937)

A drive cylinder is sketched in Figure 3-26. A drop weight is used to drive a thin-walled tube sampler into the soil. The sampler is removed from the soil, and the

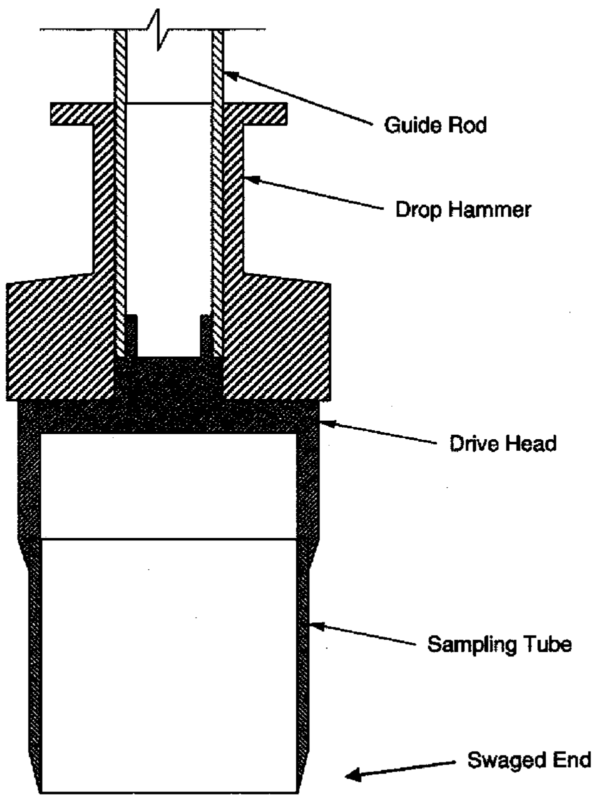

Figure 3-26. Schematic Diagram of Drive Cylinder.

soil sample is trimmed flush to the bottom and top of the sampling tube. The soil-filled tube is weighed, and the known weight of the sampling tube is subtracted to determine the gross weight of the soil sample. The dimensions of the sample are measured to enable calculation of volume. The unit weight is calculated by dividing the known weight by the known volume of the sample. A portion of the sample is oven-dried (e.g., in a microwave oven) to determine water content. The dry unit weight is computed from Eq. 3-1.

The primary problems with the drive cylinder are sampling disturbance caused by rocks or stones in the soil, densification of the soil caused by compression that results from driving the tube into the soil, and nonuniform driving of the tube into the soil. The drive cylinder method is not recommended for soils containing gravel or stones. The drive cylinder method works best for relatively soft, wet clays that do not tend to densify significantly when the tube is driven into the soil and for soils that are free of gravel or stones. However, even under favorable circumstances, densification of the soil caused by driving the ring into the soil can cause an increase in total unit weight of 0.3 to 0.8 kN/m^3 (2–5 pcf).

3.3.2.4 Nuclear Method (ASTM D2922)

Unit weight can be measured with a nuclear device operated in two ways as shown in Figure 3-27. The most common usage is called *direct transmission,* in which a source of gamma radiation is lowered down a hole made into the soil to be tested (Figure 3-27(a)). Detectors located in the nuclear density device sense the intensity of gamma radiation at the ground surface. The intensity of gamma radiation detected at the surface is a function of the intensity of gamma radiation emitted at the source and the total unit weight of the soil material. The second mode of operation of the nuclear density device is called *backscattering*. With this technique, the source of gamma radiation is located at the ground surface (Figure 3-27(b)). The intensity of gamma radiation detected at the surface is a function of the density of the soil as well as the radioactivity of the source. With the backscattering technique, the measurement depends heavily on the density of the soil within the upper 25 to 50 mm of soil and is not affected much by the density of soil deeper than about 50 mm (2 in.) below the surface. The direct transmission method is the recommended technique for soil liners because direct transmission provides a measurement averaged over a greater depth (approximately 150–200 mm, or 6–8 in., which is approximately equal to the thickness of a typical lift of compacted soil) than backscattering.

The operation of a nuclear density device in the direct transmission mode is as follows. First, the area to be tested is smoothed, and a hole is made into the soil liner material by driving a rod (called the *drive rod*) into the soil. The diameter of the hole is approximately 25 mm (1 in.), and the depth of the hole is typically 50 mm (2 in.) greater than the depth to which the gamma radiation source will be lowered below the surface. The nuclear device is then positioned with the source rod directly over the hole in the soil liner material. The source rod is then lowered to a depth of approximately 50 mm (2 in.) above the base of the hole. The source is then pressed against the surface of the hole closest to the detector by

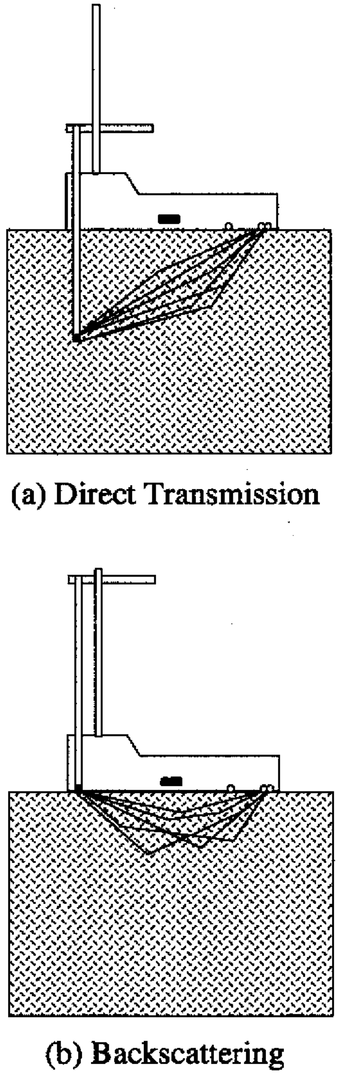

Figure 3-27. Measurement of Density with Nuclear Device by (a) Direct Transmission and (b) Backscattering.

pulling on the nuclear device and forcing the source to bear against the side of the hole closest to the detector. The intent is to have good contact between the source and soil along a direct line from source to detector. The intensity of radiation at the detector is measured for a fixed period of time, e.g., 30 or 60 s. The operator can select the period of counting; the longer the counting period, the more accurate the measurement. However, the counting period cannot be extended too much because productivity in density testing will suffer.

After total unit weight has been determined, the measured water content is used to compute dry unit weight (Eq. 3-1). The potential sources of error with the nuclear device are fewer and less significant in the density-measuring mode compared to the water content measuring mode. The most serious potential source of error is improper use of the nuclear density device by the operator. One gross error that is sometimes made is to drive the source rod into the soil rather than inserting the source rod into a hole that had been made earlier with the drive rod. Improper separation of the source from the base of the hole, an inadequate period of counting, inadequate warmup, spurious sources of gamma radiation, and inadequate calibration are other potential sources of error.

3.4 Recommended Procedure for Developing Water Content–Density Specification

One of the most important aspects of CQC and CQA for soil liners is documentation of the water content and dry unit weight of the soil immediately after compaction and verification of conformance with specifications, which almost always set tight restrictions on water content and dry unit weight. The specification for water content may be developed by the designer or may be left for the CQA engineer to determine. Historically, the method used to specify water content and dry unit weight has been based on experience with structural fill. Design engineers often require that soil liners be compacted within a specified range of water content and to a minimum dry unit weight. The acceptable zone shown in Figure 3-28 repre-

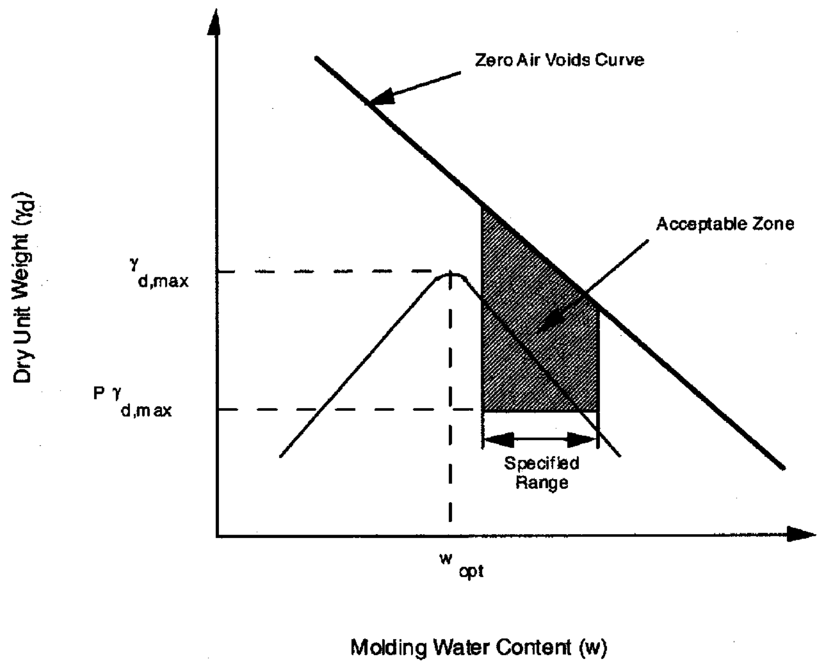

Figure 3-28. Form of Water Content–Dry Unit Weight Specification Often Used in the Past.

sents the zone of acceptable water content–dry unit weight combinations that is often prescribed. The shape of the acceptable zone shown in Figure 3-28 evolved empirically from construction practices applied to roadway bases, structural fills, embankments, and earthen dams. The specification is based primarily on the need to achieve a minimum dry unit weight for adequate strength and limited compressibility. As discussed by Mundell and Bailey (1985), Boutwell and Hedges (1989), and Daniel and Benson (1990), this method of specifying water content and dry unit weight is not necessarily the best method for compacted soil liners.

The recommended approach is intended to ensure that the soil liner will be compacted to a water content and dry unit weight that will lead to low hydraulic conductivity and adequate engineering performance with respect to other considerations (e.g., shear strength). Rational specification of water content–dry unit weight criteria should be based on test data developed for each particular borrow soil. Field test data would be better than laboratory data, but the cost of determining compaction criteria in the field through a series of test sections would almost always be prohibitive. Because the compactive effort will vary in the field, a logical approach is to select several compactive efforts in the laboratory that span the range of compactive effort that might be anticipated in the field. If this selection is done, the water content–dry unit weight criterion that evolves would apply to any reasonable compactive effort.

For most earthwork projects, modified Proctor effort represents a reasonable upper limit on the compactive effort likely to be delivered to the soil in the field. Standard compaction effort (ASTM D698) represents a medium compactive effort. It is conceivable that soil in some locations will be compacted with an effort less than that of standard Proctor compaction. A reasonable lower limit of compactive energy is the "reduced compaction" procedure in which standard compaction procedures (ASTM D698) are followed, except that only 15 drops of the hammer per lift are used instead of the usual 25 drops. The reduced compaction procedure is the same as the 15-blow compaction test described by the U.S. Army Corps of Engineers (1970). The reduced compactive effort corresponds to a reasonable minimum level of compactive energy for a typical soil liner or cover. Other compaction methods (e.g., kneading compaction) could be used. The key is to span the range of compactive effort expected in the field with laboratory compaction procedures.

The recommended approach is as follows:

1. Prepare and compact soil in the laboratory over a range of water content with modified, standard, and reduced compaction procedures to develop compaction curves, as shown in Figure 3-29(a). Make sure that the soil preparation procedures are appropriate; factors such as clod size may influence the results (Benson and Daniel 1990). Other compaction procedures can be used if they better simulate field compaction and span the range of compactive effort expected in the field. Also, as few as two laboratory compaction methods can be used if field construction procedures make either the lowest or highest compactive energy irrelevant.

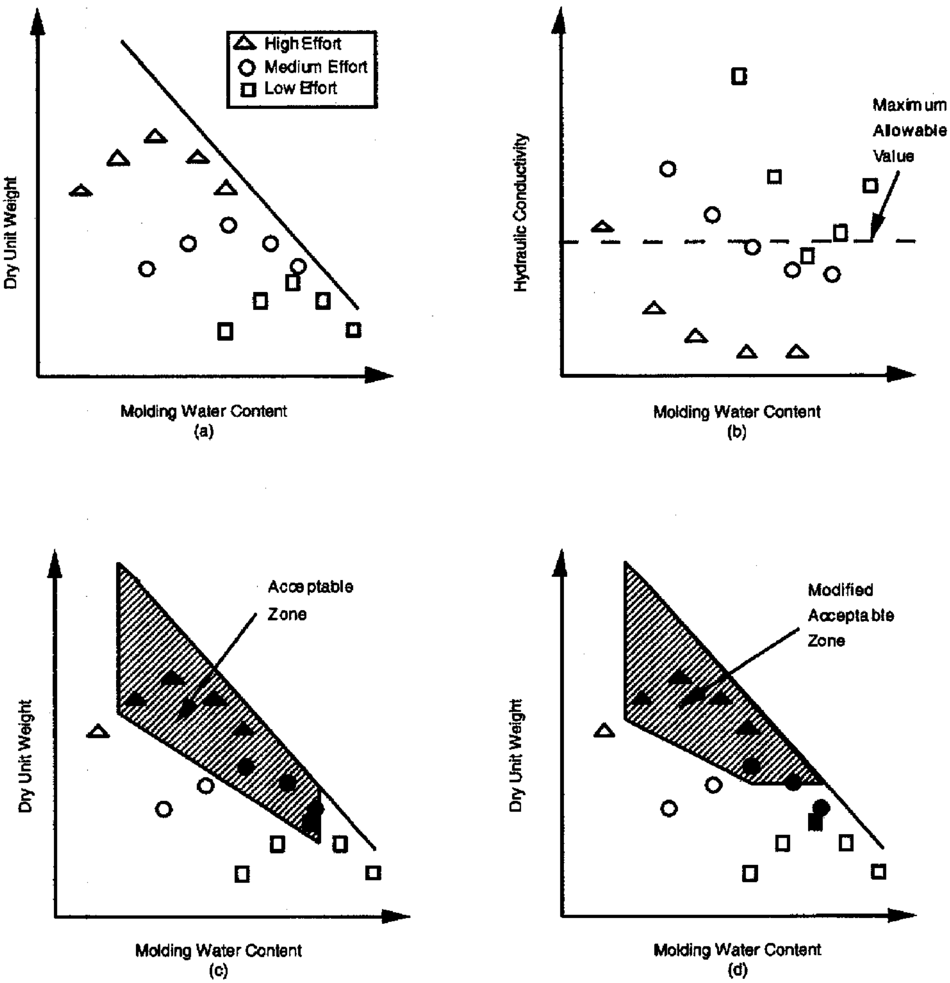

Figure 3-29. Recommended Procedure To Determine Acceptable Zone of Water Content–Dry Unit Weight Values Based on Hydraulic Conductivity Considerations.

Source: After Daniel and Benson 1990, ASCE.

2. The compacted specimens should be permeated (per ASTM D5084 or ASTM D5856). Care should be taken to ensure that permeation procedures are correct, with important details such as degree of saturation and effective confining stress carefully selected. The measured hydraulic conductivity should be plotted as a function of molding water content, as shown in Figure 3-29(b).
3. As shown in Figure 3-29(c), the dry unit weight–water content points should be replotted with different symbols used to represent compacted specimens that had hydraulic conductivities greater than the maximum acceptable value and specimens with hydraulic conductivities less than or equal to the maximum acceptable value. An acceptable zone should be drawn to encompass the

data points representing test results meeting or exceeding the design criteria. Some judgment is usually necessary in constructing the acceptable zone from the data points. Statistical criteria (Boutwell and Hedges 1989) may be introduced at this stage.

4. The acceptable zone should be modified (Figure 3-29(d)) based on other considerations such as shear strength. Additional tests are usually necessary to define the acceptable range of water content and dry unit weight that satisfy both hydraulic conductivity and shear strength criteria. Figure 3-30 illustrates how one might overlap acceptable zones defined from hydraulic conductivity and shear strength considerations to define a single acceptable zone. The same procedure can be applied to take into consideration other factors such as shrink–swell potential relevant to any particular project.

The same general procedure just outlined may also be used for soil–bentonite mixtures. However, to keep the scope of testing reasonable, the required amount of bentonite should be determined before the main part of the testing program is

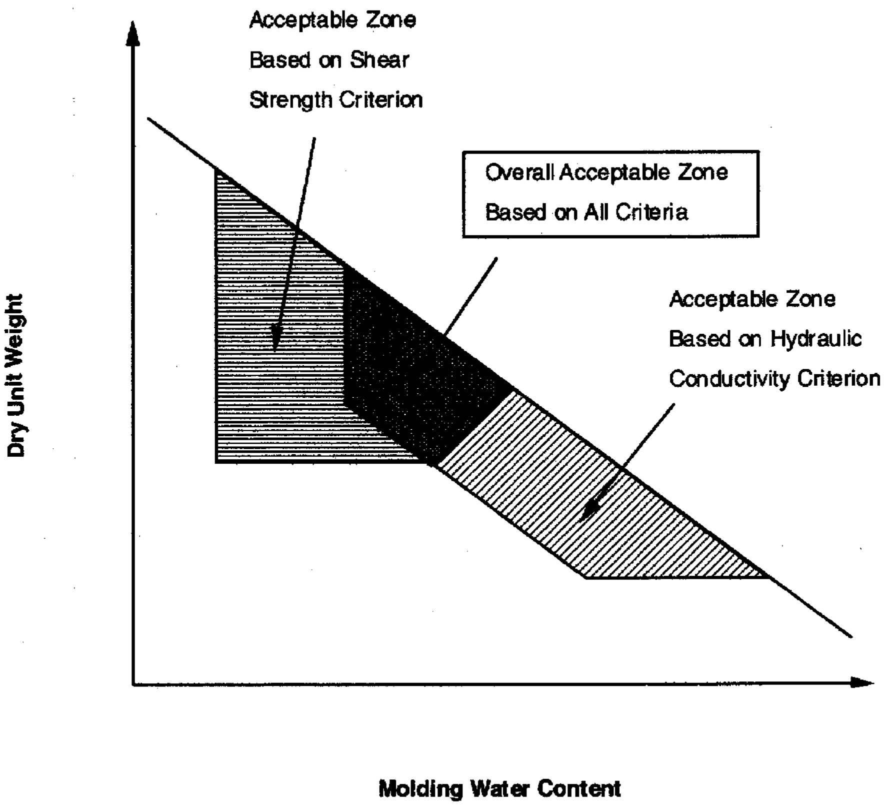

Figure 3-30. Acceptable Zone of Water Content–Dry Unit Weights Determined by Superimposing Hydraulic Conductivity and Shear Strength Data.

Source: After Daniel and Benson 1990, ASCE.

initiated. The recommended procedure for soil–bentonite mixtures may be summarized as follows:

1. Determine the type, grade, and gradation of bentonite that will be used. This process usually involves estimating costs from several potential suppliers. A sufficient quantity of the bentonite likely to be used for the project should be obtained and tested to characterize the bentonite (characterization tests are discussed later). Great care must be used to employ the same type and gradation of bentonite in the laboratory testing procedure that will be used in the field for construction.
2. Obtain a representative sample of the soil to which the bentonite will be added.
3. Prepare batches of soil–bentonite mixtures by blending in bentonite at several percentages (e.g., 2%, 4%, 6%, 8%, and 10% bentonite). Bentonite content is defined as the weight or mass of bentonite divided by the weight or mass of soil mixed with bentonite. For instance, if 5 kg of bentonite are mixed with 100 kg of soil, the bentonite content is 5%. Some people use the gross weight of bentonite rather than oven-dry weight. Since air-dry bentonite usually contains 10% to 15% hygroscopic water by weight, the use of oven-dry, air-dry, or damp weight can make a difference in the percentage. Similarly, the weight of soil may be defined as either moist or dry (air- or oven-dry) weight. The contractor would rather work with total (moist) weights because the materials used in forming a soil–bentonite blend do contain some water. However, the engineering characteristics are controlled by the relative amounts of dry materials. A dry-weight basis is recommended for definition of bentonite content, but CQC and CQA personnel must recognize that the project specifications may or may not be on a dry-weight basis.
4. Develop compaction curves for each soil-bentonite mixture prepared from Step 3 using the method of compaction appropriate to the project (ASTM D698 or ASTM D1557).
5. Compact samples for each percentage of bentonite at 2% wet of optimum using the same compaction procedure used in Step 4.
6. Permeate the soils prepared from Step 5 using ASTM D5084 or ASTM D5856. Graph hydraulic conductivity versus percentage of bentonite.
7. Decide how much bentonite to use based on the minimum required amount determined from Step 6. The minimum amount of bentonite used in the field should always be greater than the minimum amount suggested by laboratory tests because mixing in the field is usually not as thorough as in the laboratory. Typically, the amount of bentonite used in the field is 1 to 4 percentage points greater than the minimum percent bentonite indicated by laboratory tests.
8. Prepare a master batch of material by mixing bentonite with a representative sample of soil at the average bentonite content expected in the field. The procedures described earlier for determining the acceptable zone of water content and dry unit weight are then applied to the master batch.

On some projects, it may not be practical to compact numerous samples of soil using the procedures recommended above. In most practical situations, the construction specification for moisture–density is likely to be similar to the type of specification illustrated in Figure 3-28. For a conventional-type specification, the most critical point (i.e., the point that is least likely to conform to the maximum allowable hydraulic conductivity) is the lower left corner of the specified zone, as illustrated in Figure 3-31. The lower left corner represents the point within the specified zone with the lowest degree of saturation and the driest point relative to the line of optimums. If one can check only one moisture–density point, this is the point that should be checked. If soil compacted at the lower left corner of the specified moisture–density zone meets the hydraulic conductivity requirements for the project, the chances are good that the other moisture–density combinations will meet the project objectives, too.

The lower limit of a typical acceptable zone, such as sketched in Figure 3-29(c), is often similar to a line of constant degree of saturation. In many cases, specifying a minimum degree of saturation defines the lower boundary of the acceptable zone. It is recommended that if degree of saturation is used, the appropriate minimum degree of saturation be determined or at least verified by a testing program such as that described in previous paragraphs. A minimum dry density may also be specified along with degree of saturation to define the acceptable zone.

Various experiences illustrate the interplay between control parameters, such as compaction wet of the line of optimums and grain size, as illustrated by Leroueil et al. (2002).

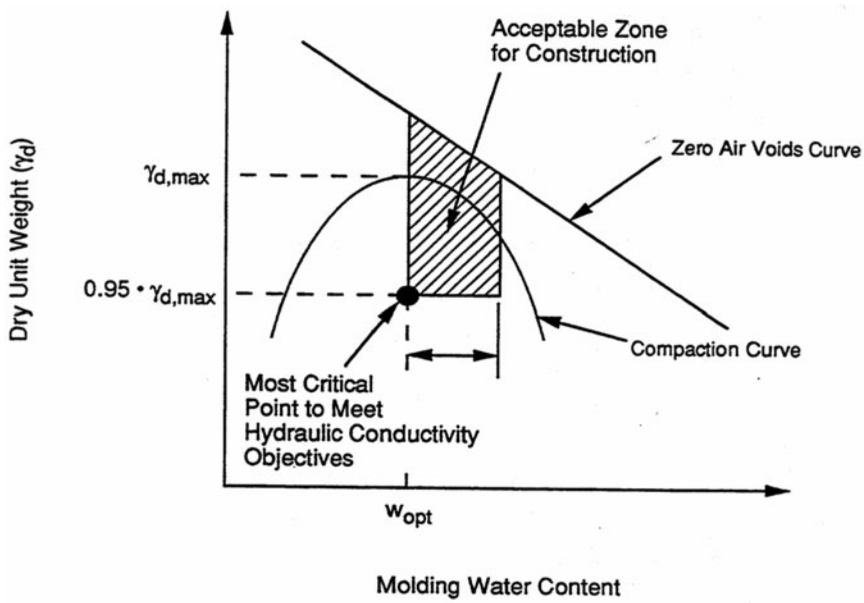

Figure 3-31. Example of a Conventional Construction Specification and Identification of the Most Critical Compliance Point at the Lower Left Corner of the Specified Zone.

3.5 Inspection of Borrow Sources before Excavation

In principle, the suitability of materials need only be confirmed once. The most logical point of confirmation is just after placement of a loose lift, just before compaction. However, at this point, substantial time and money have been spent to excavate, haul, process, and place the soil. If there is a problem with the material, it may be extremely difficult and expensive to haul away the unsuitable soil and replace it. Also, after an unsuitable soil has been placed, the CQA engineer is usually pressured (sometimes extremely pressured) to approve the material, even though it is in fact unsuitable.

The purpose of borrow material tests is to prequalify the soil. If there is a problem with the soil (e.g., presence of deleterious material in the soil), the best time to identify the problem is before or during excavation. Omission of borrow source inspection will not always constitute a problem, but it will occasionally create severe problems. For this reason, the authors strongly recommend inspection of the borrow soil before (or alternatively, during) excavation.

3.5.1 Sampling for Material Tests

To determine the properties of the borrow soil, samples should be obtained from the potential borrow area for laboratory analysis before excavation. Samples may be obtained in several ways. One method of sampling is to drill soil borings and recover samples of soil from the borings. This procedure can be effective in identifying major strata and substrata within the borrow area. Small samples obtained from the borings are excellent for index property testing but often do not provide a good indication of subtle stratigraphic changes. Test pits excavated into the borrow soil with a backhoe, front-end loader, or other excavation equipment can expose a large cross section of the borrow soil. A much better idea of the variability of soil in the potential borrow area can be obtained by examining exposed cuts, rather than viewing small soil samples recovered from borings.

Large bulk samples of soil are required for compaction testing in the laboratory. Small samples of soil taken with soil-sampling devices do not provide a sufficient volume of soil for laboratory compaction testing. Some engineers combine samples of soil taken at different depths or from different borings to produce a composite sample of adequate volume. This technique is not recommended because a degree of mixing takes place in forming the composite laboratory test sample that would not take place in the field. Other engineers prefer to collect material from auger cuttings (if an auger is used to drill bore holes) for use in performing laboratory compaction tests. This technique is generally not recommended without careful borrow pit control because vertical mixing of material takes place during auguring in a way that would not be expected to occur in the field unless controlled vertical cuts are made. The best method for obtaining large bulk samples of material for laboratory compaction testing is to take a large sample of material from one location in the borrow source. A large, bulk sample can be taken from the wall or floor of a test pit that has been excavated into the borrow area.

Alternatively, a large piece of drilling equipment such as a bucket auger can be used to obtain a large volume of soil from a discreet location in the ground.

Inspectors should also watch out for deleterious materials in the borrow soil. Sticks, grass roots, organic matter, and debris are normally not allowed in soil liner materials.

3.5.2 Material Tests

Samples of soil must be taken for laboratory testing to ensure conformance with specifications for parameters such as percentage of fines and plasticity index. The samples are sometimes taken in the borrow pit or from the loose lift just before compaction, or they are sometimes taken from both. If samples are taken from the borrow area, CQA inspectors track the approximate volumes of soil excavated and sample at the frequency prescribed in the CQA plan. Sometimes borrow-source testing is performed before issuing a contract to purchase the borrow material. A CQA program cannot be implemented for work already completed. The CQA personnel will have ample opportunity to check the properties of soil materials later, during excavation and placement of the soils. If the CQA personnel for a project did not observe borrow soil testing, the CQA personnel should review the results of borrow soil testing to ensure that the required tests have been performed. Additional testing of the borrow material may be required during excavation of the material.

The material tests that are normally performed on borrow soil are water content, Atterberg limits (liquid limit, plastic limit, and plasticity index), particle-size distribution, compaction curve, and hydraulic conductivity (Table 3-2). Each of these tests is discussed below.

3.5.2.1 Water Content

It is important to know the water content of the borrow soils so that the need for wetting or drying the soil before compaction can be identified. The water content of the borrow soil is usually measured after the procedures outlined in ASTM D2216 if one can wait overnight for results. If not, other test methods described in Section 3.3.1 and listed in Table 3-2 can be used to produce results faster.

3.5.2.2 Atterberg Limits

Construction specifications for compacted soil liners often require a minimum value for the liquid limit, plasticity index, or both of the soil. These parameters are measured in the laboratory with the procedures outlined in ASTM D4318.

3.5.2.3 Particle-Size Distribution

Construction specifications for soil liners often place limits on the minimum percentage of fines, the maximum percentage of gravel, and in some cases the minimum percentage of clay. Particle-size analysis is performed after the procedures in ASTM D422. Usually, the requirements for the soil material are explicitly stated in the construction specifications. An experienced inspector can often judge the

Table 3-2. Materials Tests

Parameter	*ASTM Test Method*	*Title of ASTM Test*
Water content	D2216	Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
	D4643	Standard Test Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Method
	D4944	Standard Test Method for Field Determination of Water (Moisture) Content of Soil by the Calcium Carbide Gas Pressure Tester Method
	D4959	Standard Test Method for Determination of Water (Moisture) Content of Soil by Direct Heating Method
Liquid limit, plastic limit, and plasticity index	D4318	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
Particle size distribution	D422	Standard Test Method for Particle-Size Analysis of Soils
Compaction curve	D698	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))
	D1557	Standard Test Methods for Laboratory Compaction Characteristics of Soils Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3))
Hydraulic conductivity	D5084	Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter
	D5856	Standard Test Method for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter

percentage of fine material and the percentage of sand or gravel in the soil. However, compliance with specifications is best documented by laboratory testing.

3.5.2.4 Compaction Curve

Compaction curves are developed by using the method of laboratory compaction testing required in the construction specifications. Standard compaction (ASTM D698) and modified compaction (ASTM D1557) are two common methods of laboratory compaction specified for soil liners. However, other compaction methods (particularly those unique to state highway or transportation departments) are sometimes specified.

Great care should be taken to follow the procedures for soil preparation outlined in the relevant test method. In particular, the drying of a cohesive material can change the Atterberg limits, as well as the compaction characteristics of the soil. If the test procedure recommends that the soil not be dried, the soil should not be dried. Also, care must be taken when sieving the soil not to remove clods of cohesive material. Rather, clods of soil retained on a sieve should be broken apart by hand if necessary to cause them to pass through the openings of the sieve. Sieves should only be used to remove stones or other large pieces of material following ASTM procedures.

3.5.2.5 Hydraulic Conductivity

The hydraulic conductivity of compacted samples of borrow material may be measured periodically to verify that the soil liner material can be compacted to achieve the required low hydraulic conductivity. Several methods of laboratory permeation are available, and most tests are either rigid-wall hydraulic conductivity tests (ASTM D5856) or flexible-wall hydraulic conductivity tests (ASTM D5084). Care should be taken not to apply excessive effective confining stress to flexible-wall test specimens. If no effective confining stress is specified in the CQA plan for flexible-wall tests, a maximum effective stress of 35 kPa (5 psi) is recommended for both liner and cover systems.

Care should be taken to prepare specimens for hydraulic conductivity testing properly. In addition to water content and dry unit weight, the method of compaction and the compactive energy can have a significant influence on the hydraulic conductivity of laboratory-compacted soils. It is particularly important not to deliver too much compactive energy to attain a desired dry unit weight. The purpose of the hydraulic conductivity test is to verify that borrow soils can be compacted to the desired hydraulic conductivity using a reasonable compactive energy.

No ASTM compaction method exists for preparation of hydraulic conductivity test specimens. The following procedure is recommended:

1. Obtain a large, bulk sample of representative material with a mass of approximately 20 kg.
2. Develop a laboratory compaction curve using the procedure specified in the construction specifications for compaction control (ASTM D698 or ASTM D1557).
3. Determine the target water content (w_{target}) and dry unit weight ($\gamma_{d,target}$) for the hydraulic conductivity test specimen. The value of w_{target} should be the lowest acceptable water content, and the value of $\gamma_{d,target}$ should be the minimum acceptable dry unit weight.
4. Mix enough soil to make several test specimens to w_{target}. The compaction procedure used in Step 2 is used to prepare a compacted specimen, except that the energy of compaction is reduced (e.g., by reducing the number of drops of the ram per lift). The dry unit weight (γ_d) is determined. If $\gamma_d = \gamma_{d,target}$, the compacted specimen may be used for hydraulic conductivity testing. If $\gamma_d >$

$\gamma_{d,target}$, then another test specimen is prepared with a larger or smaller (as appropriate) compactive energy. Trial and error preparation of test specimens is repeated until $\gamma_d \approx \gamma_{d,target}$. The procedure is illustrated in Figure 3-32. The actual compactive effort should be documented, along with hydraulic conductivity.

5. Atterberg limits and percentage fines should be determined for each bulk sample. Water content and dry density should be reported for each compacted specimen.

3.5.2.6 Testing Frequency

The CQA plan should stipulate the frequency of testing. Recommended minimum values are shown in Table 3-3. The tests listed in Table 3-3 should be performed before construction as part of the characterization of the borrow source. However, if time or circumstances do not permit characterization of the borrow source before construction, the samples for testing are obtained during excavation or delivery of the soil materials.

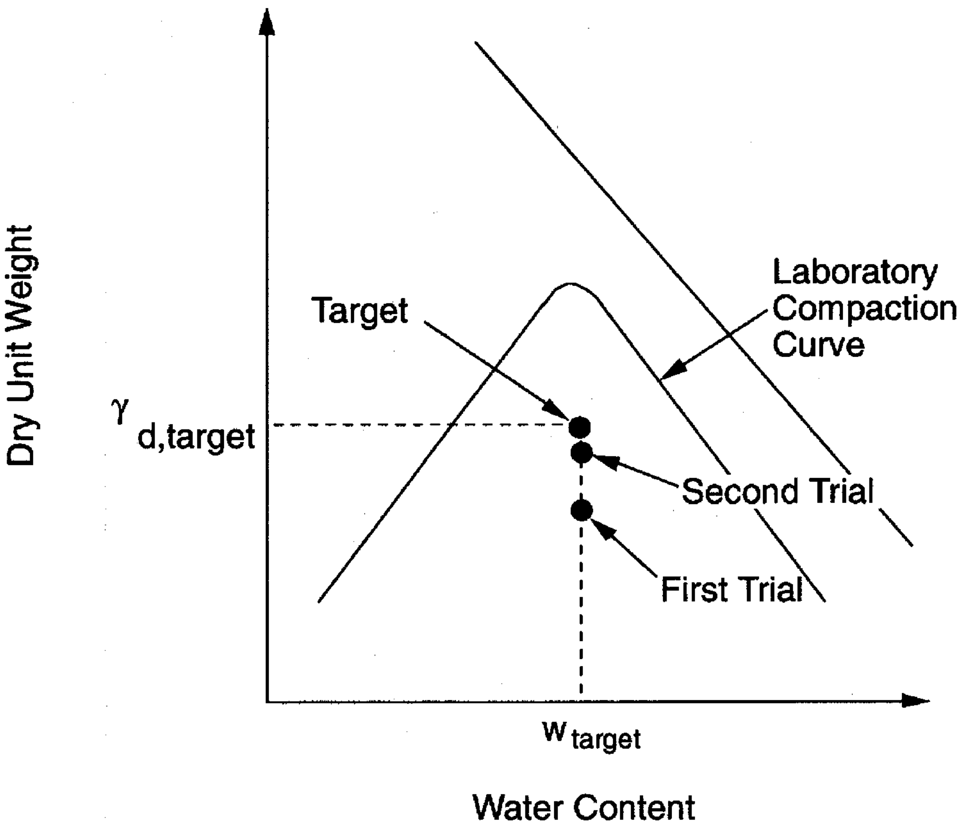

Figure 3-32. Recommended Procedure for Preparation of a Test Specimen Using Variable (and Documented) Compactive Energy for Each Trial.

Table 3-3. Recommended Minimum Testing Frequencies for Investigation of Borrow Source

Parameter	*Frequency*
Water content	1 test per 2000 m^3
Atterberg limits	1 test per 5,000 m^3
Percentage fines	1 test per 5,000 m^3
Percent gravel	1 test per 5,000 m^3
Compaction curve	1 test per 5,000 m^3
Hydraulic conductivity	1 test per 10,000 m^3

Notes: 1 yd^3 = 0.76 m^3. Each frequency includes "each change in material type."

3.6 Inspection during Excavation of Borrow Soil

It is strongly recommended that a qualified inspector who reports directly to the CQA engineer observe all excavation of borrow soil in the borrow pit. Often the best way to determine whether deleterious material is present in the borrow soil is to observe the excavation of the soil directly.

A key factor for inspectors to observe is the plasticity of the soil. Experienced technicians can often determine whether or not a soil has adequate plasticity by carefully examining the soil in the field. A useful practice for field identification of soils is ASTM D2488, "Standard Test Method for Description and Identification of Soils (Visual–Manual Procedure)." The following procedure is used for identifying clayey soils:

- Dry strength: The technician selects enough soil to mold into a ball about 25 mm (1 in.) in diameter. Water is added if necessary to form three balls that each have a diameter of about 12 mm (1/2 in.). The balls are allowed to dry in the sun. The strength of the dry balls is evaluated by crushing them between the fingers. The dry strength is described with the criteria shown in Table 3-4. If the dry strength is none or low, inspectors should be alerted to the possibility that the soil lacks adequate plasticity.

Table 3-4. Criteria for Describing Dry Strength

Description	*Criteria*
None	The dry specimen crumbles into powder with mere pressure of handling.
Low	The dry specimen crumbles into powder with some finger pressure.
Medium	The dry specimen breaks into pieces or crumbles with considerable finger pressure.
High	The dry specimen cannot be broken with finger pressure. The specimen will break into pieces between the thumb and a hard surface.
Very high	The dry specimen cannot be broken between the thumb and a hard surface.

Source: ASTM D2488

- Plasticity: The soil is moistened or dried so that a test specimen can be shaped into an elongated pat and rolled by hand on a smooth surface or between the palms into a thread about 3 mm (1/8 in.) in diameter. If the sample is too wet to roll easily, it should be spread into a thin layer and allowed to lose some water by evaporation. The sample threads are rerolled repeatedly until the thread crumbles at a diameter of about 3 mm (1/8 in.). The thread will crumble at a diameter of 3 mm when the soil is near the plastic limit. The plasticity is described from the criteria shown in Table 3-5, based on observations made during the toughness test. Nonplastic soils are usually unsuitable for use as soil liner materials without use of amendments such as bentonite.

3.7 Preprocessing of Materials

Some soil liner materials are ready to be used for final construction immediately after they are excavated from the borrow pit. However, many materials require some degree of processing before placement and compaction of the soil.

3.7.1 Water Content Adjustment

Soils that are too wet must first be dried. If the water content needs to be reduced by no more than about 3 percentage points, the soil can be dried after it has been spread in a loose lift just before compaction. If the water content must be reduced by more than about 3 percentage points, it is recommended that drying take place in a separate processing area. The reason for drying in a separate processing area is to allow adequate time for the soil to dry uniformly and to facilitate mixing of the material during drying. The soil to be dried is spread in a lift about 225 to 300 mm (9–12 in.) thick and allowed to dry. Water content is periodically measured using one or more of the methods listed in Table 3-2. The contractor's CQC per-

Table 3-5. Criteria for Describing Plasticity

Description	*Criteria*
Nonplastic	A 3-mm (1/8-in.) thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

Source: ASTM D2488.

sonnel should check the soil periodically to determine when the soil has reached the proper water content.

The CQA inspectors should check to be sure that the soil is periodically mixed with a disk tiller or rototiller to ensure uniform drying. The soil cannot be considered ready for placement and compaction unless the water is uniformly distributed; water content measurements alone do not ensure that water is uniformly distributed within the soil.

If the soil must be moistened before compaction, the same principles discussed above for drying apply; water content adjustment in a separate preprocessing area is recommended if the water content must be increased by more than about 3 percentage points. Inspectors should be careful to verify that water is distributed uniformly to the soil (a spreader bar on the back of a water truck is the recommended device for moistening soil uniformly), that the soil is periodically mixed with a disk tiller or rototiller, and that adequate time has been allowed for uniform hydration of the soil. If the water content is increased by more than 3 percentage points, at least 24 hours would usually be required for uniform absorption of water and hydration of soil particles. The construction specifications may limit the type of water that can be used; contaminated water, brackish water, or seawater is usually not allowed.

3.7.2 Removal of Oversize Particles

Oversized stones and rocks should be removed from the soil liner material. Stones and rocks interfere with compaction of the soil and may create undesirable pathways for fluid to flow through the soil liner. In the top lift, oversized stones could puncture a geomembrane placed above the soil liner. The construction specifications should stipulate the maximum allowable size of particles in the soil liner material. The specifications may be more stringent in the top lift.

Oversized particles can be removed with mechanical equipment (e.g., large screens) or by hand. Inspectors should examine the loose lift of soil while the contractor is removing oversized particles to verify that oversized particles are not present. Sieve analyses alone do not provide adequate assurance that oversized materials have been removed—careful visual inspection for oversized material should be mandatory.

3.7.3 Pulverization of Clods

Some specifications for soil liners place limitations on the maximum size of chunks or clods of clay present in the soil liner material. Disk tillers, rototillers, and road recyclers are examples of mechanical devices that will pulverize clods in a loose lift. Visual inspection of the loose lift of material is usually performed to ensure that clods of soil have been pulverized to the extent required in the construction specifications. Inspectors should be able to visually examine the entire surface of a loose lift to determine whether clods have been adequately processed. No standard method exists for determining clod size. Inspectors usually measure the dimensions of an individual clod with a ruler.

3.7.4 Homogenizing Soils

CQC and CQA are difficult to perform for heterogeneous materials. It may be necessary to blend and homogenize soils before their use in constructing soil liners to perform meaningful CQC and CQA. Soils can be blended and homogenized in a pugmill. The best way to ensure adequate mixing of materials is through visual inspection of the mixing process itself.

3.7.5 Bentonite

Bentonite is a common additive to soil liner materials that do not contain enough clay to achieve the desired low hydraulic conductivity. Inspectors must ensure that the bentonite being used for a project is in conformance with specifications (i.e., is of the proper type, quality, and gradation) and that the bentonite is uniformly mixed with soil in the required amounts.

The parameters that are specified for the bentonite quality vary considerably from project to project. The construction specifications should stipulate the criteria to be met by the bentonite and the relevant test methods.

One test of water absorbency is the Atterberg (liquid and plastic) limit tests, per ASTM D4318. The higher the quality of the bentonite, the higher the liquid limit and plasticity index. Although liquid and plastic limit tests are common tests for natural soils, they have not been frequently used as indicators of bentonite quality in the bentonite industry (although they are sometimes used, and bentonite suppliers are familiar with the test).

The two most commonly used index tests to measure bentonite quality for waste containment applications are the free swell test (ASTM D5890) and fluid loss test (ASTM D5891). Although D5890 and D5891 are applicable to bentonite used for GCLs, the testing methodology can also be applied to bentonite used for soil–bentonite admixtures. The free swell test is used to determine the amount of swelling of bentonite when bentonite is submerged in water in a glass beaker. Calcium bentonites usually have a free swell of less than 6 cc. Low-grade sodium bentonites typically have a free swell of 8 to 15 cc. High-grade bentonites often have free swell values in the range of 15 to 18 cc, and sometimes >25 cc.

The fluid loss test is performed by mixing bentonite with water and forcing it under pressure through a sheet of filter paper. High-quality bentonites form a thick, viscous slurry when mixed with water and exhibit relatively low fluid loss. Lower quality bentonites do not gel as well with water and lose a greater amount of water in the fluid loss test.

The CQA inspector should be particularly careful to ensure that the bentonite has been pulverized to the extent required in the construction specifications. The degree of pulverization is frequently overlooked. Finely ground, powdered bentonite will behave differently when blended into soil than more coarsely ground, granular bentonite. CQC/CQA personnel should be particularly careful to make sure that the bentonite is sufficiently finely ground and is not delivered in too coarse a form (per project specifications); sieve tests on the raw bentonite received at a job site are recommended to verify gradation of the bentonite.

The bentonite supplier is expected to certify that the bentonite meets the specification requirements. However, CQA inspectors should perform their own tests to ensure compliance with the specifications. The recommended CQA tests and testing frequencies for bentonite quality and gradation are summarized in Table 3-6.

3.7.5.1 Pugmill Mixing

A pugmill is a device for mixing dry materials. A conveyor belt feeds soil into a mixing unit, and bentonite drops downward into the mixing unit. The materials are mixed in a large box that contains rotating rods with mixing paddles. Water may be added to the mixture in the pugmill as well.

The degree of automation of pugmills varies. The most sophisticated pugmills have computer-controlled devices to monitor the amounts of the ingredients being mixed. CQA personnel should monitor the controls on the mixing equipment.

3.7.5.2 In-Place Mixing

An alternative mixing technique is to spread the soil in a loose lift, distribute bentonite on the surface, and mix the bentonite and soil using a rototiller or other mixing equipment. There are several potential problems with in-place mixing. The mixing equipment may not extend to an adequate depth and may not fully mix the loose lift of soil with bentonite. Alternatively, the mixing device may dig too deeply into the ground and actually mix the loose lift with underlying materials. Bentonite (particularly powdered bentonite) may be blown away by wind when it is placed on the surface of a loose lift, thus reducing the amount of bentonite that is actually incorporated into the soil. The mixing equipment may fail to pass over all areas of the loose lift and may inadequately mix certain portions of the loose lift.

In general, pugmill mixing provides the more reliable means for mixing bentonite with soil. CQA personnel should carefully examine the mixing process to ensure that the problems outlined above, or other problems, do not compromise the quality of the mixing process. Visual examination of the mixture to verify plasticity (see Section 3.6 and Table 3-5), as well as Atterberg limit tests on mixed soils, are recommended.

3.7.5.3 Measuring Bentonite Content

The best way to control the amount of bentonite mixed with soil is to measure the relative weights of soil and bentonite blended together at the time of mixing. After bentonite has been mixed with soil, there are several techniques available to esti-

Table 3-6. Recommended Tests on Bentonite To Determine Bentonite Quality and Gradation

Parameter	*Test Method*
Free swell	ASTM D5890
Fluid loss	ASTM D5891
Grain size of dry bentonite	ASTM D422

Note: For all tests, frequency is 1 per truckload or 2 per rail car.

mate the amount of bentonite in the soil. None of the techniques is particularly easy to use in all situations.

The two most commonly used techniques for measuring bentonite content are the methylene blue test (Alther 1983; ASTM C837) and hydrometer analysis (ASTM D422). The methylene blue test is a type of titration test. Methylene blue is slowly titrated into a material. Methylene blue is strongly absorbed by bentonite or other negatively charged clay minerals. The amount of methylene blue required to saturate the material is determined. The more bentonite in the soil, the greater the amount of methylene blue that must be added to achieve saturation. A calibration curve is developed between the amount of methylene blue needed to saturate the material and the bentonite content of the soil.

The methylene blue test works well when bentonite is added into a nonclayey soil. However, the amount of methylene blue that must be added to the soil is a function of the amount of clay present in the soil. If clay minerals other than bentonite are present, the clay minerals also absorb methylene blue and interfere with the determination of the bentonite content. There is no standard methylene blue test; the procedure outlined in Alther (1983) is suggested until such time as a standard test method is developed.

Hydrometer analysis can be used to measure the amount of clay in a soil. The technique can work reasonably well if the soil to which bentonite is being added contains virtually no fine-grained material (e.g., a clean sand). However, most natural soils contain some fines. The greater the amount of fines in the soil, the more the bentonite is masked by those fines in hydrometer analysis, and the less useful hydrometer analysis becomes.

Another type of test that has been used to estimate bentonite content is the electrical conductivity method (Abu-Hassanein et al. 1996). Fifty grams of soil–bentonite mixture is mixed with 1 L of water, and the electrical conductivity (EC) of the mixture is measured. Soil–bentonite mixtures with varying amounts of bentonite are tested in this way to develop a linear calibration curve. The bentonite content can then be determined directly from EC measurements performed using the actual soil–bentonite mixture.

Chapuis and Pouliot (1996) discuss use of X-ray diffraction to determine bentonite content in soils. The method can be effective, but the technique requires a well-equipped laboratory and, for this reason, may not be as cost-effective a CQA tool compared to other, simpler methods.

Measurement of hydraulic conductivity provides a means for verifying that enough bentonite has been added to the soil to achieve the desired low hydraulic conductivity. If insufficient bentonite has been added, the hydraulic conductivity will be unacceptably large. However, just because the hydraulic conductivity is acceptably low for a given sample does not necessarily mean that the required amount of bentonite has been added to the soil at all locations. Extra bentonite, beyond the minimum amount required, should be added to soil so that there will be sufficient bentonite present, even at those locations that are “lean” in bentonite.

The recommended tests and testing frequencies to verify proper addition of bentonite are summarized in Table 3-7. However, the CQA personnel should realize that the amount of testing depends on the degree of control in the mixing

Table 3-7. Recommended Tests to Verify Bentonite Content

Parameter	*Frequency*	*Test Method*
Methylene blue test	1 per 1,000 m^3	Alther (1983) and ASTM C837
Compaction curve for soil–bentonite mixture (needed to prepare hydraulic conductivity test specimen)	1 per 5,000 m^3	Per project specifications (e.g., ASTM D698 or D1557)
Hydraulic conductivity of soil–bentonite mixture compacted to appropriate water content and dry unit weight	3/ha/lift (1/acre/lift)	ASTM D5084, "Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible-Wall Permeameter"

Note: 1 yd^3 = 0.76 m^3.

process: The more control during mixing, the less is the need for testing to verify the proper bentonite content.

3.7.6 Stockpiling Soils

After the soil has been preprocessed, it is usually necessary to ensure that the water content does not change before use. The stockpiles can be of any size or shape. Small stockpiles should be covered so that the soil cannot dry or wet. For large stockpiles, it may not be necessary to cover the stockpile, particularly if the stockpile is sloped to promote drainage, moisture is added occasionally to offset drying at the surface, or other steps are taken to minimize wetting or drying of the stockpiled soil.

3.8 Placement of Loose Lift of Soil

After a soil has been fully processed, the soil is hauled to the final placement area. Soil should not be placed in adverse weather conditions (e.g., heavy rain). Inspectors are usually responsible for documenting weather conditions during all earthwork operations. The surface on which the soil will be placed should be properly prepared, and the material should be inspected after placement to make sure that the material is suitable. Inspectors should observe the soil while it is being placed in a lift. Any undesirable, deleterious materials (e.g., sticks, roots, or debris) should be immediately identified and appropriate corrective actions taken. Then the CQA inspectors should verify that the lift is not too thick. For side slopes, construction specifications should clearly state whether lifts are parallel to the slope or horizontal. For slopes inclined at 3(H):1(V) or flatter, lifts are usually parallel to the slope. For slopes inclined at 2(H):1(V) or steeper, lifts are usually horizontal. However, horizontal lifts may present problems because the hydraulic conductivity for flow parallel to lifts is expected to be somewhat greater than for flow perpendicular to lifts. Details of testing are described in the following subsections.

Transport vehicles can pick up deleterious materials on their tires while hauling material from the borrow source or preprocessing area. If this occurs, measures should be taken to prevent such materials from falling off transport vehicles into the soil liner material. These measures may include restricting vehicles to certain haul roads or removing contaminants before the vehicle enters the placement area.

Special care should be exercised when the soil is placed on a sensitive layer (e.g., drainage layer). It may be critical that the soil liner material not become mixed with the underlying material. A layer of nominally compacted soil is commonly placed above a drainage layer to ensure that the soil liner construction does not damage the drainage layer. The CQA personnel should be particularly diligent in their inspection of the placement of the first lift of soil liner material when the soil liner is placed on a critical layer such as a drainage layer.

3.8.1 Surface Scarification

Before placement of a new lift of soil, the surface of the previously compacted lift of soil liner should be roughened to promote good contact between the new and old lifts. Inspectors should observe the condition of the surface of the previously compacted lift to make sure that the surface has been scarified as required in the construction specifications. When soil is scarified, it is usually roughened to a depth of about 25 mm (1 in.). The surface does not usually require scarification if the surface is already rough after the end of compaction of a lift. CQA inspectors should ensure that the soil has been properly scarified if construction specifications require scarification. If the soil is scarified, the scarified zone becomes part of the loose lift of soil and should be counted in measuring the loose lift thickness.

3.8.2 Material Tests and Visual Inspection

3.8.2.1 Material Tests

After a loose lift of soil has been placed, samples are periodically taken to confirm the properties of the soil liner material. These samples are in addition to samples taken from the borrow area (Table 3-3). The types of tests and frequency of testing are normally specified in the CQA documents. Table 3-8 summarizes recommended minimum tests and testing frequencies. Samples of soils can be taken either on a grid pattern or on a random sampling pattern (see Section 3.9.3.2). Statistical tests and criteria could be used but are generally not applied to soil liners, in part because enough data have to be gathered to apply statistics, and yet pass/fail decisions have to be made immediately, before many data are collected.

3.8.2.2 Visual Observations

Inspectors should position themselves near the working face of soil liner material as it is being placed. Inspectors should look for deleterious materials such as stones,

Table 3-8. Recommended Material Tests for Soil Liner Materials Sampled after Placement in a Loose Lift

Parameter	*Test Method*	*Minimum Testing Frequency*
Percent fines[a]	ASTM D1140	1 per 800 cubic meters[b,c]
Percent gravel[d]	ASTM D422	1 per 800 cubic meters[b,c]
Liquid and plastic limits	ASTM D4318	1 per 800 cubic meters[b,c]
Percent bentonite[e]	Alther (1983)	1 per 800 cubic meters[b,c]
Compaction curve	As specified	1 per 4,000 cubic meters[3]
Construction oversight	Observation	Continuous

[a]Percent fines is defined as percent passing the No. 200 sieve.
[b]In addition, at least one test should be performed each day that soil is placed, and additional tests should be performed on any suspect material observed by CQA personnel.
[c]1 $yd^3 = 0.76\ m^3$.
[d]Percent gravel is defined as percent retained on the No. 4 sieve.
[e]This test is only applicable to soil–bentonite liners.

debris, and organic matter. Continuous inspection of the placement of soil liner material is recommended to ensure that the soil liner material is of the proper consistency.

3.8.2.3 Allowable Variations

Tests on soil liner materials may occasionally fail to conform to required specifications. It is unrealistic to think that 100% of a soil liner material will be in complete conformance with specifications. For example, if the construction documents require a minimum plasticity index, it may be anticipated that a small fraction of the soil (such as tiny pockets of sandy material) will fail to conform to specifications. It is neither unusual nor unexpected that occasional failing material will be encountered in soil liners; occasional imperfections in soil liner materials are expected. Indeed, one of the reasons that multiple lifts are used in soil liners is to account for the inevitable variations in the materials of construction used in building soil liners. Occasional deviations from construction specifications are not harmful. Recommended maximum allowable variations (failing tests) are listed in Table 3-9.

3.8.2.4 Corrective Action

If it is determined that the materials in an area do not conform to specifications, the first step is to define the area requiring repair. A sound procedure is to require the contractor to repair the lift of soil out to the limits defined by passing CQC/CQA tests. The contractor should not be allowed to guess at the extent of the area that requires repair. To define the limits of the area that requires repair, additional tests are often needed. Alternatively, if the contractor chooses not to request additional tests, the contractor should repair the area that extends from the failing test out to the boundaries defined by passing tests.

Table 3-9. Recommended Maximum Percentage of Failing Material Tests

Parameter	*Maximum Allowable Percentage of Outliers (percent)*
Atterberg limits	5
Percent fines	5
Percent gravel	10
Clod size	10
Percent bentonite	5
Hydraulic conductivity of laboratory-compacted soil	5

Note: All figures in the second column are in addition to outliers not concentrated in one lift or one area.

The usual corrective action is to wet or dry the loose lift of soil in place if the water content is incorrect. The water must be added uniformly, which requires mixing the soil with a disk or rototiller (see Section 3.7.1). If the soil contains oversized material, oversized particles are removed from the material (see Section 3.7.2). If clods are too large, clods can be pulverized in the loose lift (see Section 3.7.3). If the soil lacks adequate plasticity, contains too few fines, contains too much gravel, contains deleterious material, or lacks adequate bentonite, the material is usually excavated and replaced.

3.8.3 Control of Loose Lift Thickness

Construction specifications usually place limits on the maximum thickness of a loose lift of soil (e.g., 225 mm (9 in.)). Except in extremely unusual cases, the thickness of a loose lift should not exceed this value. The thickness of a loose lift may be determined in several ways. One technique is for an inspector standing near the working face of soil being placed to observe the approximate thickness of the lift. This is the most reliable technique for verifying loose lift thickness. If there is a question about loose lift thickness, a pit should be dug through the loose lift of soil and into the underlying layer. A cross beam is used to measure the depth from the surface of a loose lift to the top of the previously compacted lift. If the previously compacted lift was scarified, the zone of scarification should be counted in the loose lift thickness for the new layer of soil. Continuous observation of loose lift thickness is recommended during placement of soil liners.

Some earthwork contractors control lift thickness by driving grade stakes into the subsoil and marking the grade stake to indicate the proper thickness of the next layer. This practice is convenient for equipment operators because they can tell at a glance whether the loose lift thickness is correct. However, this practice is strongly discouraged because the penetrations into the previously compacted lift made by the grade stakes and the hole left by the grade stake in the new lift must be repaired. Also, any fragments from grade stakes left in a soil liner could puncture overlying geosynthetics. Repair of holes left by grade stakes is difficult because one must dig through the loose lift of soil to expose the grade stake, remove the grade stake without breaking the stake and leaving some of the stake in the soil, backfill the hole left by the grade stake, and then replace the loose soil in the freshly placed lift. For the first lift of soil liner, repair of grade stake holes in the underlying soil may not be relevant (depending on the subgrade and its function), but grade stakes are discouraged even for the first lift of soil because the stakes may be broken off and incorporated into the soil. Grade stakes resting on a small platform or base do not need to be driven into the underlying material and are, therefore, much more desirable than ordinary grade stakes. If grade stakes are used, it is recommended that they be numbered and accounted for at the end of each shift; this method will provide verification that grade stakes are not being abandoned in the fill material.

The recommended survey procedure for control of lift thickness involves laser sources and receivers. A laser beam source is set at a known elevation, and reception devices held by hand on rods or mounted to grading equipment are used to monitor lift thickness. However, lasers cannot be used at all sites. For instance, the liner may need to be a minimum distance above rock, and the grade lines may follow the contours of underlying rock. Furthermore, every site has areas such as slopes, which may preclude the use of lasers.

For those areas where lasers cannot be used, it is recommended that other survey techniques be used. Ordinary surveys of the position of the top of the loose lift, flexible plastic grade stakes, or metallic grade stakes (numbered and inventoried as part of the QA/QC process) is recommended. It is preferable that the stakes be mounted on a base so that the stakes do not have to be driven into the underlying lift. Repair of grade stake holes should be required; the repairs should be periodically inspected and then documented. Alternatively (and preferably for small areas), spot elevations can be obtained on the surface of a loose lift with conventional level and rod equipment, and adjustments can be made by the equipment operator based on the levels.

When soil is placed, it is usually dumped into a heap at the working face and spread with bulldozers. QA/QC personnel should stand in front of the working face to observe the soil for oversized materials or other deleterious material, to visually observe loose lift thickness, and to make sure that the bulldozer does not damage an underlying layer.

3.9 Remolding and Compaction of Soil

3.9.1 Compaction Equipment

The important parameters concerning compaction equipment are the type and weight of the compactor, the characteristics of any feet on the drum, and the weight of the roller per unit length of drummed surface. Sometimes construction specifications will stipulate a required type of compactor or minimum weight of compactor. If this is the case, inspectors should confirm that the compaction equipment is in conformance with specifications. Inspectors should be particularly cognizant of the weight of compactor and length of feet on drummed rollers. Heavy compactors with long feet that fully penetrate a loose lift of soil are generally thought to be the best type of compactor to use for soil liners. Footed rollers may not be necessary or appropriate for some bentonite–soil mixes; smooth-drum rollers or rubber tired rollers may produce best results for soil–bentonite mixtures that do not require kneading or remolding to achieve low hydraulic conductivity but only require densification.

The first lift of a soil liner often constitutes a special case because of concern that the compactor might force the downward migration of the soil liner material into the underlying layer (e.g., if the underlying layer is a drainage layer) or the upward migration of the foundation soil into the clay liner material. In such cases, a fully penetrating roller is not recommended for the first lift. The first lift may be a nominally compacted (e.g., with a bulldozer) sacrificial lift that is not counted as part of the soil liner.

Some compactors are self-propelled, whereas other compactors are towed. Towed, footed rollers are usually ballasted by filling the drum with water to provide weight that will enable significant compactive effort to be delivered to the soil. Inspectors should be careful to determine whether or not all drums on towed rollers have been filled with liquid. This detail is too often overlooked.

Compacting soil liners on side slopes can present special challenges, particularly for slopes inclined at 3(H):1(V) or steeper. Inspectors should observe side-slope compaction carefully and watch for any tendency of the compactor to slip downslope or for slippage or cracking to take place in the soil. Inspectors should also be watchful to make sure that adequate compactive effort is delivered to the soil. For soils compacted in lifts parallel to the slope, the first lift of soil should be "knitted" into existing subgrade to minimize a preferential flow path along the interface and to minimize development of a potential slip plane.

Footed rollers can become clogged with soil between the feet. Inspectors should examine the condition of the roller to make sure that the space between feet is not plugged with soil. In addition, compaction equipment is intended to be operated at a reasonable speed. The maximum speed of the compactor should be specified in the construction specifications. CQC and CQA personnel should make sure the speed of the equipment is not too great.

When soils are placed directly on a fragile layer, such as a geosynthetic material or a drainage material, great care must be taken in placing and compacting the first lift so as not to damage the fragile material or mix clay in with the underlying drainage material. In such cases, the first lift of soil is often considered a sacrificial lift that is placed, spread with bulldozers, and only nominally compacted with the bulldozers or a smooth-drum or rubber-tire roller. QA/QC personnel should be particularly careful to observe all placement and compaction operations of the first lift of soil for compacted soil liners placed directly on a geosynthetic material or drainage layer.

It is not improper for a contractor to use more than one type of compaction equipment on a project. For example, initial compaction may be with a heavy roller having long feet that fully penetrate a loose lift of soil. Later, the upper part of a lift may be compacted with a heavy rubber-tired roller or other equipment that is particularly effective in compacting near-surface materials.

3.9.2 Number of Passes

The compactive effort delivered by a roller is a function of the number of passes of the roller over a given area of soil. A pass may be defined as one pass of the construction equipment or one pass of a drum over a given point in the soil liner. It does not matter whether a pass is defined as a pass of the equipment or a pass of a drum, but the construction specifications and/or CQA plan should define what is meant by a pass. Normally, one pass of the vehicle constitutes a pass for self-propelled rollers and one pass of a drum constitutes a pass for towed rollers.

Some construction specifications require a minimum coverage. Coverage (C) is defined as follows:

$$C = [A_f/A_d] \times N \times 100\% \tag{3-8}$$

where N is the number of passes of the roller, A_f is the sum of the area of the feet on the drums of the roller, and A_d is the area of the drum itself. Construction specifications sometimes require 150 to 200% coverage of the roller. The purpose of specifying a minimum percent coverage is to ensure that the surface of virtually the entire lift has received direct compaction by a "foot" on the roller at least once. For a given roller and minimum percent coverage, the minimum number of passes (N) may be computed.

The number of passes of a compactor over the soil can have an important influence on the overall hydraulic conductivity of the soil liner. It is recommended that periodic observations be made of the number of passes of the roller over a given point. Approximately 3 observations per hectare per lift (one observation per acre per lift) is the recommended frequency of measurement. The minimum number of passes that is reasonable depends on many factors and cannot be stated in general terms. However, experience has been that at least 5 to 15 passes of a compactor over a given point are usually necessary to remold and compact clay liner materials thoroughly. On some projects, an excessive number of passes (e.g., >50) have been required. This excess can occur when the construction specifications require an unrealistically high density, when the soil is too wet, or when the contractor's compactor is too light.

3.9.3 Water Content and Dry Unit Weight

3.9.3.1 Water Content and Unit Weight Tests

One of the most important CQA tests is measurement of water content and dry unit weight. Methods of measurement were discussed in Section 3.3. Recommended testing frequencies are listed in Table 3-10. It is stressed that the recommended testing frequencies are the minimum values. Some judgment should be applied to

Table 3-10. Recommended Tests and Observations on Compacted Soil

Parameter	*Test Method*	*Minimum Testing Frequency*
Water content (rapid)[a]	ASTM D3017 ASTM D4643 ASTM D4944 ASTM D4959	13/ha/lift (5/acre/lift)[b,c]
Water content tests	ASTM D2216	One in every 10 rapid water content tests[d]
Total density (rapid)[e]	ASTM D2922 ASTM D2937	13/ha/lift (5/acre/lift)[b,c,e]
Total density[f]	ASTM D1556 ASTM D1587 ASTM D2167	One in every 20 rapid density tests[c,f,g]
Number of passes	Observation	3/ha/lift (1/acre/lift)[b,c]
Construction oversight	Observation	Continuous

[a]ASTM D3017 is a nuclear method, ASTM D4643 is microwave oven drying method, ASTM D4944 is a calcium carbide gas pressure tester method, and ASTM D4959 is a direct heating method. Direct water content determination (ASTM D2216) is the standard against which nuclear, microwave, or other methods of measurements are calibrated for on-site soils.
[b]In addition, at least one test should be performed each day soil is compacted, and additional tests should be performed in areas for which CQA personnel have reason to suspect inadequate compaction.
[c]1 acre = 0.4 ha.
[d]Every tenth sample tested with ASTM D3017, D4643, D4944, or D4959 should be also tested by direct oven-drying (ASTM D2216) to aid in identifying any significant, systematic calibration errors.
[e]ASTM D2922 is a nuclear method, and ASTM D2937 is the drive-cylinder method. These methods, if used, should be calibrated against the sand-cone method (ASTM D1556) or rubber balloon method (ASTM D2167) for on-site soils. Alternatively, the sand-cone or rubber balloon method can be used directly.
[f]Every 20th sample tested with D2922 should also be tested (as close as possible to the same test location) with the sand-cone method (ASTM D1556) or rubber balloon method (ASTM D2167) to aid in identifying any systematic calibration errors with D2922.
[g]ASTM D1587 is the method for obtaining an undisturbed sample. The section of undisturbed sample can be cut or trimmed from the sampling tube to determine bulk density. This method should not be used for soils containing any particles >1/6th the diameter of the sample.

these numbers, and the testing frequencies should be increased or kept at the minimum depending on the specific project and other QA/QC tests and observations. For example, if hydraulic conductivity tests are not performed on undisturbed samples (see Section 3.9.4.2), more water content–density tests may be required than the usual minimum.

3.9.3.2 Sampling Patterns

There are several ways in which sample locations may be selected for water content and unit weight tests. The simplest and least desirable method is for someone in the field to select locations at the time samples must be taken. This is undesirable because the selector may introduce a bias into the sampling pattern. For example, perhaps, on the previous project, soils of one particular color were troublesome. If the individual were to focus most of the tests on the current project on soils of that same color, a bias might be introduced.

A common method of selecting sample locations is to establish a grid pattern. The grid pattern is simple and ensures a high probability of locating defective areas as long as the defective areas are of a size greater than or equal to the spacing between the sampling points. It is important to stagger the grid patterns in successive lifts so that sampling points are not at the same location in each lift. One would not want to sample at the same location in successive lifts because repaired sample penetrations would be stacked on top of one another. A minimum horizontal separation of 3 m (10 ft) is suggested. A third alternative for selecting sampling points is to locate sampling points randomly. Tables and examples are given in Richardson (1992). It is recommended that no sampling point be located within 2 m (7 ft) of another sampling point. If a major portion of the area to be sampled has been omitted as a result of the random sampling process, CQA inspectors can add additional points to make sure that the area receives some testing. Random sampling is sometimes preferred on large projects where statistical procedures will be used to evaluate data. However, it can be demonstrated that for a given number of sampling points, a grid pattern will be more likely to detect a problem area, provided that the dimensions of the problem area are greater than or equal to the spacing between sampling points. If the problem area is smaller than the spacing between sampling points, the probability of locating the problem area is approximately the same with both a grid pattern and a random pattern of sampling.

No matter which method of determining sampling points is selected, it is imperative that CQA inspectors have the authority to require additional tests on any suspect area. The number of additional testing locations that are appropriate varies considerably from project to project.

3.9.3.3 Tests with Different Devices To Minimize Systematic Errors

Some methods of measurement may introduce a systematic error. For example, the nuclear device for measuring water content may consistently produce a water content measurement that is too high if there is an extraneous source of hydrogen atoms besides water in the soil. It is important that devices that may introduce a significant systematic error be periodically correlated with measurements that do not have such error. Water content measurement tests have the greatest potential for systematic error. Both the nuclear method and microwave oven-drying can produce significant systematic error under certain conditions. Therefore, it is recommended that if the nuclear method or any of the rapid methods of water content measurement (Table 3-2) are used to measure water content, periodic correlation tests should be made with conventional overnight oven-drying (ASTM D2216).

It is suggested that at the beginning of a project, at least 10 measurements of water content be made on representative samples of the site-specific soil using any rapid measurement method to be employed on the project as well as ASTM D2216.

After this initial correlation, it is suggested (see Table 3-10) that 1 in 10 rapid water content tests be cross-checked with conventional overnight oven-drying. At the completion of a project, a graph should be presented that correlates the measured water content with a rapid technique against the water content from conventional overnight oven-drying.

Some methods of unit weight measurement may also introduce bias. For example, the nuclear device may not be properly calibrated and could lead to measurement of a unit weight that is either too high or too low. It is recommended that unit weight be measured independently on occasion to provide a check against systematic errors. For example, if the nuclear device is the primary method of density measurement being used on a project, periodic measurements of density with the sand cone or rubber balloon device can be used to check the nuclear device. Again, a good practice is to perform about 10 comparative tests on representative soil before construction. During construction, 1 in every 20 density tests (see Table 3-10) should be checked with the sand cone or rubber balloon. A graph should be made of the unit weight measured with the nuclear device versus the unit weight measured with the sand cone or rubber balloon device to show the correlation. One could either plot dry unit weight or total unit weight for the correlation. Total unit weight is more sensible because the methods of measurement are actually total unit weight measurements; dry unit weight is calculated from the total unit weight and water content (Eq. 3-1).

3.9.3.4 Allowable Variations and Outliers

There are several reasons why a field water content or density test may produce a failing result (i.e., value outside of the specified range). Possible causes for a variation include a human error in measurement of water content or dry unit weight, natural variability of the soil or the compaction process leading to an anomaly at an isolated location, limitations in the sensitivity and repeatability of the test methods, or inadequate construction procedures that reflect broader scale deficiencies.

Measurement errors are made on every project. From time to time, it can be expected that CQC and CQA personnel will incorrectly measure either the water content or the dry unit weight. Periodic human errors are to be expected and should be addressed in the CQA plan.

If it is suspected that a test result is in error, the proper procedure for rectifying the error should be as follows. CQC or CQA personnel should return to the point where the questionable measurement was obtained. Several additional tests should be performed close to the location of the questionable test. If all of the repeat tests provide satisfactory results, the questionable test result may be disregarded as an error. CQA documents should specify the number of tests required to negate a blunder. It is recommended that approximately 3 passing tests be required to negate the results of a questionable test.

One of the main reasons why soil liners are built of multiple lifts is a realization that the construction process and the materials themselves vary. With multiple lifts, no one particular point in any one lift is especially significant, even if that point consists of unsatisfactory material or improperly compacted material. It should be expected that occasional deviations from construction specifications will be encountered for any soil liner. If one were to take enough soil samples, one can rest assured that a failing point on some scale would be located.

Measurement techniques for compacted soils are imperfect and produce variable results. Turnbull et al. (1966) discuss statistical quality control for compacted soils. Noorany (1990) describes three sites in the San Diego area for which nine testing laboratories measured water content and percent compaction on the same fill materials. The ranges in percent compaction were large: 81% to 97% for Site 1, 77% to 99% for Site 2, and 89% to 103% for Site 3.

Hilf (1991) summarizes statistical data from 72 earth dams; the data show that the standard deviation in water content is typically 1% to 2%, and the standard deviation in dry density is typically 0.3 to 0.6 kN/m^3 (2–4 pcf). Because the standard deviations are themselves on the same order as the allowable range of these parameters in many earthwork specifications, it is statistically inevitable that there will be some failing tests no matter how well built the soil liner is.

It is unrealistic to expect that 100% of all CQA tests will be in compliance with specifications. Occasional deviations should be anticipated. If there are only a few randomly located failures, the deviations in no way compromise the quality or integrity of a multiple-lift liner.

The CQA documents may provide an allowance for an occasional failing test. The documents may stipulate that failing tests not be permitted to be concentrated in any one lift or in any one area. It is recommended that a small percentage of failing tests be allowed rather than insisting on the unrealistic requirement that 100% of all tests meet project objectives. Statistically based requirements provide a convenient yet safe and reliable technique for handling occasional failing test results. However, statistically based methods require that enough data be generated to apply statistics reliably. Sufficient data to apply statistical methods may not be available, particularly in the early stages of a project.

Another approach is to allow a small percentage of outliers but to require repair of any area where the water content is significantly different or the dry unit weight is far too low. This approach is probably the simplest to implement—recommendations are summarized in Table 3-11.

3.9.3.5 Corrective Action

If it is determined that an area does not conform to specifications and that the area needs to be repaired, the first step is to define the extent of the area requiring repair. The recommended procedure is to require the contractor to repair the lift

Table 3-11. Recommended Maximum Percentage of Failing Compaction Tests

Parameter	*Maximum Allowable Percentage of Outliers*
Water content	3% and outliers not concentrated in one lift or area, and no water content less than 2% or more than 3% of the allowable value
Dry density	3% and outliers not concentrated in one lift or area, and no dry density less than 0.8 kN/m^3 (5 lb/ft^3) below the required value
Number of passes	5% and outliers not concentrated in one lift or area

of soil out to the limits defined by passing CQC and CQA tests. The contractor should not be allowed to guess at the extent of the area that requires repair. To define the limits of the area that requires repair, additional tests are often needed. Alternatively, if the contractor chooses not to request additional tests, the contractor should repair the area that extends from the failing test out to the boundaries defined by passing tests.

A relatively common problem is inadequate compaction of the soil. The contractor is usually able to rectify the problem with additional passes of the compactor over the problem area.

3.9.4 Hydraulic Conductivity Tests on Undisturbed Samples

Hydraulic conductivity tests are often performed on "undisturbed" samples of soil obtained from a single lift of compacted soil liner. Test specimens are trimmed from the samples and are permeated in the laboratory, usually following ASTM D5084. Compliance with the stated hydraulic conductivity criterion is checked. Occasionally, problems with sidewall leakage occur with extremely impermeable materials. Applying a thin layer of silicone vacuum grease to the sides of the specimen can be helpful in preventing sidewall leakage (Bowders et al. 2002).

This type of test is given far too much weight in most QA programs. Low hydraulic conductivity of samples taken from the liner is necessary for a well-constructed liner but is not sufficient to demonstrate that the large-scale, field hydraulic conductivity is adequately low. For example, Elsbury et al. (1990) measured hydraulic conductivities on undisturbed samples of a poorly constructed liner that averaged 1×10^{-9} cm/s, and yet the actual in-field value was 1×10^{-5} cm/s. The cause for the discrepancy was the existence of macroscale flow paths in the field that were not simulated in the small (75-mm- or 3-in.-diameter) laboratory test specimens.

Not only does the flow pattern through a 75-mm-diameter test specimen not necessarily reflect flow patterns on a larger field scale, but the process of obtaining a sample for testing inevitably disturbs the soil. Layers are distorted, and gross alterations occur if significant gravel is present in the soil. The process of pushing a sampling tube into the soil densifies the soil, which lowers its hydraulic conductivity. The harder and drier the soil, the greater the disturbance. As a result of these various factors, the large-scale, field hydraulic conductivity is almost always greater than or equal to the small-scale, laboratory-measured hydraulic conductivity. The difference between values from a small laboratory scale and a large field scale depends on the quality of construction; the better the quality of construction, the less the difference.

Laboratory hydraulic conductivity tests on undisturbed samples of compacted liner can be valuable in some situations. For instance, for soil–bentonite mixes, the laboratory test provides a useful check on whether enough bentonite has been added to the mix to achieve the desired hydraulic conductivity. For soil liners in which a test pad has not been constructed, the laboratory tests provide some verification that appropriate materials have been used and compaction was reasonable. But small-scale hydraulic conductivity tests *by themselves* do not prove that the liner is well constructed.

Laboratory hydraulic conductivity tests constitute a major inconvenience because the tests usually take at least several days, and sometimes a week or two, to complete. Their value as QA tools is greatly diminished by the long testing time; field construction personnel simply cannot wait for the results of the tests to proceed with construction, nor would the QA personnel necessarily want them to wait because opportunities exist for damage of the liner as a result of desiccation. Thus, one should carefully consider whether the laboratory hydraulic conductivity tests are truly needed for a given project and will serve a sufficiently useful purpose to make up for the inconvenience of this type of test.

Research has demonstrated that larger samples (≥300 mm in diameter) from field-compacted soils can give more reliable results than the usual 75-mm (3-in.) diameter samples (Benson et al. 1994). The following recommendations are made concerning the approach to using laboratory hydraulic conductivity tests for QA on field-compacted soils:

1. For gravelly soils or other soils that cannot be consistently sampled without causing significant disturbance, laboratory hydraulic conductivity tests should not be a part of the QA program because representative samples cannot realistically be obtained. A test pad (Section 3.11) is recommended to verify hydraulic conductivity.
2. If a test pad is constructed and it is demonstrated that the field-scale hydraulic conductivity is satisfactory on the test pad, the QA program for the actual soil liner should focus on establishing that the actual liner is built of similar materials and to equal or better standards compared to the test pad; laboratory hydraulic conductivity testing is not necessary to establish this.
3. If no test pad is constructed and it is believed that representative samples can be obtained for hydraulic conductivity testing, then laboratory hydraulic conductivity tests on undisturbed samples from the field are recommended.

3.9.4.1 Sampling for Hydraulic Conductivity Testing

A thin-walled tube is pushed into the soil to obtain a sample. Samples of soil should be taken in the manner that minimizes disturbance (such as described in ASTM D1587). Samples should be sealed and carefully stored to prevent drying and transported to the laboratory in a manner that minimizes soil disturbance (as described in ASTM D4220).

It is particularly important that the thin-walled sampling tube be pushed into the soil in the direction perpendicular to the plane of compaction. Many CQA inspectors will push the sampling tube into the soil using the blade of a bulldozer or compactor. This practice is not recommended because the sampling tube tends to rotate when it is pushed into the soil. The recommended way of sampling the soil is to push the sampling tube straight into the soil using a jack to achieve a smooth, straight push.

Sampling of gravelly soils for hydraulic conductivity testing is often a futile exercise. The gravel particles that are encountered by the sampling tube tend to tumble and shear during the push, which causes major disturbance of the soil sample. Our experience has been that QA/QC personnel may take several samples of gravelly soil before a suitable, intact sample is finally obtained; in these cases, the badly disturbed, gravelly samples are discarded. The process of discarding samples that contain too much gravel introduces bias into the process. Gravelly soils are not amenable to undisturbed sampling.

The sampling tube should not be pushed deeper than one lift (150 mm, or 6 in.) into the liner. A deeper push will create potential problems with sampling disturbance and will leave a crucial penetration that may represent a potential pathway for leakage. Sampling should be done lift by lift rather than at the completion of the liner.

3.9.4.2 Hydraulic Conductivity Testing

Hydraulic conductivity tests are performed using a flexible-wall permeameter and the procedures described in ASTM D5084. Inspectors should be careful to make sure that the effective confining stress used in the hydraulic conductivity test is not excessive. Application of excessive confining stress can produce an artificially low hydraulic conductivity. Kodikara and Rahman (2002) discuss theoretical and experimental assessments of the effect of hydraulic conductivity reduction with increasing hydraulic gradient and effective stress during testing. The CQA plan should prescribe the maximum effective confining stress that will be used; if none is specified, a value of 35 kPa (5 psi) is recommended.

Rapid hydraulic conductivity tests that give results quickly (some in fewer than 24 hours) are possible with special equipment (e.g., the flow pump method (Daniel 1994)). Only a few commercial laboratories have this type of equipment available.

3.9.4.3 Frequency of Testing

The CQA plan should stipulate the frequency of testing. Hydraulic conductivity tests are typically performed at a frequency of 3 tests/ha/lift (1 test/acre/lift) or, for very thick liners (≥1.2 m or 4 ft), every other lift. This is the recommended frequency of testing if hydraulic conductivity testing is required.

3.9.4.4 Outliers

The results of laboratory hydraulic conductivity tests performed for CQA purposes are often given far too much weight. A passing rate of 100% does not necessarily prove that the liner was well-built, yet some inexperienced individuals falsely believe this to be the case. Hydraulic conductivity tests are performed on small samples; even though small samples may have low hydraulic conductivity, inadequate construction can leave remnant macroscale defects, such as fissures and pockets of poorly compacted soil. The fundamental problem is that laboratory hydraulic conductivity tests are usually performed on 75-mm (3-in.) diameter samples, and these samples are too small to contain a representative distribution of macroscale defects (if any such defects are present). By the same token, an occasional failing test does not necessarily prove that a significant problem exists. Even on the best built liners, occasional failing test results should be anticipated.

It is recommended that a multiple-lift soil liner be considered acceptable even if a small percentage (approximately 5%) of the hydraulic conductivity tests fail. However, one should allow a small percentage of hydraulic conductivity failures only if the overall CQA program is thorough. Furthermore, it is recommended that failing samples have a hydraulic conductivity that is no greater than one-half to one order of magnitude above the target maximum value. If the hydraulic conductivity at a particular point is more than one-half to one order of magnitude too high, the zone should be retested or repaired regardless of how isolated the area is.

3.9.5 Repair of Holes from Sampling and Testing

A number of tests (e.g., from nuclear density tests and soil sampling for hydraulic conductivity tests) require that a penetration be made into a lift of compacted soil. It is extremely important that all penetrations be repaired. The recommended procedure for repair is as follows. The backfill material should first be selected. Backfill may consist of the soil liner material itself, granular or pelletized bentonite, or a mixture of bentonite and soil. The backfill material should be placed in the hole that requires repair with a loose lift thickness not exceeding about 50 mm (2 in.). The loose lift of soil should be tamped several times with a steel rod or other suitable device that compacts the backfill and ensures no bridging of material that would leave large air pockets. Next, a new lift of backfill should be placed and compacted. The process is repeated until the entire hole has been backfilled.

Because it is critical that holes be properly backfilled, it is recommended that periodic inspections and written records be made of the repair of holes. It is suggested that approximately 20% of all the repairs be inspected and that the backfill procedures be documented for these inspections. The inspector of the backfilling process should not be the same person who backfilled the hole.

3.9.6 Final Lift Thickness

Construction documents may place restrictions on the maximum allowable final (after-compaction) lift thickness. Typically, the maximum thickness is 150 mm (6 in.). One rationale for not using thick lifts is that more lifts add redundancy—imperfections in one lift are not likely to align with imperfections in adjacent lifts. Thus, for example, a 0.6-m (2-ft.) thick liner made up of four lifts of 150-mm (6-in.) thickness might be expected to offer better overall performance than three lifts of 225-mm (9-in.) thickness (Benson and Daniel 1993). Final elevation surveys should be used to establish thicknesses of completed earthwork segments. The specified maximum lift thickness is a nominal value. The actual value may be determined by surveys on the surface of each completed lift, but an acceptable practice (provided there is good CQA on loose lift thickness) is to survey the liner after construction and calculate the average thickness of each lift by dividing the total thickness by the number of lifts.

Tolerances should be specified on final lift thickness. Occasional outliers from these tolerances are not detrimental to the performance of a multilift liner. It is recommended by analogy to Table 3-9 that no more than 5% of the final lift thickness determinations be out of specification.

3.9.7 Pass/Fail Decision

After all CQA tests have been performed, a pass/fail decision must be made. Procedures for dealing with materials problems were discussed in Section 3.8.2.4. Procedures for correcting deficiencies in compaction of the soil were addressed in Section 3.9.3.5. A final pass/fail decision is made by the CQA engineer based on all the data and test results. The hydraulic conductivity test results may not be available for several days after construction of a lift has been completed. Sometimes the contractor proceeds at risk with placement of additional lifts before all test results are available. On occasion, construction of a liner proceeds without final results from a test pad on the assumption that results will be acceptable. If a "fail" decision is made at this late stage, the defective soil and any overlying materials that have been placed should be removed and replaced.

3.10 Protection of Compacted Soil

3.10.1 Desiccation

3.10.1.1 Preventive Measures

There are several ways to prevent compacted soil liner materials from desiccating. The soil may be rolled smooth with a steel-drummed roller to produce a thin, dense skin of soil on the surface. This thin skin of dense soil helps to minimize evaporation of water from the underlying material. However, the smooth-rolled surface should be scarified before placement of a new lift of soil.

An obvious preventive measure is to water the soil periodically. Care should be taken to deliver water uniformly to the soil and not to create zones of excessively wet soil. Adding water by hand is not usually recommended because water is not delivered uniformly to the soil.

An alternative preventive measure is to cover the soil temporarily with a geomembrane, moist geotextile, or moist soil. The geomembrane or geotextile should be weighted down with sandbags or other materials to prevent transfer of air between the geosynthetic cover and soil. If a geomembrane is used, care should be taken to ensure that the underlying soil does not become heated and desiccate; a light-colored geomembrane may be needed to prevent overheating. If a geomembrane is placed on a completed liner, the geomembrane can become warm during the day and cause the underlying soil liner to desiccate. To minimize this potential problem, the geomembrane should be covered as quickly as possible. CQA personnel should be concerned about possible desiccation if the geomembrane is left exposed for more than about two weeks.

One of the most effective means to limit desiccation of the completed soil liner is to place 150 to 300 mm of moist soil on the surface of the liner. If moist soil is placed over the soil liner, the moist soil is removed using grading equipment. If a geomembrane is to be placed on the surface of the soil liner, the layer of protective soil should be removed just before deployment of the geomembrane.

3.10.1.2 Observations

Visual observation is the best way to ensure that appropriate preventive measures have been taken to minimize desiccation. Inspectors should realize that soil liner materials can dry out quickly (sometimes in a matter of just a few hours). Inspectors should be aware that drying may occur over weekends, and provisions should be made to provide appropriate observations.

3.10.1.3 Tests

If there are questions about degree of desiccation, tests should be performed to determine the water content of the soil. The surface should be examined for cracks. If cracks penetrate more than about 50 mm (2 in.) below the surface, the cracks may be considered significant enough to require repair of the soil. If cracks penetrate more than 100 mm (4 in.), the entire lift may need reworking. A decrease in water content of 1 to 2 percentage points is not considered particularly serious and is within the general accuracy of testing. However, larger reductions in water content provide clear evidence that desiccation has taken place. If the water content of the soil has dropped by more than 1 to 2 percentage points, additional observation and tests are warranted. The physical condition of the soil should be carefully examined by digging shallow pits to determine the extent of obvious cracking. Laboratory hydraulic conductivity tests may also be used to evaluate desiccation, but great care must be taken to obtain a large, undisturbed, representative sample. The procedures described in Section 3.10.2.3 provide one possible means for obtaining high-quality samples.

If the soil has desiccated, CQA personnel may wish to compare the water content and dry unit weight of the desiccated soil with the acceptable range used to control the compaction process. If the water content or dry unit weight of the desiccated soil lies significantly outside the range required immediately after compaction, this problem may be a strong indicator that excessive desiccation has occurred. However, for soil liners placed at a high water content, the soil can dry and crack substantially, even though the water content has not dropped below the minimum value required during compaction. Thus, it is not correct to assume, just because the water content and dry unit weight of the desiccated soil are within the acceptable range established for compaction, that desiccation has not caused significant damage.

3.10.1.4 Corrective Action

If soil has been desiccated to a depth less than or equal to the thickness of a single lift, the desiccated lift may be disked, moistened, and recompacted. However, disking may produce large, hard clods of clay that will require pulverization. Also, it should be recognized that if the soil is wetted, time must be allowed for water to be absorbed into the clods of clay and hydration to take place uniformly. For this reason, it may be necessary to remove the desiccated soil from the construction area, to process the lift in a separate processing area, and to replace the soil accordingly.

3.10.2 Freezing Temperatures

3.10.2.1 Compacting Frozen Soil

Frozen soil should never be used to construct soil liners. Frozen soils form hard pieces that cannot be properly remolded and compacted. Inspectors should be on the lookout for frozen chunks of soil when construction takes place in freezing temperatures.

3.10.2.2 Protection after Freezing

Freezing of soil liner materials can produce significant increases in hydraulic conductivity. Soil liners must be protected from freezing before and after construction. If superficial freezing takes place on the surface of a lift of soil, the surface may be scarified and recompacted. If an entire lift has been frozen, the entire lift should be disked, pulverized, and recompacted. If the soil is frozen to a depth greater than one lift, it may be necessary to strip away and replace the frozen material.

3.10.2.3 Investigating Possible Frost Damage

Inspectors cannot determine from an examination of the surface the depth to which freezing took place. The extent of damage is difficult to determine even from examination of the soil below the surface. Freezing temperatures cause the development of microcracks in the soil. Tests can be performed to assess the vulnerability of soils to damage from freeze–thaw (per Othman et al. (1994) or ASTM D6035). Soils that have been damaged by frost action develop fine cracks that lead to the formation of chunks of soil when the soil is excavated. The pushing of a sampling tube into the soil will tend to close these cracks and mask the damaging effects of frost on hydraulic conductivity. The recommended procedure for evaluating possible frost damage to soil liners involves three steps:

1. Measure the water content of the soil within and beneath the zone of suspected frost damage. Density may also be measured, but freeze–thaw has little effect on density and may actually cause an increase in dry unit weight. Freeze–thaw is often accompanied by desiccation; water content measurements will help to determine whether drying has taken place.
2. Investigate the morphology of the soil by digging into the soil and examining its condition. Soil damaged by freezing usually contains hairline cracks, and the soil breaks apart in chunks along larger cracks caused by freeze–thaw. Soil that has not been frozen should not have tiny cracks, nor should it break apart in small chunks. The morphology of the soil should be examined by excavat-

ing a small pit into the soil liner and peeling off sections from the wall of the pit. A distinct depth may be obvious; above this depth the soil breaks into chunks along frost-induced cracks, and below this depth there is no evidence of cracks produced by freezing.

3. One or more samples of soil should be carefully hand-trimmed for hydraulic conductivity testing. The soil is usually trimmed with the aid of a sharpened section of tube of the appropriate inside diameter. The tube is set on the soil surface with the sharpened end facing downward, soil is trimmed away near the sharpened edge of the trimming ring, the tube is pushed a few millimeters into the soil, and the trimming is repeated. Samples may be taken at several depths to delineate the depth to which freeze–thaw damage occurred. The minimum diameter of a cylindrical test specimen should be 300 mm (12 in.). Small test specimens, e.g., 75-mm (3-in.) diameter specimens, should not be used because freeze–thaw can create morphological structure in the soil on a scale too large to permit representative testing with small samples (Othman et al. 1994). Hydraulic conductivity tests should be performed as described in ASTM D5084. The effective confining stress should not exceed the smallest vertical effective stress to which the soil will be subjected in the field, which is usually the stress at the beginning of service for liners. If no compressive stress is specified, a value of 35 kPa (5 psi) is recommended for both liner and cover system.

The test pit and all other penetrations should be carefully backfilled by placing soil in lifts and compacting the lifts. The sides of the test pit should be sloped so that the compactor can penetrate through to newly placed material without interference from the walls of the pit.

3.10.2.4 Repair

If damage is restricted to a single lift, the lift may be disked, processed to adjust water content or to reduce clod size, if necessary, and recompacted. If the damage extends deeper, damaged materials should be excavated and replaced.

3.10.3 Excessive Surface Water

In some cases, exposed lifts of liner material, or the completed liner, are subjected to heavy rains that soften the soil. Surface water creates a problem if the surface is uneven; for example, if a footed roller has been used and the surface has not been smooth-rolled with a smooth, steel-wheeled roller, numerous small puddles of water will develop in the depressions. Puddles of water should be removed before further lifts of material, or other components of the liner or cover system, are constructed. The material should be disked repeatedly to allow the soil to dry, and when the soil is at the proper water content, the soil should be compacted. Alternatively, the wet soil may be removed and replaced.

Even if puddles have not formed, the soils may be too soft to permit construction equipment to operate on the soil without creating ruts. To deal with this problem, the soil may be allowed to dry slightly by natural processes (but care must be taken to ensure that it does not dry too much and does not crack excessively during the drying process). Alternatively, the soil may be disked, allowed to dry while it is periodically disked, and then compacted.

If soil is reworked and recompacted, QA/QC tests should be performed at the same frequency as for the rest of the project. However, if the area requiring reworking is very small (e.g., in a sump), tests should be performed in the confined area to confirm proper compaction even if this process requires sampling at a greater frequency.

3.11 CQA Procedures for Test Pads

3.11.1 Purpose of Test Pads

The purpose of a test pad is to verify that the materials and methods of construction proposed for a project will lead to a soil liner with the required large-scale, in situ hydraulic conductivity. Unfortunately, it is impractical to perform large-scale hydraulic conductivity tests on the actual soil liner for two reasons: (1) the testing would produce significant physical damage to the liner and the repair of the damage would be questionable; and (2) the time required to complete the testing would be too long; the liner could become damaged because of desiccation while one waited for the test results.

A test pad may also be used to demonstrate that unusual materials or construction procedures will work. The process of constructing and testing a test pad is usually a good learning experience for the contractor and CQC/CQA personnel; overall quality of a project is usually elevated as a result of building and testing the test pad.

A test pad is constructed with the soil liner materials proposed for a project using preprocessing procedures, construction equipment, and construction practices that are proposed for the actual liner. If the required hydraulic conductivity is demonstrated for the test pad, it is assumed that the actual liner will have a similar hydraulic conductivity, provided the actual liner is built of similar materials and to standards that equal or exceed those used in building the test pad. If a test pad is constructed and hydraulic conductivity is verified on the test pad, a key goal of CQA/CQC for the actual liner is to verify that the actual liner is built of similar materials and to standards that equal or exceed those used in building the test pad.

3.11.2 Dimensions

Test pads (see Figure 3-33) usually measure about 10 to 15 m wide by 15 to 30 m long. The width of the test pad is typically at least 3 to 4 times the width of the compaction equipment, and the length must be adequate for the compactor to reach normal operating speed in the test area. The thickness of a test pad is usually no less than the thickness of the soil liner proposed for a facility, but it may be as little as 0.6 to 0.9 m (2–3 feet) if thicker liners are to be used at full scale. A

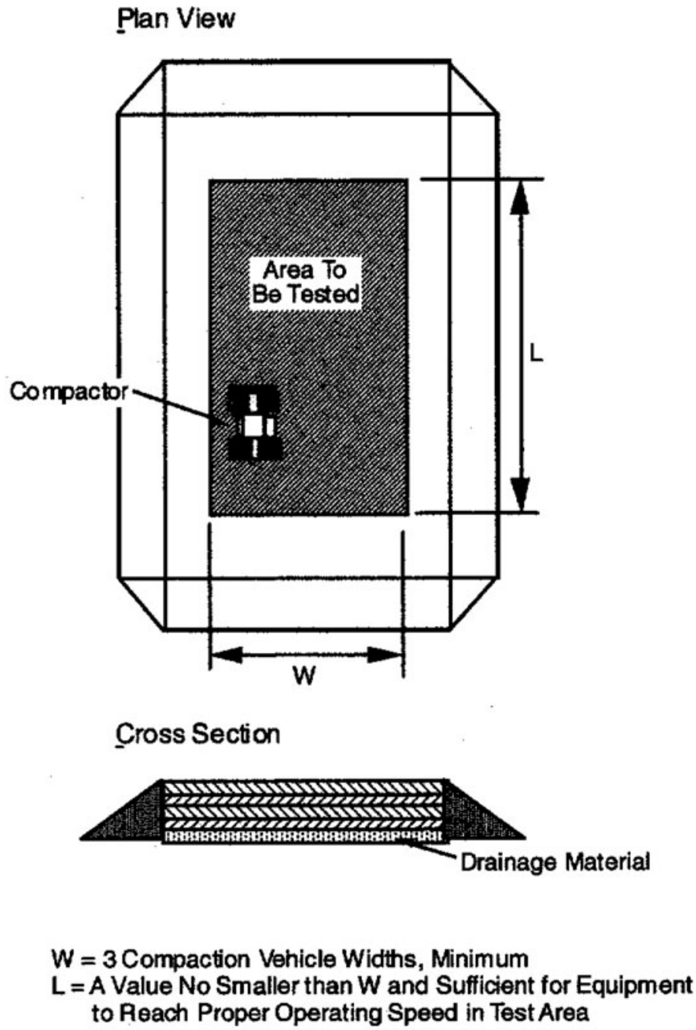

Figure 3-33. Schematic Diagram of Soil Liner Test Pad.

freely draining material such as sand is often placed beneath the test pad to provide a known boundary condition in case infiltrating water from a surface hydraulic conductivity test (e.g., a sealed double-ring infiltrometer) reaches the base of the liner. The drainage layer may be drained with a pipe or other means. However, infiltrating water will not reach the drainage layer if the hydraulic conductivity is low; the drainage pipe would only convey water if the hydraulic conductivity turns out to be high. The sand drainage material may not provide adequate foundation support for the first lift of soil liner unless the sand is compacted sufficiently. Also, the first lift of soil liner material on the drainage layer is often viewed as a sacrificial lift and is only compacted nominally to avoid mixing clayey soil in with the drainage material.

3.11.3 Materials

The test pad is constructed of the same materials that are proposed for the actual project. Processing equipment and procedures should be identical, too. The same types of CQC/CQA tests that will be used for the soil liner are performed on the test pad materials. If more than one type of material will be used, one test pad should be constructed for each type of material.

3.11.4 Construction

It is recommended that test strips be built before constructing the test pad. Test strips allow for the detection of obvious problems and provide an opportunity to fine-tune soil specifications, equipment selection, and procedures so that problems are minimized and the probability of the required hydraulic conductivity being achieved in the test pad is maximized. Test strips are typically two lifts thick, one to two equipment widths wide, and about 10 m (30 ft) long.

The test pad is built using the same loose lift thickness, type of compactor, weight of compactor, operating speed, and minimum number of passes that are proposed for the actual soil liner. It is important that the test pad not be built to standards that will exceed those used in building the actual liner. For example, if the test pad is subjected to 15 passes of the compactor, the actual soil liner should also receive at least 15 passes. It is critical that CQA personnel document the construction practices that are used in building the test pad. It is best if the same contractor builds the test pad and actual liner so that experience gained from the test pad process is not lost. The same applies to CQC and CQA personnel.

3.11.5 Protection

The test pad must be protected from desiccation, freezing, and erosion in the area where in situ hydraulic conductivity testing is planned. The recommended procedure is to cover the test pad with a sheet of white or clear plastic and then either spread a thin layer of soil on the plastic if no rain is anticipated or, if rain may create an undesirably muddy surface, cover the plastic with hay or straw.

3.11.6 Tests and Observations

The same types of CQA tests that are planned for the actual liner are usually performed on the test pad; however, the frequency of testing is usually somewhat greater for the test pad. Material tests such as liquid limit, plastic limit, and percent fines are often performed at the rate of one per lift. Several water content–density tests are usually performed per lift on the compacted soil. A typical rate of testing would be one water content–density test for each 40 m^2 (400 ft^2). The CQA plan should describe the testing frequency for the test pad.

There is a danger in overtesting the test pad; excessive testing could lead to a greater degree of construction control in the test pad than in the actual liner. The purpose of the test pad is to verify that the materials and methods of construction proposed for a project can result in compliance with performance objectives concerning hydraulic conductivity. Too much control over the construction of the test pad runs counter to this objective.

3.11.7 In Situ Hydraulic Conductivity

3.11.7.1 Sealed Double-Ring Infiltrometer

The most common method of measuring in situ hydraulic conductivity on test pads is the sealed double-ring infiltrometer (SDRI). A schematic diagram of the SDRI is shown Figure 3-34. The test procedure is described in ASTM D5093.

With this method, the quantity of water that flows into the test pad over a known period of time is measured. This flow rate, which is called the infiltration rate (I), is computed as follows:

$$I = Q/At \tag{3-9}$$

where Q is the quantity of water entering the surface of the soil through a cross-sectional area A and over a period of time t.

Hydraulic conductivity (K) is computed from the infiltration rate and hydraulic gradient (i) as follows:

$$K = I/i \tag{3-10}$$

Three procedures have been used to compute the hydraulic gradient. These procedures are called (1) apparent gradient method, (2) suction head method, and (3) wetting front method. The equation for computing hydraulic gradient from each method is shown in Figure 3-35.

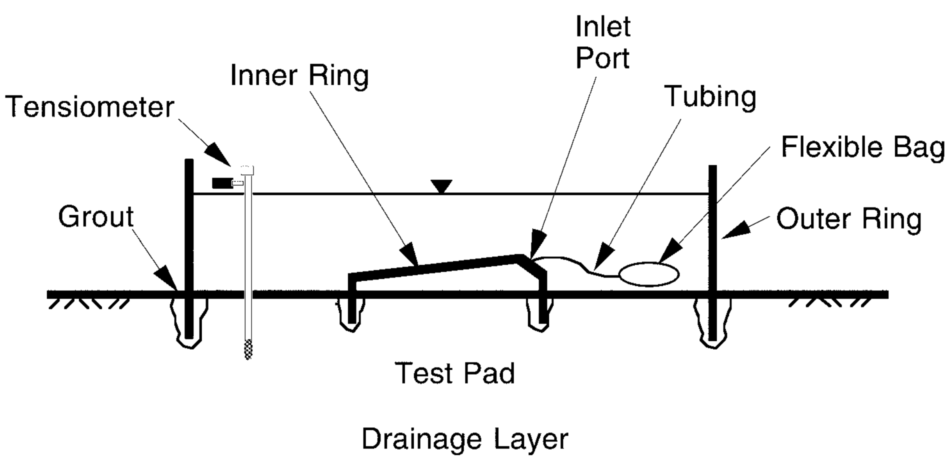

Figure 3-34. Schematic Diagram of Sealed Double-Ring Infiltrometer (SDRI).

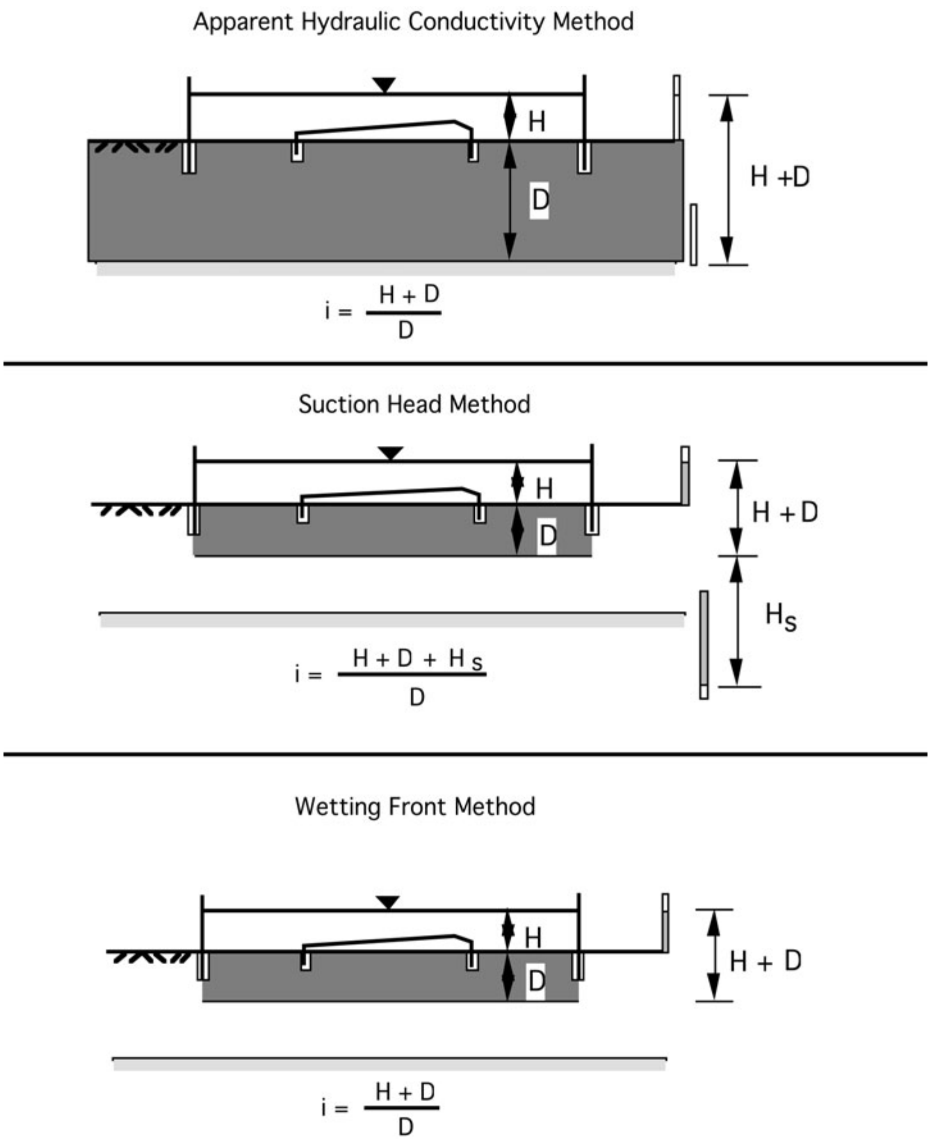

Figure 3-35. Three Procedures for Computing Hydraulic Gradient from Infiltration Test.

The apparent gradient method is the most conservative of the three methods because this method yields the lowest estimate of i and, therefore, the highest estimate of hydraulic conductivity. The apparent gradient method assumes that the test pad is fully soaked with water over the entire depth of the test pad. For relatively permeable test pads, the assumption of full soaking is reasonable, but for soil liners with $K < 1 \times 10^{-7}$ cm/s, the assumption of full soaking is excessively conservative and should not be used unless verified.

The second and most widely used method is the wetting front method. The wetting front is assumed to partly penetrate the test pad (Figure 3-35), and the water pressure at the wetting front is conservatively assumed to equal atmospheric pressure. Tensiometers are used to monitor the depth of wetting of the soil over time, and the variation of water content with depth is determined at the end of the test. The wetting front method is conservative but in most cases not excessively so. The wetting front method is the method that is usually recommended.

The third method, called the suction head method, is the same as the wetting front method, except that the water pressure at the wetting front is not assumed to be atmospheric pressure. The suction head (which is defined as the negative of the pressure head) at the wetting front is H_s and is added to the static head of water in the infiltration ring to calculate hydraulic gradient (Figure 3-35). The suction head H_s is identical to the wetting front suction head used in analyzing water infiltration with the Green-Ampt theory. The suction head H_s is *not* the ambient suction head in the unsaturated soil and is generally difficult to determine (Brakensiek 1977). Reimbold (1988) determined that H_s is close to zero for two compacted soil liner materials. Wang and Benson (1995) determined that the wetting front suction varied with soil, compaction water content, and method of data interpretation, but were generally 0 to 1 m. Because proper determination of H_s is difficult, the suction head method is not recommended unless the testing personnel take the time and make the effort to determine H_s reliably. Typically, the inclusion of a small wetting front suction head in the analysis will decrease the value of calculated hydraulic conductivity by a factor of roughly 1.5 (sometimes more). If the hydraulic conductivity is just above the regulatory limit when the wetting front suction head is assumed to be 0, including the wetting front suction head in the analysis may result in a passing rather than failing result.

Corrections may be made to account for various factors. For example, if the soil swells, some of the water that infiltrated into the soil is absorbed into the expanded soil. No consensus exists on various corrections, and these results should be evaluated case by case.

3.11.7.2 Two-Stage Borehole Test

The two-stage borehole test (ASTM D6391) was developed by Gordon Boutwell. The device is installed by drilling a hole (which is typically 100–150 mm in diameter), placing a casing in the hole, and sealing the annular space between the casing and borehole with grout as shown in Figure 3-36. A series of falling head tests is performed, and the hydraulic conductivity from this first stage (k_1) is computed. Stage 1 is complete when k_1 ceases to change significantly. The maximum vertical hydraulic conductivity may be computed by assuming that the vertical hydraulic conductivity is equal to k_1. However, the test may be continued for a second stage by removing the top of the casing and extending the hole below the casing as shown in Figure 3-36(b). The casing is reassembled, the device is again filled with water, and falling head tests are performed to determine the hydraulic conductivity from stage 2 (k_2). Both horizontal and vertical hydraulic conductivity may be computed from the values of k_1 and k_2. Further details on methods of calculation

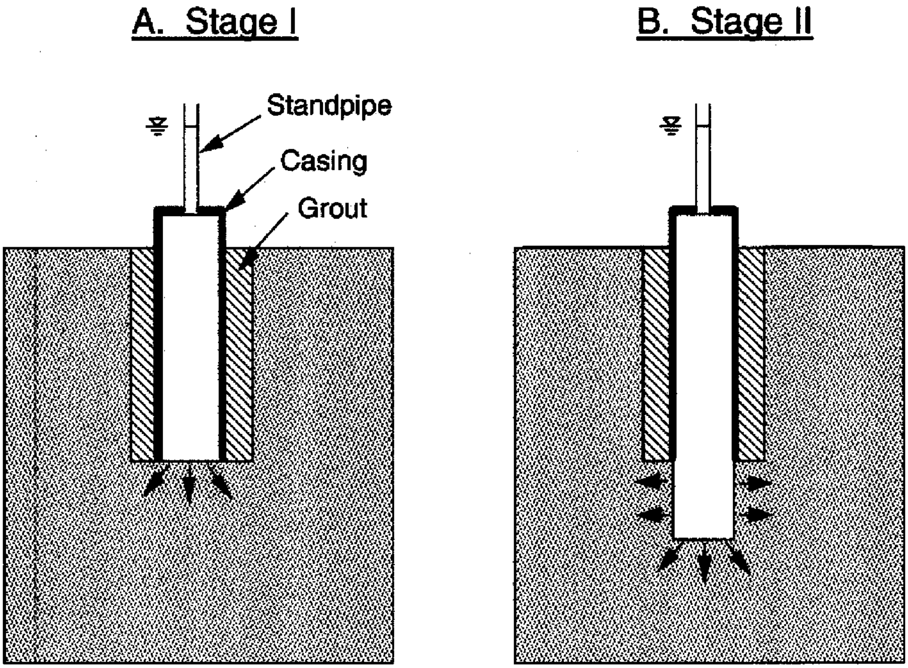

Figure 3-36. Schematic Diagram of Two-Stage Borehole Test.

are provided by Boutwell and Tsai (1992), although ASTM D6391 is recommended for practical use.

The two-stage borehole test permeates a smaller volume of soil than the sealed double-ring infiltrometer. At the present time, it is recommended that at least five two-stage borehole tests be performed on a test pad if the two-stage test is used. If five two-stage borehole tests are performed, then one might require that all five of the measured vertical hydraulic conductivities be less than or equal to the required maximum hydraulic conductivity for the soil liner. However, if one of the tests slightly fails, this is not cause for concern.

3.11.7.3 Other Field Tests

Several other methods of in situ hydraulic conductivity testing are available for soil liners. These methods include open infiltrometers, borehole tests with a constant water level in the borehole, porous probes, and air-entry permeameters. The methods are described by Daniel (1989) but are much less commonly used than the SDRI and two-stage borehole test.

3.11.7.4 Laboratory Tests

Laboratory hydraulic conductivity tests may be performed for three reasons:

1. If a large sample of soil is taken from the field and permeated in the laboratory, the result may be representative of field-scale hydraulic conductivity. The

question of how large the laboratory test specimen needs to be lacks a clear-cut answer, but experience has generally shown that a specimen with a diameter of approximately 300 mm (12 in.) may be sufficiently large (Benson et al. 1994).

2. If laboratory hydraulic conductivity tests are a required component of QA/QC for the actual liner, the same sampling and testing procedures are used for the test pad. Usually, undisturbed soil samples are obtained following the procedures outlined in ASTM D1587, and soil test specimens with diameters of approximately 75 mm (3 in.) are permeated in flexible-wall permeameters in accordance with ASTM D5084.
3. Laboratory tests may be used to determine the effect of compressive stress on hydraulic conductivity. Field hydraulic conductivity tests are usually conducted on a test pad that is subjected to a small compressive stress. Laboratory tests may be used to estimate the field hydraulic conductivity at larger compressive stresses. The shape of the field curve is assumed to be the same as the laboratory curve. This is a conservative assumption; the actual field hydraulic conductivity is expected to lie on or below the assumed curve.

3.11.8 Documentation

A report should be prepared that describes all of the test results from the test pad. The test pad documentation provides a basis for comparison between test pad results and the CQA data developed on an actual construction project.

3.12 Final Approval

Upon completion of the soil liner, the soil liner should be accepted and approved by the CQA engineer before deployment or construction of the next overlying layer.

3.13 References

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CHAPTER 4

Geomembranes

This chapter focuses on the manufacturing quality assurance (MQA) aspects of geomembrane formulation, manufacture, and fabrication, as well as the construction quality assurance (CQA) aspects of the complete installation of the geomembranes in the field. CQA includes seaming and joining as well as protection and backfilling. In early literature, these barrier materials were called flexible membrane liners (FMLs), but the currently accepted term *geomembranes* will be used throughout this book.

The specific geomembranes discussed herein are those used most often currently. There are, however, comments on other polymer types that are used. Aspects of quality assurance of these materials can be inferred from information contained in this book.

4.1 Types of Geomembranes and Their Formulations

All geomembranes are actually formulations of a parent resin (from which they derive their generic name) and varying amounts of other ingredients. The most commonly used geomembranes for solid- and liquid-waste containment are listed in Table 4-1. They are listed according to their commonly referenced acronyms, which will be explained in the text. Other geomembranes in limited use or under initial field trials will also be mentioned where appropriate but will be covered in less detail than the types listed below.

It should also be recognized that Table 4-1 and the references to it in the text are meant to reflect on the current state of the practice. The values mentioned are not meant to be prescriptive, and future research and development may result in substantial changes.

4.1.1 High-Density Polyethylene (HDPE)

As noted in Table 4-1, high density polyethylene (HDPE) geomembranes are made from polyethylene resin, carbon black, and additives; the additives are processing aids and long-term antioxidants. HDPE geomembranes are smooth on both sides, smooth on one side and textured on the other, or textured on both sides.

Table 4-1. Types of Commonly Used Geomembranes and Their Approximate Weight Percentage Formulations

Type	*Resin*	*Plasticizer*	*Fillers*	*Carbon Black*	*Additives*
HDPE	95–98	0	0	2–3	0.25–1
LLDPE	94–96	0	0	1–3	0.5–4
fPP	85–98	0	0–13	2–4	0.5–2
PVC	50–70	25–35	0–10	2–5	2–5
CSPE	40–60	0	40–50	5–10	5–15
EPDM	25–30	0	20–40	20–40	1–5

Notes: This table should not be directly used for MQA or CQA documents because the authors of the book do not intend to provide prescriptive formulations for manufacturers and their respective geomembrane products. HDPE, high-density polyethylene; LLDPE, linear low-density polyethylene; fPP, flexible polypropylene; PVC, polyvinyl chloride; CSPE, chlorosulfonated polyethylene; and EPDM, ethylene propylene diene terpolymer.

4.1.1.1 Resin

The polyethylene resin used for HDPE geomembranes is prepared by low-pressure polymerization of ethylene as the principal monomer with the characteristics listed in ASTM D1248. As seen in Figure 4-1, the resin is usually supplied to the manufacturer or formulator in a clear, colorless pellet form, which is then mixed with the master batch (carbon black and antioxidants in a carrier resin) and "let down" to meet the designated formulation.

Regarding the preparation of a specification or MQA document for the resin component of an HDPE geomembrane, the following items should be considered:

1. The polyethylene resin, which is covered in ASTM D1248, is to be made from virgin, uncontaminated ingredients.
2. The quality control (QC) tests performed on the incoming resin will typically include density, either ASTM D792 or D1505, and melt flow index, which follows ASTM D1238.
3. Typical natural densities of the various resins used are between 0.932 and 0.940 g/cm^3. According to ASTM D1248, this geomembrane is Type II polyethylene and is classified as medium-density polyethylene.
4. Typical melt flow index values are between 0.05 and 1.0 g per 10 min using ASTM D1238, Condition 190/2.16.
5. Other tests that can be considered for QC of the resin are melt flow ratio (comparing high- to low-weight melt flow values), notched constant tensile load test (per ASTM D5397), and a single-point notched constant load test (per ASTM D5397, Appendix A). The last two stress crack resistance tests require a plaque to be made from the resin from which test specimens are taken. The single-point notched constant load test is then performed at 30% yield strength, and the test specimens are recommended not to fail within a specified time limit.

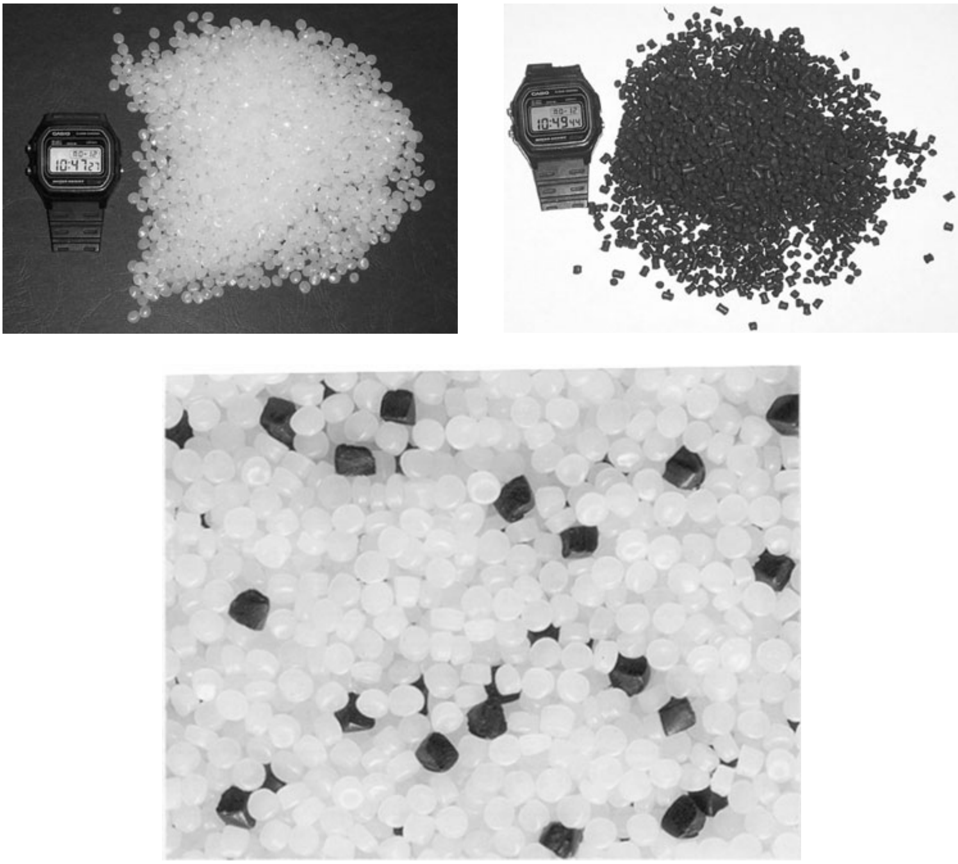


Figure 4-1. HDPE Resin Pellets (Left) and Master Batch (Right) and Approximate Mixed Proportions (Bottom).

6. Additional QC certification procedures (if any) of the manufacturer should be implemented and followed.
7. The frequency of performing each of the preceding tests should be covered in the manufacturer's QC document and should be implemented and followed.
8. An HDPE geomembrane formulation should consist of at least 95% polyethylene resin. As seen in Table 4-1, the balance is carbon black and additives consisting of stabilizers and antioxidants. No fillers, extenders, or other materials should be mixed into the formulation.
9. By adding carbon black and additives to the resin, the density of the final formulation is generally 0.941–0.950 g/cm^3. Because values greater than 0.941 g/cm^3 are in the high-density polyethylene category according to ASTM D1248, geomembranes of this type are commonly referred to in the industry as high-density polyethylene (HDPE).
10. Regrind or rework chips (which have been previously processed by the same manufacturer but have never been used in the field), are often added to the extruder during processing. This topic will be discussed more fully later.

11. Reclaimed, recycled, or postconsumer material (which is polymer material that has seen previous service life) should never be allowed in the formulation in any quantity.

4.1.1.2 Carbon Black

Carbon black is added into an HDPE geomembrane formulation for general stabilization, particularly for UV light stabilization. It is generally added (called "letdown") as a preformulated concentrate in pellet form. In this master batch is the carbon black and possibly the antioxidants contained in a carrier resin (usually a low-density polyethylene). Figure 4-1 shows a photograph of the concentrated pellets and the subsequent mixture with resin, which generally consists of the designated amount of carbon black and antioxidant additives.

Regarding the preparation of a specification or MQA document for the carbon black component of HDPE geomembranes, the following items should be considered.

1. The carbon black used in HDPE geomembranes should be a Group 5 category or lower, as described in ASTM D1765.
2. Typical amounts of carbon black are from 2.0 to 3.0% by weight, per ASTM D1603. Values less than 2.0% do not appear to give adequate long-term UV protection, whereas values greater than 3.0% begin to adversely affect physical and mechanical properties.
3. Current carbon black dispersion requirements in the final HDPE geomembrane are based on ASTM D5596. This test method uses a thin section of the finished geomembrane and examines it under a magnification of 100×. By comparing the microscopic view to a chart, a laboratory technician can estimate the degree of dispersion. Specifications are developed around Categories 1, 2, and (perhaps) 3.
4. The type of carrier resin for the carbon black and antioxidant concentrate master batch should be identified.

4.1.1.3 Additives

Additives are introduced into an HDPE geomembrane formulation for oxidation prevention, long-term durability, and as a lubricant or processing aid during manufacturing. It is difficult to write a specification for HDPE geomembranes around a particular additive or group of additives because they are generally proprietary. Furthermore, there is ongoing research and development in this area; thus, additives are subject to change on a regular basis.

If additives are included in a specification or MQA document, the description must be general as to the type and amount. However, the amount can probably be bracketed to an upper and lower value.

1. Although generally not part of a specification or MQA document, the general nature of the additive package used in the HDPE compound may be requested of the manufacturer.

2. The indirect assessment of additives remaining in the manufactured geomembrane is by the oxidative induction time (OIT) test. Both standard OIT, per ASTM D3895, and high-pressure OIT, per ASTM D5885, are used in this regard.

4.1.2 Linear Low-Density Polyethylene (LLDPE)

As with HDPE, linear low-density polyethylene (LLDPE) geomembranes are made from polyethylene resin, carbon black, and additives for processing stabilization and long-term antioxidation. In the low-pressure polymerization of LLDPE, the random incorporation of alpha olefin comonomers produces sufficient short-chain branching to yield densities in the range of 0.915 to 0.930 g/cm^3. This range results in resin properties quite different from HDPE. In particular, LLDPE has lower density, greater flexibility, less tendency toward stress cracking, greater elongation at break (both in-plane and out-of-plane), and lower modulus values at all levels of elongation. Like HDPE, LLDPE geomembranes are smooth on both sides, smooth on one side and textured on the other, or textured on both sides.

4.1.2.1 Resin

The polyethylene resin used for LLDPE geomembranes is a linear polymer of ethylene with other alpha olefins. As with HDPE, the resin is generally supplied to the manufacturer in the form of pellets that look similar to those shown in Figure 4-1.

Some specification or MQA document items for LLDPE resins follow:

1. The linear low-density polyethylene resin is to be made from completely virgin materials. The natural density of the resin is usually between 0.915 and 0.930 g/cm^3.
2. An LLDPE geomembrane formulation should consist of at least 94% polymer resin. As seen in Table 4-1, the balance is carbon black and additives for processing stabilization and long-term antioxidation.
3. Typical QC tests for LLDPE resin are density, via ASTM D792 or D1505, and melt flow index, via ASTM D1238.
4. Additional QC certification procedures (if any) of the manufacturer should be implemented and followed.
5. The frequency of performing each of the preceding tests should be covered in the MQC plan, and the plan should be implemented and followed.
6. Regrind or rework chips (which have been previously processed by the same manufacturer but never used as a geomembrane) are often added to the formulation during processing. This topic will be more fully discussed later.
7. Reclaimed, recycled, or postconsumer material (which is polymer that has seen previous service life) should never be allowed in the formulation in any quantity.

4.1.2.2 Carbon Black

Carbon black is added to LLDPE geomembrane formulations for general stabilization, particularly for UV light stabilization. It is generally added as a preformulated concentrate in pellet form (recall Figure 4-1).

Some items to be included in a specification or MQA document follow:

1. The carbon black used in LLDPE geomembranes should be a Group 5 category or lower, as defined in ASTM D1765.
2. Typical amounts of carbon black are from 1.0 to 3.0% by weight as per ASTM D1603. Values less than 1.0% do not appear to give adequate long-term UV protection, whereas values greater than 3.0% begin to affect physical and mechanical properties negatively.
3. Current carbon black dispersion requirements in the final LLDPE geomembrane are based on ASTM D5596. This test method uses a thin section of the finished geomembrane and examines it under a magnification of 100×. By comparing the microscopic view to a chart, laboratory technicians can estimate the degree of dispersion. Specifications are developed around Categories 1, 2, and (perhaps) 3.
4. The type of carrier resin of the concentrate should be identified, and a statement as to its past successful use can be requested.

4.1.2.3 Additives

Additives are introduced into an LLDPE formulation for antioxidation, long-term durability, and as a lubricant or processing aid during manufacturing. It is quite difficult to write a specification for LLDPE geomembranes around a particular additive or group of additives because they are generally proprietary. Furthermore, there is ongoing research and development in this area; thus, additives are subject to change over time.

If additives are included in a specification or MQA document, the description must be general as to the type and amount. However, the amount can probably be bracketed to an upper value.

1. Although generally not part of a specification or MQA document, the general nature of the additive package used in the LLDPE compound may be required of the manufacturer.
2. The indirect assessment of additives remaining in the manufactured geomembrane is by the OIT test. Both standard OIT, per ASTM D3895, and high-pressure OIT, per ASTM D5885, are used in this regard.

4.1.3 Flexible Polypropylene (fPP)

As seen in Table 4-1, flexible polypropylene (fPP) geomembranes are made from polypropylene resin, carbon black, and additives. For nonblack geomembranes, TiO_2 is added along with the colorant in amounts up to 13%. As defined in ASTM D4439, flexible polypropylene is "a material produced by polymerization of propylene with or without other alpha olefin monomers having a 2% secant modulus of less than 300 MPa (40,000 lb/in.^{2}) as determined by ASTM D5323." This polymer results in properties with significantly greater flexibility than HDPE and slightly greater than LLDPE. Flexible polypropylene geomembranes are either nonreinforced (as is always the case for HDPE and LLDPE) or reinforced with a fabric scrim between individual plies of the material. Reinforced fPP is designated as fPP-R. Occasionally, a light texturing is associated with the nonreinforced product.

4.1.3.1 Resin

The polypropylene resin used for fPP geomembranes is a linear polymer of ethylene with other alpha olefins. As with HDPE and LLDPE, the resin is generally supplied to the manufacturer in the form of pellets that look similar to those shown in Figure 4-1.

Some specification or MQA document items for fPP resins follow:

1. The polypropylene resin is to be made from completely virgin materials. The natural density of the resin is usually between 0.890 and 0.930 g/cm^3.
2. An fPP geomembrane formulation should consist of 85 to 98% polymer resin. As seen in Table 4-1, the balance is carbon black, colorants, and additives. Note that a roofing membrane, called thermoplastic polyolefin, or TPO, is a formulation that has at least 50% fPP included. The remainder, however, can consist of numerous other blended resins. TPO should not be used as a geomembrane in landfill containment systems.
3. Typical QC tests for fPP resin are density, via ASTM D792 or D1505, and melt flow index, via ASTM D1238.
4. Additional QC certification procedures of the manufacturer (if any) should be implemented and followed.
5. The frequency of performing each of the preceding tests should be covered in the MQC plan, and the plan should be implemented and followed.
6. Regrind or rework chips (which have been previously processed by the same manufacturer but never used as a geomembrane) are often added to the formulation during processing. This topic will be more fully discussed later.
7. Reclaimed, recycled, or postconsumer material (which is polymer that has seen previous service life) should never be allowed in any quantity.

4.1.3.2 Carbon Black or Colorants

Carbon black is usually added to fPP geomembrane formulations for general stabilization, particularly for UV light stabilization. It is generally added as a preformulated concentrate in pellet form (recall Figure 4-1). Alternatively, various colorants can be used to obtain a nonblack material.

Some items to be included in a specification or MQA document follow:

1. The carbon black used in fPP geomembranes should be a Group 3 category or lower, as defined in ASTM D1765.
2. Typical amounts of carbon black are from 2.0 to 4.0% by weight, per ASTM D1603. Values less than 2.0% do not appear to give adequate long-term UV protection, whereas values greater than 4.0% begin to affect physical and mechanical properties negatively.

3. Current carbon black dispersion requirements in the final fPP geomembrane are based on ASTM D5596. This test method uses a thin section of the finished geomembrane and examines it under a magnification of 100×. By comparing the microscopic view to a chart, laboratory technicians can estimate the degree of dispersion. Specifications are developed around Categories 1, 2, and (perhaps) 3.
4. Flexible polypropylene geomembranes are often required to be a color other than black. If this is the case, carbon black is not used; instead, a colorant (tan and green additives are common) is used. The nature and percentage of the colorant can be required of the manufacturer.
5. The type of carrier resin of the additive package should be identified, and a statement as to its past successful use can be requested.

4.1.3.3 Additives

Additives are introduced into an fPP formulation for antioxidation, long-term durability, and as a lubricant or processing aid during manufacturing. It is difficult to write a specification for fPP geomembranes around a particular additive or group of additives because they are generally proprietary. Furthermore, there is ongoing research and development in this area, and thus additives are subject to change over time.

If additives are included in a specification or MQA document, the description must be general as to the type and amount. However, the amount can probably be bracketed to an upper value.

1. Although generally not part of a specification or MQA document, the general nature of the additive package used in the fPP compound may be required of the manufacturer.
2. The indirect assessment of additives remaining in the manufactured geomembrane is by the OIT test. Both standard OIT, per ASTM D3895, and high-pressure OIT, per ASTM D5885, are used in this regard.
3. Alternatively, incubation in a weatherometer, oven, or water bath can be undertaken, and the time for 50% change, the "halflife," can be specified accordingly.

4.1.3.4 Reinforcing Scrim

Flexible polypropylene geomembranes are often manufactured with a fabric, called "reinforcing scrim," between two plies of the sheet polymer. This arrangement results in a three-ply laminated geomembrane consisting of geomembrane–scrim–geomembrane, which is laminated together under pressure to form a unitized system. The geomembrane is said to be reinforced and carries the designation fPP-R. Other options of multiple plies are also available. The scrim imparts dimensional stability to the material, which is important during storage, placement, and seaming. It also imparts a major increase in mechanical properties over the unreinforced type, particularly in the tensile strength, modulus of elasticity, and resistance to tears and punctures of the final geomembrane.

The reinforcing scrim for fPP-R geomembranes is a woven fabric generally made from polyester yarns manufactured in a standard basket weave. There are many fibers (of fine diameter) per individual yarn (e.g., 100 to 200 fibers per yarn, depending on the desired strength). The yarns, or strands, as they are referred to in the industry, are spaced close enough to one another to achieve the desired properties but far apart enough to allow open space between them so that the opposing geomembrane sheet surfaces can adhere together. This arrangement is sometimes referred to as *strike-through* and is measured by a ply-adhesion test. The designation of reinforcing scrim is based on the number of yarns, or strands, per inch of woven fabric. The general range is from 6×6 to 20×20; 10×10 is the most common. A 10×10 scrim refers to 10 strands per inch in the machine (or warp) direction and an equal number of 10 strands per inch in the cross-machine (or weft) direction.

It must also be mentioned that the polyester scrim yarns must be coated to allow for good bonding to the upper and lower fPP sheets. Various coatings, including latex and polyvinyl chloride, have been used. The exact formulation of the coating material (or "ply enhancer") is usually proprietary.

Regarding a specification or MQA document for the fabric scrim in fPP-R geomembranes, the following applies.

1. The type of polymer used for the scrim is usually specified as polyester, although nylon has been used in the past. It should be identified.
2. The strength of the fabric scrim can be specified and, when done, is best given in tensile strength units of force per unit width, rather than individual yarn strength.
3. The strike-through is indirectly quantified in specifications on the basis of ply-adhesion requirements. This specification will be discussed later.

4.1.4 Polyvinyl Chloride (PVC)

As seen in Table 4-1, polyvinyl chloride (PVC) geomembranes are made from polyvinyl chloride resin, plasticizer(s), fillers, carbon black, and additives. As with HDPE and LLDPE, PVC geomembranes can be smooth on both sides, smooth on one side and textured on the other, or textured on both sides. Texturing of PVC geomembranes is sometimes referred to as "faille."

4.1.4.1 Resin

The polyvinyl chloride resin used for PVC geomembranes is made by cracking ethylene dichloride into a vinyl chloride monomer. It is then polymerized to make PVC resin. The PVC resin (in the form of a porous white powder) is then compounded with other components to form a PVC compound.

In the preparation of a specification or MQA document, the following items concerning the PVC resin should be considered.

1. The polyvinyl chloride resin should be made from completely virgin materials.
2. A PVC compound will generally consist of 50 to 70% PVC resin by weight.

3. Typical QC tests on the resin powder are contamination, relative viscosity, resin gels, color, and dry time. The specific test procedures are specified by the manufacturer. They are often tests other than ASTM tests.
4. The frequency of performing each of the preceding tests should be covered in the MQC plan, and the plan should be implemented and followed.
5. Regrind or rework chips (which have been previously processed by the same manufacturer but never used as a geomembrane) are often added to the formulation during processing. This topic will be more fully discussed later.
6. Reclaimed, recycled, or postconsumer material (which is polymer that has seen previous service life) should never be allowed in any quantity.
7. QC certification procedures used by the manufacturer should be implemented and followed.

4.1.4.2 Plasticizer

Plasticizers are added to PVC formulations to impart flexibility, improve handling, and modify physical and mechanical properties. When blended with the PVC resin, the plasticizers must be completely mixed into the resin. Because the resin is a porous powder and the plasticizers are liquid, mixing of the two components continues until the liquid is completely absorbed by the powder. The result is a plasticized resin powder that can be readily mixed with other ingredients. It is also possible to wet blend with acceptable results. There are two general categories of possible plasticizers: monomeric plasticizers and polymeric plasticizers. There are many specific types within each category. For example, monomeric plasticizers are sometimes phthalates, epoxides, or phosphates, and polymeric plasticizers are sometimes polyesters, ethylene copolymers, or nitrile rubber.

For a specification or MQA document written for PVC plasticizers, the following items should be considered.

1. If more than one type of plasticizer is used in a PVC formulation, they must be compatible with one another.
2. The plasticizer or placticizers in a PVC compound is generally from 25 to 35% of the total compound by weight.
3. The exact type of plasticizers used by the manufacturers can be requested. Because of the sensitive nature of proprietary properties, however, this request is generally not made. Current thinking is to consider stipulating an indirect property (e.g., a molecular weight of 400 or greater) as an alternative to direct plasticizer identification.
4. The plasticizers should be certified by the manufacturer as having a successful past performance for the particular application or as having been used on a specific number of similar projects.

4.1.4.3 Filler

The filler used in a PVC formulation is a relatively small component (recall Table 4-1), and if used at all its identification can be requested. Calcium carbonate, in powder form, has been used, but other options also exist. Also, a small amount of carbon black is usually added to obtain the characteristic gray color of PVC geomembranes. Certification as to successful past performance should be requested.

4.1.4.4 Additives

Other additives (for ease of manufacturing, coloring, and stabilization) are also added to the formulation in relatively small amounts. They are generally not identified but can be requested. Certification as to successful past performance may be requested.

4.1.4.5 Reinforcing Scrim

In circumstances in which enhanced tensile, tear, and dimensional stability are required, a three-ply PVC geomembrane should be used. It consists of two plies of PVC film with a polyester scrim sandwiched between the plies. It is then designated as PVC-R. (See Section 4.1.5.5 for additional details.)

4.1.5 Chlorosulfonated Polyethylene (CSPE)

As seen in Table 4-1, chlorosulfonated polyethylene (CSPE) geomembranes consist of chlorosulfonated polyethylene resin, fillers, carbon black (or colorants), and additives. The finished geomembrane is usually fabricated with a woven textile, called a "reinforcing scrim," between the individual plies of the material. It is designated as CSPE-R.

4.1.5.1 Resin

There are two types of chlorosulfonated polyethylene resin used to make CSPE geomembranes. One is a completely amorphous polymer, and the other is a thermoplastic material containing a controlled amount of crystallinity to provide useful physical properties in the uncured state while maintaining flexibility without plasticizers. The second type is generally used to manufacture geomembranes. CSPE is made directly from branched polyethylene by adding chlorine and sulfur dioxide. The chlorosulfonic groups act as preferred cross-linking sites during the polymer aging process. In the typical commercial polymer, there is one chlorosulfonyl group for each 200 backbone carbon atoms.

CSPE resin pieces usually arrive at the sheet manufacturing facility in large cartons. They are somewhat pillow-shaped, with dimensions of 10 to 20 mm. The resin pieces are relatively spongy in their resistance to finger pressure. Alternatively, CSPE can be premixed with carbon black in slab form, which is then referred to as a master batch. The master batch is usually made by a formulator and shipped to the manufacturing facility in a prepared form.

In preparation of a specification or MQA document, the following items concerning the CSPE resin should be considered.

1. The CSPE resin should be made from completely virgin materials.
2. The formulation is usually based on 40 to 60% resin by weight.

3. Typical MQC tests on the CSPE resin are Mooney viscosity, chlorine content, sulfur content, and a series of vulcanization properties (e.g., rheometry and high-temperature behavior).
4. The CSPE resin can be premixed with carbon black in slab form (referred to as a master batch) and shipped to the manufacturer's facility.
5. Additional QC certification procedures used by the manufacturer should be implemented and followed.
6. The frequency of performing each of the preceding tests should be covered in the MQC plan, and the plan should be implemented and followed.

4.1.5.2 Carbon Black

The amount of carbon black in CSPE geomembranes varies from 5 to 10% by weight. The carbon black functions as a UV light blocking agent and a filler, and it aids in processing. The usual types of carbon black used in CSPE formulations are N 630, N 774, N 762, and N 990, per ASTM D1765. With formulations containing low percentages of carbon black, N 110 to N 220 should be used, both of which have high specific surface areas. When the carbon black is premixed with the resin and produced in the form of a master batch of pellets, it is fed directly into the mixer with the other components, such as fillers, stabilizers, and processing aids.

A specification on carbon black in CSPE geomembranes may be framed around the type and amount of carbon black as just described, but this is rarely the case. Specific MQC certification procedures should be available and implemented.

4.1.5.3 Fillers

The purposes of blending fillers into the CSPE compound are to provide workability and processability. The common types of fillers are clay and calcium carbonate. Both are added in powder form and in relatively large quantities, ranging from 40 to 50%.

Specifications are rarely written for this aspect of the material; however, MQC certification procedures should be available and implemented.

4.1.5.4 Additives

Additives, in the amount of 5 to 15% by weight, are used in CSPE compounds for stabilization, which is used to distinguish the various grades. The industrial grade of CSPE geomembranes uses lead oxide as a stabilizer, whereas the potable-water grade uses magnesium oxide or magnesium hydroxide. These stabilizers function as acid acceptors during the polymer aging process. During aging, hydrogen chloride or sulfur dioxide releases from the polymer and the metal oxides react with these substances, inducing cross-linking over time. This cross-linking is a unique feature of CSPE, wherein the as-manufactured material is initially a thermoplastic material and after in-service aging (3–7 years) it becomes a thermoset material.

Specifications are rarely written for the type and quantity of additives used in CSPE; however, MQC certification procedures should be written for each additive, be available to the specifier, and be implemented accordingly.

4.1.5.5 Reinforcing Scrim

CSPE geomembranes are usually fabricated with a "reinforcing scrim" between two plies of the polymer sheets. This results in a three-ply laminated geomembrane consisting of geomembrane–scrim–geomembrane, which is laminated together under pressure to form a unitized system. The geomembrane is said to be reinforced and carries the designation CSPE-R. Other options of multiple plies are also available (e.g., two layers of scrim reinforcement). The scrim imparts dimensional stability to the material, which is important during storage, placement, and seaming. It also imparts a major increase in mechanical properties over the unreinforced type, particularly in the tensile strength, modulus of elasticity, and resistance to tears and punctures of the final geomembrane.

The reinforcing scrim for CSPE-R geomembranes is a woven fabric, generally made from polyester yarns oriented in a standard basket weave. There are many fibers (of fine diameter) per individual yarn (e.g., 100 to 200 fibers per yarn, depending on the desired strength). The yarns, or strands, as they are referred to in the industry, are spaced close enough to one another to achieve the desired properties but far apart enough to allow open space between them so that the opposing geomembrane sheet surfaces can adhere together. This arrangement is sometimes referred to as strike-through and is measured by a ply-adhesion test. The designation of reinforcing scrim is based on the number of yarns, or strands, per inch of woven fabric. The general range is from 6×6 to 20×20; 10×10 is the most common. A 10×10 scrim refers to 10 strands per inch in the machine (or warp) direction and an equal number of 10 strands per inch in the cross-machine (or weft) direction.

It must also be mentioned that the polyester scrim yarns must be coated for them to have good bonding to the upper and lower sheets or plies. Various coatings, including latex and polyvinyl chloride, have been used. The exact formulation of the coating material (or "ply enhancer") is usually proprietary.

Regarding a specification or MQA document for the fabric scrim in CSPE-R geomembranes, the following applies:

1. The type of polymer used for the scrim is usually specified as polyester, although nylon has been used in the past. The polymer should be identified accordingly.
2. The strength of the fabric scrim can be specified and, when done, is best given in tensile strength units of force per individual yarn rather than individual yarn strength.
3. The strike-through is indirectly quantified in specifications on the basis of ply-adhesion requirements. This specification will be discussed later.

4.1.6 Other Geomembrane Types

There are other possible geomembranes that have not been described thus far. They will be briefly noted here, along with similarities and differences to those just described.

Ethylene propylene diene terpolymer (EPDM) is a terpolymer of ethylene, propylene, and a diene with the residual unsaturated portion of the diene in the side chain (ASTM D1418). It is sometimes reinforced with a fabric scrim similar to that described for other reinforced geomembranes. In such cases, it is properly designated as EPDM-R. EPDM is a thermoset plastic, which results in some unique properties. It is also possible to blend EPDM with fPP to have a hybrid material capable of heat bonding of field seams.

Ethylene interpolymer alloy (EIA) is always used as a reinforced geomembrane; thus EIA-R is its proper designation. The resin is a blend of ethylene vinyl acetate and polyvinyl chloride, resulting in a thermoplastic elastomer. The fabric reinforcement is usually a tightly woven polyester, which requires the polymer to be spread coated on both sides of the fabric. However, other related products are being developed under different trademarks in this general category.

Chlorinated polyethylene (CPE) has been used as a geomembrane in the past and was generally scrim reinforced, hence CPE-R. The resin producer no longer markets to the geomembrane industry; thus the material will not be discussed further.

Manufactured bituminous geomembranes have been used to line and cover waste facilities, but are rarely (if ever) used in the United States. They will not be discussed further.

Finally, field-fabricated geomembranes have been occasionally referenced in the literature. The process is usually one of placing a geotextile (generally a needle-punched nonwoven) and spraying it with a polymer or bitumen. The impregnated fabric is the geomembrane. Construction concerns are paramount for such products, and they will not be discussed further.

4.2 Manufacturing

Once the specific type of geomembrane formulation has been thoroughly mixed, it is then manufactured into a continuous sheet. The major processes used for manufacturing of the various types of sheets of geomembranes are variations of either extrusion (either flat die or blown film) or calendering. Autoclaving and spread coating will also be briefly mentioned.

Blending, compounding, mixing, or masticating of the various components described in Section 4.1 is conventionally done on a weight percentage basis. However, each geomembrane's processing is somewhat unique in its equipment and procedures. Even for a particular type of geomembrane, manufacturers will use different procedures for blending or mixing, e.g., batch methods versus continuous-feed systems.

Nevertheless, a few general considerations are important to follow in the preparation of a specification or MQA document.

1. The blending, compounding, mixing, and masticating equipment must be clean and completely purged from previously produced materials of a different formulation. This cleaning might require sending a complete cycle of purging material through the system (sometimes referred to as a "blank").

2. The various components of the formulation are added on a weight percentage basis to an accuracy set by industry standards. Different components are often added to the mixture at different stages in the processing, that is, the entire batch is not necessarily added at the outset.
3. By the time the complete formulation is ready for extrusion or calendering it must be completely homogenized. No traces of segregation, agglomeration, streaking, or discoloration should be visually apparent.

"Regrind," "reworked," or "trim" materials are all terms which can be defined as a finished geomembrane sheet that has been cut from edges or ends of rolls or is off-specification because of a surface blemish, thickness, or other property.

These materials can be reintroduced during the blending, compounding, or mixing stage in controlled amounts as a matter of cost efficiency on the part of the manufacturer. Regrind, rework, and trim material must be clearly distinguished from "recycled," or "reclaimed," material, which is finished sheet material that has actually seen some type of service performance and has subsequently been returned to the manufacturing facility for reuse for new sheet material.

In preparing a specification or MQA document on the use of reprocessed material, the following items should be considered:

1. Regrind, reworked, or trim materials in the form of chips or edge strips may be added if the material is from the same manufacturer and is exactly the same formulation as the geomembrane being produced.
2. Generally, HDPE, LLDPE, fPP, and PVC will be added in chip form as regrind in controlled amounts into the hopper of the extruder or mixer.
3. Generally, CSPE-R, EPDM-R, and EIA-R will be added in the form of a continuous strip of edge trimmings into the roll mill, which precedes calendering. For scrim-reinforced geomembranes, it is important that the edge trim does *not* contain any amount of the fabric scrim.
4. The maximum amount of regrind, reworked, or trim material to be added is currently limited to 10% by weight. Its occurrence in the completed sheet is extremely difficult, if not impossible, to identify, much less to quantify by current chemical fingerprinting methods (Hsuan et al. 2001). If regrind is not permitted to be used, the manufacturer may charge a premium over current practice.
5. It is generally accepted that no amount of recycled, reclaimed, or postconsumer sheet materials (in any form whatsoever) should be added to the formulation.

4.2.1 Flat Die Extrusion

HDPE, LLDPE, and fPP geomembranes are manufactured by taking the mixed components described earlier and feeding them into a hopper that leads to a horizontal extruder (Figure 4-2). Many extruders are 200-mm (8.0-in.) diameter systems, which are quite large, for example, up to 9 m (30 ft) long. In an extruder, the components enter a feed hopper, are heated to melting temperature, and are horizontally transported via a continuous screw through a feed section, compres-

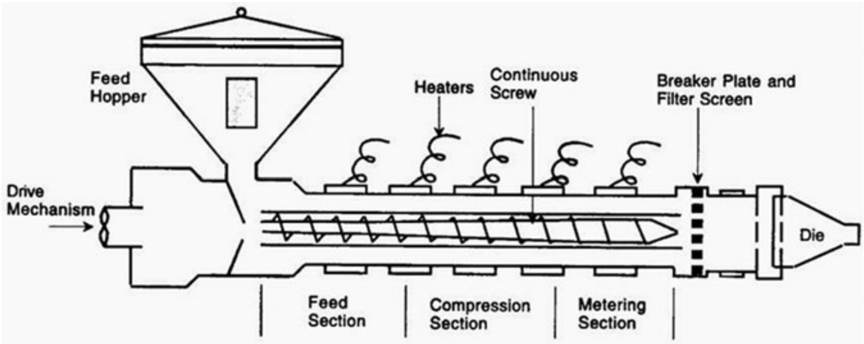


Figure 4-2. Cross-Section Diagram of a Horizontal Single-Screw Extruder.

sion section, metering section, and filtering screen, and are then pressure fed into a die. The die opening can be either a horizontal outlet flat die or a vertical outlet circular die. Flat die extrusion will be described here, and blown film die extrusion will be described in the next section.

4.2.1.1 Smooth Sheet

The thoroughly mixed molten polymer enters the die and is forced by pressure to flow laterally (in a coat hanger configuration), exiting the die gap along its full width at a thickness that is precisely controlled. A typical sheet width is approximately 7.0 m (23 ft). Wider sheets can be made by using two side-by-side extruders such that the melt streams meld together within the common die block (Figure 4-3). The finished dimensions, particularly thickness, can be tightly controlled ($\pm$1.0%).

Insofar as a specification or MQA document for finished geomembranes made by flat die extrusion is concerned, the following items should be considered:

1. The finished geomembrane sheet must be free from pinholes, surface blemishes, scratches, or other defects (e.g., nonuniform color, streaking, roughness, carbon black agglomerates, or visually discernible regrind).

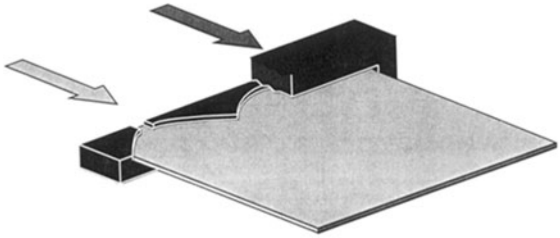



Figure 4-3. Sketch and Photograph of Twinned Flat Die Extrusion.

Source: Struve 1995, with permission from Geosynthetic Information Institute.

2. The nominal and minimum thicknesses of the sheet should be specified. The minimum value is usually related to the nominal thickness as a percentage.
3. The maximum thickness of the sheet is rarely, if ever, specified because if a manufacturer wishes to supply sheet thicker than specified, it is generally acceptable. It is also done, however, so that those manufacturers with unique variations of flat die extrusion (such as horizontal ribs or factory-fabricated seams) are not excluded from the market.
4. The finished sheet width should be controlled so that it is within a set tolerance. This control is usually achieved by creating a sheet larger than that called for and trimming the edges immediately before final rolling onto the wind-up core. (The edge trim is subsequently ground into chips and used as regrind, as previously described).
5. Other MQC tests, such as strength, puncture, or tear, should be part of a certification program that should be available and implemented.
6. The frequency of performing each of the preceding tests should be covered in the MQC plan. and the plan should be implemented and followed.
7. The trimmed and finished sheet is wound onto a hollow wind-up core (usually heavy cardboard or sometimes plastic pipe). The outside diameter of the core should be at least 150 mm (6.0 in.). The core must be stable enough to support the roll without buckling or otherwise failing during handling, storage, and transportation.
8. Partial rolls may be cut and prepared for shipment per the contract drawings for details within a specific project.

4.2.1.2 Textured Sheet

By creating a roughened surface on a smooth sheet, a process called "texturing" in this book, a high-friction surface can be created. Texturing can be done on one or both surfaces of the sheet. There are currently two methods used to texture smooth geomembranes: coextrusion (most common in North America) and structuring via a patterned set of cooling rollers (most common in Europe). In both methods, the textured surface is formed during processing. Thus, there is no possibility of the texturing wearing or rubbing off. Texturing sheet by coextrusion is usually accomplished using the blown film extrusion method, which will be described in Section 4.2.2.2. To provide a textured surface using flat die extrusion, the smooth sheet leaves the die gap and (while still warm) is sent between counterrotating rolls, which are patterned accordingly. Thus, the still-viscous sheet takes the surface configuration of the roller pattern (Figure 4-4). Note that the edge is smooth to facilitate welding of the sheets in the field.

Regarding the writing of a specification or MQA document for textured geomembranes, the following points should be considered:

1. Using the coextrusion process, the surface texturing should be of the same type of polymer and formulation as the base sheet polymer and its formulation. If other materials are added to the texturing material, they must be identified in case of subsequent seaming difficulties.

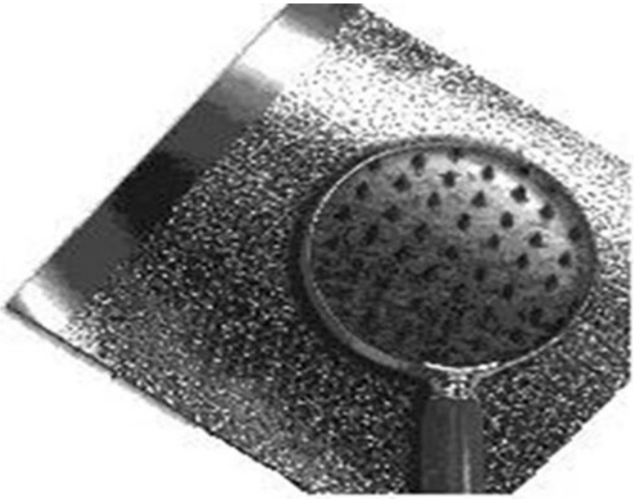


Figure 4-4. Textured Sheet from Flat Die Extrusion.

Source: Courtesy of Agru America, Inc.

2. For all types of texturing, the pattern should be uniform and consistent across the roll width and length.
3. The QC of the texturing process can be assessed for uniformity using an asperity height measurement, per GRI-GM12. (The Geosynthetic Research Institute (GRI) provides interim test methods for a variety of geosynthetic related topics until such time as consensus organizations (such as ASTM or ISO) adopt a standard on the same topic. When ASTM or ISO adopts standards, the GRI standard is depreciated.) The test, however, only measures the height of the textured peaks and not their sharpness, configuration, or spacing. Furthermore, it has not been correlated to an interface shear test, which is the ultimate purpose of the texturing process.
4. The effectiveness of the texturing must be sufficient to develop the amount of interface shear strength as required for the project design and specifications. This shear strength is, however, a design issue. The actual interface shear strength for design purposes should come from a large-scale direct shear test simulating site-specific conditions as closely as possible (e.g., ASTM D5321). This test, although critically important in its associated stability analyses, is not an MQC test.
5. The thickness of the base geomembrane should be measured using a micrometer according to ASTM D5199 along the smooth edge strips of textured geomembranes and within the textured portion of the sheet by using a tapered-point micrometer, per ASTM D5994. This test is referred to as a core thickness measurement and is determined by moving the micrometer within a defined area to obtain the lowest measurement.
6. Other MQC tests, such as tensile strength, puncture, tear, NCTL, OIT, should be part of a certification program, which should be available and implemented.

Generic specifications, such as GRI-GM13, GRI-GM17, and GRI-GM18, are available in this regard.

7. The frequency of performing each of the preceding tests should be covered in the MQC plan or generic specification, and the plan should be implemented and followed.

4.2.1.3 Coextruded Sheet

Flat die extrusion can be configured to provide for coextrusion of different melt streams made by different extruders. Figure 4-5 shows such a situation using a special feed block ahead of the die. HDPE–LLDPE–HDPE sheet has been made in this manner, as well as white and black sheet.

Regarding the writing of a specification or MQA document on coextruded flat die sheets, the following points should be considered:

1. All of the considerations in Section 4.2.1.1 apply to coextruded sheet as well.
2. Additionally, the individual thickness of each component of the coextruded sheet is of interest. However, there is no practical method to separate each material because primary bonding has occurred and there is no possibility to delaminate the material. A thin section under a transmission microscope or a scanning electron micrograph is necessary to determine each component's thickness. In this regard, the manufacturer's process control is all important.

4.2.1.4 Fabric-Reinforced Sheet

It is possible to have two extruders produce relatively thin sheets of the formulated material, one on top of the other, with a reinforcing fabric inserted between

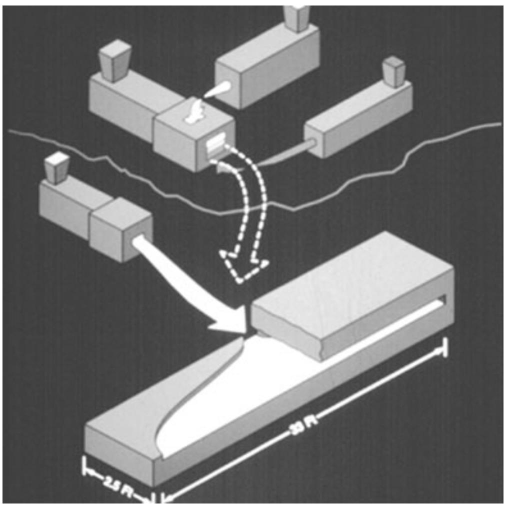


Figure 4-5. Sketch of a Flat Die, Center Fed, with Coextrusion Block.

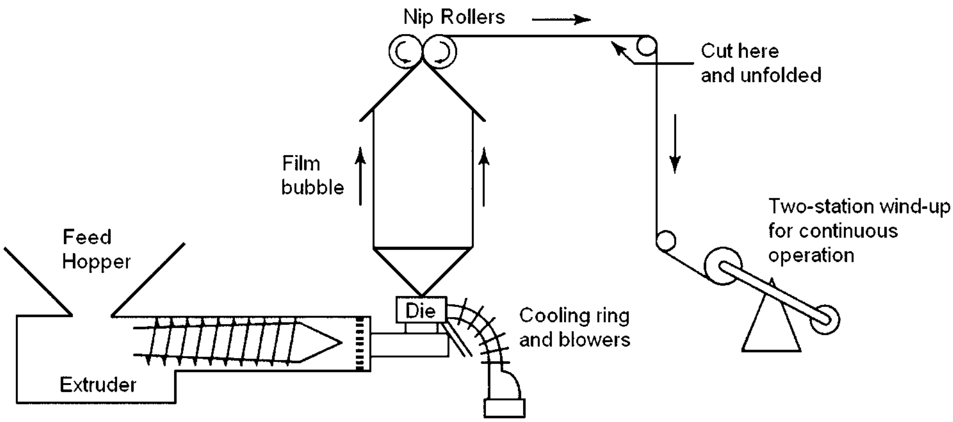
Source: Struve 1995, with permission from Geosynthetic Information Institute.

them. The three-ply material, geomembrane–scrim–geomembrane, must be passed through counterrotating rollers, so that the still-warm polymer sheets can strike through the scrim and adhere to one another.

One product (fPP-R) has been produced in such a manner. However, the much more common method of manufacturing scrim-reinforced geomembranes is by calendering and will be described later.

4.2.2 Blown Film Extrusion

By using a vertically oriented circular die, the extruder can feed molten polymer upward, creating a large cylinder of polyethylene sheet (Figure 4-6). Because the



(a)

(b)

Figure 4-6. Blown Film Cylinder of Polyethylene Geomembranes, (a) Sketch of Blown Film Manufacturing; (b) Photograph of Upward Rising Geomembrane Cylinder.

cylinder of polymer is closed at the top where it passes over a set of nip rollers that advances the cylinder, air pressure is generally maintained within it to control its dimensional stability. Upward moving air is also outside of the cylinder to aid further in stability of the gradually cooling cylinder of geomembrane. After passing through the nip rollers at the top of the system, the collapsed cylinder is cut longitudinally, opened to its full width, brought down to floor level, and rolled onto a wind-up core. Collapsing the cylinder and passing it through the nip rollers results in two subtle creases. After slitting the collapsed cylinder and opening it to full width, remnants of the two creases remain at quarter distances of the sheet width from the edges.

Blown film extrusion is the most common method of manufacturing HDPE, LLDPE, and fPP geomembranes in North and South America. Using this process, such geomembranes can be made smooth, one-sided textured, two-sided textured, and coextruded, using different polymers, colorants, or materials.

4.2.2.1 Smooth Sheet

A large single extruder is used to make smooth sheet by the blown film extrusion method. All thicknesses can be produced, and the widths of the sheet can be as large as 10.7 m (35 ft).

Regarding a specification or MQA document for blown film produced geomembranes, the following applies:

1. The finished geomembrane sheet shall be free from pinholes, surface blemishes, scratches, or other defects (e.g., nonuniform color, streaking, roughness, carbon black agglomerates, visually discernible regrind, etc.). Note that two machine direction creases from nip rollers are automatically induced into the finished sheet at the quarter distances from each edge.
2. The nominal and minimum thicknesses of the sheet should be specified. The minimum value is usually related to the nominal thickness as a percentage. Thickness control is more difficult than with flat die extrusion and values referenced range from 5 to 15% of the nominal thickness.
3. The maximum thickness of the sheet is rarely, if ever, specified. This is for the obvious reason that if a manufacturer wishes to supply sheet thicker than specified, it is generally acceptable.
4. The finished sheet width should be controlled to be within a set tolerance. Geomembranes made from the blown film extrusion method should meet a ±2.0% width specification.
5. Other MQC tests such as tensile strength, puncture, tear, etc., should be part of a certification program which should be available and implemented.
6. The finished sheet is wound onto a core, which is usually heavy cardboard or plastic pipe. The outside diameter of the core should be at least 150 mm (6.0 in.). It must be stable enough to support the roll without buckling or otherwise failing during handling, storage, and transportation.
7. Partial rolls may be cut and prepared for shipment as per the contract drawings for specific project details.

4.2.2.2 Textured Sheet

As mentioned in Section 4.2.1.2, the texturing of HDPE, LLDPE, and fPP sheet produced by blown film is the most common method. To provide a textured surface on one or both sides of the sheet, a spiral mandrel die is used (Figure 4-7).

The *coextrusion* texturing method uses a blowing agent (usually nitrogen gas) in the molten extrudate and delivers it from a small extruder immediately adjacent to the main extruder. When both sides of the sheet are to be textured, two small extruders (one internal and one external to the main extruder) are necessary. As the extrudate from these smaller extruders leaves the die and meets the cool air, the blowing agent expands, opens to the atmosphere, and creates the textured surfaces (Figure 4-8). Depending on the amount of nitrogen added, different degrees of texturing can be produced.

Regarding the writing of a specification or MQA document on textured geomembranes, the following points should be considered:

1. All points raised in Section 4.2.1.2 should be applied here as well.
2. Some manufacturers of textured blown film extruded sheet produce a smooth edge on both sides of the sheet to aid in seaming.

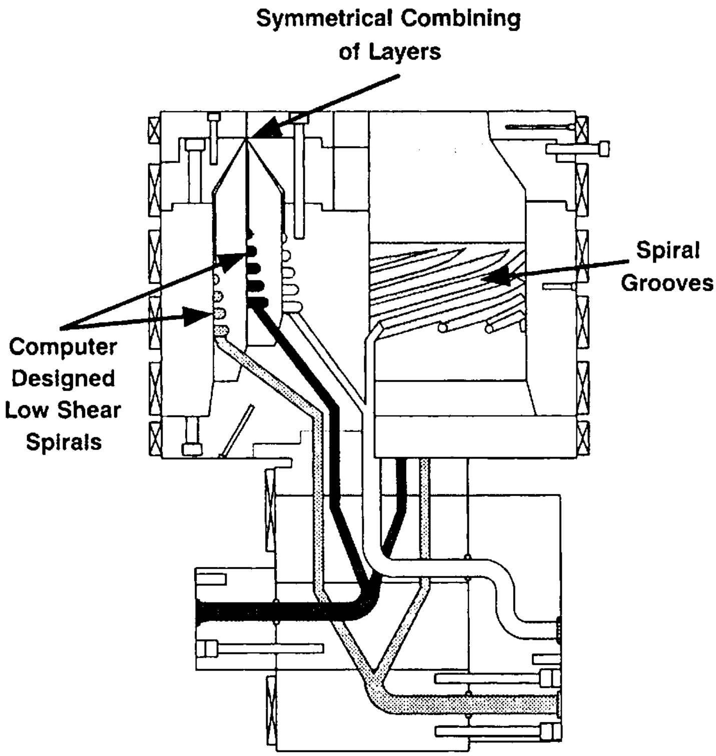


Figure 4-7. Sketch of a Circular Spiral Mandrel Die.

Source: Struve 1995, with permission from Geosynthetic Information Institute.

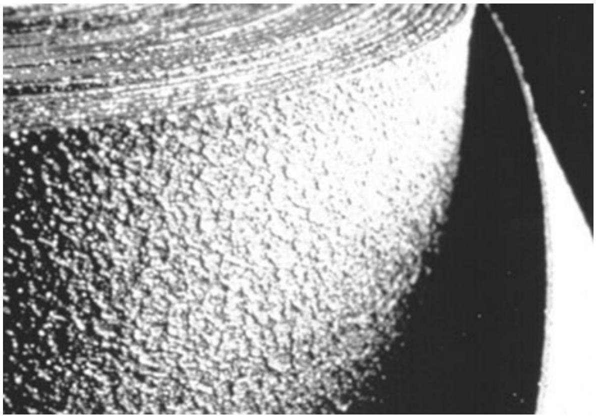


Figure 4-8. Textured Sheet from Blown Film Extrusion.

Source: Courtesy of GSE Lining Technology, Inc.

4.2.2.3 Coextruded Sheet

In much the same way as with textured sheet using blown film extrusion, the individual extruder feeds can produce HDPE–LLDPE–HDPE coextruded sheet, white/black sheet, conductive layering, and many other combinations.

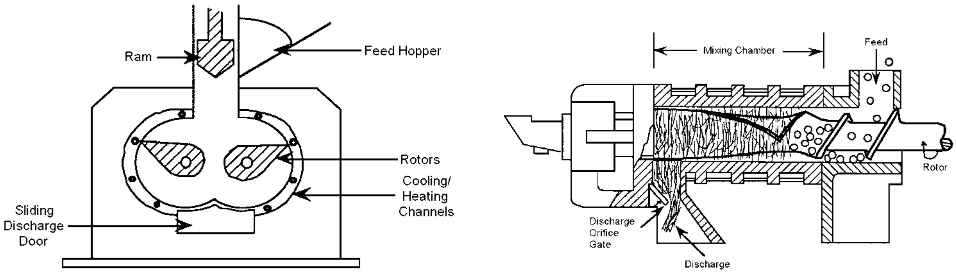
Regarding the writing of a specification or MQA document on coextruded blown film sheet, the following points should be considered:

1. All of the considerations in Section 4.2.1.2 apply to coextruded sheet as well.
2. Additionally, the individual thickness of each component of the coextruded sheet is of interest. However, there is no practical method to separate each material because primary bonding has occurred and there is no possibility to delaminate the material. A thin section under a transmission microscope or a scanning electron microscope is necessary to determine each component's thickness. In this regard, the manufacturer's process control is all important.

4.2.3 Calendering

Polyvinyl chloride (PVC) geomembranes are manufactured by taking proportional weights of PVC resin (a dry powder) and plasticizer (a liquid) and premixing them until the plasticizer is absorbed into the resin. Filler (in the form of a dry powder) and other additives (also usually dry powders) are then added to the plasticized resin, and the total formulation is mixed in a blender. Various types of high-intensity or low-intensity blenders can be used. Note that PVC rework in the form of chips, rather than edge trim, can be introduced at this point.

The resulting free-flowing powder compound is fed into a mixer, where heat is introduced, thereby initiating a reaction among the various components. These mixers can be either batch-type or continuous types (Figure 4-9(a) and (b)). In these mixers, the temperature is approximately 180 °C (350 °F), which melts the



(a) (b)

Figure 4-9. Sketches of Various Process Mixers: (a) Batch Process Mixer and (b) Continuous-Type Mixer.

mixture into a viscous mass. The mixed material is then removed from the discharge door or port onto a conveyor belt. From the conveyor belt, the viscous material is further worked (called "masticating") in a rolling mill into a consistent, uniform color, continuous mass of 100 to 150 mm (4–6 in.) diameter. Finished product edge trim can also be introduced into the rolling mill. The fully mixed formulation is then fed by conveyor directly into the sizing calender.

Chlorosulfonated polyethylene geomembranes are made in the same way by mixing CSPE resin with carbon black (or their colorants), thereby making a "master batch" of these two components. Added to this master batch are fillers, additives, and lubricants in a batch-type mixer (Figure 4-9(a)). Within the mixer, the shearing action of the rotors against the ingredients generates enough heat to cause melting, and subsequent chemical reactions occur. After the mixing cycle is complete, the batch is dropped from the mixer onto a two-roll mill, then to a conveyor leading to a second two-roll mill. In moving through the roll mill, it is further mixed into a completely homogenized material with a uniform color and texture. Edge trim is often taken from finished sheet and routed back to the roll mill for mixing and reuse.

4.2.3.1 Nonreinforced Sheet

PVC and nonreinforced CSPE formulations, irrespective of the preprocessing procedures, are manufactured into continuous geomembrane sheets by a calendering process. The viscous feed of polymer coming from the rolling mills is worked and flattened between counterrotating rollers into a geomembrane sheet. Most calenders are "inverted-L" configurations (Figure 4-10(b)), but other options are also available The rollers are usually smooth stainless steel cylinders and are up to 2.0 m (80 in.) wide. The opening distance between adjacent cylinders is set for the desired thickness of the final sheet. A rolling bank of molten material is formed between adjacent rolls. In an inverted four-roll "L" calender, several such banks are formed. They act as reservoirs for the molten material and help to fill the sheet to full thickness as it passes between the rolls. As the geomembrane exits from the calender, it enters an additional series of rollers for pickoff, embossing, stripping,

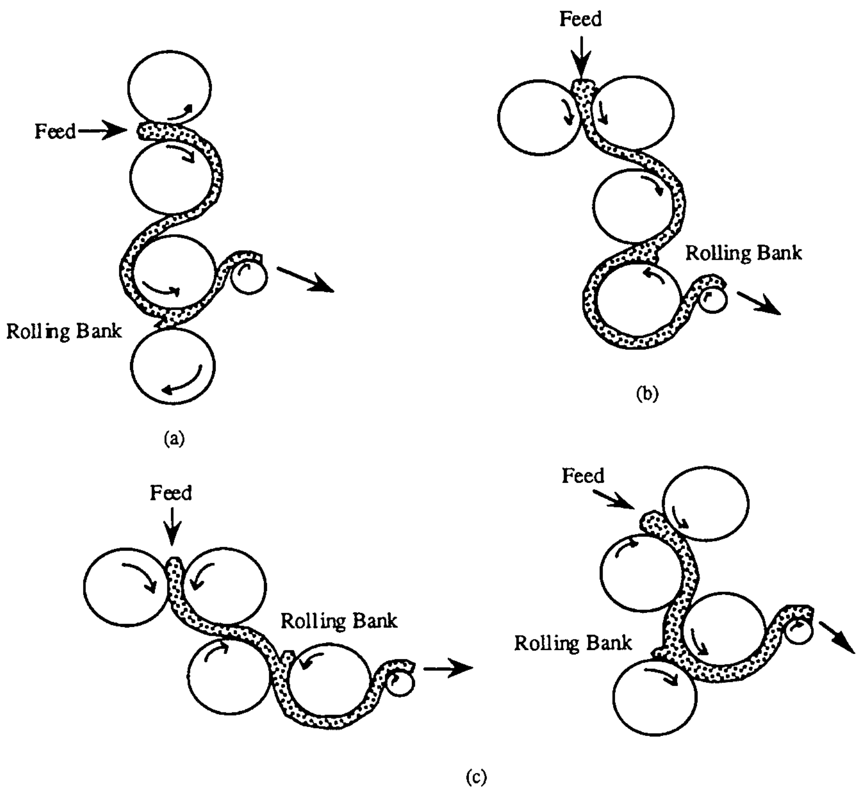


Figure 4-10. Various Types of Four-Roll Calenders: (a) Vertical, (b) Inverted L, (c) and Inclined Z.

cooling, and cutting. At least one, and perhaps two, rollers are embossed, or "faille," to impart a slight surface texture on the geomembrane. This embossing is meant to prevent the rolled geomembrane from sticking together (i.e., "blocking") during wind-up, storage, and transportation.

In developing a specification or MQA document for the manufacturing of nonreinforced PVC and CSPE geomembranes, the following considerations are important:

1. The finished geomembrane sheet should be free from pinholes, surface blemishes, scratches, or other defects (e.g., agglomerates of various additives or fillers or visually discernible rework).
2. The finished geomembrane sheet surfaces should be a uniform color and texture.
3. The addition of a dusting powder, such as talc, to eliminate blocking is *not* an acceptable practice. The powder will invariably attach to the sheet or it will be trapped within the embossed irregularities and eventually be contained in the seamed area as a potential contaminant, which could affect the adequacy of the seam.

4. The nominal and minimum thickness of the sheet should be specified.
5. The maximum thickness of the finished geomembrane sheet is generally not specified.
6. The width of the finished geomembrane depends on the type of calender used by the manufacturer.
7. The geomembrane sheet should be edge-trimmed to result in a specified width. This trim should be controlled to within ±0.25%.
8. Various MQC tests, such as tensile strength, puncture, and tear, should be part of a certification program, which should be available and implemented.
9. The frequency of performing each of the preceding tests should be covered in the MQC plan, and the plan should be implemented and followed.
10. The finished geomembrane sheet should be rolled onto stable wind-up cores of at least 75 mm (3.0 in.) diameter.
11. The rolls must be protected with a plastic film (Figure 4-11) to prevent exposure to sunlight during storage and shipment to a fabrication facility to make larger panels.

4.2.3.2 Reinforced Sheet

As mentioned earlier, CSPE formulations are manufactured into geomembrane sheet by a calendering process. The viscous continuous mass of polymer is worked and flattened into a geomembrane sheet. Most calenders are "inverted-L" config-

Figure 4-11. Rolls of Calendered Geomembrane Being Stored before Fabrication into Panels.

urations (recall Figure 4-10(b)), but other options are also illustrated. The inverted-L type calender provides an opportunity to introduce two simultaneous ribbons of the mixed and masticated polymeric compound, thereby making two individual sheets of geomembranes. Although this section of the manual is written for CSPE, it should be recognized that many other geomembrane types that are calendered can be made in multiple-ply form as well. Because they are separately formed geomembrane sheets, they are brought together immediately upon exiting the calender to provide a laminated geomembrane consisting of two or more plies of the material.

While producing the two separate plies in an inverted-L calender as described above, a woven fabric, called a reinforcing scrim, can be introduced between the two plies (Figure 4-12). The geomembrane is then said to be reinforced and is designated accordingly (e.g., CSPE-R). The scrim is usually a woven polyester yarn with 6×6, 10×10, or 20×20 count. These numbers refer to the number of yarns per inch in the machine and cross-machine directions, respectively. Other scrim counts are also possible.

Regarding the preparation of a specification or MQA document for multiple-ply scrim-reinforced calendered geomembranes, the following should be considered:

1. The finished geomembrane should be free from surface blemishes, scratches, and other defects (e.g., additive agglomerates or visually discernible rework).
2. The finished geomembrane sheet should be of a uniform color (which may be black, or, by the addition of colorants, such colors as white, tan, gray, or blue), gloss, and surface texture.

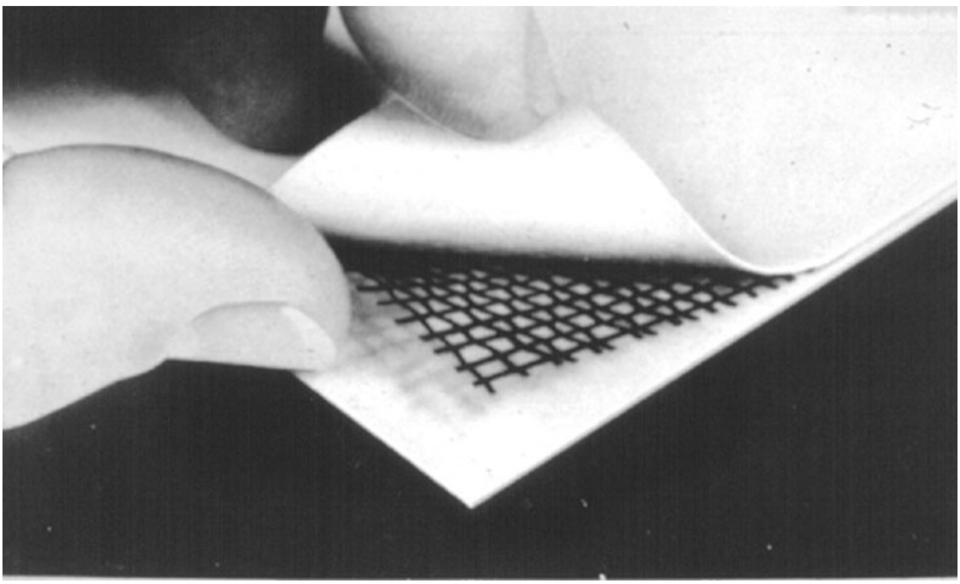


Figure 4-12. Multiple-Ply Scrim-Reinforced Geomembrane.

3. A uniform reinforcing scrim pattern should be reflected on both sides of the geomembrane; the pattern should be free from such anomalies as knots, gathering of yarns, delaminations, cross-over yarns, or nonuniform and deformed scrim.
4. The sheet should not be embossed because the surface irregularities caused by the scrim are adequate to prohibit blocking.
5. The thickness of the sheet should be measured over the scrim and, at a minimum, should be the nominal thickness minus 10%.
6. The geomembrane should have a nonreinforced selvage (i.e., geomembrane ply directly on geomembrane ply with no intermediate fabric scrim) on both edges. This selvage shall be approximately 6 mm (0.25 in.) wide.
7. Various MQC tests, such as strength, puncture, tear, or ply adhesion, should be part of a certification program, which should be available and implemented.
8. The frequency of performing each of the preceding tests should be covered in the MQC plan, and the plan should be implemented and followed.
9. The finished geomembrane sheet should be rolled onto stable wind-up cores of at least 75 mm (3.0 in.) in diameter.

4.2.3.3 Panel Fabrication

The geomembranes just described (recall Figure 4-11) as manufactured by the calendering process, are typically 1.0 to 2.0 m (40–80 in.) wide and are transported in rolls weighing up to 6.7 kN (1500 lb) to a panel fabrication facility. When a specific job order is placed, the rolls are unwound and placed directly on top of one another for factory seaming into a panel (Figure 4-13). A panel will typically con-

Figure 4-13. Factory Fabrication of Calendered Rolls into Accordion-Folded Large Panels for Field Deployment.

Source: Courtesy of Watersaver Co., Inc.

sist of 5 to 10 rolls, which are alternately seamed to one another, that is, the left side of a particular roll is seamed to the underlying roll while the right side is seamed to the overlying roll. After seaming, the completed panel is accordion-folded (in a lengthwise direction) and placed on a wooden pallet. It is then covered with a protective wrapper and shipped to the job site for deployment. Note that some fabricators use other procedures for panel preparation.

Regarding a specification or MQA document for factory fabrication of calender-produced geomembrane panels, the following items should be considered:

1. The factory seaming of geomembrane rolls into panels should be performed by approved seaming methods (see ASTM D4545). Dielectric seaming is a factory-seaming method for joining the rolls. This is a thermal (or heat fusion) method that is acceptable and is unique to factory seaming of flexible thermoplastic geomembranes.
2. Hot wedge welding, another thermal seaming method, is also used as a factory-seaming method. It is fully acceptable because it is a field seaming method as well.
3. When factory seams are made by solvent seaming methods, they are generally protected against blocking by covering them with a 100 mm (4 in.) wide strip of thin polyethylene film. When the panels are unfolded in the field, these strips are discarded.
4. Factory seams should be subjected to the same type of destructive and nondestructive tests as field seams (described in Section 4.4).
5. The finished and folded panels must be protected against accidental damage and excessive exposure during handling, transportation, and storage. Usually they are protected by covering them in a heavy cardboard enclosure and placed on wooden pallets for shipping.
6. The cardboard enclosures should be labeled and coded according to the specific job specifications.

4.2.4 Autoclaving

Thermoset polymers are also used for geomembranes, both nonreinforced and reinforced (e.g., EPDM and EPDM-R). Immediately after the rubber compound passes through the calender, the temperature of the calendered liner (with or without fabric reinforcement) is quickly reduced to room temperature when it passes over a series of cooling drums. Eventually, the rubber sheet passes through the accumulator on a wide belt conveyor and eventually goes to the splicing table. At the splicing table, even wider sheets are produced by cutting the calendered sheet (without any fabric) and splicing it to the previously cut sheet of the same dimensions. Fabric-reinforced liner is not converted to wider sheets. The two cut sheets are spliced together (in a matter of seconds) as they pass under a splicing beam. After the factory splice has been formed, both sides of the liner are dusted with a release agent, and the dusted liner is wound onto a metal curing mandrel. Once the mandrel of rubber has been properly prepared for curing and placed on a curing rack, it is conveyed to the autoclave for curing.

The curing equipment consists of the standard autoclave, preferably with a heated jacket to reduce condensation, and a closed chamber in which a rack containing several curing mandrels is placed and steam is slowly introduced. In operating a closed-chamber steam vulcanizer, the curing cycle consists of a rise to the predetermined pressure, a definite period at the required curing pressure, and a blow down to atmospheric pressure. The rack of curing mandrels is allowed to set approximately 30 to 45 minutes before preparing each mandrel for observation (e.g., inspection or cut plan) on the finishing and inspection floor. Finally, the inspected geomembranes are packaged (rolled onto a core) for distribution.

4.2.5 Spread Coating

As mentioned previously, an exception to the calendering method of producing reinforced geomembranes is the spread coating process. This process is currently unique to a geomembrane type called ethylene interpolymer alloy (EIA-R), but it has been used to produce other specialty geomembranes in the past. The process uses a dense fabric substrate, commonly either a woven or nonwoven textile, and spread coats the molten polymer on its surface. Because of the dense structure of the fabric, penetration of the viscous polymer to the opposite side is usually not complete. When cooled, the sheet is turned over, and the process is repeated on the opposite side. Adherence of the polymer to the fabric is essential.

Geomembranes produced by the spread coating method are indeed multiple-ply reinforced materials, but they are produced by a method different than calendering. MQC and MQA plans and specifications should be framed in a similar manner as described previously for reinforced geomembranes.

4.2.6 Generic Specifications

Since the publication of the first edition of this book, a number of generic specifications intended for most of the geomembranes described in this section have become available. Such generic specifications cover designated test methods, limiting properties, and minimum testing frequencies. Currently, they are as follows:

GRI-GM13:	High-Density Polyethylene (HDPE): Smooth and Textured
GRI-GM17:	Linear Low-Density Polyethylene (LLDPE): Smooth and Textured
GRI-GM18:	Flexible Polypropylene (fPP): Nonreinforced and Reinforced
GRI-GM21:	Ethylene Propylene Diene Terpolymer (EPDM): Nonreinforced and Reinforced
PGI 1104:	Polyvinyl Chloride (PVC)

The most recent versions of these generic specifications can be found on the websites of the institutes that developed and maintain these documents, that is, the Geosynthetic Institute and the PVC Geomembrane Institute.

4.3 Handling

Although there should be great concern and care focused on the manufacturers and installers of geomembranes, it is also incumbent that they are handled, packaged, stored, transported, re-stored, rehandled, and deployed so as not to cause any damage. This section is written with these ancillary considerations in mind.

Different types of geomembranes require different types of packaging after they are manufactured. Generally, extrusion-manufactured geomembranes (HDPE, LLDPE, and fPP) are wrapped around a core in roll form, whereas calendered geomembranes (PVC, CSPE-R, and autoclaved geomembranes, such as EPDM-R) are accordion-folded in two directions and packaged onto wooden pallets.

4.3.1 Rolls

The extrusion-manufactured geomembranes are produced and fed directly to a wind-up core in full-width rolls. No external wrapping or covering is generally needed, nor provided. These rolls, which weigh up to 22 kN (5000 lb), are moved either by forklifts using a long rod inserted into the core (called a "stinger") or they are picked up by high-strength polymer slings with a crane or hoist. The slings are often dedicated to each particular roll and follow along with it until its actual deployment. The rolls are usually stored outdoors. They are stacked such that one roll is nested into the valley of the two underlying rolls (Figure 4-14).

Regarding a specification or MQA document for finished rolls of HDPE geomembranes, the following applies:

1. The cores on which the rolls of geomembranes are wound should be at least 150 mm (6.0 in.) in diameter.

Figure 4-14. Rolls of Extrusion-Manufactured Geomembranes Awaiting Shipment to a Job Site.

2. The cores should have a sufficient diameter that forklift stingers can be used for lifting and movement without damaging the geomembrane.
3. The cores should be sufficiently strong that the roll can be lifted by a stinger or with slings without excessively deflecting or structurally buckling the roll.
4. The stacking of rolls at the manufacturing facility should not cause buckling of the cores or flattening of the rolls. In general, the maximum stacking limit is five rolls high.
5. If storage at the manufacturer's facility is for longer than 12 months, the rolls should be covered by a sacrificial covering or placed within a temporary or permanent enclosure.
6. The manufacturer should identify all rolls with the manufacturer's name, product identification, thickness, roll number, roll dimensions, and date manufactured.

4.3.2 Panels

Calendered geomembranes are initially manufactured in relatively narrow rolls and are then sent to a fabricator for factory seaming into much wider panels. At the fabrication facility, they are unrolled directly on top of one another, factory seamed along alternate edges of the rolls, and then accordion-folded both widthwise and lengthwise and placed onto wooden pallets for packaging and shipment. Such geomembranes are generally not stored longer than a few weeks at the fabrication facility.

Regarding items for a specification or MQA document, the following applies:

1. The wooden pallets on which the accordion-folded geomembranes are placed should be structurally sound and of good workmanship so that forklifts or cranes can transport and maneuver them without structurally failing or causing damage to the geomembrane.
2. The wooden pallets should extend at least 75 mm (3 in.) beyond the edge of the folded geomembrane panel on all four sides.
3. The folded geomembrane panel should be packaged in treated cardboard or plastic wrapping for protection from precipitation and direct UV exposure.
4. Banding straps around the geomembrane and pallet should be properly cushioned so as not to cause damage to any part of the geomembrane panel.
5. Palleted geomembranes should be stored only on level surfaces because the folded material is susceptible to shifting and possible damage.
6. The stacking of palleted geomembrane panels on top of one another should not be permitted.
7. If storage at the fabricator's facility is for longer than 6 months, the palleted panels should be covered with a sacrificial covering or temporary shelter or placed within a permanent enclosure.

4.3.3 Shipping and Site Storage

The geomembrane rolls or pallets are shipped to the job site, offloaded, and temporarily stored at a remote location on the job site. The manufacturing designations of MQC and MQA will now shift to construction designations of CQC and CQA because field construction personnel are now involved. These designations will carry forward throughout the remainder of this chapter.

Regarding items for a specification of CQA document, the following applies:

1. Unloading of rolls or pallets at the job site's temporary storage location should be such that no damage to the geomembrane occurs.
2. Pushing, sliding, or dragging of rolls or pallets of geomembranes should not be permitted.
3. Offloading at the job site should be performed with cranes or forklifts in a careful manner so that damage does not occur to any part of the geomembrane.
4. Temporary storage at the job site should be in an area where standing water cannot accumulate at any time.
5. The ground surface should be suitably prepared such that no stones or other rough objects are present that could damage the geomembranes.
6. Temporary storage of rolls of HDPE, LLDPE, or fPP geomembranes in the field should not be so high that crushing of the core or flattening of the rolls occurs. This limit is typically five rolls high.
7. Temporary storage of pallets of PVC, CSPE-R, EPDM-R, or EIA-R geomembranes by stacking should not be permitted.
8. Suitable means of securing the rolls or pallets should be used such that shifting, abrasion, or other adverse movement does not occur.
9. If storage of rolls or pallets of geomembranes at the job site is longer than 6 months, a sacrificial covering or temporary shelter should be provided for protection against precipitation, UV exposure, and accidental damage.

4.3.4 Acceptance and Conformance Testing

It is the primary duty of the installation contractor, via their CQC personnel, to ensure that the geomembrane supplied to the job site is the proper material that was specified in the contract, as stipulated in the plans and specifications. It is also the duty of the CQA engineer to verify this material to be appropriate. Clear marking should identify all rolls or pallets with the information described previously. A complete list of roll numbers should be prepared for each material type.

Before or on delivery of the rolls or pallets of geomembrane, the CQA engineer should ensure that conformance test samples are obtained and sent to the proper laboratory for testing. This laboratory will generally be the laboratory of the CQA firm but may be that of the CQC firm, if so designated in the CQA documents. Alternatively, conformance testing could be performed at the manufacturer's facility and, when completed, the particular lot should be marked for the particular site under investigation.

The following items should be considered for a specification or CQA document with regard to acceptance and conformance testing:

1. The particular tests selected for acceptance and conformance testing can be all of those mentioned in the material's specification, but this is rarely the case be-

cause MQC and MQA testing should have preceded the field operations. However, at a minimum, the tests indicated in Table 4-2 are recommended for field acceptance and conformance testing for the particular geomembrane type.

2. The method of geomembrane sampling should be prescribed. For geomembranes on rolls, a strip approximately 45 cm (18 in.) from the entire width of the roll on the outermost wrap is usually cut and removed. For geomembranes folded on pallets, the protective covering must be removed, the uppermost accordion-folded section opened, and an appropriate size sample taken. Alternatively, factory-seamed "retains" can be shipped on top of fabricated panels for easy access and use in conformance testing.
3. The machine direction must be indicated using a permanent marker with an arrow on all samples taken for conformance testing.
4. Samples are usually taken on the basis of a stipulated area of geomembrane (e.g., one sample per 10,000 m^2 (100,000 ft^2)). Alternatively, one could take samples at the rate of one per lot; however, a lot must be clearly defined. One possible definition could be that a lot is a group of consecutively numbered rolls or panels from the same manufacturing line.
5. All conformance test results should be reviewed, accepted, and reported by the CQA monitor before deployment of the geomembrane.
6. Any nonconformance of test results should be reported to the owner/operator. The method of a resolution of such differences should be clearly stated in the CQA document.

4.3.5 Placement

When the subgrade or subbase (either soil or some other geosynthetic) is approved as being acceptable, the rolls or pallets of the temporarily stored geomembranes are brought to their intended location, unrolled or unfolded, and accurately positioned, or ("spotted") for field seaming (Figure 4-15).

Table 4-2. Suggested Minimum Conformance Testing and Current ASTM Methods

Geomembrane Type	*Thickness*	*Tensile Strength and Elongation*	*Puncture*	*Tear*	*Ply Adhesion*
HDPE-S	D 5199	D 6693	D 4833	D 1004	N/A
-T	D 5994	D 6693	D 4833	D 1004	N/A
LLDPE-S	D 5199	D 6693	D 4833	D 1004	N/A
-T	D 5994	D 6693	D 4833	D 1004	N/A
fPP-S	D 5199	D 6693	D 4833	D 1004	N/A
-T	D 5994	D 6693	D 4833	D 1004	N/A
-R	D 5199	D 751	D 4833	D 5884	D 6636
PVC	D 5199	D 882	D 4833	D 1004	N/A
CSPE-R	D 5199	D 751	D 4833	D 5884	D 6636
EPDM-R	D 5199	D 751	D 4833	D 5884	D 6636

Note: N/A, not applicable.

Figure 4-15. Photographs Showing the Unrolling (upper) and Unfolding (lower) of Geomembranes.

4.3.5.1 Subgrade (Subbase) Conditions

Before beginning to move the geomembrane rolls or pallets from their temporary storage location at the job site, the soil subgrade (or other subbase material) should be inspected for its preparedness.

Some items recommended for a specification or CQA document include the following:

1. The soil subgrade shall be according to the specified grading, moisture content, and density as required by the installer and as approved by the CQA engineer for placement of the geomembrane. See Chapter 3 for such details for compacted clay liner subgrades.
2. Construction equipment deploying rolls or pallets of geomembranes shall not deform or rut the soil subgrade excessively. This is a site-specific situation and may require limiting the ground contact pressure of the deployment vehicles. Tire or track deformations beneath the geomembrane should not be greater than 25 mm (1.0 in.) deep.
3. The geomembrane shall not be deployed on frozen subgrade where ruts are greater than 12 mm (0.5 in.) deep.
4. When placing the geomembrane on another geosynthetic material (geosynthetic clay liner, geotextile, geonet, or geocomposite), construction placement equipment should not be permitted to ride directly on the lower geosynthetic material. In cases where rolls must be moved over previously placed geosynthetics, it is necessary to move materials by hand or by using small pneumatic-tire lifting units. Other techniques, such as use of block and tackle, have also been used.
5. If not specifically excluded in the regulations, and if agreed on by the CQA engineer, all-terrain vehicles (ATVs) or equipment with smooth, oversized tires of maximum ground contact pressure of 28 to 41 kPa (4–6 lb/in.^{2}) can be used; however, restrictions should be imposed. Considerations in this regard are as follows:
   - The vehicle can be operated on the previously placed geosynthetics only when deploying materials.
   - There should be no sudden stops or starts.
   - There should be no spinning of tires or sliding at any time.
   - Vehicle tires must be smooth and clean of mud, dirt, and debris that could potentially puncture or damage the underlying geosynthetic material.
   - All entering and exiting on the geosynthetic material should be done at 90-degree angles to the material.
   - There should be no excessive turning while driving on the geosynthetic material. Movement should be primarily forward and backward while deploying, and turning should be minimized to the greatest extent possible.
   - There should be no driving over wrinkles in geosynthetics.
   - There should be no more than one person riding on vehicle.
   - Vehicles should not be used on slopes.
6. Underlying geosynthetic materials should have all folds, wrinkles, and other undulations removed before placement of the overlying geomembrane.
7. Care and planning should be taken to unroll or unfold the geomembrane close to its intended and final position. Dragging (particularly textured geomembranes) should be minimized.

4.3.5.2 Temperature Effects—Sticking and Cracking

High temperatures can cause geomembrane surfaces on rolls, or accordion-folded on pallets, to stick together, a process commonly called “blocking.” At the other extreme, low temperatures can cause geomembrane sheets to crease, or even crack, when unrolled or unfolded. Comments on unrolling or unfolding of geomembranes at each of these temperature extremes follow.

For example, a specification or CQA document should include the following items:

1. Geomembranes, when unrolled or unfolded, should not stick together to the extent where tearing or visually observed straining of the surface of the geomembrane occurs. The upper temperature limit is specific to the particular type of geomembrane. A sheet temperature of 50 °C (122 °F) is the upper limit that a geomembrane should be unrolled or unfolded unless it is shown otherwise to the satisfaction of the CQA engineer.
2. Geomembranes, when unrolled or unfolded in cold weather, should not crease, crack, craze, or distort in texture. A sheet temperature of 0 °C (32 °F) is the lower limit that a geomembrane should be unrolled or unfolded unless it is shown otherwise to the satisfaction of the CQA engineer.
3. Geomembranes that have torn, crazed, or cracked, or have been excessively deformed should be rejected or repaired per the CQA document.

4.3.5.3 Temperature Effects—Expansion and Contraction

All geomembranes expand when they are heated and contract when they are cooled. This effect is particularly visible for HDPE because of its relative stiffness and thickness in comparison to other geomembranes. This expansion and contraction must be considered when placing, seaming, and backfilling geomembranes in the field. Figure 4-16 shows an excessively large wrinkle in a polyethylene liner that has expanded due to thermal warming from the sun.

Either the contract plans and specifications or the CQA documents should cover the expansion and contraction situation on the basis of site-specific and geomembrane-specific conditions. Some items to consider include the following:

1. The geomembrane shall be placed to compensate for the coldest temperatures envisioned so that no thermally induced tensile stresses are generated in the geomembrane or in its seams either during installation or subsequently after the geomembrane is covered.
2. The geomembrane shall be placed in such a manner that it does not lift up off the subgrade or substrate material at any location within the facility (i.e., no “trampolining” of the geomembrane shall be allowed to occur).
3. The amount of slack to be added, if any, to the deployed and seamed geomembrane should be carefully considered and calculated, taking into account the type of geomembrane and the geomembrane’s temperature during seaming versus its final temperature in the completed facility.

Figure 4-16. Excessively Large HDPE Geomembrane Wrinkle.

4. The geomembrane shall not have excessive slack to the point where waves can accumulate or fold over and be entombed by the overlying soil (Soong and Koerner 1998). This is particularly the case when the geomembrane is the upper component of a composite liner (i.e., when it is underlain by a compacted clay liner or geosynthetic clay liner). Such type of composite liner requires the geomembrane to be in intimate contact with the underlying soil component.
5. Control of waves or wrinkles in geomembranes may require backfilling to be limited to the early morning hours of the workday. In extreme cases, backfilling may have to be done at night. This is a site-specific situation, largely at the discretion of the installation contractor so long as the product of a flat geomembrane with intimate contact to the underlying soil results. Methods of backfilling will be described later.

4.3.5.4 Spotting

When a geomembrane roll or panel is deployed, it is generally required that some shifting will be necessary before field seaming begins. This is called "spotting" by many installers.

Some items for a specification or CQA document should include the following:

1. Spotting of deployed geomembranes should be done with no disturbance to the soil subgrade or geosynthetic materials on which they are placed.

2. Spotting should be done with a minimum amount of dragging of the geomembrane on soil subgrades or on other geosynthetics. Textured geomembranes are of particular concern because the texturing can easily dislodge stones in the proof-rolled soil subgrade, leaving them as potential puncturing points beneath the geomembrane. The dragging of textured geomembranes when placed on other geosynthetics can cause folds or creases and likewise cause stress concentrations beneath the geomembrane. Use of rub sheets to facilitate movement is an acceptable practice, particularly when using textured geomembranes.
3. Temporary tack welding (usually with a hand-held hot air gun) of all types of thermoplastic geomembranes should be allowed at the CQA engineer's discretion.
4. When temporary tack welds of geomembranes are used, the welds should not interfere with the primary seaming method or with the ability to perform subsequent destructive seam tests.

4.3.5.5 Seam Orientation

Because rolls or panels of geomembranes vary greatly in both length and width dimensions, firm statements about seam orientation are difficult. There are, however, some generalized items for a specification or CQA document. They are as follows:

1. Whenever possible, seams should be oriented parallel to the maximum slope. This is particularly important for sideslopes.
2. If at all possible, full roll length should be used on sideslopes, thereby avoiding edge seams.
3. If not possible (i.e., for extremely long sideslopes), the edge seams along the ends of adjacent rolls should be toward the bottom of the slope rather than the top. These edge seams should always be staggered between adjacent rolls.
4. The corners of cells will necessarily be tailored with seams running in different orientations. This shaping is generally unavoidable. A fan pattern is a common configuration.
5. The installer should submit a roll or panel layout plan for approval by the design engineer and/or CQA organization. Geomembrane should not be placed before approval of the site-specific layout plan.

4.3.5.6 Wind Considerations

Unfortunately, wind damage to geomembranes is not an uncommon occurrence (Figure 4-17). Many deployed geomembranes have been lifted by wind and have been damaged. In some cases, the geomembranes have even been torn out of anchor trenches. This problem is sometimes referred to as "blow-out" by field personnel. Generally, but not always, the unseamed geomembrane rolls or panels acting individually are most vulnerable to wind uplift and damage.

Figure 4-17. Wind Damage to Deployed Geomembrane.

The contract plans and specification, or at least the CQA documents, must be specific as to resolutions regarding geomembranes that have been damaged due to shifting by wind. Some suggestions follow:

1. Geomembrane rolls or panels that have been displaced by wind should be inspected and approved by the CQA engineer before any further field operations commence.
2. Geomembrane rolls or panels that have been damaged (torn, punctured, abraded, or deformed excessively and permanently) shall be rejected or repaired as directed in the contract plans, specifications, or CQA documents.
3. Permanent crease marks or severely folded (crimped) locations in geomembranes should not be permitted unless it can be shown that such distortions have no adverse effect on the properties of the geomembrane. A large-scale multiaxial burst test, per ASTM D5617, has been used to assess possible damage and to provide a guide between potentially damaged and as-received geomembrane samples. If this test cannot be done, these areas should be cut out and properly patched as per the contract documents and approved by the CQA engineer.
4. If patching of wind-damaged geomembranes becomes excessive (to the limit set forth in the specifications or CQA plan), the entire roll or panel should be rejected.

4.4 Seaming and Joining

The field seaming of the deployed rolls or panels is a critical aspect of the successful functioning of the geomembrane as a barrier to liquids and gases. This section describes the various seaming methods in current use, the concept and importance of test strips (or trial seams), and destructive and nondestructive test methods; it concludes with other emerging methods (e.g., electrical leak location surveys).

4.4.1 Field Seaming Methods

The fundamental mechanism of seaming overlapped polymeric geomembrane sheets together is to temporarily reorganize, that is, melt, the polymer structure of the two surfaces to be joined in a controlled manner that, after the application of pressure and a certain amount of time, results in the two sheets being bonded together. This reorganization results from an input of energy that originates from either thermal or chemical methods. These methods may involve the addition of extra polymer in the bonded area.

Ideally, seaming two geomembrane sheets would result in no net loss of tensile strength across the two sheets, and the joined sheets would perform as one single geomembrane sheet. However, because of geometric irregularities and possible loss of material, current seaming techniques result in minor tensile strength loss relative to the parent geomembrane sheet. The characteristics of the seamed geomembranes are a function of the type of geomembrane and the seaming technique used. These characteristics, such as required strength, geomembrane type, and seaming type, should be recognized by the designer when applying the appropriate design factors of safety for the overall geomembrane function and site-specific system performance.

The seam can be the location of the highest tensile stress in a geomembrane liner. Designers and inspectors should be aware of the importance of seeking only the highest quality geomembrane seams. The minimum seam tensile strengths (as determined by design) for various geomembranes must be predetermined by laboratory testing, knowledge of past field performance, manufacturer literature, various journals and conference proceedings papers, or other standard-setting organizations that maintain current information on seaming techniques and technologies.

The available methods of seaming thermoplastic and thermoset geomembranes discussed herein are given in Table 4-3 and shown schematically in Figure 4-18.

Within the group of geomembranes that are being discussed in this manual, there are four general categories of seaming methods; *thermal fusion*, *extrusion welding*, *chemical processes*, and *adhesive processes*. Each type will be explained, along with their specific variations so as to give an overview of field seaming technology.

Table 4-3. Various Methods of Joining Geomembranes

Thermal Processes	*Chemical and Adhesive Processes*
Fusion:	Chemical:
• Hot Wedge	• Chemical
• Hot Air (Rarely)	• Bodied Chemical
Extrusion:	Adhesive:
• Fillet	• Chemical Adhesive
• Flat (Depreciated)	• Contact Adhesive

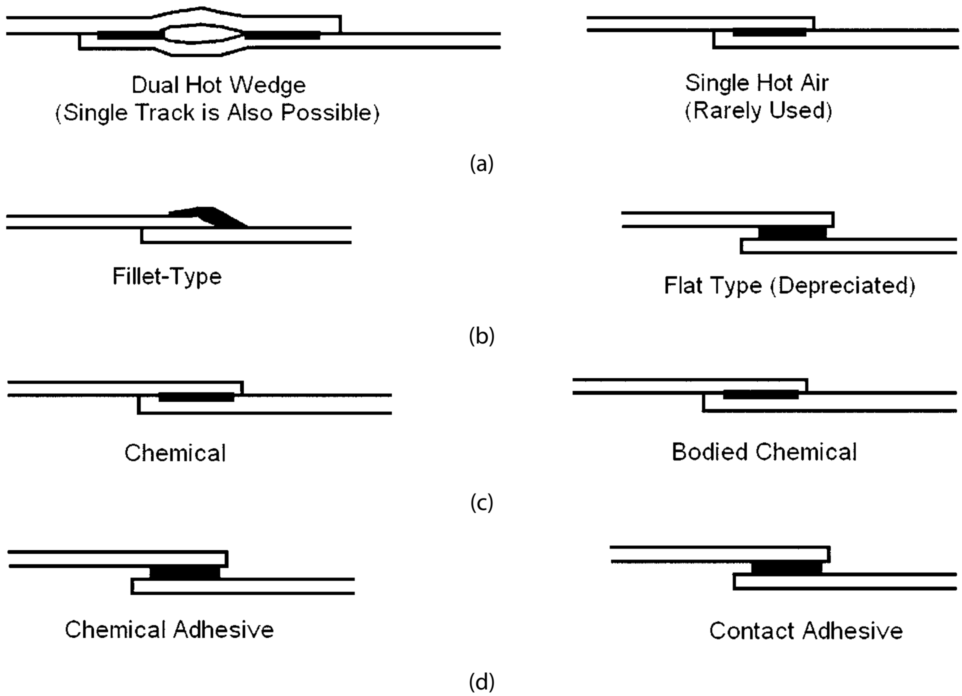


Figure 4-18. Various Methods Available To Fabricate Geomembrane Seams: (a) Fusion Seams, (b) Extrusion Seams, (c) Chemical Seams, and (d) Adhesive Seams.

Two *thermal fusion methods* can be used on all thermoplastic geomembranes. In both of them, portions of the opposing surfaces are truly melted. Temperature, pressure, and seaming rate all play important roles because excessive melting weakens the geomembrane and inadequate melting results in low seam strength. The *hot wedge*, or hot shoe, method consists of an electrically heated resistance element in the shape of a wedge that travels between the two sheets to be seamed. As it melts the surfaces of the two sheets being seamed, a shear flow occurs across the upper and lower surfaces of the wedge. No grinding of surfaces is required with this method. Roller pressure is applied as the two sheets converge at the tip of the wedge to form the final seam. Hot wedge units are controllable as far as temperature, amount of pressure applied, and travel rate. A standard hot wedge creates a single uniform width seam, and a dual hot wedge (or "split" wedge) forms two parallel seams with a uniform unbonded space between them. This space is then used to evaluate seam quality and continuity by pressurizing the unbonded space with air and monitoring any drop in pressure that may signify a leak in the seam. Dual-track hot wedge seams are considered the premier seaming method for all types of thermoplastic geomembranes, i.e., all of the geomembranes listed in Table 4-1 except for EPDM, which is a thermoset polymer. Figure 4-19 shows the split wedge and the resulting seam.

The *hot air* method makes use of a device consisting of a resistance heater, a blower, and temperature controls to force hot air between two sheets to melt the

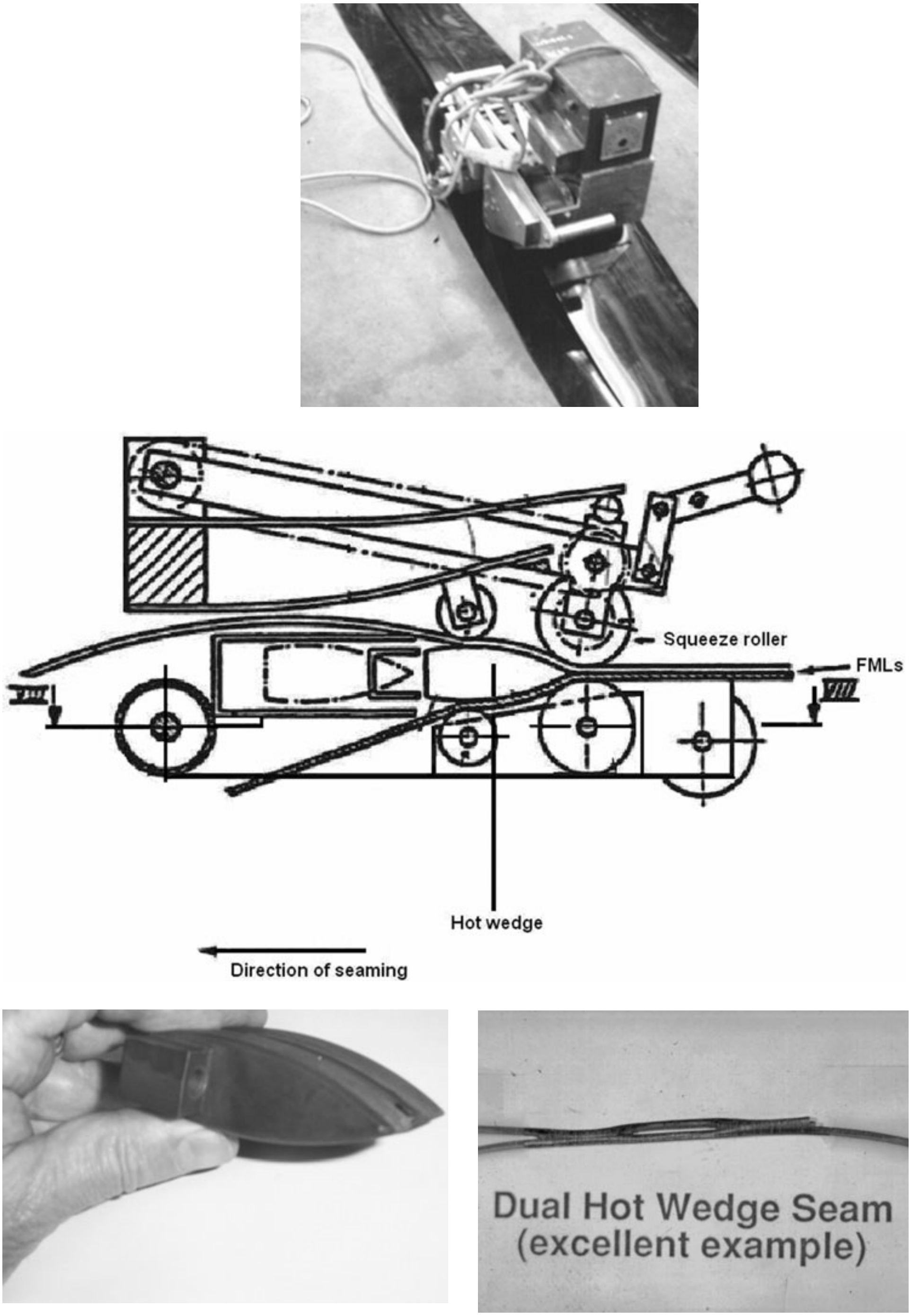


Figure 4-19. Photographs of Wedge Welding Devices, Details of the Device, a Dual-Track Wedge, and a Cross Section of the Subsequent Seam.

opposing surfaces. Immediately following the melting of the surfaces, pressure is applied to the seamed area to bond the two sheets. As with the hot wedge method, both single and dual seams can be produced. In selected situations, this technique may also be used to temporarily "tack" weld two sheets together until the final seam or weld is made and accepted. The method is not often used for production seams.

Extrusion welding is currently used on geomembranes made from polyolefins (HDPE, LLDPE, and fPP). After preparing the surfaces of the two geomembranes by light surface grinding, a ribbon of molten polymer is extruded over the edge of the two surfaces to be joined. The molten extrudate causes the prepared surfaces of the sheets to become hot and melt, after which the entire mass cools and bonds together. The extrudate should completely cover all surfaces that have been ground. The technique is called *extrusion fillet* seaming when the extrudate is placed over the leading edge of the seam and is called *extrusion flat* seaming when the extrudate is placed between the two sheets to be joined. The latter method is essentially depreciated at this time. Extrusion fillet seaming is essentially the only practical method for seaming polyolefin geomembrane patches, for seaming in poorly accessible areas such as sump bottoms and around pipes, and for seaming extremely short seam lengths. Temperature and seaming rate both play important roles in obtaining an acceptable bond; excessive melting weakens the geomembrane, and inadequate melting results in poor extrudate flow across the seam interface and low seam strength. The polymer used for the extrudate is also important and should be the same type and formulation used to make the geomembrane sheets being joined. The designer should specify acceptable extrusion compounds and how to evaluate them in the specifications and CQA documents.

Regarding the *chemical* seam types that are used on nonpolyolefin thermoplastic geomembranes, *chemical seams* make use of a liquid solvent (usually methyl ethyl keytone) applied between the two geomembrane sheets to be joined. After a few seconds (required to soften the surfaces), pressure is applied to make complete contact and bond the sheets together. As with any of the chemical seaming processes to be described, the two adjacent materials to be bonded are transformed into a viscous phase. Care must be used to see that the proper amount of chemical is applied to achieve the desired results. *Bodied chemical* seams are similar to chemical seams, except that 1 to 20% of the parent lining resin or compound is dissolved in the chemical and then is used to make the seam. The purpose of adding the resin or compound is to increase the viscosity of the liquid for slope work or adjust the evaporation rate of the chemical. This viscous liquid is applied between the two opposing surfaces to be bonded. After a few seconds, pressure is applied to make complete contact.

For thermoset geomembranes such as EPDM, *chemical adhesive* seams make use of a dissolved bonding agent (an *adhesive*) in the chemical or bodied chemical that is left after the seam has been completed and cured. The adhesive thus becomes an additional element in the system. *Contact adhesives* are applied to the lower mating surface. After reaching the proper degree of tackiness, the upper sheet is placed on top of the lower, followed by application of roller pressure. The adhesive forms the bond and is an additional element in the system.

Other seaming methods use ultrasonic, electrical conduction, and magnetic induction energy methods. Because these methods are in the developmental stage, they will not be described further in this book. See U.S. EPA (1991) for details.

To gain an overview as to which seaming methods can be used on the various geomembranes described in this book, Table 4-4 is offered. It is generalized, but it is used to introduce the primary seaming methods versus the type of geomembrane that can be seamed by that method.

4.4.2 Field Seaming Details

Full details of field seaming methods for the edges and ends of geomembrane rolls or panels has been described in the U.S. EPA Technical Guidance Document entitled: "Inspection Techniques for the Fabrication of Geomembrane Field Seams." In that document (U.S. EPA 1991), there are separate chapters devoted to the following field seaming methods:

- hot wedge seams,
- hot air seams,
- extrusion fillet seams,
- extrusion flat seams,
- chemical and bodied chemical seams, and
- chemical and contact adhesive seams.

There is also a section on emerging technologies for geomembrane seaming. The interested reader should consult the document for additional details regarding all of these seaming methods.

Table 4-4. Possible Field Seaming Methods for Various Geomembranes Listed in This Book

	Type of Geomembrane					
Seaming Method	*HDPE*	*LLDPE*	*fPP & fPP-R*	*PVC*	*CSPE-R*	*EPDM & EPDM-R*
Thermal fusion (hot wedge and hot air)	A	A	A	A	A	N/A
Extrusion (fillet and flat)	A	A	A	N/A	N/A	N/A
Chemical (chemical and bodied chemical)	N/A	N/A	N/A	A	A	N/A
Adhesive (chemical and contact)	N/A	N/A	N/A	A	A	A

Note: A, method is applicable and N/A, method is not applicable.

Whenever the plans and specifications are not written around a particular seaming method, the actual method that is used becomes a matter of choice for the installation contractor. As seen in Table 4-4, a number of available choices are available for each geomembrane type. Furthermore, even when the installation contractor selects the particular seaming method to be used, its specific details are rarely stipulated, even in the specification or CQA documents. This freedom gives the installation contractor the maximum latitude in selecting such options as seaming temperatures, travel rates, mechanical roller pressures, chemical type, tack time, and hand rolling pressure. The plans, specifications, and CQA documents adequately provide for destructive tests (on test strips and on production seams) and nondestructive tests (on production seams) to ensure that the seams are fabricated to the highest quality and uniformity and are in compliance with the project's documents.

This is not to say that the specification never influences the type of seaming method. For example, if the specifications call for a nondestructive air-pressure channel test to be conducted, the installation contractor must use a thermal fusion technique, such as the dual-track hot wedge method, because it is the only method (except for "hot air," which is rarely used) that can produce such a seam.

4.4.3 Trial Seams and Test Strips

Trial seams and test strips, also called qualifying seams, are an important aspect of CQC/CQA procedures. They are meant to serve as a prequalifying experience for personnel, equipment, and procedures for making seams on the identical geomembrane material under the same climatic conditions as the actual field production seams will be made. The trial seams are usually made on two narrow pieces of excess geomembrane varying in length between 1.0 and 3.0 m (3–10 ft) (Figure 4-20). The trial seams should be made in sufficient lengths, preferably as a single continuous seam, for all required testing purposes.

These trial seams are meant to reproduce all aspects of the actual production field seaming activities that are intended to be performed in the immediately up-

Figure 4-20. Fabrication of a Geomembrane Trial Seam.

coming work session to determine equipment and operator proficiency. Ideally, trial seams can be used to estimate the quality of the production seams while minimizing damage to the installed geomembrane through destructive mechanical testing. Trial seams are typically made every four hours (for example, at the beginning of the work shift and after the lunch break). They are also made whenever personnel or equipment are changed, when climatic conditions reflect wide changes in geomembrane temperature, or when other conditions occur that could affect seam quality. These details should be stipulated in the contract specifications or CQA.

The destructive testing of the trial seams should be done as soon as the installation contractor feels that the strength requirements of the contract specification or CQA documents can be met. Thus, it behooves the contractor to have all aspects of the trial seam fabrication in complete working order just as would be done in the case of fabricating production field seams. For extrusion and thermal fusion seams, destructive testing can be done as soon as the seam cools. For chemical and adhesive seams, this testing could take several days; the use of a field oven to accelerate the curing of the seam is advisable.

From two to six test specimens are cut from the trial seam using a 25 mm (1.0 in.) wide die. They are selected at random by the CQA inspector. The specimens are then tested in both peel and shear using a field tensiometer, as shown in Figure 4-21. Generally, peel tests are more informative in assessing the quality of the seam. If any of the test specimens fail, a new trial seam is fabricated. If additional test specimens fail, the seaming apparatus and seamer should not be accepted and should not be used for seaming until the deficiencies are corrected and successful trial welds are achieved. The CQA personnel should observe all trial seam procedures and tests. If the specimens pass, seaming operations can move directly to production seams in the field. Pass/fail criteria for destructive seam tests will be described later.

The flow chart illustrated in Figure 4-22 gives an idea of the various decisions that can be reached depending upon the outcome of destructive tests on trial seam

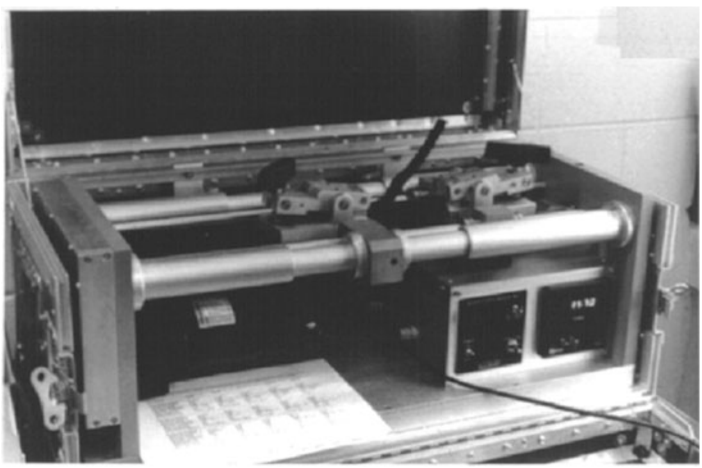


Figure 4-21. A Field Tensiometer Used To Assess Trial Seams.

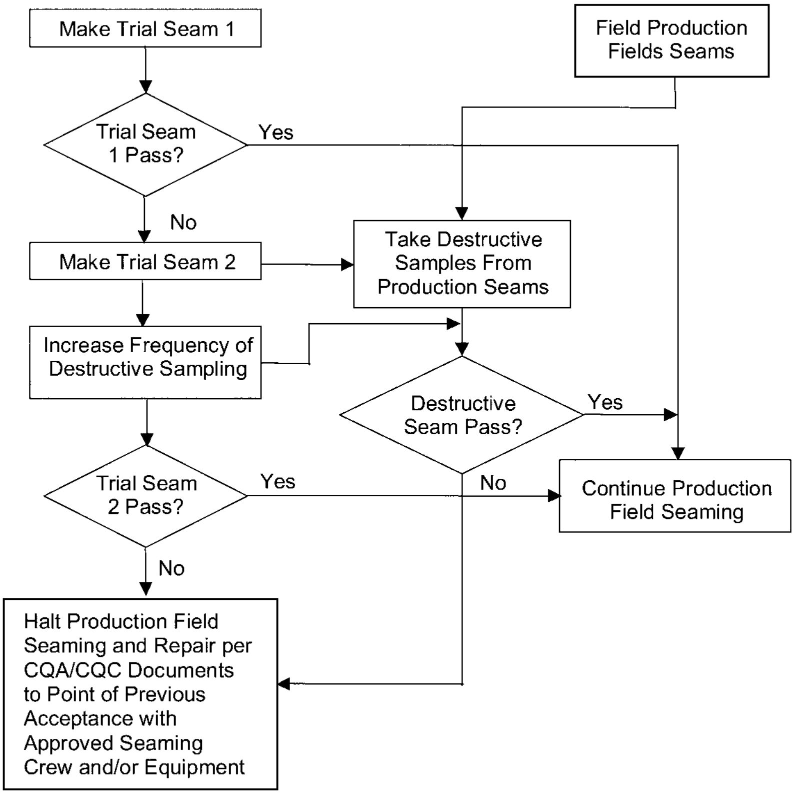


Figure 4-22. Trial Seam Process Flow Chart.

Source: U.S. EPA 1993.

specimens. Failed trial seams are linked to an increased frequency of destructive tests taken on production field seams made during the time between making the trial seams and their testing. Furthermore, there are only two chances at making adequate trial seams before production field seaming is stopped and retraining or equipment repairs are initiated. These details should be covered in either the project specification or the CQA documents.

GRI-GM19 is a generic specification available for HDPE, LLDPE, and fPP seams. Some specification or CQA document items regarding the fabrication of geomembrane trial seams (or test strips) include the following:

1. The frequency of making trial seams should be clearly stated. Typically this is at the beginning of the day, after the lunch break, and whenever changed conditions are encountered (e.g., changes in weather, equipment, or personnel).
2. The CQA personnel should have the option of requesting trial seams of any field seaming crew or device at any time.
3. The procedure for sampling and evaluating the trial seam samples should be clearly outlined (i.e., the number of peel and shear test specimens to be cut

and tested from the trial seam sample, the rate of testing, and what the required strength values are in these two different modes of testing).

4. The fabrication of the trial seam and testing of test specimens should be observed by the CQA personnel.
5. The time for testing after the trial seam is fabricated varies between seam types. For extrusion and thermal fusion fabricated seams, the testing can commence immediately after the seam cools to ambient temperature. For chemical and adhesive fabricated seams, the testing must wait until adequate curing of the seam occurs. This can take as long as one to seven days. During this time, all production seaming must be tracked and documented.
6. Accelerated oven curing of chemical and adhesive fabricated seams is acceptable to hasten the curing process and obtain test results as soon as possible.
7. The required inspection protocol and implications of failed test specimens from the trial seams must be clearly stated. The protocol outlined in Figure 4-22 is suggested.
8. Trial seam test specimens are usually discarded after they have been evaluated. If this is not the case, it should be clearly indicated who receives the test specimens and what should be the use (if any) of these spent specimens.

4.4.4 Destructive Test Methods for Seams

The major reason that plans and specifications do not have to be specific about the type of seaming methods and their particular details is that geomembrane seams can be readily evaluated for their quality by taking samples of the production seams and destructively testing them either at the job site or rapidly at a testing laboratory.

4.4.4.1 Overview

Destructive testing of geomembrane seams is the process of actually cutting out (i.e., to sample) and removing a portion of the completed production seam, and then further die cutting the sample into appropriately sized test specimens. These specimens are then tested, according to a specified procedure, to failure or to yield, depending on the type of geomembrane.

A possible procedure is to select the sampling location and cut two closely spaced 25 mm (1.0 in) wide test specimens from the seam. The distance between these two test specimens is defined later. The individual specimens are then tested in a peel mode using a field tensiometer (recall Figure 4-21). If the results are acceptable, the complete seam between the two field test specimens is removed, identified, and properly distributed. If either test specimen fails, two new locations on either side of the failed specimen are selected until acceptable seams are located. The seam distance between acceptable seams is usually repaired by cap stripping (see Section 4.4.4.6), but other techniques are also possible. The exact procedure must be stipulated in the specifications or CQA document.

The length dimension of the field seam sample between the two test specimens just described varies according to whatever is stipulated in the plans and specifications, or in accordance with the CQA documents. Some common options are to sample the seam for a distance of 36 cm (14 in.), 71 cm (28 in.), or 106 cm (42 in.) along its length. Because the usual destructive seam tests are either shear or peel tests and both types are 25 mm (1.0 in.) wide test specimens, this allows for approximately 10, 20, or 30 tests (half shear and half peel) to be conducted on the respective lengths cited above. The sample width perpendicular to the seam is usually 30 cm (12 in.), with the seam centrally located within this dimension.

The options for the seam sample lengths between the two peel test specimens mentioned above that are seen in various plans, specifications, and CQA documents, are as follows:

- A 36 cm (14 in.) sample is taken from the seam and cut into five shear and five peel specimens. ASTM D6392 gives a recommended template in this regard. The tests are conducted in the field or at a remote laboratory by, or under the direction of, the responsible CQA organization.
- A 71 cm (28 in.) long sample is taken from the seam and cut in half. One half is further cut into five shear and five peel test specimens, which are tested in the field or at a remote laboratory by the CQA organization (see ASTM D6392). The other half is sent to a remote laboratory for testing by the CQC organization, which also does five shear and five peel tests. Alternatively, sometimes only the CQA organization does the testing, and the second half of the sample is left intact and archived by the owner/operator.
- A 106 cm (42 in.) long sample is taken from the seam and cut into three individual 36 cm (14 in.) samples. Individual samples go to the CQA organization, the CQC organization, and the owner/operator. The CQA and CQC organizations each cut their respective samples into five shear and five peel test specimens and conduct the appropriate tests immediately (see ASTM D6392). The remaining sample is archived by the owner/operator.

Whatever the strategy for taking samples from the production seams for destructive testing, it must be clearly outlined in the contract plans and specifications and further defined or corroborated in the CQA documents.

Obviously, the hole created in the production seam from which the test sample was originally taken must be patched in an appropriate manner (see Figure 4-23 for such a patched sampling location). The seams of such patches are themselves candidates for field sampling and testing. If this sampling is done, one would have the end result of a patch on a patch, which is a rather unsightly and quite undesirable condition. Tie-in seams to the original production seam on both sides of the patch are particularly important because they require excellent care and workmanship.

4.4.4.2 Sampling Strategies

The sample spacing of production seams of installed geomembranes represents a dilemma of major proportions. Too few samples results in a poor statistical representation of the strength of the seam, and too many samples requires an additional cost and a risk of having the necessary repair patches being problems in them-

Figure 4-23. Completed Patch on a Geomembrane Seam That Had Previously Been Sampled for Destructive Tests.

selves. Unfortunately, there is no clear strategy for all cases, but the following are some of the choices that one has in formulating a specification or CQA plan.

Also, in selecting a sampling strategy, the sampling frequency is tied directly into the performance of the trial seams as described in Section 4.4.3. If the trial seams fail during the time that production seaming is ongoing, the spacing of destructive sampling and testing must be decreased. The following strategies, however, are for situations in which geomembrane trial seams are being made in an acceptable manner.

4.4.4.3 Fixed-Increment Sampling

A commonly used sampling strategy is the fixed-increment sampling method. In this method, a seam sample is taken at fixed increments along the total length of the seams. Increments usually range from one destructive sample in 75 m to one in 225 m (250–750 ft); a commonly specified value is one sample every 150 m (500 ft). This value can be applied directly to the record drawings during layout of the seams, to each seaming crew as they progress during the work period, or to each individual seaming device. Once the increment is decided on, it can be adjusted in accordance to the CQA inspector. For example, the CQA documents should allow exceptions such as avoiding sumps, connections, and protrusions. The CQA documents should also clearly allow the field inspector to take a destructive sample wherever he or she feels it is appropriate.

4.4.4.4 Randomly Selected Sampling

In random selection of destructive seam sample locations, it is first necessary to preselect a preliminary estimate of the number of samples to be taken. This estimate is done by taking the total seam length of the facility and dividing it by an arbitrary interval (e.g., 150 m (500 ft)) to obtain the total number of samples that are required. Two choices to define the actual sampling locations are now available: stratified random sampling and strict random sampling. The stratified method takes each preselected interval (e.g., a 150 m (500 ft) length) and randomly selects a single sample location within this interval. Thus, with stratified random sampling, one has location variability within a fixed increment (unlike fixed frequency sampling, which is always at the exact end of the increment). The strict method uses the total seam length of the facility (or cell) and randomly selects sample locations throughout the facility up to the desired number of samples. Thus, with strict random sampling, a group of samples may be taken close to one another, which necessarily leaves other areas with sparse sampling.

There are various ways of randomly selecting the specific location within an interval (e.g., in a specific region of great concern) or within the total project seam length. These methods are as follows:

- Use a random number generator from statistical tables to predetermine the sampling locations within each interval or for the entire project.
- Use a programmable pocket calculator with a random number generator program to select the sampling location in the field for each interval or for the entire project.
- Use a random number obtained by multiplying two large numbers together to form an 8-digit result. A pocket calculator with an adequate register will suffice. The center two digits in such a procedure are quite randomly distributed and can be used to obtain the sampling location. For example, multiplication of the following two sequences of numbers, "4567 by 4567," gives 20857489, where the central two digits, i.e., the "57," are used to select the location within the designated sampling interval. If this interval were 500 ft, the sampling location within it would be at $0.57 \times 500 = 285$ ft from the beginning of the interval. The next location of the sample would require a new calculation, resulting in a different central two-digit number somewhere within the next 500-ft sampling interval and would be located in a similar fashion.

4.4.4.5 Other Sampling Strategies

The *method of attributes* uses a statistical procedure to vary the distance between destructive samples in accordance to the past and ongoing performance of the seaming crew. Beginning with a given failure rate based on past history (e.g., 2.0%), a listing of failures is accumulated for approximately the first 30 destructive test results. If the initial failure rate is lower than the initially assumed rate, the sampling interval is increased. If the initial failure rate is higher, the sample interval is decreased. The entire project is controlled in this manner; this method is best suited for large projects. Thus, good seaming is rewarded by fewer destructive samples, and poor seaming is penalized by more destructive samples. The procedure has been formalized as GRI-GM14.

Use of *control charts* is similar to attributes insofar as rewarding good seaming and penalizing poor seaming. It can be used for jobs of any size, large or small. Upper and lower bounds are initially set based on the historic performance of the seaming crew. When the failure rate crosses the upper or lower limits, the sampling interval is shortened or lengthened accordingly. The method is described by Richardson (1992) and is formalized in GRI-GM20.

4.4.4.6 Shear Testing of Geomembrane Seams

Shear testing of specimens taken from field-fabricated geomembrane seams represents a reasonably simulated performance test. The exception is that a normal stress is not applied to the surfaces of the test specimen, thus it is an "unconfined" tension test. A slight rotation may be induced during tensioning of the specimen, making the actual test results tend toward conservative values. The conducting of a shear test in the grips of a tension-testing machine is shown in Figure 4-24.

Commonly recommended shear tests for various geomembrane seams, along with the methods of testing the unseamed sheet material in tension, are given in Table 4-5. Major differences are in the test specimen shapes, sizes, and strain rates. It is difficult to compare results from one type of geomembrane to another.

Insofar as the shear testing of nonreinforced geomembrane seams, all use a 25 mm (1.0 in.) wide test specimen with the seam centrally located within the testing grips. For the reinforced geomembranes, a "grab" test specimen is used. In a grab tension test, the specimen is 200 mm (4.0 in.) wide but is only gripped in the

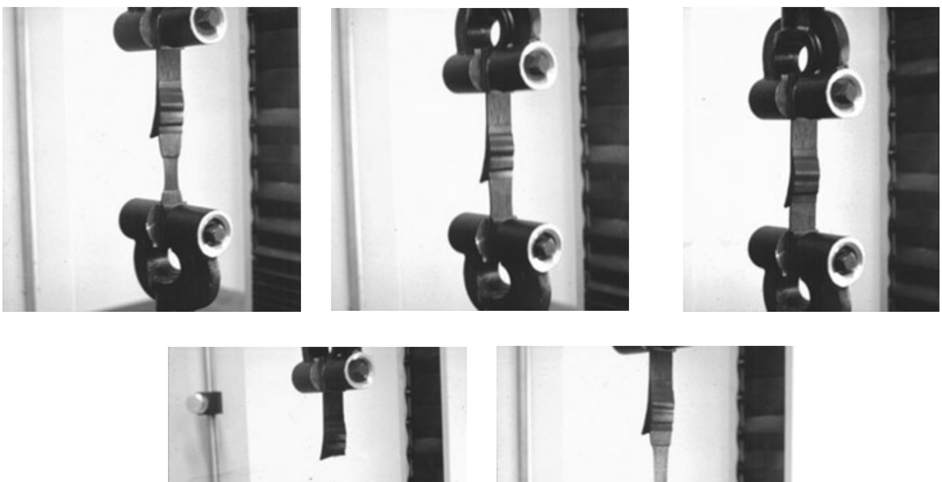


Figure 4-24. Shear Test of an HDPE Geomembrane Seam Evaluated in a CQC/CQA Laboratory.

Table 4-5. Recommended Test Method Details for Geomembrane Seams in Shear and in Peel and for an Unseamed Sheet

	HDPE		*LLDPE, fPP*		*PVC, EPDM*		*fPP-R, CSPE-R, EPDM-R, EIA-R*	
Type of Test	*S.I.*	*Customary*	*S.I.*	*Customary*	*S.I.*	*Customary*	*S.I.*	*Customary*
Shear Test on Seams								
ASTM test method	D6392		D6392		D882		D751	
Specimen shape	Strip		Strip		Strip		Grab	
Specimen width	25 mm	1.00 in.	25 mm	1.00 in.	25 mm	1.00 in.	100 mm (25 mm grab)	4.00 in. (1.00 in. grab)
Specimen length	150 mm + seam	6.00 in. + seam	150 mm seam	6.00 in. + seam	150 mm seam	6.00 in. + seam	225 mm + seam	9.00 in. + seam
Gage length	100 mm + seam	4.00 in. + seam	100 mm seam	4.00 in. + seam	100 mm seam	4.00 in. + seam	150 mm seam	6.00 in. + seam
Strain rate	50 mm/min	2.0 in./min	500 mm/min	20 in./min	500 mm/min	20 in./min	300 mm/min	12 in./min
Strength	Force (N)/ (25 mm × t mm)	Force (lb)/ (1.00 in. × t in.)	Force (N)/ (25 mm × t mm)	Force (lb)/ (1.00 in. × t in.)	Force (N)/ (25 mm × t mm)	Force (lb)/ (1.00 in. × t in.)	Force (N)	Force (lb)
Peel Test on Seams								
ASTM test method	D6392		D6392		D882		D413	
Specimen shape	Strip		Strip		Strip		Strip	
Specimen width	25 mm	1.00 in.	25 mm	1.00 in.	25 mm	1.00 in.	25 mm	1.00 in.
Specimen length	100 mm	4.00 in.	100 mm	4.00 in.	100 mm	4.00 in.	100 mm	4.00 in.
Gage length	N/A		N/A		N/A		N/A	
Strain rate	50 mm/min	2.0 in./min	500 mm/min	20 in./min	500 mm/min	20 in./min	50 mm/min	(2.0 in./min)
Strength (N/mm^2; lb/in.^{2})	Force (N)/ (25 mm × t mm)	Force (lb)/ (1.00 in. × t in.)	Force (N)/ (25 mm × t mm)	Force (lb)/ (1.00 in. × t in.)	Force (N)/ (25 mm × t mm)	Force (lb)/ (1.00 in. × t in.)	Force (N)	Force (lb)
Tensile Test on Sheet								
ASTM test method	D6693		D6693		D882		D751	
Specimen shape	Dumbbell		Dumbbell		Strip		Grab	
Specimen width	6.3 mm	0.25 in.	6.3 mm	0.25 in.	25 mm	1.00 in.	100 mm (25 mm grab)	4.00 in. (1.00 in. grab)
Specimen length	115 mm	4.50 in.	115 mm	4.50 in.	150 mm	6.00 in.	150 mm	6.00 in.
Gage length	33 mm	1.30 in.	33 mm	1.30 in.	50 mm	2.00 in.	75 mm	3.00 in.
Strain rate	50 mm/min	2.0 in./min	50 mm/min LLDPE; 500 mm/min fPP	2.0 in./min LLDPE; 20 in./min fPP	500 mm/min	20 in./min	300 mm/min	12 in./min
Strength	Force (N)/ (6.3 mm × t mm)	Force (lb)/ (0.25 in. × t in.)	Force (N)/ (6.3 mm × t mm)	Force lb)/ (0.25 in. × t in.)	Force (N)/ (25 mm × t mm)	Force (lb)/ (1.00 in. × t in.)	Force (N)	Force (lb)
Strain	Elong. (mm)/33 mm	Elong. (in.)/1.30 in.	Elong. (mm)/33 mm	Elong. (in.)/1.30 in.	Elong. (mm)/50 mm	Elong. (in.)/2.00 in.	Elong./75 mm/mm	Elong./3.00 in./in.
Modulus	From Graph		From Graph		From Graph		N/A	

Note: mm/min, millimeters per minute; in./mm, inches per minute; N/mm^2, Newtons per square millimeter of specimen cross section; N/mm, Newtons per linear millimeter width of specimen; lb/in.^{2}, pounds per square inch of specimen cross section; lb/in., pounds per linear inch width of specimen; Force, maximum force attained at specimen yield or break; t, geomembrane thickness; and N/A, not applicable.

central 25 mm (1.0 in.). The test specimen is tensioned at its designated strain rate until failure occurs. If the seam delaminates (i.e., pulls apart in a seam separation mode), the seam fails because the objective is to have the sheet fail on either side of the seam. In this case, the specimen is rejected as a failed seam. Details on various types of seam failures and on the interpretation of locus-of-break codes are found in ASTM D6392. Conversely, if the seam does not delaminate but fails in the adjacent sheet material on either side of the seam, it is an acceptable failure mode, and the seam strength and elongation are compared to the specified values.

There are usually three criteria to be met for a passing seam shear test (see GRI-GM19):

1. The pattern of failure must be identified. These patterns, called locus-of-break codes, are identified in ASTM D6392. In general, the geomembrane on either side of the seamed area must fail. The seam itself cannot delaminate in whole or part.
2. The seam strength (i.e., its maximum recorded value) must equal or exceed the specified value for that precise material and thickness. Generic specifications are available from the Geosynthetic Institute (for HDPE, LLDPE, and fPP), the PVC Geomembrane Institute (for PVC), and some state regulatory agencies.
3. The seam elongation before break must exceed a specified value. This requirement is meant to ensure that embrittlement of the seam has not occurred during its joining or welding. The procedure is described in ASTM D6392; the minimum value should be 50% or higher.

The test is difficult to perform on the inside of the tracks facing the air channel of dual-channel thermal fusion seams. For small air channels, the tab available for gripping will be considerably less than that required in test methods as given in Table 4-5. Regarding the testing of the inside or outside tracks (away from the air channel) of a dual-channel thermal fusion seam, or even both tracks, the specification or CQA document should be specific.

Finally, the number of failures allowed per number of tests conducted should be addressed. If sets of five shear test specimens are performed for each field sample, many specifications allow for one failure of the five tested. Furthermore, this outlier must be at least 80% of the specified value. If the failure number is larger, then the plans, specifications, or CQA documents must be clear on the implications.

When a destructive seam test sample fails, many specifications and CQA documents require two additional destructive samples to be taken, one on each side of the original sample and spaced 3 m (10 ft) from it. If either one of these samples fails, the iterative process of sampling every 3 m is repeated until passing test results are observed. In this case, the entire seam length between the two successful test samples must be questioned. Remedies are to cover the entire seam with a narrow piece of the same type of geomembrane, called a “cap strip” or, if the seam is made with a thermal fusion method (e.g., hot wedge), to extrude a fillet weld over the outer seam edge. When such repairs are concluded, the seams on the cap strip or extrusion fillet weld may be sampled and tested as just described.

4.4.4.7 Peel Testing of Geomembrane Seams

Peel testing of specimens taken from field-fabricated geomembrane seams represents a QC type of index test. Such tests are not meant to simulate in situ performance but are important indicators of the overall quality of the seam. The conducting of a peel test in the grips of a tension-testing machine is shown in Figure 4-25.

The recommended peel tests for various geomembrane seams, along with the unseamed sheet material in tension are given in Table 4-5. Major differences between geomembrane types are in the test specimen shapes, sizes, and strain rates. As such, it is difficult to compare results from one type of geomembrane to another.

Insofar as the peel testing of geomembrane seams is concerned, all of the nonreinforced geomembranes listed have a 25 mm (1.0 in.) width test specimen. Furthermore, the specimen lengths and strain rate are also equal for all geomembrane types. In a peel test for nonreinforced geomembranes, the test specimen is tensioned at its appropriate strain rate until failure occurs. The seam strength is the maximum force attained divided by the specimen width (resulting in units of force per unit width) or by the specimen cross-sectional area (resulting in units of stress). The former procedure is the most common (i.e., peel strengths are measured in force per unit width). If stress units are desired, the thickness of the geomembrane sheet must be included. The nominal sheet thickness is then used. If the actual sheet thickness is used, many thickness measurements will be required to obtain a statistically reliable value. It is difficult to measure and is not a recommended procedure. The difference for reinforced geomembranes is that a

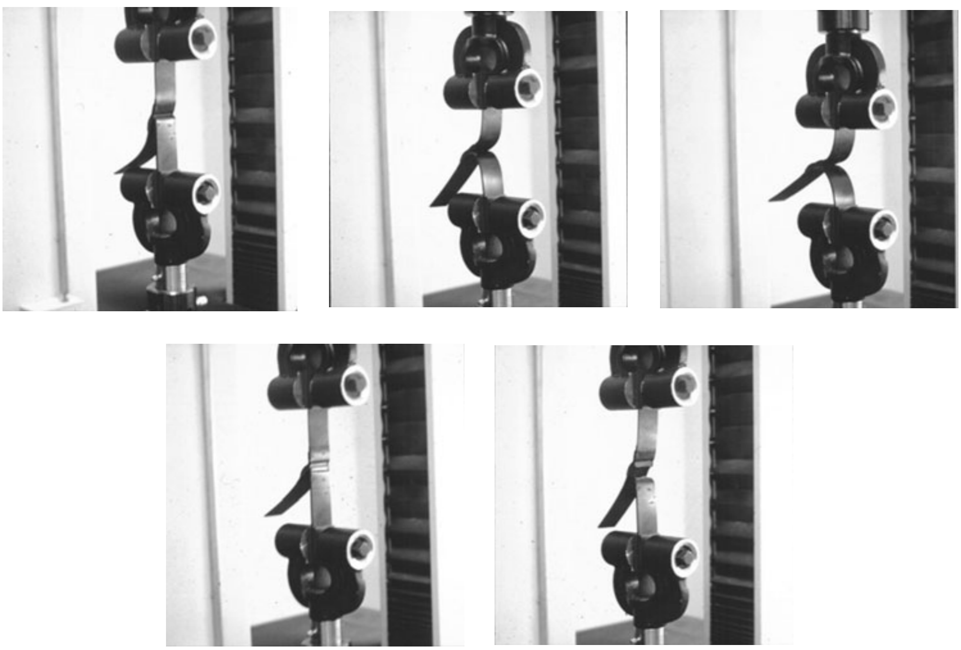


Figure 4-25. Peel Test of an HDPE Geomembrane Seam Evaluated in a CQC/CQA Laboratory.

grab tension test is used in which only the central 25 mm (1.0 in.) is tensioned. The breaking force is then reported in units of Newtons or pounds.

There are usually three criteria to be met for a passing seam peel test (see GRI-GM19):

1. The pattern of failure must be identified. These patterns, called locus-of-break codes, are identified in ASTM D6392. In general, the geomembrane on either side of the seamed area must fail. The seam itself cannot delaminate in whole or part.
2. The seam strength (i.e., its maximum recorded value) must equal or exceed the specified value for that precise material and thickness. Generic specifications are available from the Geosynthetic Institute (for HDPE, LLDPE, and fPP), the PVC Geomembrane Institute (for PVC), and many state regulatory agencies. Manufacturers also have specifications for their particular materials.
3. The seam separation (also called "incursion") before break must not exceed a specified value. Essentially, this requirement is meant to ensure that the majority of the seam is contributing to the imposed tensile force. The procedure is described in ASTM D6392, and the maximum value should be 25% or lower.

For dual-channel seams, both insides of the tracks facing the air channel can be tested, but because of the narrow width of most air channels the tab available for gripping will be considerably less than that given in Table 4-5. Regarding the testing of the inside or outside tracks (away from the air channel) of a dual-channel seam, or even both tracks, the specification or CQA document should be specific.

Finally, the number of failures allowed per number of tests conducted should be addressed. If sets of five peel test specimens are performed for each field sample, many specifications allow for one failure of the five tested. Furthermore, this outlier must be at least 80% of the specified value. If the failure number is larger, then the plans, specifications, or CQA documents must be clear on the implications.

When a destructive seam test sample fails, many specifications require an additional two destructive samples to be taken, one on each side of the original, spaced 3 m (10 ft) from it. If either one of these samples fails, the iterative process of sampling every 3 m (10 ft) is repeated until successful samples result. In this case, the entire seam length between the last successful test samples must be questioned. Remedies are to cap strip the entire seam or, if the seam is made with a thermal fusion method (e.g., hot wedge), to extrude a fillet weld over the outer seam edge. When this procedure is done, the seams on the cap strip or extrusion fillet weld may be sampled and tested as just described.

4.4.4.8 General Specification Items

Regarding field sampling of geomembrane seams and their subsequent destructive testing, a specification or CQA document should consider the following items:

1. CQA personnel should observe the cutting, removal, and distribution of all production seam samples.

2. All samples should be adequately numbered and marked with permanent identification.
3. All sample locations should be indicated on the geomembrane layout (and record) drawings.
4. The reason for taking the sample should be indicated (e.g., statistical routine, suspicious feature, or change in temperature).
5. The sample length and width should be given. The seam will generally be located along the center of the length of the sample.
6. The distribution of various portions of the sample (if more than one) should be specified.
7. The number of shear and peel tests to be conducted on each sample (field tests and laboratory tests) should be specified.
8. If a generic specification is being used, the particular version must be identified or at least a comment that the most recent revision is to be used.
9. If a modification to a generic specification is used, the modification must be clearly stated in the specifications or CQA documents.
10. The CQA personnel should witness all field tests and see that proper identification and details accompany the test results. Details should be provided in the CQA documents. Such details, as follows, are often required:
    - date and time;
    - ambient temperature;
    - identification of seaming unit, group, or machine;
    - name of master seamer;
    - welding apparatus temperature and pressure, or chemical type and mixture;
    - pass or fail description; and
    - a copy of the report should be attached to the remaining portion of the sample.
11. The CQA personnel should verify that samples sent to the testing laboratory are properly marked, packaged, and shipped so as not to cause damage.
12. Results from the laboratory tests should be sent to the CQA engineer within a stipulated time. For extrusion and thermally bonded seams, verbal test results are sometimes required 24 to 72 hours after the laboratory receives the samples. For chemically bonded seams, the time frame is longer and depends on whether or not accelerated heat curing of the seams is required (see ASTM D6214 or GRI-GM7 in this regard). In all cases, the CQA engineer must inform the owner's representative of the results and make appropriate recommendations.
13. The procedures for seam remediation in the event of failed destructive tests should be clear and unequivocal. Options usually are (i) to repair the entire seam between acceptable sampling locations or (ii) to retest the seam on both sides in the vicinity of the failed sample. If these tests are acceptable, only this section of the seam is repaired. If they are not, wider spaced samples are to be taken and tested.
14. Repairs to locations where destructive samples were removed should be stipulated. These repairs are specific to the type of geomembrane and to the seaming method. Guidance in this regard is available in U.S. EPA (1991).

15. Each repair of a patched seam where a test sample had been removed should be verified. This repair is usually done visually and by an appropriate nondestructive test. If, however, the sampling strategy selected calls for a destructive test to be made at the exact location of a patch, it should be accommodated. Thus, the final situation will require a patch to be placed on an earlier patch. If this (unsightly) detail is to be avoided, it should be stated outright in the specifications or CQA document.
16. The time required to retain and store the remains of destructive test samples on the part of the CQC and CQA organizations should be stipulated.

4.4.5 Nondestructive Test Methods for Seams

Although it is obviously important to conduct destructive tests on the fabricated seams, such tests do not give adequate information on the continuity and completeness of the entire seam between sampling locations. It does little good if one section of a seam meets the specification requirements, only to have the section next to it missed completely by the field-seaming crew. Thus, continuous methods of nondestructive testing (NDT) are discussed in this section (Table 4-6). In each of these methods, the goal is to validate 100% of the seams or at least a major percentage of them.

4.4.5.1 Currently Available Methods

The currently available NDT methods for evaluating the adequacy of geomembrane field seams are listed in Table 4-6 in the order that they will be discussed.

The *dual-seam* method using positive air pressure was mentioned earlier in connection with the dual hot wedge or dual hot air thermal seaming methods. The air channel that results between the two bonded tracks is inflated using a hypodermic needle and is pressurized to approximately 200 kPa (30 lb/in.^{2}) (Figure 4-26). There is no limit as to the length of the seam that can be tested. If the pressure drop is within an allowable amount in the designated time period (usually 5 min), the seam is acceptable; if an unacceptable drop occurs, a number of actions can be taken:

1. The seam distance can be systematically halved and the process repeated until the leak is located,
2. the seam length in question can be tested by some other leak detection method,
3. an extrusion fillet weld can be placed over the entire leading edge of the seam, or
4. a cap strip can be seamed over the entire seam length involved.

Details of the test can be found in ASTM D6392 and GRI-GM6. This test is well suited for long, straight seam lengths. It is generally performed by the installation contractor, with CQA personnel viewing the procedure and documenting the results. It is considered by most people to be the premier nondestructive testing method for geomembrane seams.

Table 4-6. Nondestructive Geomembrane Seam Testing Methods

	Primary User		*General Comments*					
Nondestructive Test Method	*CQC*	*CQA*	*Cost of Equipment*	*Speed of Tests*	*Costs of Tests*	*Type of Result*	*Recording Method*	*Operator Dependency*
1. Dual-seam (positive pressure)	Yes	—	\$500	Fast	Moderate	Yes–no	Manual	Low
2. Vacuum chamber (negative pressure)	Yes	Yes	\$1,200	Slow	Very high	Yes–no	Manual	High
3. Air lance	Yes	—	\$200	Fast	Low	Yes–no	Manual	High
4. Mechanical point stress (pick test)	Yes	—	\$0	Fast	\$0	Yes–no	Manual	Very high
5. Electric sparking	Yes	Yes	\$3,000	Fast	Low	Yes–no	Manual	Moderate
6. Electric wire	Yes	Yes	\$500	Fast	Low	Yes–no	Manual	Moderate
7. Ultrasonic pulse echo	—	Yes	\$5,000	Moderate	High	Yes–no	Automatic	Moderate
8. Ultrasonic impedance plane	—	Yes	\$7,000	Moderate	High	Qualitative	Automatic	Unknown
9. Ultrasonic shadow	—	Yes	\$5,000	Moderate	High	Qualitative	Automatic	Moderate

Note: The electrical leak survey (ELLS) method is not included in this table. The ELLS method evaluates the entire geomembrane (seams and sheet) and will be described in Section 4.4.6.

Source: Adapted from Richardson and Koerner 1988.

The *vacuum chamber* (or *box*) method uses a box up to 1 m (3 ft) long with a transparent top that is placed over the seam; a vacuum of approximately 20 kPa (3 lb/in.^{2}) is applied. When a leak is encountered, the soapy solution originally placed over the seam shows bubbles, thereby reducing the vacuum. This reduction is due to air entering from beneath the geomembrane and passing through the unbonded zone. The test is slow to perform (a 10-s dwell time is currently recommended), and it is often difficult to make a vacuum-tight joint at the bottom of the box where it passes over the seam edges. Because of upward deformations of the liner into the vacuum box, only geomembrane thickness greater than 1.0 mm (40 mils) should be tested in this manner. For thinner, more flexible geomembranes, an open grid wire mesh can be used along the bottom of the box to prevent uplift. All of the field seams cannot be inspected by the vacuum chamber method. The test cannot cover portions of sumps, anchor trenches, and pipe penetrations with any degree of assurance. The method is also awkward to use on

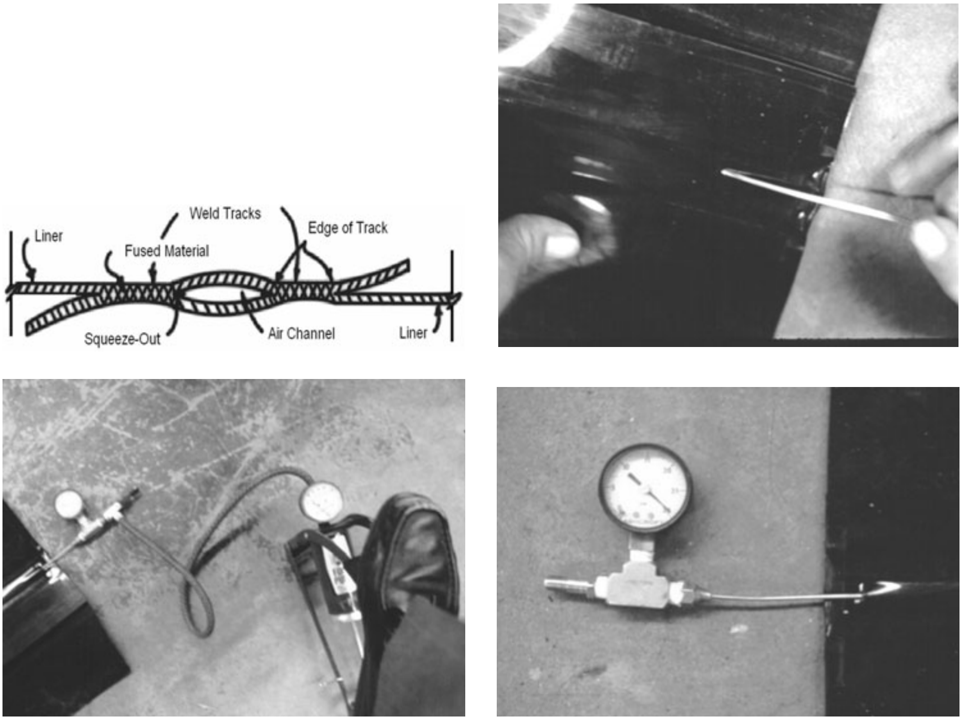


Figure 4-26. Various Steps in Setup and Performance of Air Pressure Test on a Dual-Track Hot Wedge Seam.

sideslopes. The adequate downward pressure required to make a good seal is difficult to mobilize because it is usually done by standing on top of the box.

The *air lance* method uses a jet of air at approximately 350 kPa (50 lb/in.^{2}) pressure coming through an orifice of 5 mm (3/16 in.) diameter. The jet of air is directed beneath the upper edge of the overlapped seam at a distance of about 100 mm (4.0 in.) from the edge of the seamed area to detect unbonded areas. When such an area is located, the air passes through the opening in the seam, causing inflation and fluttering in the localized area. A distinct change in the sound emitted can generally be heard. The method works best on relatively thin (less than 1.1 mm (45 mils)) flexible geomembranes, but it works only if the defect is open at the front edge of the seam where the air jet is directed. It is essentially a geomembrane installer's method to be used in a CQC manner.

The *mechanical point stress* or *pick test* uses a dull tool, such as a blunt screwdriver, under the top edge of a seam. With care, an individual can detect an unbonded area, which would be easier to separate than a properly bonded area. This rapid test depends completely on the care and sensitivity of the person doing it. Detectability is similar to that of the air lance method, but both are very operator dependent. This test is to be performed only by the geomembrane installer as a CQC method. Design or CQA personnel should not use the pick test because of potential damage to the geomembranes.

Electric sparking is a technique used to detect pinholes in thermoplastic liners. The method uses a high-voltage (15–30 kV) current, and any leakage to ground (through an opening or hole) results in sparking. The method is used effectively with coextruded conductive geomembranes.

The *electric wire* method places a copper or stainless steel wire between the overlapped geomembrane regions and actually embeds it into the completed seam. After seaming, a charged probe of about 20,000 V is connected to one end of the wire and slowly moved over the length of the seam. A seam defect between the probe and the embedded wire results in an audible alarm from the unit.

The last group of nondestructive test methods noted in Table 4-6 are collectively called ultrasonic methods. Although they have been shown in the laboratory to be technically viable, they are rarely used in practice. The *ultrasonic pulse echo* technique is basically a thickness measurement technique and is only for use with nonreinforced geomembranes. Here a high-frequency pulse is sent into the upper geomembrane and (in the case of good acoustic coupling and good contact between the upper and lower sheets) reflects off the bottom of the lower one. If, however, an unbonded area is present, the reflection will occur at the unbonded interface. The use of two transducers, a pulse generator, and a CRT monitor are required. It cannot be used for extrusion fillet seams because of their nonuniform thickness. The *ultrasonic impedance plane* method works on the principle of acoustic impedance. A continuous wave of 160 to 185 kHz is sent through the seamed geomembrane, and a characteristic dot pattern is displayed on a CRT screen. Calibration of the dot pattern is required to signify a good seam. The method has potential for all types of geomembranes but still needs additional developmental work. The *ultrasonic shadow* method uses two roller transducers: One sends a signal into the upper geomembrane, and the other receives the signal from the lower geomembrane on the other side of the seam (Richardson and Koerner 1988). Good seams receive a strong signal, whereas poor seams receive a weak signal or none at all. The technique can be used for all types of seams, even those in difficult locations, such as those around manholes, sumps, or appurtenances. It is best suited for semicrystalline geomembranes, such as HDPE, and will not work for scrim-reinforced liners. Field trials to date have not been very successful.

4.4.5.2 Recommendations for Various Seam Types

The various NDT methods listed in Table 4-6 have certain uniqueness and applicability to specific seam and geomembrane types. Thus a specification should only be framed around the particular seam type and geomembrane type for which it has been developed. Table 4-7 gives guidance in this regard. Even within this table, there are certain historical developments. For example, the air lance method is used routinely on the flexible geomembranes seamed by chemical methods, whereas the vacuum chamber method is used routinely on the relatively stiff HDPE geomembranes. Also, the dual seam can technically be used on all geomembranes, but only when they are seamed by a dual-track thermal fusion method (typically by the hot wedge rather than hot air seaming method). Thus, by requiring such a dual-seam pressure test method, one mandates the type of seam that is to be used by the installation contractor.

Table 4-7. Applicability of Various Nondestructive Test Methods to Different Seam Types and Geomembrane Types

NDT Method	*Seam Types*	*Geomembrane Types*
1. Dual-seam (positive pressure)	HW, HA	All except EPDM
2. Vacuum chamber	All	All
3. Air lance	C, BC, Chem A, Cont. A	All except HDPE
4. Mechanical point stress	All	All
5. Electric sparking	All	All
6. Electric wire	All	All
7. Ultrasonic pulse echo	HW, HA, C, BC, Chem. A, Cont. A	All except reinforced
8. Ultrasonic impedance plane	HW, HA, C, BC, Chem. A., Cont. A	All except reinforced
9. Ultrasonic shadow	E Fil., E Flt., HW, HA	All except reinforced

Note: HW, hot wedge; HA, hot air; EPDM, ethylene propylene diene terpolymer; C, chemical; BC, bodied chemical; Chem. A, chemical adhesive; Cont. A, contact adhesive; E Fil., extrusion fillet; E Flt., extrusion flat.

Finally, it should be mentioned that only three of the nine methods listed in Tables 4-6 and 4-7 are used routinely at this time. They are the dual-seam with positive air pressure, vacuum chamber, and air lance methods. The others are either uniquely used by installation contractors (pick test and electric wire) or are in the research and development stage (the various ultrasonic test methods). An additional method, the *electrical leak location survey* method, will be described in Section 4.4.6. It identifies leaks in both seams and sheet and can be used after backfilling of the geomembrane occurs.

4.4.5.3 General Specification Items

Regarding field evaluation of geomembrane seams and their nondestructive testing, a specification or CQA document should consider the following items:

1. The purpose of nondestructive testing should be clearly stated. For example, nondestructive testing is meant to verify the continuity of field seams and not to quantify seam strength.
2. Generally, nondestructive testing is conducted as the seaming work progresses or as soon as a suitable length of seam is available.
3. Generally, nondestructive testing of some type is required for 100% of the field seams. For geomembranes supplied in factory-fabricated panels, the factory seams may or may not be specified to be nondestructively tested in the field. This decision depends on the degree of MQC (and MQA) required on the factory-fabricated seams.
4. The specification should recognize that the same type of nondestructive test cannot be used in every location. For example, in sumps and at pipe penetrations, the dual-seam and vacuum chamber methods may not be possible for use.
5. It must be recognized that many of the methods mentioned have no standardized test method available. In such cases, referencing to consensus documents is not possible.

6. CQA personnel should observe all nondestructive testing procedures.
7. The location, date, test number, name of test person, and outcome of tests must be recorded.
8. The owner's representative should be informed of any deficiencies.
9. The method of repair of deficiencies found by nondestructive testing should be clearly outlined in the specifications or CQA documents, as should the retesting procedure.

4.4.6 Electrical Leak Location Surveying

The electrical leak location survey method (or ELL method) impresses a high voltage across the geomembrane and then detects the precise locations where electrical current flows through leaks in the electrically insulating geomembrane. The development of the technology was initiated by the U.S. EPA in 1980 (Peters et al. 1982). The first commercial leak location surveys for earth-covered and water-covered geomembranes were performed in 1985. Numerous organizations currently provide such surveys.

Figure 4-27 illustrates the technique for the primary liner system of soil-covered geomembranes. If the secondary liner system is to be investigated, it must be done before the primary liner system is placed. Most importantly, the method can be performed after the soil materials have been placed on the geomembrane, for example, after placement of a leachate collection and removal layer. As shown, electrodes are placed in contact with conducting material above and below the geomembrane. A voltage is applied between these electrodes using an isolated power supply. Because the geomembrane is an electrical insulator, electrical current will flow only if there are holes in the geomembrane. Electrical potential data is collected using two additional moveable electrodes on the surface material in a

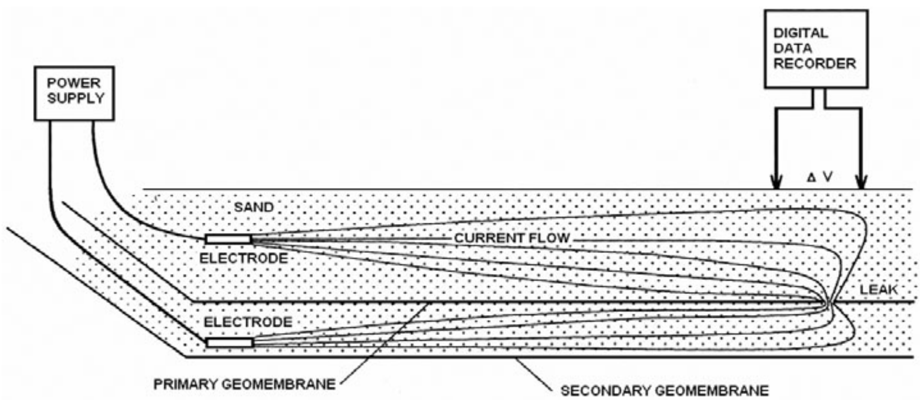


Figure 4-27. Principle of the Electrical Leak Location Survey Method for Sand-Covered Geomembrane Liners.

Source: Darilek and Miller 1998, with permission from Industrial Fabrics Association International.

grid or survey line pattern within the facility. The data are recorded manually or with portable data loggers for plotting and analysis. Characteristic leak signals caused by current flowing through the leaks indicate the locations of the leaks.

Depending on the conductivity of the materials above and below the geomembrane, the power required can be as low as 1 W. For surveys with earth cover, the maximum power usually needed is on the order of a few hundred watts.

The geomembrane leak location method is standardized in ASTM D6747 and D7002. These standards describe several electrical leak location implementation methods. These are as follows:

- a method for water-covered geomembranes;
- a method using an electrically conductive layer on one surface of a specialty geomembrane;
- a method for geomembranes covered with earth materials;
- a preinstalled grid system for monitoring leaks throughout the active life of the facility;
- a water puddle method for uncovered geomembranes; and
- use of electrically conductive sheet.

Papers by Nosko et al. (2002), Rollin et al. (2002), Darilek and Laine (2005), and Peggs (2006) have been published describing the leak survey methods and results of surveys. The method's implementation will be discussed in Section 4.6.

4.5 Protection and Backfilling

The field-deployed and -seamed geomembrane must be backfilled with soil or covered with a subsequent layer of geosynthetics in a timely manner after its acceptance by the CQA personnel. If the covering layer is soil, it will generally be a drainage material such as sand or gravel depending on the required permeability of the overlying layer. Depending on the particle size, hardness, and angularity of this soil, a geotextile or other type of protection layer will often be necessary. If the covering layer is a geosynthetic, it will generally be a geonet or geocomposite drainage material, which is usually placed directly on the geomembrane. This placement is a critical step because geomembranes are relatively thin materials with puncture and tear strengths of finite proportions. Specifications should be clear and unequivocal regarding this final step in the installation survivability of geomembranes.

4.5.1 Equipment Considerations

Most earthwork equipment used in heavy construction is too large and heavy to provide safety to the underlying geomembrane. Furthermore, such equipment cannot negotiate tight areas, particularly when those areas are lined with geosynthetics. Equipment practices (e.g., speed, orientation, turning, stopping, and starting) must be greatly limited depending on the site-specific circumstances. Some general specification items will be given at the end of this section. These specifications are quite generalized; however, under no circumstances should construction equipment drive directly on any geomembrane.

4.5.2 Geosynthetic Coverings

Various geosynthetic materials may be used to cover the deployed and seamed geomembrane. Often a geotextile, a geonet, or a geocomposite will be the covering material. Sometimes, however, it will be a geogrid for cover soil reinforcement on sideslopes. Construction equipment cannot be allowed to operate or drive directly on any of the geosynthetics. Generators, low-tire-inflation ATVs, and other seaming-related equipment are allowed as long as they do not damage the geosynthetics. (Recall Section 4.3.5.1 in this regard.) As a result, the movement of large rolls of geotextile, geonet, or geocomposite becomes labor intensive. Proper planning and sequencing of the operations is important for logistical control. The geosynthetic materials are laid directly on the geomembrane with no bonding allowed of any type to the geomembrane. For example, thermally fusing a geonet to a geomembrane should not be permitted.

The geosynthetics placed above the geomembrane will either be overlapped (as with some geotextiles), sewn (as with other geotextiles), connected with plastic ties (as with geonets), mechanically joined with rods or bars (as with geogrids), or male–female joined (as with drainage composites). These details will be described in Chapter 7 on geosynthetic materials other than geomembranes.

4.5.3 Soil Coverings

There are at least three important considerations concerning soil backfilling of geomembranes: type of soil backfill material, type of placement equipment, and considerations for waves or wrinkles in the geomembrane. The last of these will be treated separately in Section 4.5.4.

Concerning the type of soil backfilling material, its particle size characteristics, hardness, and angularity are important with regard to the puncture and tear resistance of the geomembrane. In general, the maximum soil particle size is important, with additional concerns over poorly graded soils, increased angularity, and increased hardness. Research on the puncture resistance of geomembranes has shown that HDPE and scrim-reinforced geomembranes are more sensitive to puncture than are PVC, LLDPE, fPP, and other nonreinforced flexible geomembranes for conventional thicknesses of the respective types of geomembranes. Using truncated cones in laboratory tests to simulate the puncturing phenomenon (Hullings and Koerner 1991), the critical cone height values that were obtained are listed in Table 4-8. However, these values are not based on actual soil subgrades, nor on geostatic type stresses. The values are meant to give relative performance among the geomembrane types.

Although the truncated cone hydrostatic test is an extremely challenging index-type test, the data of Table 4-8 do not reflect creep or stress relaxation of

Table 4-8. Critical Cone Heights for Selected Geomembranes in Simulated Laboratory Puncture Studies

Geomembrane Type	*Thickness in mm (mils)*	*Critical Cone Height in mm (inches)*
HDPE	1.5 (60)	12 (0.50)
CSPE-R	0.9 (36)	15 (0.60)
PVC	0.5 (20)	70 (2.75)
LLDPE	1.0 (40)	89 (3.50)

Source: Hullings and Koerner 1991.

the geomembrane. In reviewing numerous CQA documents, it appears that the maximum backfill particle size for use with HDPE and CSPE-R geomembranes should not exceed 6 mm (0.25 in.). LLDPE, PVC, fPP, and EPDM geomembranes appear to be able to accommodate larger soil backfill particle sizes. If the soil particle size must exceed the approximate limits given (e.g., to provide high permeability in a drainage layer), then a protection material must be placed on top of the geomembrane and beneath the soil. Nonwoven, needle punched geotextiles are generally used in this regard. They are often of a high mass per unit area, up to 1000 g/m^2 (32 oz/yd^2), and have been made from both virgin (usually) and recycled (occasionally) fibers.

Concerning the type of placement equipment, the initial lift height of the backfill soil is important. (Construction equipment should never be allowed to move directly on any deployed geomembrane. This restriction includes rubber-tired vehicles such as automobiles and pickup trucks, but it does not include lightweight equipment like ATVs.) The minimum initial lift height should be determined for the type of placement equipment and soil under consideration; however, 150 mm (6 in.) is the minimum thickness. Between this value and approximately 300 mm (12.0 in.), low ground pressure placement equipment should be specified. Ground contact pressure equipment no greater than 40 kPa (6.0 lb/in.^{2}) is recommended. For lift heights of greater than 300 mm (12.0 in.), proportionately heavier placement equipment can be used.

Placement of soil backfilling should proceed from a stable working area adjacent to the deployed geomembrane and gradually progress outward. Soil is never to be dropped from dump trucks or front-end loaders directly onto the geomembrane. The soil should be pushed forward in an upward tumbling action so as not to fall directly on the geomembrane. It should be placed by a bulldozer or front-end loader, never by a motor grader, which would necessarily have its front wheels riding directly on the geomembrane. Figure 4-28 shows a photograph of this type of soil covering placement over a protection geotextile on a geomembrane in which the indicated entombed waves are not acceptable. The exact procedure depends on site-specific materials and conditions; a discussion follows.

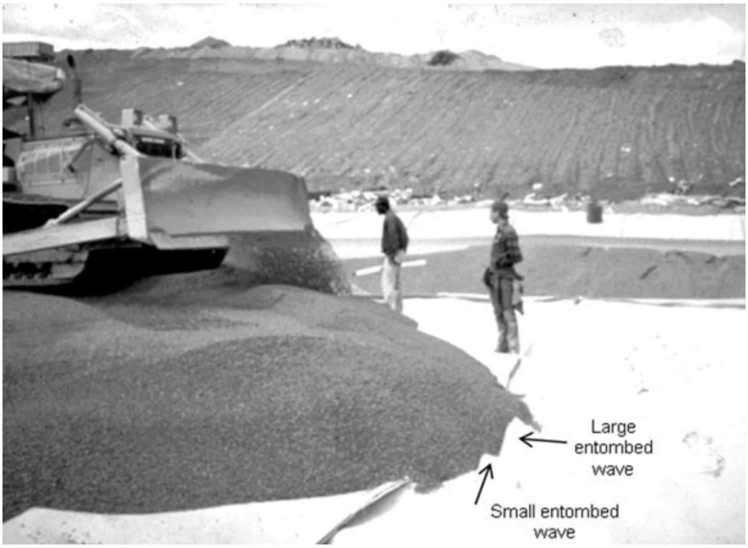


Figure 4-28. Advancing Primary Leachate Collection Gravel Over the Deployed Geotextile Covering of a Geomembrane Illustrating the Unacceptable Practice of Entombing Waves.

4.5.4 Wave or Wrinkle Management

It is difficult to achieve intimate contact of a geomembrane to the subgrade under conditions of high temperature and bright sunlight. These conditions result in expansion of all geomembrane types, with the stiffer, thicker, and black types such as HDPE resulting in fewer, but much larger, waves then other types of geomembranes (recall Figure 4-16). The management of such waves or wrinkles is difficult. Clearly, waves that are entombed in the soil backfill covering the geomembrane, as shown in Figure 4-28, will not flatten out even under extreme normal pressures (Soong and Koerner 1998). Thus, the geomembrane must be flat as it is backfilled.

From a CQA perspective, the installer has a number of options with respect to the backfilling process (Table 4-9). Commentary on each method follows:

1. It is not uncommon to push waves in front of the advancing backfill soil, allowing them to accumulate. When too large, or no longer possible, the covering geotextile (if present) and the geomembrane are cut, overlapped, and properly seamed. This new seam must then be inspected by a nondestructive test, quite often the vacuum chamber method.
2. By fixing the geomembrane at its end or intermediate points, it can be forced to lay flat so that backfilling can proceed. It must be done one roll at a time, or in discrete sections, and is slow and tedious.
3. White-surfaced geomembrane (coextruded or laminated) helps reduce waves or wrinkles by at least 50%. In this regard, covering a black geomembrane with a white geotextile is also helpful.

Table 4-9. Methods of Avoiding Waves or Wrinkles in Geomembranes during Backfilling

Method	*Advantages*	*Disadvantages*
Push-accumulate-cut-seam	Quick and low cost	Additional welds and inspection
Fixing berms or piles	Helps considerably	Slow and expensive
White sheet or white geotextile	Quick and easy	Does not completely avoid waves or wrinkles
Temporary tent	Shield from sun	Low productivity and high cost
Morning or night placement	Working with nature	Limits productivity somewhat

4. Geomembranes have been placed, seamed, and backfilled beneath a temporary tent. It obviously avoids sun, but it results in low productivity and high costs.
5. Backfilling in the early morning hours when the geomembrane has cooled overnight and thus becomes taut is an obvious way of working with nature. In some locations, it might be necessary to work at night. Both situations limit productivity somewhat but result in a flat geomembrane, which is the ultimate goal.

In summary, it is the duty of the CQA organization to see that intimate contact via flat geomembranes against the subgrade is achieved when backfilled. The method of obtaining this goal is that of the geomembrane installer and earthwork contractor. It is not up to CQA personnel to direct the field operation in this regard. CQA personnel should focus only on the result of achieving intimate contact with the subgrade via flat geomembranes.

4.5.5 General Specification Items

The specification or CQA document for backfilling should be written around the concept that the geomembrane must be protected against damage by the overlying material. Because soil, usually sand or gravel, is the most common backfilling material, the items that follow should be considered:

1. The temperature during soil backfilling should be considered. Expansion and contraction, as well as mechanical properties, vary in accordance with the geomembrane temperature.
2. In general, backfilling in warm climates or during summer months should be performed at the coolest part of the day.
3. In extreme cases of excessively high temperatures, backfilling may be required during nontypical work hours (e.g., sunrise to approximately 10:00 a.m., or possibly at night).
4. If soil backfilling is to be done at night, the work area should be suitably lit for safety, constructability, and inspection considerations.
5. If soil backfilling is to be done at night, excessive equipment noise may not be tolerated by people in the local neighborhood. This important and obviously site-specific condition should be properly addressed.

6. When a geotextile or other protection layer is to be placed above the geomembrane, it should be done according to the plans and specifications.
7. Soil placement equipment should never move or drive directly on the geomembrane or the covering geotextile if one is present (recall Section 4.3.5.1).
8. Personnel or material vehicles (e.g., automobiles or pickup trucks) should never drive directly on the geomembrane or covering geotextile.
9. The particle size characteristics of the backfill soil should be stipulated as part of the design requirements.
10. The maximum ground contact pressure of the placement equipment should be stipulated in the design requirements. (Recall Section 4.3.5.1, Item No. 5.)
11. The minimum soil lift thickness should be stipulated in the design requirements. Furthermore, the thickness should be clear as to whether it is loose or compacted thickness. In general, the first lift should be at least 300 mm (12 in.) compacted thickness.
12. For areas regularly traversed by heavy equipment (e.g., the access route for loaded dump trucks), a larger than usual backfill height should be required.
13. The CQA personnel should be available at all times during backfilling of the geomembrane. It is the last time when anyone will see the completely installed material.
14. Documentation should include the soil type, lift thickness, total thickness, density, and moisture conditions (as appropriate).

4.6 Complete System (Sheet and Seams) Leak Prevention

The goal of this chapter is to manufacture geomembranes without holes or flaws and then to seam them into a completely leak-free system. Additionally, the backfilling must be done so as not to compromise the installed liner system. Toward this end, the various subsections of the chapter have been written. With particular reference to field installation (i.e., placement, seaming, destructive testing, nondestructive testing, covering, and backfilling), the concept that is presented in Figure 4-29 is recommended.

The initial premise is based on seams being made by the dual-channel wedge method followed by a successfully completed air channel pressure test, according to the project specification. Under these conditions, a traditional destructive test sampling frequency of one sample per 150 m (500 ft) of seam should be imposed. If, however, an added level of quality is shown or offered by the installer (e.g., certified welder, taped edges, automatic devices, or infrared or ultrasonic testing), the spacing can be increased, for example, to one sample per 300 m (1000 ft). On establishing the initial spacing, statistical methods, such as attributes via GRI-GM14 or control charts via GRI-GM20, either open up the spacing for good destructive seam test results or close it for poor destructive seam test results as the job progresses. Thus, good seaming is rewarded and poor seaming is penalized. Even further, the bold option is not to have routine destructive test sampling and to use the electrical leak location survey method (Section 4.4.6) to test the entire

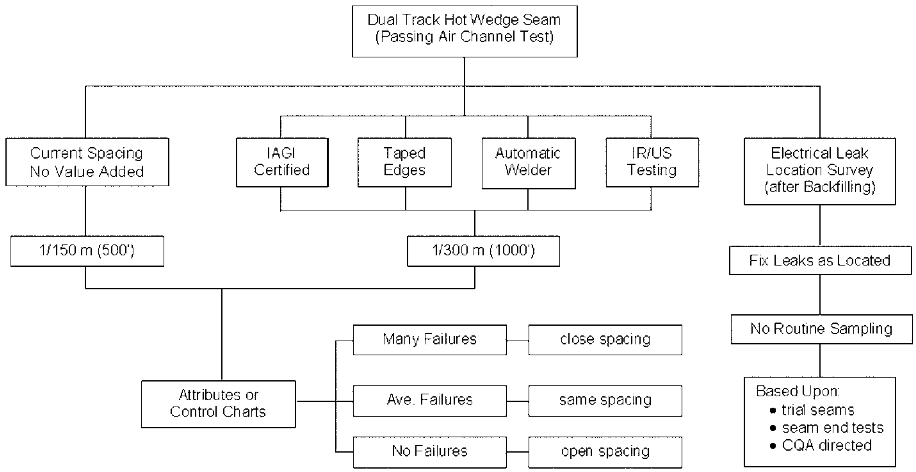


Figure 4-29. Recommended Strategy for Destructive and Nondestructive Testing Leading toward Complete Leak Prevention.

facility, seams and sheet, *after backfilling*. Leaks that are discovered are fixed and retested by the vacuum chamber method as they are detected. Of course, trial seams and destructive tests at the anchor trench and panel ends of long seams and as directed by the CQA inspector must always be accommodated.

Using such a concept, the quality of the completed geomembrane-lined facility will be consistently upgraded via a feedback system of checks and balances. The elements embodied in this concept are all within the state of the practice and are highly recommended for widespread implementation by the CQC/CQA industry.

4.7 References

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ASTM D792. "Test method for specific gravity and density of plastics by displacement."

ASTM D882. "Test methods for tensile properties of thin plastic sheeting."

ASTM D1004. "Test method for initial tear resistance of plastic film and sheeting."

ASTM D1238. "Test method for flow rates of thermoplastics by extrusion plastometer."

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ASTM D4833. "Test method for index puncture resistance of geotextiles, geomembranes, and related products."

ASTM D5046. "Specification for fully crosslinked elastomeric alloys."

ASTM D5199. "Test method for measuring nominal thickness of geotextiles and geomembranes."

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Geosynthetic Clay Liners

Beginning in 1985, relatively thin layers of bentonite were factory-manufactured with cover and carrier geotextiles on top and bottom surfaces, respectively. Thus, a cap geotextile, bentonite, and a carrier geotextile form a composite barrier material. The first uses of such a barrier material in solid-waste containment systems were as the lower component of primary liners in double-lined landfills. The resulting decreased leakage rates were observed almost immediately. Since that time, geosynthetic clay liners (as they are currently called) have developed into a separate category of geosynthetic materials. This chapter addresses this category of materials.

5.1 Types and Composition of Geosynthetic Clay Liners

As with most types of manufactured products within a given category, there are sufficient differences such that no two products are truly equal to one another. Geosynthetic clay liners (GCLs) are no exception. Yet, there are a sufficient number of common characteristics that the current commercially available products deserve a separate category and a separate treatment in this book. GCLs can be defined as follows:

> Geosynthetic clay liners (GCLs) are factory-manufactured hydraulic and gas barriers typically consisting of bentonite clay or other low-permeability clay materials, supported by geotextiles or geomembranes or both, which are held together by needling, stitching, or chemical adhesives.

Other names that GCLs have been listed under are "clay blankets," "clay mats," "bentonite blankets," "bentonite mats," and "prefabricated bentonite clay blankets." They are also called "clay geosynthetic barriers" in current ISO terminology. GCLs are hydraulic barriers to water, leachate, or other liquids. When saturated, they are also a barrier to gases. As such, they are used to augment or replace compacted clay liners or geomembranes or they are used in a composite manner to enhance the more traditional clay liner or geomembrane materials.

Cross-sectional sketches of the currently available GCLs at the time of writing are shown in Figure 5-1. Sketches in Figs. 5-1(a) and (d) are considered nonreinforced,

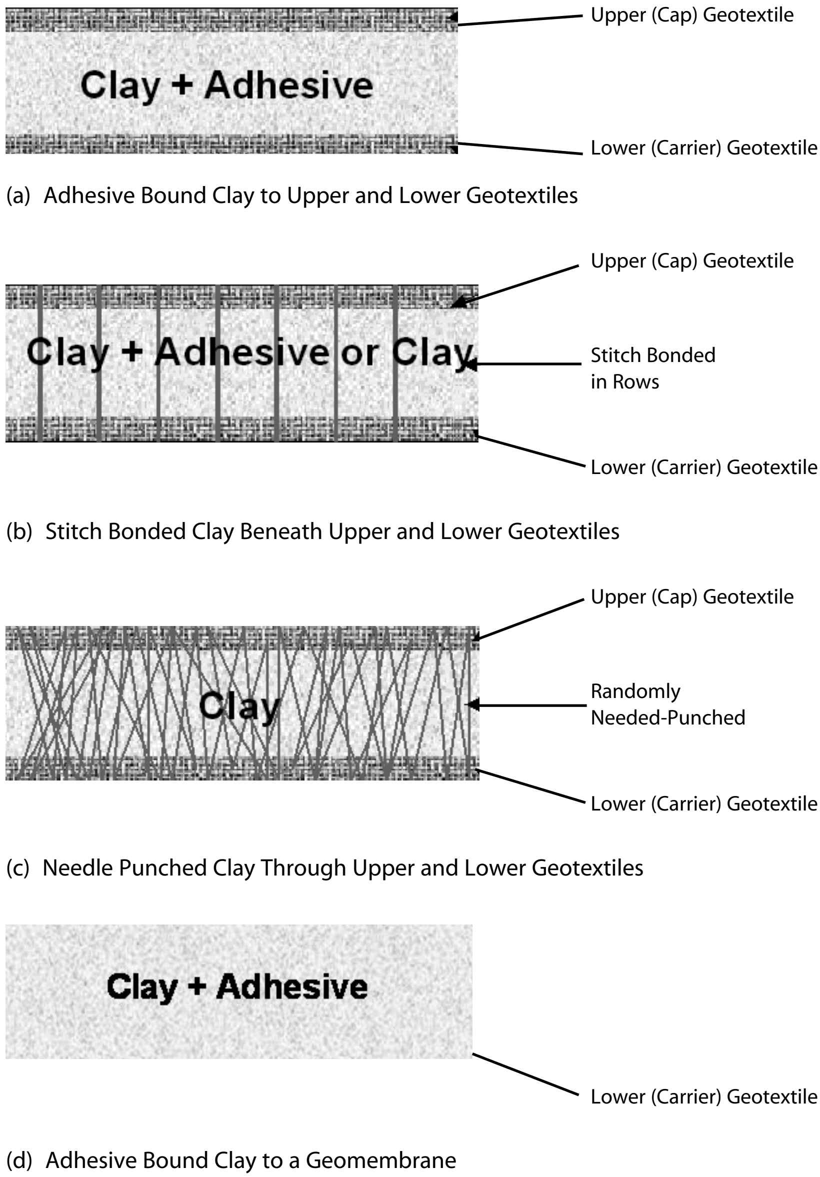


Figure 5-1. Cross-Sectional Sketches of Currently Available Geosynthetic Clay Liners.

and Figs. 5-1(b) and (c) are (internally) reinforced. General comments regarding each type follow:

- Figure 5-1(a) illustrates a bentonite clay mixed with a water-soluble adhesive that is contained by individual geotextiles on its upper and lower surfaces alike.

- Figure 5-1(b) illustrates a stitch-bonded variation of the above type of product in which the upper and lower woven geotextiles are joined by continuous sewing in discrete rows throughout the machine direction of the product as well as a recent product that consists of bentonite powder alone with no admixed adhesive.
- Figure 5-1(c) illustrates bentonite clay powder or granules, containing no adhesives, supported by individual geotextiles on its upper and lower surfaces; it is needle punched throughout to provide for internal reinforcement. The upper geotextile is nonwoven, and the lower geotextile is nonwoven, woven, or composite. Several variations of this type of GCL are available.
- Figure 5-1(d) illustrates a bentonite clay that is admixed with an adhesive and is supported by a geomembrane on its lower surface, as shown, or it can be used in an inverted manner with the geomembrane side facing upward. Variations of this product are also available.

All the GCL products available in North America use sodium bentonite clay (predominantly smectite) powder or granules at as-manufactured mass per unit areas in the range of 3.2 to 6.0 kg/m^2 (0.66–1.2 lb/ft^2). The clay thicknesses in the various products vary between 4.0 and 6.0 mm (160–320 mils). GCLs are delivered to the job site at moisture contents from 10% to 40%, depending on the manufacturer purposely wetting the clay or not and the humidity at the manufacturing facility. The types of geotextiles used with the different products vary widely in their manufacturing style (e.g., woven slit film, woven yarn, needle-punched nonwovens, heat-bonded nonwovens, and composite nonwovens/woven) and in their mass per unit area (e.g., varying from 85 to 1000 g/m^2 (2.5–30 oz/yd^2)). Other variations in products have a thin plastic film either under or over the covering geotextile, as well as some products with a polymer or bentonite infill in the covering geotextile. The particular product with a geomembrane backing can also vary in its type, thickness, and surface texture.

GCLs are factory-made in widths of 2.2 to 5.2 m (7–17 ft) and lengths of 30 to 61 m (100–200 ft). Typical roll weights are approximately 1,200 kg (2,650 lb). On manufacturing, GCLs are rolled onto a core and are enclosed within a plastic film to prevent additional moisture gain during factory storage, transportation, and field storage before placement and their final covering with an overlying layer.

5.2 Manufacturing

This section on manufacturing GCLs will discuss the various raw materials, manufacturing the rolls, and covering the rolls.

5.2.1 Raw Materials

The bentonite clay materials currently used in the manufacture of GCLs are all of the sodium montmorillonite variety, which is a naturally occurring mineral in the

Wyoming and North Dakota regions of the United States. After the clay is mined, it is dried, pulverized, sieved, and stored in silos until it is transported to a GCL-manufacturing facility.

The other raw material ingredient used in the manufacture of certain GCLs (recall Section 5.1) is an adhesive that is a proprietary product among the manufacturers that produce this type of GCL. Also proprietary are newly emerging types of polymer-modified bentonites (internally or externally). Additionally, geotextiles or geomembranes are used as carrier (below the clay) or cap (above the clay) layers that are product-specific, as was mentioned in the previous section.

Regarding a specification or MQA document for the various raw materials used in the manufacture of GCLs, the following items should be considered:

1. The clay should meet the GCL manufacturer's specification for quality control. This specification is often 70% to 90% sodium montmorillonite clay from bentonite deposits in Wyoming and North Dakota. A certificate of analysis should be submitted by the vendor for each lot of clay supplied. Although the situation is far from established, the certificate may include the various compounds of the clay (per X-ray diffraction or methylene-blue absorption), particle size (per ASTM C136), bulk density (per ASTM B417), swell index (per ASTM D5890), or fluid loss (per ASTM D5891).
2. The GCL manufacturer should have an MQC document that describes the procedures for accomplishing quality in the final product, various tests to be conducted, and their frequency. This MQC document should be fully implemented and followed.
3. The MQC test methods that the GCL manufacturer should perform on the clay component on a regular basis are swell index (per ASTM D5890) and fluid loss (per ASTM D5891). These values are stipulated in the GRI-GCL3 specification.
4. For those products that use adhesives, the composition of the proprietary adhesive is rarely specified. Likewise, the nature of polymer-modified bentonites is not specified. If a statement is required in either case, it should signify that the material or modification selected has been successfully used in the past and to what extent.
5. The geotextiles or the geomembrane used as the cap or the carrier fabric vary according to the particular style of product. The GRI-GCL3 specification gives minimum values in this regard. If a statement is required, it should signify that the products selected have been successfully used in the past and to what extent.
6. If further detail is needed as to a specification for the geotextiles, see Chapter 7. Similarly, specifications for geomembranes are found in Chapter 4.
7. The type of sewing thread (or yarn) that is used in joining stitch-bonded products is rarely specified. If a statement is required, it should signify that the materials selected have been successfully used in the past and to what extent.
8. Any other component of the GCL, such as thin film or polymer infill, that is used should have a statement available as to the extent of successful past use.

5.2.2 *Manufacturing*

The raw materials just described are used to make the final GCL product. The production facilities are all relatively large operations in which the products are made continuously. Process quality control is necessary and is practiced by all GCL manufacturers. Figure 5-2 illustrates, in schematic form, the various processing methods used for those GCLs that have adhesives mixed with the bentonite and those that are stitch-bonded and needle-punched through the bentonite. Figure 5-2(a) illustrates an adhesively bonded clay product that has an adhesive sprayed in a number of layers with intermittent additions of bentonite. The clay is placed either between geotextiles or on a geomembrane. Figure 5-2(b) illustrates the needle punching or stitch bonding of a bentonite clay (in either powder or granule form) after it is placed between the covering geotextiles. In some cases, thin polymer films are included in the process, as are polymer infills and heat bonding of

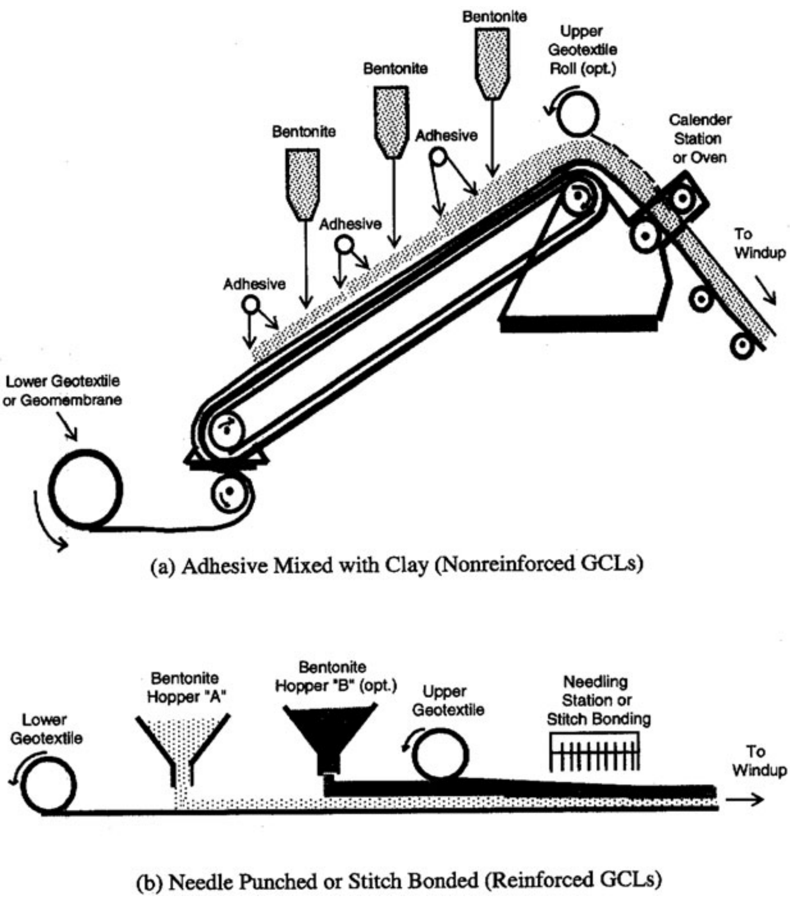


Figure 5-2. Schematic Diagrams of the Manufacture of Different Types of Geosynthetic Clay Liners (GCLs).

one or both of the external surfaces. Polymer impregnation or bentonite infill of the cap fabric is a secondary operation, as is melt bonding of needled fibers in the carrier fabric. The final step of windup around a core and placement of the protective covering is common among all GCL manufacturers.

Numerous items should be included in a specification or MQA document focused on the manufactured GCL product. The following items are written around the GRI-GCL3 specification, but others are at the discretion of the design engineer, per the site-specific plans, specification, and MQA document.

1. The following MQC tests should be conducted by the manufacturer at the testing frequency stipulated in the specification or quality manual of the manufacturer.
   - clay (as received)
     swell index (mL/2 g), per ASTM D5890
     fluid loss (mL), per ASTM D5891
   - geotextiles (as received)
     cap fabric—type and mass per unit area (g/m^2), per ASTM D5261
     carrier fabric—type and mass per unit area (g/m^2), per ASTM D5261
     coating mass per unit area of cap fabric (g/m^2), per ASTM D5261
   - geomembrane/geofilm (as received)
     thickness (mm), per ASTM D5199 or D5994
     density (g/cm^3), per ASTM D1505 or D792
     break tensile strength, machine direction (MD) and cross machine direction (XMD) (kN/m), per ASTM D6693 or D882
   - GCL (as manufactured)
     mass of GCL (g/m^2), per ASTM D5993
     mass of bentonite (g/m^2), per ASTM D5993
     moisture content (%), per ASTM D5993
     tensile strength, MD (N/m), per ASTM D6768
     peel strength (N/m), per ASTM D6496
     permeability (m/s) or flux (m^3/[s $\times$ m^2]), per ASTM D5887
2. For those GCL applications requiring a long service lifetime, long-term endurance properties should be crafted into a specification. These specifications should include permeability with worst-case permeants (per ASTM D6766) for bentonite durability, geotextile and geomembrane strength retained after elevated temperature incubation, and long-term stability of thin-film and polymer infills. The last two tests would require the writing of a specific incubation and testing protocol.
3. Verification that needle-punched, nonwoven geotextiles and reinforced GCLs (by either needle-punching or stitch-bonding) have been inspected continuously for the presence of broken needles using a full-width metal detector must be available. There should also be an alarm indication (via a flashing light or loudspeaker) if broken needles occur. Current manufacturing practice dictates that there should be a strong full-width inline magnet for removal of broken needles. If excessive broken needles cannot be removed by magnets or by

hand, the product should be labeled accordingly and used at the discretion of the MQA/CQA organization.

4. It is recommended that the overlap distance on both sides of the GCL, typically 150 mm (6.0 in.), be marked with two continuous waterproof lines guiding the minimum overlap distances. This distance may have to be increased if the GCL is covered by a geomembrane that is exposed for a considerable time. This is a site-specific decision that must be addressed by the design engineer.
5. The product should be wrapped around a core that is structurally sound so that it can support the weight of the roll without excessive bending or buckling under normal handling conditions as recommended by the manufacturer.

5.2.3 Covering the Rolls

The final step in the manufacturing of GCLs is their covering with a waterproof, tightly fit, plastic covering. This covering is sometimes a spirally wound polyethylene film approximately 0.05 to 0.08 mm (2–5 mils) thick and is the final step in production. The covering can also be a plastic bag or sheet pulled over the product as a secondary operation.

Some items needed for a specification or MQA document with regard to the covering of GCLs are the following:

1. The manufacturer should clearly stipulate the type of protective covering and the manner of cover placement. The covering should be verified as to its capability for safe storage and proper transportation of the product.
2. The covering should be placed around the GCL in a workmanlike manner so as to effectively protect the product on all of its exposed surfaces and edges.
3. The central core should be accessible for handling by forklift vehicles fitted with a long pole attachment (called a "stinger"). For wide GCLs (e.g., wider than approximately 3.5 m (11.5 ft), handling should be by construction lifting equipment using two dedicated slings provided on each roll at approximately the one-third points.
4. Clearly visible labels should identify the name and address of the manufacturer, trademark, date of manufacture, location of manufacture, style, roll number, lot number, serial number, dimensions, weight, and other important items for proper identification. Refer to ASTM D4873 for proper labeling in this regard. In some cases, the roll number itself is adequate to trace the entire MQC record and documentation.

5.3 Handling

A number of activities occur between the manufacture of a GCL, its final positioning in the field, and its subsequent backfilling. Storage at the manufacturing facility, shipping, storage at the site, and acceptance and conformance testing will be described in this section. The specifier should certainly be aware of ASTM D5885, which addresses handling in general.

5.3.1 Storage at the Manufacturing Facility

Storage of GCLs at the manufacturer's facility is common. Storage times typically range from days to six months. Figure 5-3 illustrates typical GCL storage at a manufacturing facility.

Some specifications or MQA items to consider for storage and handling of GCLs are the following:

1. GCLs should always be stored indoors until they are ready to be transported to the field site.
2. Handling of the GCLs should be such that the protective wrapping is not damaged. If it is, it must be immediately rewrapped by machine or by hand. In the case of minor tears, it may be taped.
3. Placement and stacking of the rolls should be done in a manner to prevent thinning of the product at the points of contact with the storage frame or with one another. Storage in individually supported racks is common to use floor space efficiently.

5.3.2 Shipping

Rolls of GCLs are shipped from the manufacturer's storage facility to the job site via common carrier. Ships, railroads, and trucks have all been used, depending on the locations of the origin and final destination. The usual carrier within the

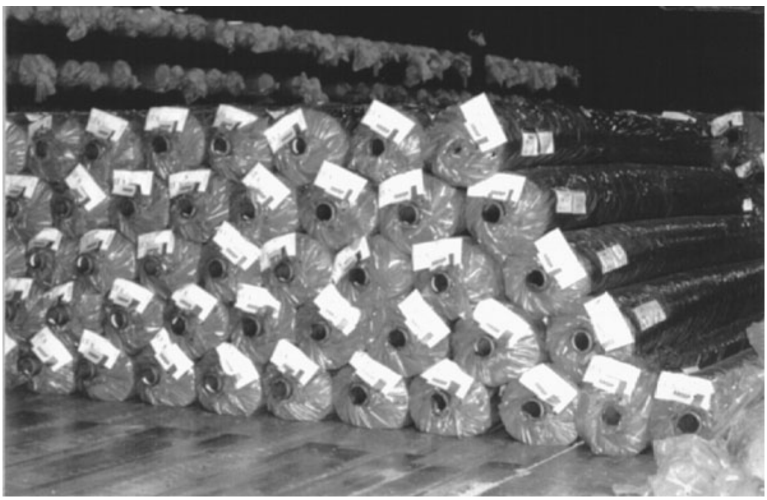


Figure 5-3. Indoor Factory Storage of GCLs Waiting for Shipment to a Job Site.

United States is truck, which should have the GCLs contained in an enclosed trailer or protected on a flat-bed trailer, as shown in Figure 5-4(a). Some manufacturers have their own dedicated fleet of trucks. The rolls are sometimes handled by forklift with a stinger attached. The “stinger” is a long tapered rod that fits inside the core on which the GCL is wrapped. Generally, however, rolls are handled using the two captive slings provided on each roll (Figure 5-4(b)).

Insofar as a specification or MQA document is concerned, a few items should be considered:

1. The GCLs should be shipped by themselves with no other cargo that could damage them in transit, during stops, or while off-loading other materials.
2. The method of loading the GCL rolls, transporting them, and off-loading them at the job site should not cause any damage to the GCL, the core, or its protective wrapping.
3. Any protective wrapping that is damaged or stripped off the rolls should be repaired immediately, or the roll should be moved to an enclosed facility until its repair can be made to the approval of the QA personnel.

(a) GCL Being Delivered to the Job Site

(b) GCLs Being Unloaded Using Dedicated Slings

(c) Temporary Storage of GCLs at Job Site

(d) Field Deployment of GCL on Soil Subgrade with Controlled Overlap

Figure 5-4. Handling of GCLs at the Job Site.

4. If any of the clay has been lost during transportation or from damage of any type, the outer layers of GCL should be discarded until undamaged product is evidenced. The remaining roll must be rewrapped in accordance with the manufacturer's original method to prevent hydration or further damage to the remaining roll.

5.3.3 Storage at the Site

Storage of GCLs at the field site is important because of the potential for moisture pickup (even through the intact plastic covering by diffusion) or accidental damage. The concept of just-in-time delivery can be used for GCLs transported from the factory to the field. When storage is required for a short period of time (i.e., days or a few weeks) and the product is delivered in trailers, the trailers can be unhitched from their tractors and used as temporary storage. Usually, however, the rolls are immediately unloaded and temporarily stored at the job site. A protective covering should be considered (Figure 5-4(c)) until deployment (Figure 5-4(d)).

If temporary storage of GCLs is permitted on the job site, off-loading the rolls must be done in an acceptable manner. Some specification or CQA document items to consider are the following. Note that the designations of MQC and MQA will now shift to CQC and CQA because field construction personnel are involved.

1. Handling of rolls of GCLs should be done in a competent manner so that damage does not occur to the product nor to its protective wrapping. In this regard, ASTM D4873, "Guide for Identification, Storage and Handling of Geotextiles," should be referenced and followed.
2. The location of temporary field storage should not be in areas where water can accumulate. The rolls should be stored on high, flat ground or elevated off the ground so as not to form a dam, creating the ponding of water. Constructing a platform is recommended so that GCL rolls are continuously supported along their length.
3. The rolls should not be stacked so high as to cause thinning of the product at points of contact. Furthermore, they should be stacked in such a way that access for conformance testing is possible.
4. If outdoor storage of rolls is to be longer than a few weeks, particular care (e.g., the use of tarpaulins or protective plastic sheeting) should be taken to minimize moisture pickup or accidental damage. For storage periods longer than one season, a temporary enclosure should be placed over the rolls or they should be moved within an enclosed facility.

5.3.4 Acceptance and Conformance Testing

On delivery of the GCLs to the field site, the CQA officer should see that conformance test samples are obtained. These samples are then sent to the CQA laboratory for testing to ensure that the GCL conforms to the project plans and specifications. The samples are taken from selected rolls by removing the protective wrapping and cutting full-width, 1 m (3 ft) long samples from the outer wrap of the selected rolls. Sometimes one complete outer revolution of GCL is discarded before the test sample is taken. The rolls are immediately rewrapped and replaced in the shipping trailers or in the temporary field storage area. Alternatively, conformance testing could be performed at the manufacturer's facility and when completed, the particular lot should be identified for the specific project under investigation.

Items to consider for a specification or CQA document in this regard are the following:

1. The samples should be identified by type, style, lot, and roll numbers. The machine direction should be noted on the samples with a waterproof marker.
2. A lot is usually defined as a group of consecutively numbered rolls from the same manufacturing line. Other definitions are also possible and should be clearly stated in the CQA documents.
3. Sampling should be done according to the project specification or CQA documents. Unless otherwise stated, sampling should be on a lot basis. Different interpretations of sampling frequency within a lot are based on total job-site area or on number of rolls. For example, sampling could be based on 10,000 m^2 (100,000 ft^2) of area or on use of ASTM D4354, which is based on total number of project rolls.
4. Testing at the CQA laboratory may include mass per unit area (per ASTM D5993) and swell index of the clay component (per ASTM D5890). The sampling frequency for these index tests should be based on ASTM D4354. Other conformance tests, which are more performance oriented, could be required by the project specifications but at a reduced frequency compared to the above-mentioned index tests. Examples are the flux or hydraulic conductivity (permeability) (per ASTM D5887) and direct shear testing (per ASTM D6243). The sampling frequency for these performance tests might be based on area, for example, one test per 10,000 m^2 (100,000 ft^2).
5. If testing of the geotextiles, or geomembrane, covering the GCLs is desired, it should be done on samples taken from the original rolls of the geotextiles, or geomembrane, before they are fabricated into the GCL product. Once fabricated, their properties will change considerably due to the needling, stitching, and/or gluing that will be undertaken during manufacturing.
6. Peel testing of needle-punched or stitch-bonded reinforced GCLs should be done in accordance with ASTM D6496. The sampling frequency is recommended to be one test per 4,000 m^2 (45,000 ft^2).
7. Conformance test results should be sent to the CQA engineer before installation of any GCL from the lot under review.
8. The CQA engineer should review the results and should report any nonconformance to the owner/operator's project manager.
9. The resolution of failing conformance tests must be clearly stipulated in the specification of CQA documents. Statements should be based on ASTM D4759, entitled "Practice for Determining the Specification Conformance of Geosynthetics."

5.4 Installation

This section will cover the placement, joining, sealing, repairing, backfilling, and covering of GCLs. The specifier should be aware of ASTM D6102, which addresses the general topic.

5.4.1 Placement

The installation contractor should remove the protective wrapping from the rolls to be deployed only after the substrate layer (soil or other geosynthetic) in the field has been approved by CQA personnel. The specification and CQA documents should be written in such a manner as to ensure that the GCLs are not damaged in any way. A CQA inspector should be present at all times during the handling, placement, and covering of GCLs.

There are several methods that are used to deploy GCLs (Table 5-1). The selection of the particular method used depends on equipment availability, subgrade material and condition, location within the facility, and ambient conditions. Figures 5-5(a) to (d) illustrate each of the methods mentioned and described in Table 5-1. Figure 5-6 is obviously an unacceptable practice and illustrates the large weight of GCLs in comparison to rolls of other geosynthetics.

Table 5-1. Field Installation Techniques

Installation Method	*Description*	*Advantages*	*Disadvantages*
Manual unroll	GCL is placed on the ground and is pushed manually.	Minimum equipment required. Applicable for confined spaces.	Low production rates. Labor-intensive.
Gravity roll release	GCL is lowered downslope by slowly releasing from a harness assembly.	Applicable for slopes that are too steep for traditional equipment.	Low production rates. May be difficult to guide GCL as it unrolls.
Stationary roll pull	Roll is suspended at site perimeter and one end is pulled out into areas to be lined.	Equipment can be kept out of lined area.	Modest production rates. Coarser subgrades could damage underside of GCL.
Moving roll pull	One end of roll is placed on the ground or is suspended from equipment that moves backward along the area to be lined.	High production rates possible.	Equipment may damage underlying geosynthetic materials or cause rutting of subgrade surfaces.

Source: Data are derived from Trauger and Tewes 1995.

(a) Field deployment of a GCL by manual unroll method

(b) Field deployment of a GCL by gravity roll release method

(c) Field deployment of a GCL by stationary roll pull method

(d) Field deployment of a GCL by moving roll pull method

Figure 5-5. Various Acceptable Methods of Field Deployment of GCLs.

The following items should be considered for inclusion in a specification or CQA document:

1. The installer should take the necessary precautions to protect the soil or geosynthetic materials underlying the GCL. If the substrate is soil, construction equipment can be used to deploy the GCL, provided that excessive rutting is not created. Excessive rutting should be clearly defined and quantified. In some cases, 25 mm (1.0 in.) is the maximum rut depth allowed. If the ground freezes, the depth of ruts might be further reduced to a specified value. If the substrate is a geosynthetic material, GCL deployment should be by hand or by use of small jack lifts or lightweight equipment on pneumatic tires with low ground contact pressure. Additional restrictions on the use of equipment should be considered (recall Section 4.3.5.1).
2. The minimum overlap distance should be specified and verified. This is typically 150 to 300 mm (6–12 in.) depending on the particular product, site temperature and humidity, and slope. For exposed geomembranes over GCLs, the overlap should be even greater. This is a design decision and must be communicated accordingly.

Figure 5-6. An Unacceptable Situation for Field Deployment of a GCL.

3. Additional bentonite clay should be introduced into the overlap region with certain types of GCLs, typically those with needle-punched, nonwoven geotextiles on both of their surfaces that do not have access to bentonite within the product itself. The clay is usually added by using a lime spreader or line chalker with the bentonite clay in a dry state. Alternatively, a bentonite clay paste, in the mixture range of four to six parts water to one part of clay, can be extruded in the overlap region. Manufacturer's recommendations on type and quantity of clay to be added should be followed.
4. During placement, care must be taken not to entrap, in or beneath the GCL, fugitive clays, stones, or sand that could damage a geomembrane, cause clogging of drains or filters, or hamper subsequent seaming of materials either beneath or above the GCL.
5. On sideslopes, the GCL should be anchored at the top and then unrolled so as to keep the material free of wrinkles and folds.
6. In general, GCL ends should be shingled downgradient.
7. Trimming of the GCL should be done with great care so that fugitive clay particles do not come in contact with drainage materials such as geonets, geocomposites, or natural drainage materials.
8. The deployed GCL should be visually inspected to ensure that no potentially harmful objects are present (e.g., stones, cutting blades, small tools, or sandbags).

9. Broken needles left in the material after manufacturing are a concern if the overlying material is a geomembrane. A hand-held metal detector is recommended to locate and remove broken needles or to reject rolls that contain too many broken needles to remove by hand.

5.4.2 Joining

Joining of GCLs is generally accomplished by overlapping without sewing or other mechanical connections. The overlap distance requirements should be clearly stated. For all GCLs, the required overlap distance should be marked on the underlying layer by a pair of continuous guidelines. The overlap distance is typically 150 to 300 mm (6–12 in.). For temporarily exposed GMs covering GCLs (for months to years), reduction or even complete loss of overlap distance, i.e., GCL panel separation, has occurred. This design issue affects selection of the type of GCL vis-à-vis the site-specific conditions. Overlap distances as much as 450 mm (18 in.) may be required; see Section 5.6 for further detail.

For those GCLs with needle-punched nonwoven geotextiles on their surfaces, dry bentonite is sometimes placed in the overlapped region. If this is the case, utmost care should be given to avoid fugitive bentonite particles from coming into contact with leachate collection systems. Other variations, however, have been to extrude a tube of moist bentonite into the overlapped region; one product has a self-sealing zone available that does not require additional bentonite.

Items to consider for a specification or CQA document follow:

1. The amount of overlap for adjacent GCLs must be stated and adhered to in field placement of the materials. Overlap distance is both site-specific and product-specific and is a design issue.
2. If reduction in overlap distance is anticipated, the overlap should be increased or the system should be covered or backfilled or both in a timely manner.
3. The overlap distance is sometimes different for the roll ends versus the roll edges. The values should be stated and followed.
4. If dry or moistened bentonite clay (or other material) is to be placed in the overlapped region, the type and amount should be stated in accordance with the manufacturer's recommendations or design considerations. Furthermore, the placement procedure should be clearly outlined so as to have enough material to make an adequately tight joint and yet not an excessive amount, which could result in fugitive clay particles.

5.4.3 Sealing around Penetrations

The placement and sealing of GCLs around penetrations such as pipes and sumps is a difficult, yet important, consideration. In general, the GCL is cut in a zigzag pattern and placed with generous overlaps against the protruding object. Hand-placed bentonite or bentonite paste is often used as an additional sealant.

Items to consider for a specification or CQA document follow:

1. The design detail of GCL placement around penetrations should follow the design drawings. Alternatively, manufacturer's recommendations can be solicited, evaluated, and followed accordingly.
2. Further sealing of the GCL to the penetration should follow the project plans or specifications.
3. Displacement of the GCL or sealant during backfilling is a concern. As a result, backfilling activities must be observed by the CQA inspector.

5.4.4 Repairs

For geotextile-related GCLs, holes, tears, or rips in the covering geotextiles made during transportation, handling, placement, or anytime before backfilling should be repaired by patching using additional geotextile. If the bentonite component of the GCL is disturbed either by loss of material or by shifting, it should be covered using a full GCL patch of the same type of product.

Some relevant specification or CQA document items follow:

1. Any patch used for repair of a tear or rip in the geotextile should be done using the same type as the damaged geotextile or other approved geotextile by the CQA engineer.
2. The size of the geotextile patch must extend at least 30 cm (12 in.) beyond any portion of the damaged geotextile and must be bonded adhesively or by heat to the product to avoid shifting during backfilling with soil or covering with another geosynthetic.
3. If bentonite particles are lost from within the GCL or if the clay has shifted, the patch should consist of the full GCL product. It should extend at least 30 cm (12 in.) beyond the extent of the damage at all locations. For those GCLs requiring additional bentonite clay in overlap seaming, a similar procedure should be used for patching.
4. Particular care should be exercised in using a GCL patch because fugitive clay can be lost and can find its way into drainage materials or onto geomembranes in areas that eventually are to be seamed together.

5.5 Backfilling or Covering

The layer of material placed above the deployed GCL will be either soil or another geosynthetic. Soils will vary from compacted clay layers to coarse aggregate drainage layers. Geosynthetics will generally be geomembranes, although other geosynthetics may also be used, depending on the site-specific design. The GCL should generally be covered before a rainfall or snow event occurs. The reason for rapid covering of nonreinforced GCLs is that hydration before covering and backfilling will cause free swelling and the rapid loss of bearing capacity of the hydrated bentonite. Thus, when even nominal loads are placed on the GCL, a rapid thinning will occur due to lateral displacement (squeezing) of the bentonite within the GCL. Hydration before covering is less of a concern for the reinforced (particularly needle-punched) GCLs, but migration of the fully hydrated clay in these products might also be possible under sustained compressive or shear loading. Figure 5-7(a) shows the premature hydration of a GCL being gathered up by hand to be discarded in the adjacent landfill. Figure 5-7(b) shows fugitive bentonite that has extruded through the woven geotextile on the bottom of the folded GCL, contaminating the subgrade and causing a low interface shear strength. This situation associated with woven geotextiles has resulted in several failures. GCLs with nonwoven geotextiles on both upper and lower surfaces avoid the situation of extruded bentonite from occurring.

Some recommended specifications or CQA document items are as follows:

1. The GCL should be covered with its subsequent layer before a rainfall or snowfall occurs.
2. The GCL should not be covered before observation and approval by the CQA personnel. This inspection requires close coordination between the installation crew and the CQA personnel.
3. If soil is to cover the GCL, it should be done such that the GCL or underlying materials are not damaged. Continuous observation of the cover material placement is recommended. The minimum thickness of soil covering must be stipulated and adhered to in the field before trafficking. Koerner and Narejo (1995) show that a minimum cover soil of 300 mm (12 in.) is required.
4. If a geosynthetic is to cover a GCL, both the underlying and the newly deployed material should not be damaged.
5. The overlying material should not be deployed such that excess tensile stress is mobilized in the GCL. On sideslopes, this stipulation requires soil backfill to proceed from the bottom of the slope upward. Other conditions are site-specific and material-specific and must be stated accordingly.

5.6 Exposed Geomembrane-Covered GCLs

For situations where a composite geomembrane–GCL liner is not backfilled in a timely manner, that is, the geomembrane is exposed to the local environment, for months or years, loss of overlap and even complete GCL panel separation has occurred (see Figure 5-8). This problem occurs mainly on sideslopes but has also been observed on flat surfaces (Koerner and Koerner 2005; Thiel and Richardson 2005). This situation is unacceptable.

Some recommended specification or CQA document items are as follows:

1. Backfill the geomembrane with at least 300 mm (12 in.) of soil cover in a timely manner, such time depending on site-specific conditions of temperature, moisture, subgrade, and slope.

(a) Premature hydration of a geosynthetic clay liner being gathered and discarded because of its exposure to rainfall before covering

(b) Fugitive bentonite resulting from extrusion of hydrated bentonite through woven geotextile of GCL

Figure 5-7. Illustrations of a GCL Being Exposed to Moisture or Hydration with Unacceptable Results.

(a) Loss of initial overlap—no separation

(b) Separation of GCL panels ≃200 mm (8 in.)

(c) Separation of GCL panels ≃300 mm (12 in.)

Figure 5-8. Loss of Overlap and Panel Separation at Several Field Sites.

Sources: (a) Koerner and Koerner 2005; (b) Koerner and Koerner 2005; (c) Thiel and Richardson 2005, with permission from Geosynthetic Information Institute.

2. If using GCLs with needle-punched nonwoven geotextiles on both sides, one must be scrim-reinforced, that is, one of the geotextiles must be a composite woven–nonwoven.
3. Increase the as-placed overlap distance to between 250 mm (10 in.) and 450 mm (18 in.), depending on the GCL product being used.
4. Protection of the exposed GM–GCL composite liner during its exposure time using thermal blankets, sprayed-on polystyrene foam, geofoam sheet, or other insulation techniques might be considered.

5.7 References

ASTM B417. "Test method for apparent density of non-free-flowing metal powders using the Carney funnel."

ASTM C136. "Test method for sieve analysis of fine and coarse aggregate."

ASTM D417. "Test method for apparent density of non free-flowing metal powders."

ASTM D792. "Test method for density and specific gravity (relative density) of plastics by displacement."

ASTM D882. "Test method for tensile properties of thin plastic sheeting."

ASTM D1505. "Test method for density of plastics by the density-gradient technique."

ASTM D4354. "Practice for sampling of geosynthetics for testing."

ASTM D4759. "Practice for determining the specification conformance of geosynthetics."

ASTM D4873. "Guide for identification, storage and handling of geotextiles."

ASTM D5199. "Test method for nominal thickness of geotextiles and geomembranes."

ASTM D5261. "Test method for measuring mass per unit area of geotextiles."

ASTM D5885. "Standard guide for storage and handling of geosynthetic clay liners."

ASTM D5887. "Test method for measurement of index flux through saturated geosynthetic clay liner specimens using flexible wall permeameter."

ASTM D5890. "Test method for swell index of clay mineral component of geosynthetic clay liners."

ASTM D5891. "Test method for fluid loss of clay component of geosynthetic clay liners."

ASTM D5993. "Test method for measuring the mass per unit area of geosynthetic clay liners."

ASTM D5994. "Test method for measuring core thickness of textured geomembranes."

ASTM D6102. "Standard guide for installation of geosynthetic clay liners."

ASTM D6243. "Test method for determining the internal and interface shear resistance of geosynthetic clay liner by the direct shear method."

ASTM D6496. "Test method for determining average bonding peel strength between top and bottom layers of needle-punched geosynthetic clay liners."

ASTM D6693. "Test method for determining tensile properties of nonreinforced polyethylene and nonreinforced flexible polypropylene geomembranes."

ASTM D6766. "Test method for evaluation of hydraulic properties of geosynthetic clay liners with potentially incompatible liquids."

ASTM D6768. "Test method for tensile strength of geosynthetic clay liners."

GRI-GCL3. "Standard specification for test methods, required properties, and testing frequencies of geosynthetic clay liners (GCLs)."

Koerner, R. M., and Narejo, D. (1995). "On the bearing capacity of hydrated GCLs." *J. Geotech. Geoenviron. Engrg.*, 121(1), 82–87.

Koerner, R. M., and Koerner, G. R. (2005). "GRI white paper No. 5—In situ separation of GCL panels beneath exposed geomembranes," Geosynthetic Institute, Folsom, PA, April 15.

Thiel, R., and Richardson, G. (2005). "Concern for GCL shrinkage when installed on slopes," *Proc. GRI-18 at GeoFrontiers,* Paper 2.31, Geosynthetic Information Institute, Folsom, Pa.

Trauger, R., and Tewes, K. (1995). "Design and installation of a state-of-the-art landfill liner system," *Proc. GCL Conf.,* April 14–15, Nürnberg, Germany, 175–182.

Soils in Drainage Layers and Alternative Cover Systems

6.1 Introduction and Background

Natural soil materials are commonly used in waste containment units for the following reasons:

1. Drainage layers in final cover systems (to reduce the hydraulic head on the underlying barrier layer and to enhance slope stability by reducing seepage forces).
2. Gas collection layers in final cover systems (to channel gas to vents for controlled removal of potentially dangerous gases). It also can act as a sideslope seep collection layer.
3. Leachate collection layers in liner systems (to enable removal of precipitation in unfilled areas or removal of leachate in filled areas).
4. Leak detection layers in double liner systems (to monitor performance of the primary liner and, if necessary, to serve as a secondary leachate collection layer).
5. Drainage trenches (to collect horizontally flowing groundwater or gas).
6. Seep collection layers in final cover system slopes.
7. Alternative final cover systems.

Drainage layers are also used in miscellaneous ways, such as to drain liquids from backfill behind retaining walls or to relieve excess water pressure in critical areas such as the toe of slopes. Natural soils may be used in alternative covers as soil water storage layers, wicking drainage layers, or capillary break layers.

6.2 Material Composition

Drainage materials usually consist of sand or gravel mined from commercial sources. Most of the alluvial sands and gravels found in North America are predominantly quartz, with minor amounts of other minerals. Quartz is a stable mineral and generally not subject to dissolution or other reactions that would cause a deterioration of the material. Gravel and cobble drainage materials are sometimes derived from processed rock, especially if suitable alluvial gravels are not available. Expanded clay and shale materials ("lightweight aggregate") have been proposed for use in landfills (Bowders et al. 1997).

The most common host rocks used in making crushed granular media are limestone and dolostone (dolomite), although other rocks such as basalt are sometimes used if they are prevalent locally. The principal concern if calcium-rich materials, such as crushed limestone, are used is that the calcium carbonate in the material may be dissolved and later precipitated at another location, causing not only a deterioration of the granular medium but also (and perhaps more importantly) precipitation and some degree of plugging of the drainage system. However, the potential for limestone and dolostone to be dissolved by landfill leachate is far from clearly established by available data (Bennett et al. 2000). Nevertheless, CQA observers should be cognizant of the need to make sure that carbonate components comply with specifications and are not present in excessive amounts. If the specifications place a limit on carbonate content, tests should be performed to confirm compliance (Table 6-1). The ASTM test for carbonate content (D4373) has been reported to be poorly reproducible for slightly carbonaceous drainage materials. Some commercial laboratories have developed alternative testing methods that have yielded more consistent results. Tests recommended for soil drainage materials are provided in Table 6-1.

Table 6-1. Recommended Tests and Testing Frequencies for Drainage Material

Location of Sample	*Type of Test*	*Minimum Frequency*
Potential borrow source	Grain size (ASTM D6913)	1 per 2,000 m^3
	Hydraulic conductivity (ASTM D2434)	1 per 2,000 m^3
	Carbonate content[a] (ASTM D4373)	1 per 2,000 m^3
On site after placement and compaction	Grain size (ASTM D6913)	1 per hectare for drainage layers; 1 per 500 m^3 for other uses
	Hydraulic conductivity (ASTM D2434)	1 per 3 hectares for drainage layers; 1 per 1,500 m^3 for other uses
	Carbonate content[a] (ASTM D4373)	1 per 2,000 m^3

[a]The frequency of carbonate content testing should be reduced to 1 per 20,000 m^3, or entirely eliminated, for those drainage materials that obviously do not and cannot contain significant carbonates (e.g., crushed basalt).

6.3 Material Gradation

Soil drainage systems are constructed of materials that have high hydraulic conductivity. High hydraulic conductivity is not only required initially, but the drainage material must also maintain a high hydraulic conductivity over time and resist plugging or clogging. Generally speaking, the larger the particles used in constructing the drainage layer, the larger the pore sizes in the medium and the slower the material will plug from suspended solids or precipitates (Rowe et al. 2000). Primarily for this reason, there has been a gradual trend in recent years by some designers toward use of more coarse-grained materials for drainage media in landfills.

The hydraulic conductivity of drainage materials depends primarily on the grain size of the finest particles present in the soil. An equation that is sometimes used to estimate hydraulic conductivity of granular materials is Hazen's formula:

$$k = (D_{10})^2 \tag{6.1}$$

where k is the hydraulic conductivity (cm/s) and D_{10} is the equivalent grain diameter (mm) at which 10% of the soil is finer by weight. To determine the value of D_{10}, a plot is made of the grain-size distribution of the soil (determined per ASTM D422), as shown in Figure 6-1. The value of D_{10} is determined from the grain-size

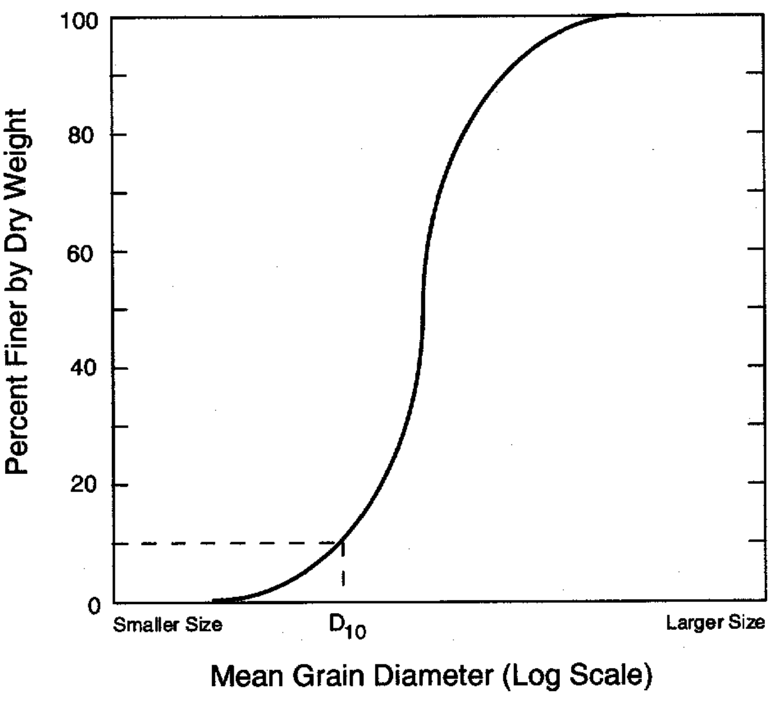


Figure 6-1. Grain-Size Distribution Curve.

distribution curve as shown in Figure 6-1. More sophisticated methods are available for predicting hydraulic conductivity from soil characteristics (Boadu 2000); however, if an empirical method is going to be used in lieu of direct measurement, the tendency is to use a simple empirical method such as Eq. 6.1.

Experimental data verify that the percentage of fine material in the soil dominates hydraulic conductivity. For example, the data in Table 6-2 illustrate the effect of a small amount of fines on the hydraulic conductivity of filter sand. The addition of just a few percent of fine material to a drainage material can reduce the hydraulic conductivity of the drainage material by 100-fold or more.

Construction specifications usually stipulate a minimum hydraulic conductivity for the drainage layer. The value specified varies considerably from project to project but is typically in the range of 0.01 to 1 cm/s. The method used to determine hydraulic conductivity in the laboratory is ASTM D2434.

Drainage materials may also be required to serve as filters. For instance, as shown in Figure 6-2, a filter layer may be needed to protect a drainage layer from plugging. Usually, one filter layer will suffice, but if the particle size differences between the soil and the drainage material are extreme, two filter layers might be necessary. The filter layer must serve three functions:

1. The filter must prevent migration of significant amounts of the protected soil through the filter (to prevent clogging of the drain from fugitive soil particles).
2. The filter must have a relatively high hydraulic conductivity (to ensure free drainage). The filter should be more permeable than the layer of soil.
3. The filter material itself must not migrate significantly into the adjacent drainage layer (again, to prevent clogging of the drain).

Filter specifications vary somewhat, but the design procedures are similar. The determination of requirements for a filter material usually proceeds as follows:

1. The grain-size distribution curve of the soil to be retained (protected) is determined following procedures outlined in ASTM D422, or as specified. The size of the protected soil at which 15% is finer ($D_{15,soil}$) and 85% is finer ($D_{85,soil}$) is determined.

Table 6-2. Effect of Fines on Hydraulic Conductivity of a Washed Filter Aggregate

Percent Passing No. 100 Sieve[a]	*Hydraulic Conductivity (cm/s)*
0	0.003–0.11
2	0.004–0.04
4	0.0007–0.02
6	0.0002–0.007
7	0.00007–0.001

[a]Opening size is 0.15 mm.

Source: Cedergren 1989, with permission from John Wiley & Sons.

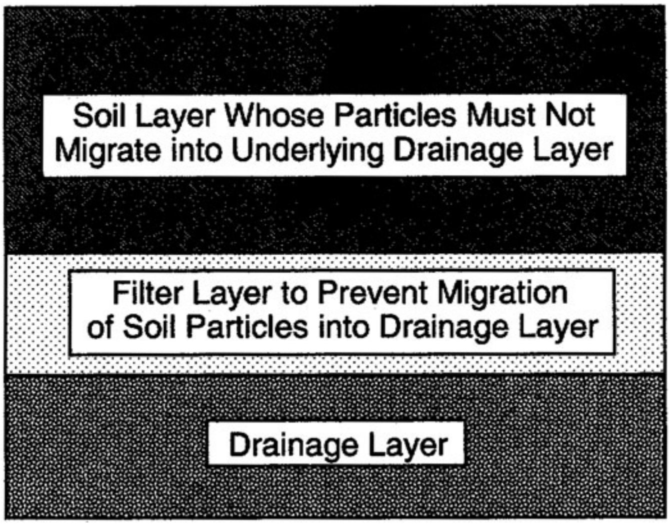


Figure 6-2. Filter Layer Used to Protect Drainage Layer from Plugging.

2. The grain-size distribution of the filter soil is determined.
3. Experience shows that the particles of the protected soil will not significantly penetrate into the filter if the size of the filter at which 15% is finer ($D_{15,\text{filter}}$) is less than four to five times D_{85} of the protected soil:

$$D_{15,\text{filter}} \leq (4 \text{ to } 5)D_{85,\text{soil}} \tag{6.2}$$

4. Experience shows that the hydraulic conductivity of the filter will be significantly greater than that of the protected soil if the following criterion is satisfied:

$$D_{15,\text{filter}} \geq 4D_{15,\text{soil}} \tag{6.3}$$

5. To ensure that the particles within the filter do not tend to migrate excessively into an adjacent drainage layer, the following criterion may be applied:

$$D_{15,\text{drain}} \leq (4 \text{ to } 5)D_{85,\text{filter}} \tag{6.4}$$

6. Experience shows that the hydraulic conductivity of the drain will be significantly greater than that of the filter if the following criterion is satisfied:

$$D_{15,\text{drain}} \geq 4D_{15,\text{filter}} \tag{6.5}$$

Thus, the construction specification usually stipulates compliance with filter criteria, such as those stated above. The CQA personnel will normally focus their attention on meeting the specific gradation specification. However, even more important in many cases is simple vigilance by field personnel to ensure that the filter is not forgotten, overlooked, or contaminated with fine materials that might wash into it, or might be improperly located or undersized.

Filter design is complicated significantly by the presence of biodegradable waste materials (e.g., municipal solid waste) placed directly on top of the filter. In such circumstances, the usual filter criteria may be modified to satisfy site-specific requirements. Some degree of reduction in hydraulic conductivity of the filter layer may be acceptable, so long as the reduction does not impair the ability of the drainage system to serve its intended function. A laboratory test method to quantify the hydraulic properties of both soil and geotextile filters that are exposed to leachate is ASTM D1987. However, regardless of specific design criteria, the gradational characteristics of the filter material control the behavior of the filter. CQC/CQA personnel should focus their attention on ensuring that the drainage material and filter material meet the grain-size-distribution requirements set forth in the construction specifications, as well as other specified requirements, such as mineralogy of the materials.

6.4 Control of Materials

The recommended procedure for verifying the hydraulic conductivity for a proposed drainage material is as follows. Representative samples of the proposed material should be obtained and shipped to a laboratory for testing. Samples should be compacted in the laboratory to a density that will be representative of the density expected in the field. Hydraulic conductivity should be measured following procedures in ASTM D2434 and compared with the required minimum values stated in the construction specifications. If the hydraulic conductivity exceeds the minimum value, the material is tentatively considered acceptable. However, it should be realized that the process of excavating and placing the drainage material will cause some degree of crushing of the drainage material and will produce additional fines. Thus, the construction process tends to increase the amount of fines in the drainage material and to decrease the hydraulic conductivity of the material. If the drainage material just barely meets the hydraulic conductivity requirements stated in the construction specifications from initial tests, there is a good possibility that the material will fail to meet the required hydraulic conductivity standard after the material has been placed. As a rule of thumb, approximately 0.5% of additional fines by weight will be generated every time a drainage material is handled. Also, the reproducibility of hydraulic conductivity tests is not well established; a material may just barely meet the hydraulic conductivity standard in one test but fail to meet minimum requirements in another test. Finally, if the drainage materials are found suitable before placement but unsuitable after placement, an extremely difficult situation arises because it is virtually impossible to remove and replace the drainage material without risking damage to underlying geosynthetic components (e.g., a geomembrane). Therefore, some margin of safety should be factored into the selection of drainage material. The authors recommend selecting a drainage material that has a hydraulic conductivity from initial screening tests that is at least an order of magnitude greater than the minimum required value.

Because it is extremely difficult to remove and replace a drainage material without damaging an underlying geosynthetic component, some testing of the drainage material should occur before placement of the material. The CQC personnel should have a high degree of confidence that the drainage material is suitable before placement of the material. Because the construction process may alter the characteristics of the drainage material, it is important that CQA tests also be performed on the material after it has been placed and compacted (if it is compacted).

The usual tests for CQA involve determination of the grain-size distribution of the soil (ASTM D422) and hydraulic conductivity of the soil (ASTM D2434). Hydraulic conductivity tests tend to be time-consuming and relatively difficult to reproduce precisely; the test apparatus that is employed, the compaction conditions for the drainage material, and other details of testing may significantly influence test results. Grain-size distribution analyses are simpler. Therefore, it is recommended that the CQA testing program emphasize grain-size distribution analyses, with particular attention paid to the amount of fines present in the drainage material rather than hydraulic conductivity testing. The percent of fines is usually defined as the percent on a dry-weight basis passing through a No. 200 sieve (openings of 0.075 mm). Wash sieve analysis should be performed on drainage materials that contain a significant amount of fine particles (especially cohesive clay particles) to ensure an accurate determination of the percentage of fines.

The recommended tests and frequency of testing are shown in Table 6-1. The same principles for sampling strategies discussed in Chapter 3 may be applied to location of tests or location of samples for drainage layer materials. Also, occasional failing tests may be allowed, but it is recommended that no more than 5% of the CQA tests be allowed to deviate from specifications, and the deviations should be relatively minor (i.e., no more than about 2–3% fines beyond the maximum value allowed and no less than about 20% the minimum allowable hydraulic conductivity).

6.5 Location of Borrow Sources

The construction specifications usually establish criteria that must be met by the drainage material. Earthwork contractors are usually given latitude in locating a suitable source of material that meets construction specifications. On occasion, the materials may be available on site or from a nearby site, but most frequently, the materials are supplied by a commercial materials company. If the materials are supplied by an existing materials processor, stockpiles of materials are usually readily available for testing, and no geotechnical investigations are required (other than to test the proposed material). Materials processors usually screen their materials and, thus, the gradation is often well-defined and closely controlled.

6.6 Processing of Materials

Materials may be processed in several ways. Oversized stones or rocks are typically removed by sieving. Fine material may also be removed by sieving. Washing the fines out of a sand or gravel can be particularly effective in removing silt and clay particles from granular material. For drainage-layer materials that are supplied from a commercial processing facility, the facility owner is usually experienced in processing the material to remove fines.

For the CQA inspector, the main problems to watch for are removal of oversized material, inappropriate use of angular material (angular stone can puncture an unprotected geomembrane), and assurance that excessive fines are not present in the material.

6.7 Placement

Drainage materials may be placed in layers (e.g., as leachate collection layers) or they may be placed in drainage trenches (e.g., to provide drainage near the toe of a slope). Placement considerations differ depending on the application.

6.7.1 Drainage Layers

Granular drainage materials are usually hauled to the placement area in dump trucks, loosely dumped from the truck, and spread with bulldozers. The contractor should dump and spread the drainage material in a manner that minimizes generation of fine material. For instance, light-contact-pressure bulldozers can be used to spread the drainage material and minimize the stress on the granular material. Granular materials placed on top of geosynthetic components on sideslopes should be placed from the bottom to the top of the slope.

When granular drainage material is placed on a previously placed geomembrane or geotextile and spread with a bulldozer, the sand or gravel should be lifted and tumbled forward so as to minimize shear forces on the underlying geosynthetic. The bulldozer should not be allowed to push the blade downward (called "crowd") into the granular material and drag it over the surface of the underlying geosynthetic material.

Granular materials are often placed with a backhoe in small, isolated areas such as sumps. Some drainage materials may even be placed by hand (e.g., in sumps and around drainage pipes).

CQA personnel should position themselves in front of the working face of the placement operation to observe the materials as they are spread and to ensure that there is no puncture of underlying materials. CQA personnel should observe placement of drainage layers to ensure that fine-grained soil is not accidentally mixed with drainage material. The wheels or tracks of construction equipment, for instance, will tend to deposit adjacent soils on the drainage layer if the equipment moves across the drainage layer.

6.7.2 Drainage Trenches

Drainage materials are often placed in trenches to provide for subsurface drainage of water (e.g., near the toe of a cover to provide an outlet for water in a drainage layer located in the cover). A typical trench configuration is shown in Figure 6-3. Often, a perforated pipe will be placed in the bottom of the trench. Geotextile filters are often required along the sidewalls to prevent migration of fine particles into the drainage material. CQA personnel should carefully review the plans and specifications to ensure that the drainage and filter components have been properly located in the trench before backfilling.

CQC/CQA personnel should be aware of all applicable safety requirements for inspection of trenches. Unsupported trenches can pose a hazard to personnel working in the trenches or inspecting the trenches. For trenches that are supported by shoring, CQA personnel should review with the contractor the plan for pulling the shoring in terms of the timing for placement of materials and ensure that the procedures are in accord with the specifications for the project.

Granular backfill is usually placed in a trench by a backhoe. For narrow trenches, a "tremie" is commonly used to direct the material into the trench without allowing the material to come into contact with soil on the sidewalls of the trench. Sometimes drainage materials are placed by hand for small trenches.

A special type of trench involves support of the trench wall with biodegradable ("biopolymer") slurry. The trench is excavated into soil using a biodegradable, viscous fluid to maintain the stability of the trench. The backfill is placed into the fluid-filled trench. An agent is introduced to promote degradation of the viscous drilling fluid, which quickly loses much of its viscosity and allows the granular backfill to attain a high hydraulic conductivity without any plugging effect from the slurry. This technology allows construction of deep, continuous drainage

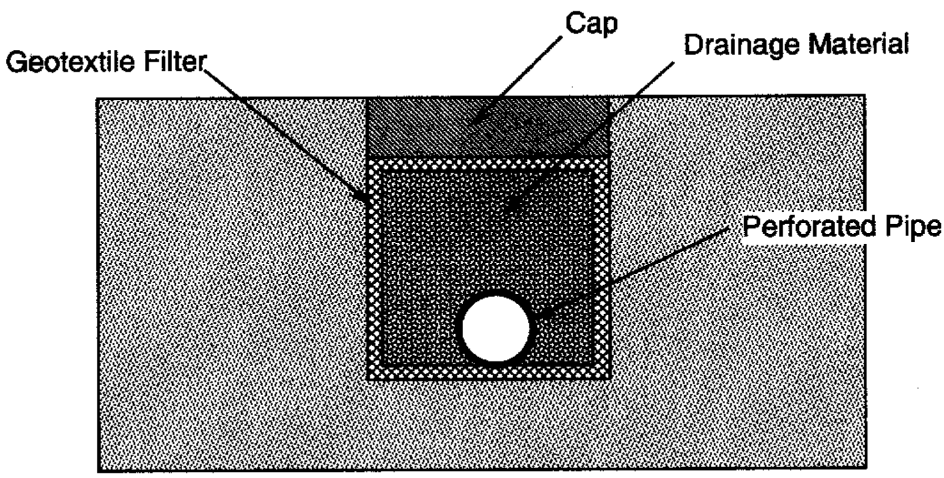


Figure 6-3. Typical Design of a Drainage Trench.

trenches but is used much more often for remediation of contaminated sites than in new waste containment facilities. Further details are given by Day (1990).

6.8 Compaction

Many construction specifications stipulate a minimum percentage compaction for granular drainage layers. There is rarely a need for more than nominal compaction of drainage materials. However, on occasion, there may be a need to compact a drainage material for one of the following reasons:

1. If a settlement-sensitive structure is to be placed on top of the drainage layer, the drainage layer may need to be compacted to minimize settlement.
2. If dynamic loads might cause loose drainage material to liquefy or settle excessively, the material may need to be compacted.
3. If the drainage material must have exceptionally high strength, the material may need to be compacted.
4. If the drainage sand "fluffs" after placement, some compaction is usually necessary.

Only in rare instances will the problems listed above be significant for gravel, rock, or stone drains. Settlement-sensitive structures are rarely built on top of liner or cover systems. Liquefaction is rarely an issue because the hydraulic conductivity of the drainage material is usually sufficiently large to preclude the possibility of liquefaction. Strength is rarely a problem with granular materials. Reasons not to compact the drainage layer are as follows:

1. Compacting the drainage material increases the amount of fines in the drainage material, which decreases hydraulic conductivity.
2. Compacting the drainage layer reduces the porosity of the material, which decreases hydraulic conductivity.
3. Dynamic compaction stresses may damage underlying geosynthetics.

Unless there is a sound reason for the drainage material to be compacted, it is recommended that the drainage material not be compacted. The main goal of the drainage layer is to remove liquids, and this can only be accomplished if the drainage layer has high hydraulic conductivity. The uncompacted drainage layer may be slightly compressible, but the amount of compression is expected to be small.

There is a potential problem with drainage layer materials placed on sideslopes. In some situations, the friction between the drainage layer and underlying geosynthetic component may not be adequate to maintain stability of the sideslope. CQA personnel should assume that the designer has analyzed slope stability and designed stable sideslopes for assumed materials and conditions. However, CQA personnel should be watchful for evidence of slippage at the interface between the drainage layer and an underlying geosynthetic component. If problems are noted, the design engineer should be notified immediately.

6.9 Protection

The main protection required for the drainage layer is to ensure that (1) large pieces of waste material do not penetrate excessively into the layer and (2) fines do not contaminate the layer. Many designs call for placement of protective soil or select waste directly on the leachate collection layer. If select waste will be placed directly on the leachate collection layer, as shown in Figure 6-4, CQA personnel should stand near the working face of the first lift of solid waste placed on the leachate collection layer to observe placement of select material. No widely accepted definition for "select waste" has been developed, but it usually refers to waste free of large objects that might puncture underlying materials. Sometimes a protective layer of soil is placed above the liner system.

Fine material can cause a significant reduction in hydraulic conductivity. Wind-borne fines may contaminate drainage materials. Soil erosion from adjacent slopes may also lead to accumulation of fines in the drainage material. The CQA personnel cannot complete their job until the drainage material is fully covered and protected.

Residual fines may be washed by rain from other soils and may plug drainage materials during construction. Reddi et al. (2000) report decreases in hydraulic conductivity on an order of magnitude from suspended clay particles in water en-

Figure 6-4. CQC and CQA Personnel Observing Placement of Select Waste on a Drainage Layer.

tering a filter medium. The accumulation of fines in sumps or other low points can reduce the effectiveness of the drainage system. CQC/CQA personnel should be aware of this potential problem and watch for (1) areas where fines may be washed into the drainage material and (2) evidence of lack of free drainage in low-lying areas (e.g., development of ponds of water in the drainage material in low-lying areas). If excessive fines are washed into a portion of the drainage material, the design engineer should be contacted for further evaluation before covering the drainage material by the next successive layer in the system.

6.10 Alternative Final Covers

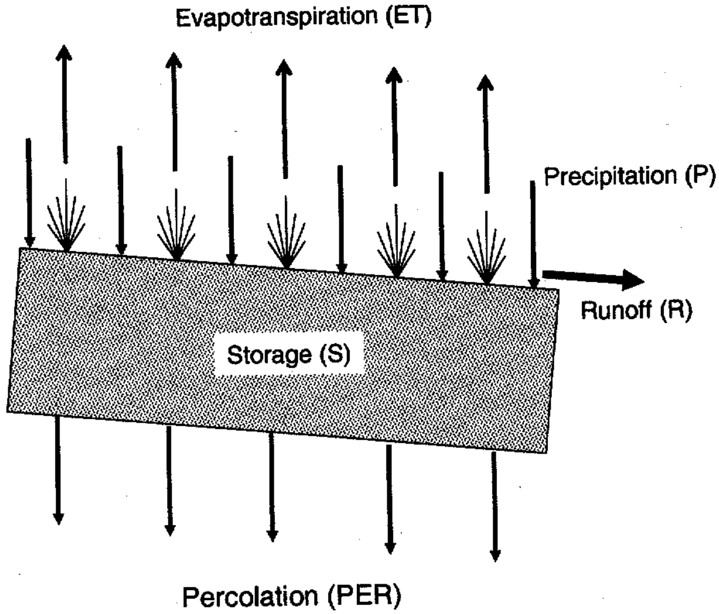
6.10.1 Introduction

An "alternative final cover" of a municipal solid waste landfill refers to a final cover that does not rely on a low-permeability layer, such as a geomembrane, compacted clay liner, or geosynthetic clay liner, to impede percolation of water. Alternative final covers are designed essentially as water storage media that will retain water until it can be returned to the atmosphere via evapotranspiration, for example, by plants. Background information about alternative final covers may be found in Benson (1999), Khire et al. (2000), Benson et al. (2001), and Albright et al. (2004).

6.10.2 Soil Moisture Retention

The basic water balance parameters for an alternative final cover are illustrated in Figure 6-5. Precipitation (P), which may include snowmelt, falls on the cover. Some of the precipitation may drain as a result of runoff (R). Each increment of precipitation that infiltrates into the cover produces a change in soil moisture storage (ΔS), as long as the soil has not exceeded its maximum storage capacity. Soil moisture is returned to the atmosphere via evapotranspiration (ET). Although evapotranspiration can include direct evaporation of water and sublimation of frozen water, the vast majority of evapotranspiration is the result of plants transpiring soil water to the atmosphere.

The "field capacity" of a soil is determined by saturating the soil and allowing it to drain by gravity until equilibrium is reached and no additional water drains from the soil. The water content in this condition is termed "field capacity." Soils with water contents below field capacity can store additional water, up to field capacity, with no drainage of water from the soil. However, when the field capacity of the soil is exceeded, gravity drainage occurs from the soil, and percolation through the cover (PER) develops. The fundamental goal in the design of alternative final covers is to keep the water content of the soil below field capacity, which will preclude gravity drainage and essentially stop percolation of water through the cover. There is no ASTM standard method for measuring field capacity, but common practice is to place the soil in a device that will enable the application of a small negative water pressure and then to measure the water con-



$$\boxed{PER = P - ET - R - \Delta S}$$

Figure 6-5. Basic Water Balance in an Alternative Final Cover.

tent of the soil when it equilibrates at this pressure. No standardized pressure exists, although historically a suction of approximately one-third atm has been used. Jury et al. (1991) suggested a suction of about 2.5 kPa, or about one-fourth atm, to define the approximate water content at field capacity.

Different soils have different field capacities and hence different abilities to store soil moisture. Generally, the finer the soil, the larger the amount of water that can be retained by the soil. Fine-grained materials such as silts make excellent soil moisture retention media. Coarse-textured materials, such as gravel, drain water readily and, thus, are poor materials for retaining moisture in a final cover.

The key to design of an effective alternative final cover is to have sufficient water storage capacity so that during the time of year when the soil is wettest, the water content of the cover soil remains below field capacity. If the water content of the cover soil never exceeds field capacity, then percolation through the cover is essentially zero. The soil is usually wettest following a period of prolonged precipitation with minimal evapotranspiration (usually at the end of winter or into the spring). Figure 6-6 illustrates how the water content in a cover system might vary over a typical year. The soil is typically wettest in the winter and spring and driest in summer and fall. The key is whether, at its wettest, the water content of the soil is above or below field capacity. If it is below field capacity, there is essentially no

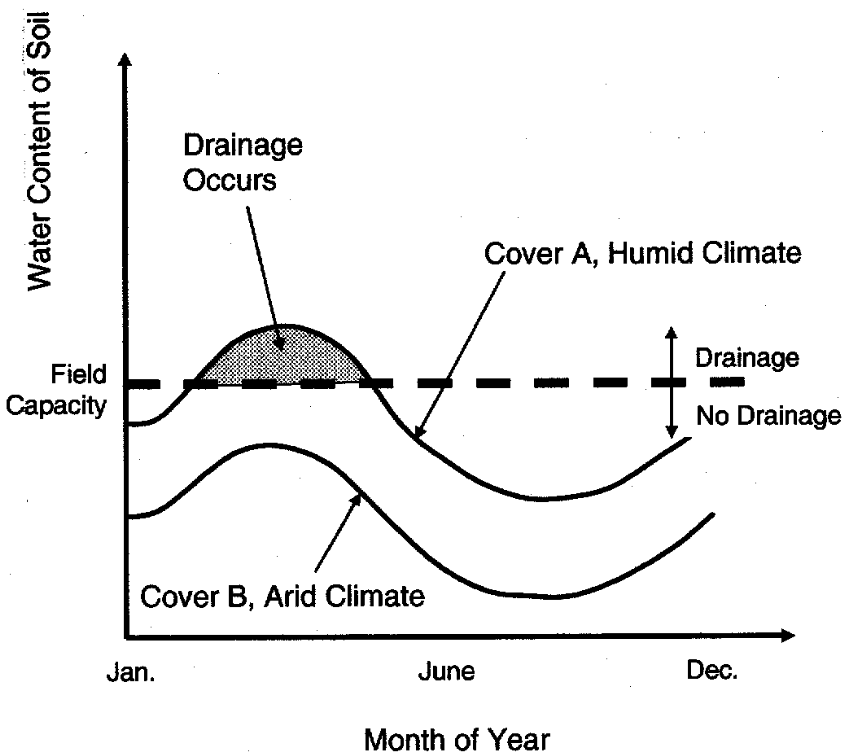


Figure 6-6. Difference in Water Balance between Humid and Arid Site.

percolation. The longer the soil remains above field capacity, the greater the percolation of water through the alternative final cover. In humid climates, the amount of precipitation, compared to evapotranspiration, is large, making it far more difficult to limit percolation to small amounts without incorporating a low-permeability geosynthetic layer in the cover.

6.10.3 Types of Final Covers

In general, two types of alternative final covers are in use (Albright et al. 2004). As shown in Fig 6-7(a), a monolithic cover consists of a single layer of water-absorbent soil with plants at the surface. The soil should be a material that absorbs and retains water well, such as silt, silty sand, clayey sand, or low-plasticity clay material. Highly plastic clays are not desirable because they can shrink and crack during a drought. Desiccation cracks are undesirable because they allow water to penetrate into and perhaps through the cap quickly, rather than being stored in the cap. The soil should support growth of plants, which are critical for maximizing evapotranspiration. Plant species are usually selected carefully to ensure a robust, drought-resistant plant cover that maximizes evapotranspiration and provides protection from erosion.

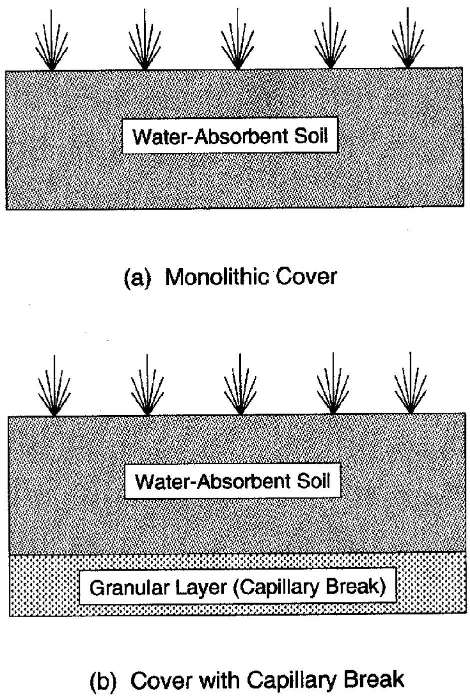


Figure 6-7. Two Types of Alternative Final Covers.

The second type of alternative cover, shown in Figure 6-7(b), consists of a water storage layer underlain by a granular material that serves as a capillary break. The water storage layer has essentially the same requirements as the soil in a monolithic alternative final cover, described in the previous paragraph. The granular layer (capillary break) is generally a coarse-grained material that is almost completely free of fines. A clean gravel (Unified Soil Classification System, symbol "GP") or coarse, clean sand ("SP") would be the typical material of choice for this layer. Figure 6-7(b) does not show a filter layer between the water-absorbent soil and capillary break material, but if the two do not meet standardized filter criteria, it is essential that these layers be separated by a soil filter or geotextile filter. This will generally be the site-specific situation.

6.10.4 Capillary Break

The granular, or capillary-break, layer functions as a low-permeability layer for transmission of liquid water so long as it is essentially dry. Design methods are described by Stormont and Morris (1997) and Khire et al. (2000). What is perhaps counterintuitive is the fact that a clean sand or gravel (which would have high hydraulic conductivity in a saturated state) can serve as a low-permeability, high-impedance layer to water percolation in an unsaturated state.

In soil, water flows in response to gradients in the energy of the soil water, that is, a gradient in hydraulic head. In unsaturated soils, the soil water is in a state of negative pressure (capillary condition). The suction in the soil water is the absolute value of the negative head or soil water pressure. The drier the soil, the larger the soil suction.

When soil water is at equilibrium and there is no flow of water, the soil suction is essentially constant throughout the soil. Even with small amounts of water movement, the gradient in suction is often small, and soil suctions in adjacent materials tend to be more or less the same at any given time. The water content in two different soils can be different, even though the suctions are the same. To illustrate, soil moisture retention curves are sketched in Figure 6-8 for a typical water-absorbent soil, for example, silt, and a typical capillary-break material, for example, gravel. For a given suction, the capillary-break material is far drier than the water-absorbent soil, that is, the water content of the capillary-break material (w_{CBM}) is far less than the water content of the water-absorbent soil (w_{WAS}).

The hydraulic conductivity of materials decreases rapidly with decreasing water content. Paths for flow of liquid water in relatively dry soils are extremely

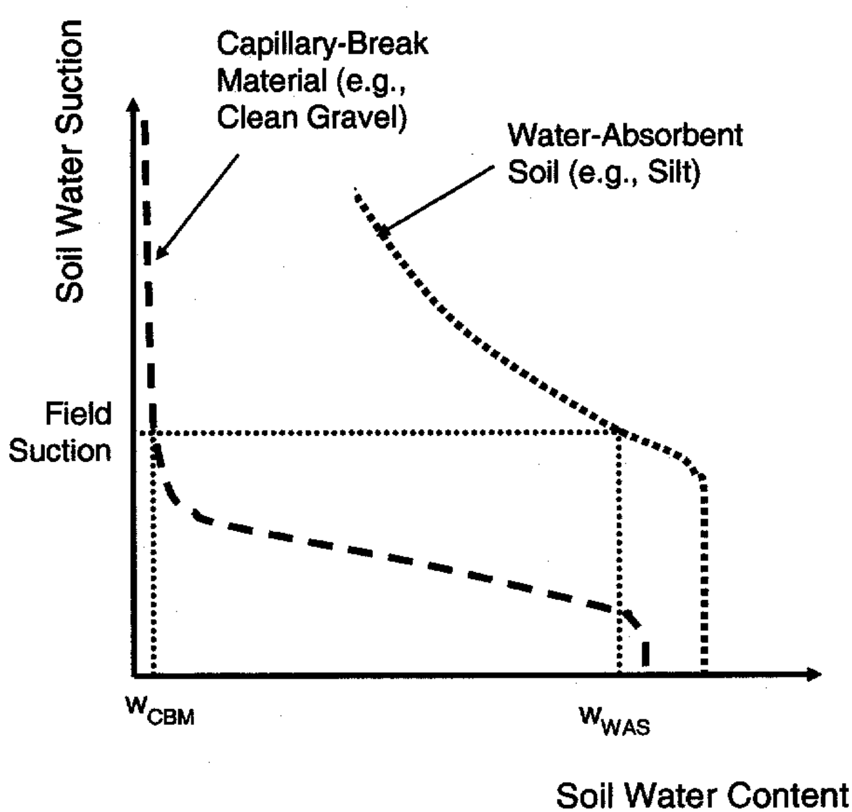


Figure 6-8. Typical Soil Moisture Characteristic Curves for an Alternative Final Cover.

tortuous and occur only through thin films of water coating the surfaces of almost dry soil particles, or receded into the tiniest capillary pores between particles. Thus, dry gravel is much less permeable to liquid water than is comparatively moist silt. Dry gravel serves to impede the flow of water, maximizing retention of water in the overlying layer and effectively breaking the hydraulic connectivity between soil water in the overlying and underlying layers. Of course, the granular material only impedes water flow if it is relatively dry; should this layer become saturated, it is no longer effective in limiting downward infiltration of water.

Capillary-break materials are commonly used. For example, they are used beneath floor slabs of buildings to keep the slab dry. They are also used in golf course greens to help retain water in the overlying soil, which supports growth of grass and minimizes irrigation needs.

A graph of water content versus suction (e.g., Figure 6-8) is termed a "soil moisture characteristic curve" and can be measured with a pressure plate (ASTM D2325) or pressure membrane device (ASTM D3152). The filter paper method (ASTM D5298) is a convenient method for estimating the suction of a soil at a given water content. Much of the science and experimental methodology developed for soil moisture retention was developed in the soil science and agronomy fields.

6.10.5 Construction Quality Assurance

6.10.5.1 Soil Moisture Retention Layer

The soil in a monolithic final cover, or the water-absorbent material in a capillary barrier, is generally not heavily compacted because excess compaction can make the soil less suitable as a growth medium for plants. Similarly, the moisture content of the soil at the time of placement is not especially critical to the moisture retention characteristics of the material, although placement of the soil dry of the field capacity is desirable to limit drainage after construction. Zornberg et al. (2003) describe an alternative final cover in which the cover soils were placed at optimum water content (ASTM D698) plus or minus two percentage points, and at a density of at least 90% of maximum density from D698 (standard Proctor). Adequate fines content was ensured by requiring that the hydraulic conductivity of saturated soil be less than 5×10^{-7} m/s ($<5 \times 10^{-5}$ cm/s).

The thickness of the monolithic cover is important and might typically be in the range of approximately 1 to 2 m (Albright et al. 2004). The important CQA parameters for the water-retention material are (1) gradation of the material, (2) classification of the material, (3) thickness of the layer, and (4) compaction (but not too much compaction).

The gradation of the material and its soil classification may be specified and, if so, are critical CQA parameters. The designer likely intends that this material contain sufficient fines to absorb water well. In the case of clay materials, the specification may limit the soils to relatively low-plasticity clays because highly plastic clays are vulnerable to desiccation cracking, which is not desirable in a water-retention layer.

No guidelines exist for the frequency of testing, but if no specification is given, the authors suggest at least one gradation test per meter of material thickness per hectare of cover, as a minimum. The same would apply to other tests to verify specified parameters, such as soil classification.

Compaction of the water-absorbent soil is usually not a critical parameter, and overcompaction can be detrimental. Thus, CQA requirements for compaction are usually minimal (nothing like those for a compacted clay liner, for example). However, the testing methods for moisture and density are the same as those described earlier for compacted clay liners. The authors suggest a testing frequency of about one moisture–density test per meter of soil thickness per hectare of final cover. Observation of the placement and compaction process by a qualified construction monitor is helpful in supplementing test data.

The designer may specify soil moisture retention characteristics and may even require a few measurements of soil moisture characteristic curves for CQA purposes, but such tests should number no more than a few for the entire project. The tests are slow and time-consuming and, if required at all, should be infrequent and for verification purposes, rather than for day-to-day CQA.

6.10.5.2 Capillary-Break Material

The important characteristic of this material is that it contains few fines. Typically, the unified soil classification of this material will be “SP” or “GP.” This gradation can be confirmed by sampling and testing the material for particle-size distribution (ASTM D6913). Soils are classified according to ASTM D2487.

No recommended testing frequencies have been published for material testing of capillary-break materials to ensure conformance with specifications. The authors recommend testing at a slightly greater frequency compared to the water-absorbent layer, for example, one test per 0.5 m of soil thickness per hectare of final cover.

Because the objective of a capillary-break material is to have large void spaces between soil particles and minimal fine material, excessive compaction should be avoided. Thus, compaction requirements are usually not critical. The authors suggest a testing frequency of about one moisture–density test per meter of soil thickness per hectare of final cover.

Care should be given to observing the placement and covering of the capillary-break material to ensure that fines are not washed into the material during construction.

Although carbonate content is a potentially important characteristic of granular drainage materials, it is comparatively insignificant for a capillary-break material because the capillary-break material should be essentially dry most of the time and should seldom, if ever, contain enough flowing water to risk buildup of carbonate precipitates.

6.10.5.3 Filter

If the overlying material does not meet filter criteria with the capillary-break material, a filter to separate the overlying material from the capillary-break material is essential (Figure 6-2). The filter may be a geotextile or soil material. Design criteria were discussed earlier for each material. If a geotextile filter is used, its durability criteria are important. Testing of grain-size characteristics to ensure conformance with specifications should occur with a suggested frequency of one test per hectare.

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ASTM D2434. "Standard test method for permeability of granular soils (constant head)."

ASTM D2487. "Standard classification of soils for engineering purposes (Unified Soil Classification System)."

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Geosynthetic Drainage Systems

7.1 Overview

The collection of liquids within waste containment systems, their transmission, and eventual removal represent an important element in the successful functioning of waste containment facilities. Focus in this chapter is on the primary and secondary leachate collection systems *beneath* solid waste and on surface water and gas removal systems in the cover *above* the waste. This chapter parallels Chapter 6 on natural soil drainage materials, but it uses geosynthetics. In actual practice, combined systems such as drainage geocomposites overlain by natural soils (usually sand) are often used; however, we will focus here on the individual geosynthetic components. The individual materials to be described are the following:

- geotextiles used as filters over various drainage systems (geonets, sands, and gravels);
- thick geotextiles used for gas collection;
- geonets and geocomposites (geotextiles laminated to geonets) used as primary or secondary leachate collection and gas collection; and
- other geosynthetic drainage materials used as surface-water collection systems and possibly as primary leachate collection and secondary leachate collection.

The locations of the various geosynthetic materials listed above are illustrated in the sketch of Figure 7-1. The chapter will also provide a discussion of geotextile protection (or cushioning) layers, which are placed over or under an associated geomembrane.

7.2 Geotextiles as Filters and Separators

Geotextiles, which some refer to as filter fabrics or even construction cloth, consist of polymeric yarns (fibers) made into woven or nonwoven textile sheets and supplied to the job site in large rolls. When ready for placement, the rolls are removed from their protective covering, properly positioned, and unrolled over the substrate

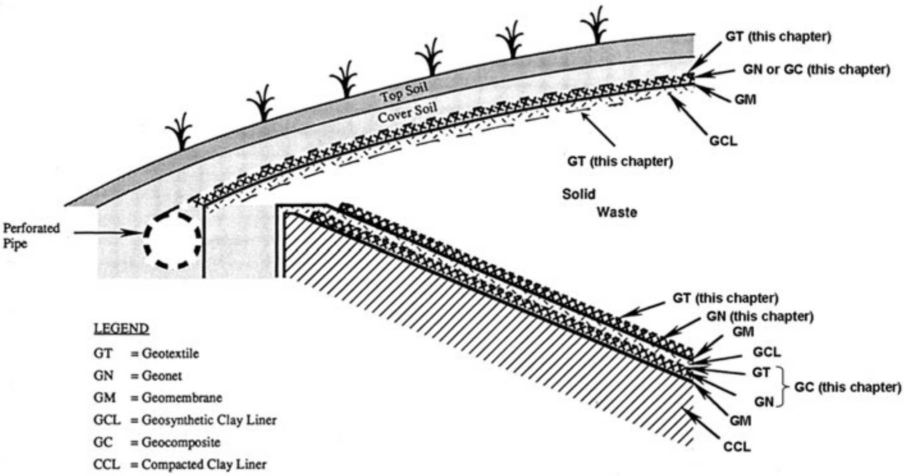


Figure 7-1. Cross Section of a Landfill Illustrating the Use of Different Geosynthetics Involved in Waste Containment Drainage Systems.

material. The substrate on which the geotextile is placed is usually a geomembrane, geonet, drainage soil, or other soil material. The roll edges and ends are either overlapped for a specified distance or are sewn together. After approval by the CQA personnel, the geotextile is covered with the overlying material. Depending on site-specific conditions, this overlying material can be a geomembrane, geosynthetic clay liner, compacted clay liner, geonet, or drainage soil.

This section presents the MQC/MQA aspects of geotextiles insofar as their manufacturing is concerned and the CQC/CQA aspects as far as handling, placement, seaming, and backfilling are concerned.

7.2.1 Manufacturing of Geotextiles

The manufacturing of geotextiles made from polymeric fibers follows traditional textile manufacturing methods and uses the same type of equipment. Most manufacturing facilities have developed their respective geotextile products to the point where product quality control procedures and programs are routine and fully developed. Many are ISO 9000 and/or ISO 14,000 certified, which is a good indication of the manufacturer's commitment to providing high-quality products.

Three discrete stages in the manufacture of geotextiles should be recognized from an MQC/MQA perspective: (1) the polymeric or resin base materials; (2) yarn or fiber; and (3) fabric. Each stage will be described.

7.2.1.1 Resins and Their Additives

Approximately 90% of geotextiles used today are made from polypropylene resin. The other 10% are polyester and a range of polymers, including polyethylene, nylon, and others used for specialty purposes. As with all geosynthetics, however, the base resin has various additives formulated with it, resulting in the final compound. Additives for UV light protection and as processing aids are common (Table 7-1).

The resin is usually supplied in the form of pellets, which are then blended with carbon black (either in the form of concentrate pellets or chips, or as a powder) or the complete additive package. The additive package is usually a powder and is proprietary with each particular manufacturer. For some manufacturers, the pellets are precompounded with carbon black or the entire additive package. Polypropylene pellets and carbon black are similar to those shown in the manufacture of polyethylene geomembranes. For nonblack geotextiles, the carbon black is omitted and titanium dioxide is substituted along with a colorant. In this case, the formulation is slightly different than that listed in Table 7-1. The UV exposure tests to be described will be used for an assessment of the final product.

The following items should be considered for a specification or MQA document for resins and additives used in the manufacture of geotextiles for waste containment applications:

1. The resin should meet the specific manufacturer's MQC requirements. This stipulation usually requires a certificate of analysis to be submitted by the resin vendor for each lot supplied. Included will be various properties, their specification limits, and the appropriate test methods. For polypropylene and polyethylene resins, the usual requirements are density (per ASTM D792 or ASTM D1505), melt flow index (per ASTM D1238), and other properties felt to be relevant by the manufacturer. For polyester resin, the usual requirements are intrinsic viscosity, solution viscosity, color, moisture content, and other properties felt to be relevant by the manufacturer.
2. The internal quality control of the manufacturer should be reported to verify that the geotextile manufactured for the project meets the proper specifications.
3. The frequency of performing each of the preceding tests should be covered in the MQC plan, which should be implemented and followed.
4. The percentage and type of carbon black, according to ASTM D1603, should be specified for the particular formulation being used, although it is usually low in comparison to geomembranes. Note that many geotextiles are not formulated with carbon black and instead use titanium dioxide, colorants, various antioxidants, and UV stabilizers. These are product-specific issues that are acceptable as long as the specification is met.

Table 7-1. Compounds Used in the Manufacture of Geotextiles (Values are Percentages Based on Weight)

Generic Name	*Resin*	*Carbon Black*	*Other Additives*
Polypropylene	95–98	0–2	1–3
Polyester	97–98	0–1	1–2
Polyethylene	95–98	1–3	1–2

5. The type and amount of additives are rarely specified. If a statement is required, it should signify that the additive package has been successfully used in the past and to what extent.

7.2.1.2 Fiber Types

The resin, carbon black, and additives are introduced to an extruder, which supplies heat, mixing action, and filtering. It then forces the molten material to exit through a die, sometimes containing many small orifices called a "spinnerette." The fibers, also called "filaments," are usually drawn (work hardened) by mechanical tension or impinged by air as they are stretched and cooled. The resulting filaments can be wound onto a bobbin or can be used directly to form the finished product. Some filaments are subsequently twisted together in the form of a "yarn" for subsequent fabric manufacturing. Other important manufacturing variations include those made from short "staple" fibers placed into a random, three-dimensional fiber network and those made from flat, tape-like yarns called "slit-film yarns" (IFAI 1990). Each type (filament, staple, or slit-film yarns) can be twisted together with others, as shown in Figure 7-2. Note that the term "yarn" can also be used as a generic term for any continuous strand (fiber, filament, or tape) used to form a textile fabric. Thus, all of the examples in Figure 7-2, except for staple, are yarns and can be used to manufacture geotextiles.

7.2.1.3 Geotextile Types

The fibers, filaments, or yarns just described are joined together to make a fabric, which, when placed in the ground, is called a "geotextile." Generic classifications are woven, nonwoven, and knit. Knit geotextiles, however, are rarely used in waste containment systems and will not be described further.

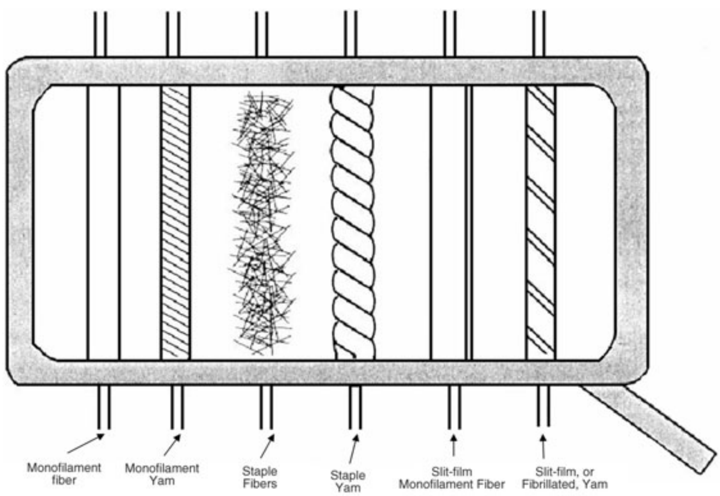


Figure 7-2. Types of Polymer Fibers Used in the Construction of Different Types of Geotextiles.

The manufacturer of a woven geotextile uses the desired type of fiber, filament, or yarn from a bobbin and constructs the fabric on a weaving loom. Fabric weaving technology is well established over literally centuries of development. Most woven fabric patterns used for geotextiles are simple, or basket-type, weaves consisting of each yarn going over and under an intersecting yarn on an alternate basis. Figure 7-3(a) shows a micrograph of a typical woven monofilament geotextile pattern.

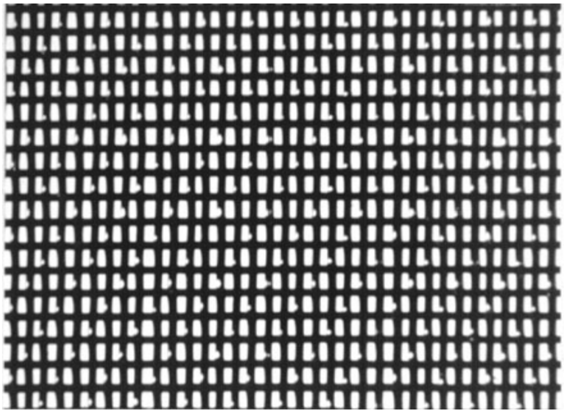
In contrast to this type of uniformly woven pattern are nonwoven fabrics as shown in Figure 7-3(b) and (c). Here the staple fibers, filaments, or yarns are used directly and laid down on a moving belt randomly. The speed of the moving belt dictates the mass per unit area of the final product. While it is positioned on the belt, the material is lofty, and the fibers, filaments, or yarns are not mechanically entangled in any way. Two variations of mechanical bonding can be used, which give rise to two unique types of nonwoven geotextiles.

- Nonwoven, *needle-punched* geotextiles go through a needling process, in which barbed needles penetrate the fabric and entangle numerous fibers transverse to the plane of the fabric (note the fiber entanglement pattern in Figure 7-3(b)). As a postprocessing step, the fabric can be passed over a heated roller, resulting in a singed or burnished surface of the fibers, filaments, or yarns on one or both sides of the fabric.
- Nonwoven, *heat bonded* geotextiles are formed by passing the unbonded filament mat through counterrotating rollers with a source of heat, usually steam or hot air, thereby melting some of the fibers at various crossover points (note the fiber bonding pattern in Figure 7-3(c)). This process compresses the mat and simultaneously joins some of the fibers at their intersections by melt bonding.

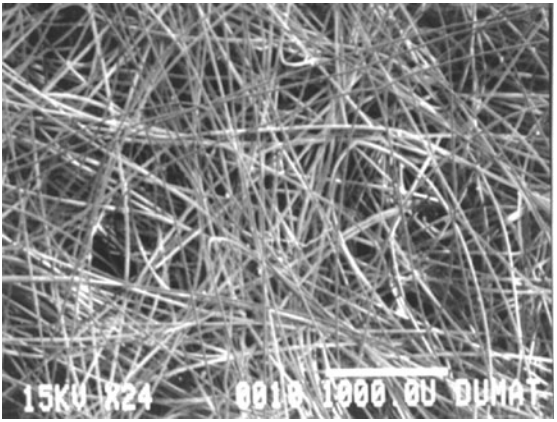
7.2.1.4 General Specification Items

There are numerous items recommended for inclusion in a specification or MQA document for geotextiles used in waste containment facilities:

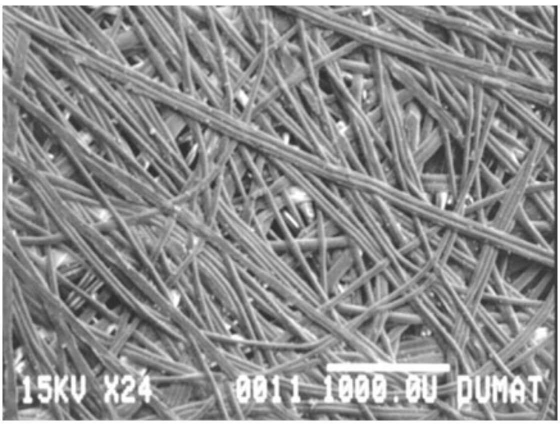
1. There should be verification and certification that the actual geotextile properties meet the manufacturer's specification for that particular type and style.
2. Quality control certifications should include mass per unit area (per ASTM D5261), grab tensile strength and elongation (per ASTM D4632), trapezoidal tear strength (per ASTM D4533), puncture strength (per ASTM D4833 or ASTM D6241), thickness (per ASTM D5199), apparent opening size, or AOS (per ASTM D4751), and permittivity (per ASTM D4491).
3. Values for each property should meet, or exceed, the project specification values (note in the case of AOS that the property listed is a maximum value; all others are minimum values). In this regard, it should be noted that there exists a widely used highway specification for both geotextile filters and separators that is often used for waste containment applications. It is designated as AASHTO M288, followed by the year of the latest revision.



(a) Woven monofilament geotextile at 4× magnification



(b) Nonwoven needle-punched geotextile at 24× magnification



(c) Nonwoven heat bonded geotextile at 24× magnification

Figure 7-3. Three Major Types of Geotextiles Used for Filtration and Separation Applications.

4. A statement should be included that the property values listed are based on the minimum average roll value, or MARV. The exception is apparent opening size, which is a maximum average opening size, hence MaxARV. These values are actually the mean value minus (or plus for AOS) two standard deviations of each of the properties of the manufactured geotextile (see Koerner 2005). The specification listing and subsequent conformance testing of the received product should be evaluated in a similar manner. It should also be noted that this concept is only used with geotextiles and not with any other type of geosynthetic material.
5. The geotextile's UV light resistance should be specified, which is usually a certain percentage of strength or elongation retained after exposure in a laboratory weathering device. Usually the xenon arc weatherometer (per ASTM D4355) is specified, and the strength retention after 500 hours is typically 50% to 90%. This property is the only geotextile property that is not based on the MARV (or MaxARV) concept; it is a minimum average value.
6. The frequency of performing each of the preceding tests should be covered in the manufacturer's MQC plan, which should be implemented and followed.
7. Verification that needle-punched, nonwoven geotextiles have been inspected continuously for the presence of broken needles using an in-line metal detector with an adequate sweep rate should be provided. Furthermore, a needle removal system (e.g., full roll width magnets) should be implemented.
8. A statement indicating if, and to what extent, reworked polymer or fibers were added during manufacturing should be included. If used, the statement should note that the rework polymer or fibers were of the same composition as the intended product.
9. Reclaimed or recycled material (i.e., fibers or polymer that have been previously used) should not be added to the formulation unless it is specifically allowed in the project specifications. Note, however, that reclaimed fibers may be used in geotextiles in certain waste containment applications. The gas collection layer above the waste and the geotextile protection layer between drainage stone and a geomembrane are likely locations. These design decisions should be approved by the regulatory agency and should be stated accordingly.

7.2.2 *Handling of Geotextiles*

A number of activities occur between the manufacture of geotextiles and their final positioning in the waste facility. These activities involve protective wrapping, storage at the manufacturing facility, shipment, storage at the site, product acceptance and conformance testing, and final placement at the facility. Each of these topics will be described in this section.

7.2.2.1 Protective Wrapping

All rolls of geotextiles, irrespective of their type, must be enclosed in a protective wrapping that is opaque and waterproof. The object is to prevent degradation from atmospheric exposure (e.g., UV light or ozone), moisture uptake (e.g., rain or snow), and to a limited extent, accidental damage. It must be recognized that geotextiles are the most sensitive of all geosynthetics to degradation induced by UV light exposure. Geotextile manufacturers use tightly wound plastic wraps or loosely fit plastic bags for this purpose. Quite often, the plastic is polyethylene in the thickness range of 0.05 to 0.13 mm (2–5 mils). Several important issues should be considered in a specification or MQA document:

1. The protective wrapping should be wrapped around (or placed around) the geotextile in the manufacturing facility and should be included as the final step in the manufacturing process.
2. The packaging should not interfere with the handling of the rolls either by slings or by the use of the central core on which the geotextile is wound.
3. The protective wrapping should prevent exposure of the geotextile to UV light, prevent it from moisture uptake, and to some extent, limit damage to the roll.
4. Every roll must be labeled with the manufacturer's name, geotextile style and type, lot and roll numbers, and roll dimensions (length, width, and gross weight). Details should conform to ASTM D4873.

7.2.2.2 Storage at the Manufacturing Facility

The manufacturing of geotextiles is such that temporary storage of rolls at the manufacturing facility is generally necessary. Storage times range from a few days to a year or longer. Figure 7-4(a) shows geotextile storage at a manufacturer's facility.

Regarding specification and MQA document items, the following should be considered:

1. Handling of rolls of geotextiles should be done in a competent manner such that damage does not occur to the geotextile or to its protective wrapping. In this regard, ASTM D4873 should be referenced and followed.
2. Rolls of geotextiles should not be stacked on one another to the extent that deformation of the core occurs or to the point where accessibility can cause damage in handling.
3. Outdoor storage of rolls at the manufacturer's facility should not be longer than six months. For storage periods longer than six months, a temporary enclosure should be put over the rolls or they should be moved within an enclosed facility.

7.2.2.3 Shipping

Geotextile rolls are shipped from the manufacturer's (or the manufacturer's representative's) storage facility to the job site via common carrier. Ships, railroads, and trucks have all been used, depending on the origin and final destination. The usual carrier from within the United States is truck. When using flat-bed trucks,

(a) Storage at a manufacturing facility

(b) Storage at a field site

Figure 7-4. Photographs of Temporary Storage of Geotextiles.

the rolls are usually loaded by means of a crane, with slings wrapped around the individual rolls. When the truck bed is closed (i.e., an enclosed trailer), the rolls are usually loaded by forklift with a "stinger" attached. The "stinger" is a long tapered rod that fits inside the core on which the geotextile is wrapped.

Insofar as specifications and MQA/CQA documents are concerned, the following items should be considered:

1. The method of loading the geotextile rolls, transporting them, and off-loading them at the job site should not cause any damage to the geotextile, its core, or its protective wrapping.
2. Any protective wrapping that is accidentally damaged or stripped off the rolls should be repaired immediately, or the roll should be moved to an enclosed facility until its repair can be made to the approval of the CQA personnel.

7.2.2.4 Storage at the Site

Unloading geotextile rolls at the site and temporary storage must both be done in an acceptable manner. Figure 7-4(b) shows typical storage at the field site. Some specification and CQA document items to consider are the following:

1. Handling of rolls of geotextiles should be done in a competent manner such that damage does not occur to the geotextile or its protective wrapping. In this regard, ASTM D4873 should be referenced and followed.
2. The location of field storage should not be in areas where water can accumulate. The rolls should be elevated off the ground so as not to form a dam, creating the ponding of water.
3. The rolls should be stacked in such a way that cores are not crushed and the geotextile is not damaged. Furthermore, they should be stacked in such a way that access for conformance testing is possible.
4. Outdoor storage of rolls should not exceed manufacturer's recommendations or longer than six months, whichever is less. For storage periods longer than six months, a temporary enclosure should be placed over the rolls or they should be moved within an enclosed facility.

7.2.2.5 Acceptance and Conformance Testing

On delivery of the rolls of geotextiles to the project site and temporary storage thereof, the CQA engineer should see that conformance test samples are obtained. These samples are then sent to the CQA laboratory for testing to ensure that the supplied geotextile conforms to the project plans and specifications. The samples are taken from selected rolls by removing the protective wrapping and cutting full-width, 1 m (3 ft) long samples off the outer wrap of the selected rolls. Sometimes the outer revolution of geotextile is discarded before the test sample is taken. The rolls are immediately rewrapped and replaced in temporary field storage. The samples must be appropriately marked for future identification. Alternatively, conformance testing could be performed at the manufacturer's facility and, when completed, the particular lot should be marked for shipment to the particular site under consideration. Items to be considered in specifications and CQA documents in this regard are the following:

1. The samples should be identified by type, style, lot, and roll numbers. The machine direction should be noted on the sample(s) with a waterproof marker of opposing color to the color of the geotextile. A "lot" is defined as a unit of production or a group of other units or rolls with one or more common properties readily separable from other similar units. Note that a lot can also be defined as 10,000 m^2 (100,000 ft^2) of geotextile or the area of the particular site under consideration. Other definitions are also possible and should be clearly stated in the CQA documents.
2. Sampling should be done according to the job specification or CQA documents. Unless otherwise stated, sampling should be based on one per lot.

ASTM D4354 may be referenced and followed in this regard, but it might result in a different value for sampling than that stated above.

3. Testing at the CQA laboratory may include mass per unit area (per ASTM D5261), grab tensile strength (per ASTM D4632), trapezoidal tear strength (per ASTM D4533), puncture strength (per ASTM D4833 or ASTM D6241), possibly apparent opening size (per ASTM D4751), and permittivity (per ASTM D4491). Other conformance tests may be required by the project specifications (ASTM D4759).
4. Conformance test results should be sent to the CQA engineer before deployment of any geotextile from the lot under review.
5. The CQA engineer should review the results and should report any nonconformance to the owner/operator's project manager.
6. The resolution of failing conformance tests must be clearly stipulated in the specifications or CQA documents. Statements should be based on ASTM D4759, entitled "Guide for Determining the Specification Conformance of Geosynthetics."
7. The geotextile rolls that are sampled should be immediately rewrapped in their protective covering to the satisfaction of the CQA personnel.

7.2.2.6 Placement

The geosynthetic installation contractor should remove the protective wrappings from the geotextile rolls to be deployed only after the substrate layer, soil, or other geosynthetic has been documented and approved by the CQA personnel. The specification and CQA documents should be written in such a manner as to ensure that the geotextiles are not damaged or excessively exposed to UV degradation. The following items should be considered for inclusion in a specification or CQA document:

1. The installer should take the necessary precautions to protect the underlying layers on which the geotextile will be placed. If the substrate is soil, construction equipment can be used, provided that excess rutting is not created. Excess rutting should be clearly defined and quantified by the design engineer. In some cases, 25 mm (1.0 in.) is the maximum rut depth allowed. If the ground freezes, the depth of ruts might be further reduced to a specified value. If the substrate is a geosynthetic material, deployment must be by hand, by use of small jack lifts on pneumatic tires with low ground contact pressure, or by use of all-terrain vehicles (ATVs) with low ground contact pressure. It is also possible to use specially adapted construction equipment, provided that the maximum ground contact pressure is not exceeded. For use of ATVs or other equipment, additional restrictions should be considered (see Section 4.3.5.1).
2. During placement, care must be taken not to entrap (either within or beneath the geotextile) stones, excessive dirt, or moisture that could damage a geomembrane, cause clogging of drains or filters, or hamper subsequent seaming.
3. On sideslopes, the geotextiles should be anchored at the top and then unrolled so as to keep the geotextile free of wrinkles and folds.

4. Trimming of the geotextiles should be performed using only upward-cutting hook blades.
5. Nonwoven geotextiles placed on textured geomembranes can be troublesome because of sticking (called a Velcro effect). They are then difficult to align or even separate after they are placed on top of one another. A thin sheet of plastic on the geomembrane during deployment of the geotextile can be helpful in this regard. This is called a "rub sheet," and it must be removed after correct positioning of the geotextile.
6. The geotextile should be weighted with sandbags, or the equivalent, to provide resistance against wind uplift. This is a site-specific procedure and completely the installer's decision. Uplifted and moved geotextiles can generally be reused and repositioned but only after approval by the CQA personnel.
7. A visual examination of the deployed geotextile should be carried out to ensure that no potentially harmful objects are present (e.g., stones, sharp objects, broken needles, or sandbags).

7.2.3 Seaming

Seaming of geotextiles by sewing is sometimes required (versus overlapping or heat bonding with no sewn seams) of the geotextiles placed in waste facilities. This method generally should be the case for geotextiles used in filtration, but it may be waived for geotextiles used as gas collection layers above the waste or as protective layers for geomembranes, as per the plans and specifications. In such cases, heat bonding is also an acceptable alternate method of joining separation geotextiles. In cases where overlapping is permitted, the overlapped distance requirements should be clearly stated in the specification and CQA documents. Geotextile seam types and procedures, seam tests, and geotextile repairs are covered in this section.

7.2.3.1 Seam Types and Procedures

Three types of sewn geotextile seams are shown in Figure 7-5. They are the "flat" or "prayer" seam, the "J" seam, and the "butterfly" seam. Although each seam can be made by a single thread or by a two-thread chain stitch, as illustrated, the latter stitch is recommended. Furthermore, a single, double, or even triple, row of stitches can be made, as illustrated by the dashed lines in the figures. Figure 7-6 shows a photograph of the fabrication of a flat seam. See Diaz (1990) for further details regarding geotextile seaming.

The project specifications or CQA documents should address the following considerations:

1. The type of seam, type of stitch, stitch count or number of stitches per inch, and number of rows should be specified based on the tendency of the fabric to fray, the strength needed, and the toughness of the fabric. For filtration and separation geotextiles, a flat seam using a two-thread chain stitch and one

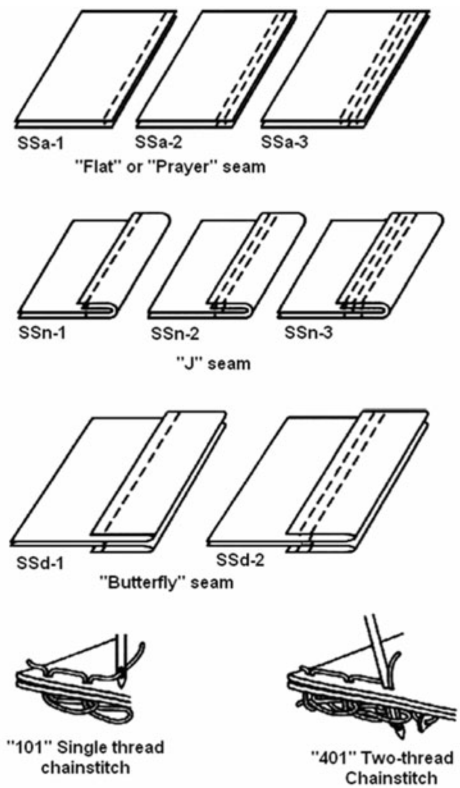


Figure 7-5. Various Types of Sewn Seams for Joining Geotextiles.

Source: Diaz 1990, with permission from Industrial Fabrics Association International.

Figure 7-6. Sewing of a Geotextile Field Seam in a "Flat" or "Prayer" Seam Type.

row is usually specified. For reinforcement geotextiles, stronger and more complex seams are used. Alternatively, a minimum seam strength (per ASTM D4884) could be specified.
2. The seams should be continuous (i.e., spot sewing is generally not allowed).
3. On slopes greater than approximately 5(H)-to-1(V), or 11.3 deg, seams should be constructed parallel to the slope gradient. Exceptions are permitted for small patches and repairs.
4. The thread type must be polymeric with chemical and UV light resistant properties equal to or greater than that of the geotextile itself.
5. The color of the sewing thread should contrast to the color of the geotextile for ease in visual inspection. This may not be possible in some cases due to polymer composition.
6. Heat seaming of geotextiles may be permitted for certain seams. A number of methods are available such as hot plate, hot knife, hot air, and ultrasonic devices.
7. Overlapping of geotextiles with no mechanical or physical joining may be permitted for certain cases. The overlap distance should be stated, depending on the site-specific conditions.

7.2.3.2 Seam Tests

For geotextiles used in filtration and separation, seam samples and subsequent strength testing are generally not required. If tests are, however, they should be stipulated in the specifications or CQA documents. Also, the sampling and testing frequency should be noted accordingly. The test method to evaluate sewn seam test specimens is ASTM D4884.

7.2.3.3 Repairs

Holes or tears in geotextiles, made during placement or any time before backfilling, should be repaired by patching. Some relevant specifications and CQA document items follow:

1. The patch material used for repair of a hole or tear should be the same type of polymeric material as the damaged geotextile, or as approved by the CQA engineer.
2. The patch should extend at least 30 cm (12 in.) beyond any portion of the damaged geotextile.
3. The patch should be sewn in place by hand or machine so as not to accidentally shift out of position or be moved during backfilling or covering operations.
4. The machine direction of the patch should be aligned with the machine direction of the geotextile being repaired.
5. The thread should be of contrasting color to the geotextile and of chemical and UV light resistance properties equal to or greater than that of the geotextile itself.
6. The repair should be made to the satisfaction of the specification and CQA documents.

7.2.4 Backfilling or Covering

The layer of material placed above the deployed geotextile will be soil, a waste material, or another geosynthetic. Soils will vary from compacted clay layers to coarse aggregate drainage layers. Solid waste (if used) should be what is commonly referred to as "select" waste (i.e., waste that has been carefully sorted to avoid large objects) and placed so as not to cause damage. Sometimes shredded tires are placed above the geotextile. Geosynthetics placed above the geotextile will vary from geomembranes to geosynthetic clay liners. Some considerations for a specification and CQA document follow:

1. If soil is to cover the geotextile, it should be done such that the geotextile is not shifted from its intended position and underlying materials are not exposed or damaged.
2. If a geosynthetic is to cover the geotextile, both the underlying geotextile and the newly deployed material should not be damaged during the process.
3. If solid waste or shredded tires are to cover the geotextile, the type of material should be specified, and visual observation of the placement by CQA personnel should be required.
4. The overlying material should not be deployed such that excess tensile stress is mobilized in the geotextile. On sideslopes, this process requires soil backfill to proceed from the bottom of the slope upward.
5. Soil backfilling or covering by another geosynthetic should be done within the time frame stipulated for the particular type of geotextile. Conservative time frames for geotextiles are within 14 days for polypropylene and polyethylene and 28 days for polyester geotextiles.

7.3 Geotextiles as Protection Materials

Much of the text in the previous section on filtration and separation geotextiles applies to geotextiles used as protection materials as well. Both are often needle-punched nonwovens. The distinguishing features for protection materials, however, is that they are always needle-punched nonwovens and are generally quite high in their mass per unit area. Also, there is more possibility of using many different fibers and fiber types than with filtration and separation geotextiles. It has been clearly shown that thick geotextiles offer substantial protection to geomembranes from gravel and other sharp objects (Koerner et al. 1996). The amount of protection provided is proportional to the mass per unit area; large, angular gravel sometimes requires geotextile weights of up to 2,000 g/m^2 (59 oz/yd^2) (Figure 7-7). Others have found that puncture strength is also an important property (Shercliff 1996). The required weight and associated properties must be stipulated in the project plans and specifications. A generic specification for this type of geotextile is available under the designation of GRI-GT12. Some considera-

Figure 7-7. Thick Needle-Punched Nonwoven Geotextiles Used as Protection.

tions for a geotextile protection material specification and CQA document are as follows:

1. Physical and mechanical properties that should be specified are the following:
   - mass per unit area (per ASTM D5261);
   - grab tensile strength (per ASTM D4632);
   - grab tensile elongation (per ASTM D4632);
   - trapezoidal tear strength (per ASTM D4533);
   - puncture strength (per ASTM D4833, ASTM D5494, or ASTM D6241); and
   - UV resistance (per ASTM D4355).
2. All of the above values should be required as being MARV, except for UV resistance, which is to be a "minimum average" value.
3. The frequency of testing should be sufficient to establish the MARV value on a statistically reliable basis.
4. The type of fiber should be given consideration in that this application can use a variety of fibers, even the possible use of postconsumer fibers. This is a project-specific issue and, if allowed, must be communicated accordingly.
5. For extremely thick protection geotextiles, it may be necessary to evaluate the thoroughness of the needling process by performing direct shear tests per ASTM D5321. This is a product-specific issue.
6. Placement of protection geotextiles is done as described in the previous section, with the exception that these thick, nonwoven geotextiles are rarely sewn together. If some type of mechanical joining is necessary, a hot wedge, hot air, or hot knife device can be used. If not mechanically joined, there must be adequate overlap to prevent stones or gravel from getting under the geotextile and impinging directly on the geomembrane.

7.4 Geonets and Geonet–Geotextile Geocomposites

Geonets are geosynthetic materials with unitized sets of parallel ribs positioned in layers such that liquid can be transmitted within and between their open spaces. Thus, the primary function of geonets is in-plane drainage (recall the application areas of Figure 7-1). Figure 7-8(a) shows a photograph of several rolls of geonets. Figure 7-8(b) shows a close-up of the configuration of a typical biplanar geonet. Figure 7-8(c) shows a close-up of a triplanar geonet. Note that open space exists both in the plane of the geonet (above, under, or along the parallel sets of ribs) and cross plane to the geonet (within the apertures between adjacent sets of ribs). In all cases, the apertures must be protected against migration and clogging by adjacent soil or other particulate materials. Thus, geonets *always* function with geomembranes or geotextiles or both on both of their planar surfaces. Whenever the geonet comes supplied with a geotextile on one or both of its surfaces, it is called a geocomposite or, more accurately, a drainage geocomposite. The geotextile is usually bonded to the surface by heat fusing in the factory per the project specifications.

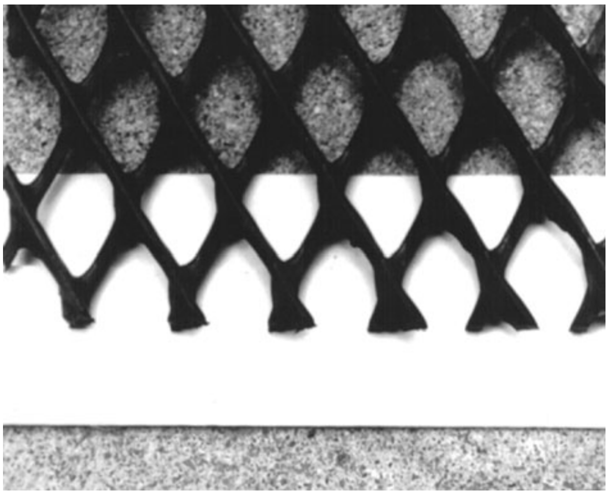
This section will describe the manufacturing and handling of geonets for waste containment facilities. Because continuity of liquid flow is necessary at the sides and ends of the rolls, joining methods will also be addressed, as will the placement of the covering layer. Also addressed will be the bonding of geotextiles to geonets in the form of drainage geocomposites.

7.4.1 Manufacturing of Geonets

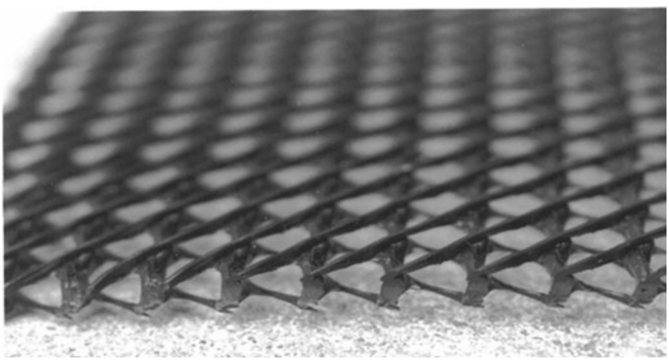
Geonets currently used in waste containment applications are formed using an extruder, which accepts the intended polymer formulation and then melts, mixes, filters, and feeds the molten material directly into a counterrotating die. This die imparts either two or three parallel sets of ribs in the preform. On exiting the die, the ribs of the preform are opened by being forced over a steel spreading mandrel. Figure 7-9 shows a small laboratory size biplanar geonet as it is formed and expands into its final shape. The fully formed geonet is then water quenched, longitudinally cut in the machine direction, spread open as it exits the quench tank, and rolled onto a handling core. The widths of the rolls are determined by the maximum circumference of the spreading mandrel. Because the process is continuous in its operation, the roll length is determined on the basis of the manageable size and weight of a roll. The thickness of the geonet is based on the slot dimensions of the opposing sections of the counterrotating mold. Thicknesses of commercially available geonets generally vary between 4.0 and 8.9 mm (160–350 mils).

Essentially all the resins used for geonets are high-density polyethylene in the natural resin density range of 0.940 to 0.955 g/cm^3. Thus, they are truly high-density polyethylene according to ASTM D1248. This density is significantly higher than that for HDPE geomembranes. The final compound is approximately 97% polyethylene

(a) Rolls of drainage geonets



(b) Biplanar geonets



(c) Triplanar geonets

Figure 7-8. Various Geonets Used in Waste Containment Facilities.

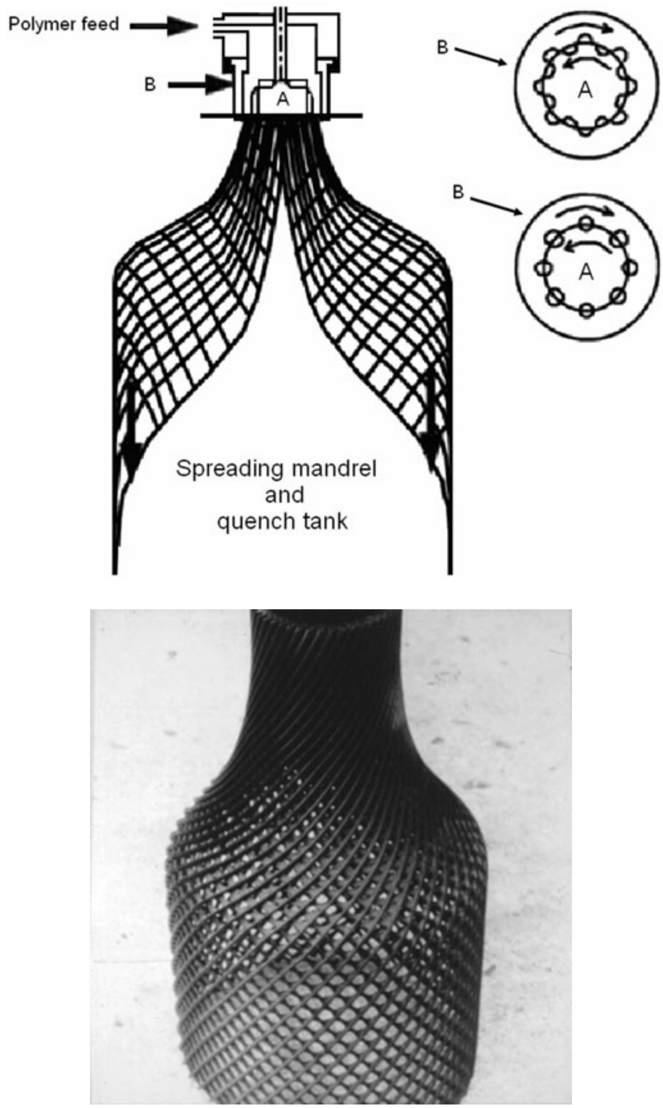


Figure 7-9. Counterrotating Die Technique (top sketch) for Manufacturing Biplanar Drainage Geonets and Example of Laboratory Prototype (bottom photo).

Note: Triplanar drainage geonets are made similarly but have three sections to the counterrotating die with stationary central slots for the main drainage ribs.

resin. An additional 2% to 3% is carbon black, added as concentrate, and the remaining 0.5% to 1.0% are additives. The additives (antioxidants and processing aids) are added to the master batch along with the carbon black, both of which are proprietary to the various geonet manufacturers. Formulations are often the same as for HDPE geomembranes (recall Chapter 4), or slight variations thereof.

Regarding the preparation of a specification or MQA document for the resin component of HDPE geonets, the following items should be considered:

1. The density of geonet resins is in the high-density range for polyethylene (i.e., its density is greater than 0.940 g/cm^3). Specifications may call for the resin to be made from virgin, uncontaminated ingredients. Geonets can be made with some off-spec geomembrane material as a part of their total composition, provided this material is of a similar formulation as the intended geonet and does not consist of recycled or reclaimed material. Recycled or reclaimed material is generally not allowed.
2. Typical quality control tests on the resin are density (via ASTM D1505 or D792) and melt flow index (via ASTM D1238).
3. An HDPE geonet formulation should consist of at least 97% polyethylene resin; the balance is carbon black and additives. No fillers, extenders, or other materials should be mixed into the formulation.
4. By adding carbon black and additives to the resin, the density of the final formulated product is generally more than 0.945 g/cm^3. Because this value is in the high-density polyethylene category according to ASTM D1248, geonets of this type are properly referred to as high-density polyethylene (HDPE).
5. Regrind or reworked polymer, which is previously processed HDPE geonet in chip form, is often added to the extruder during processing. It is acceptable if it is the same formulation as the geonet being produced. Percentages up to 10% by weight have been used successfully.
6. No amount of recycled or reclaimed material, which has seen earlier use in another product, should be added to the formulation.
7. An acceptable variation of the process just described is to add a foaming agent into the extruder, which then is processed in the standard manner. As the geonet is formed and is subsequently quenched, the foaming agent expands within the ribs, creating innumerable small spherical voids. The voids are approximately 0.01 mm (0.5 mil) in diameter. This type of geonet is called a "foamed-rib" geonet, in contrast to the standard type, which is a "solid-rib" geonet. Foamed-rib geonets are currently seen less frequently in drainage systems than previously.
8. QC certificates from the manufacturer should include proper identification of the product and style and results of QC tests.
9. The frequency of performing each of the preceding tests should be covered in the MQC plan, which should be implemented and followed.
10. At this time, there is no generic specification for either biplanar or triplanar geonets or geocomposites. Tests that should be considered for a generic specification are as follows:
    1. Geonet
        - density (per ASTM D1505 or ASTM D792),
        - thickness (per ASTM 5199),
        - carbon black content (per ASTM D4218),

   - compression strength (per ASTM D1621), and
   - transmissivity (per ASTM D4716).
2. Geotextiles
   - mass per unit area (per ASTM D5261),
   - grab tensile strength and elongation (per ASTM D4632),
   - trapezoidal tear strength (per ASTM D4533),
   - puncture strength (per ASTM D4833),
   - permittivity (per ASTM D4491),
   - apparent opening size (per ASTM D4751), and
   - UV stability (per ASTM D4355).
3. Geonet composite
   - transmissivity (per ASTM D4716) and
   - ply adhesion (per ASTM D6636).

11. The precise manner of conducting the transmissivity test is important insofar as verifying design values. GRI-GC8 provides a guide for determining the allowable, or design, flow rate of geonets and geocomposites. It is not, however, a manufacturer's MQC specification. The GRI's MQC specification for geonets and geonet composites is currently in draft form.

7.4.2 Handling of Geonets

A number of activities occur between the manufacture of geonets and their final positioning where intended at the waste facility. These activities involve packaging, storage at the manufacturing facility, shipment, storage at the site, acceptance and conformance testing, and final placement at the facility. Each of these topics will be described in this section.

7.4.2.1 Packaging

As geonets come from the quenching tank, they are wound on a core until the desired length is reached. The geonet is then cut along its width, and the entire roll is wrapped by polymer straps so as not to unwind during subsequent handling. There is generally no protective wrapping placed around geonets; however, a plastic wrapping can be provided if necessary.

Specifications or an MQA document should be formed around a few important points:

1. The core must be stable enough to support the geonet roll while it is handled by either slings around it or from a forklift "stinger" inserted in it.
2. The core should have a minimum 100 mm (4.0 in.) inside diameter.
3. The polymer banding straps around the outside of the roll should be made from materials with adequate strength yet should not damage the outer wrap of the roll.

7.4.2.2 Storage at the Manufacturing Facility

The storage of geonet rolls at the manufacturer's facility is similar to that described for geomembranes. (Refer to Section 4.3.1 for a complete description.)

7.4.2.3 Shipping

The shipment of geonet rolls from the manufacturer's facility to the project site is similar to that described for geomembranes. (Refer to Section 4.3.3 for a complete description.)

7.4.2.4 Storage at the Site

The storage of geonet rolls at the project site is similar to that described for geomembranes. Refer to Section 4.3.3 for a complete description, and see Figure 7-10. An important additional consideration is that a ground cloth should be placed under geonets if they are stored on soil for any time longer than one month. This step is meant to prevent weeds from growing into the lower rolls of the geonet stack. If weeds do grow in the geonet during storage, the broken pieces must be removed by hand on the job when the geonet is unrolled and deployed.

7.4.2.5 Acceptance and Conformance Testing

The acceptance and conformance testing of geonets is similar to that described for HDPE geomembranes. (Refer to Section 4.3.4 for a complete description.) For geonets, the usual conformance tests are the following:

- density (per ASTM D1505 or D792),
- mass per unit area (per ASTM D5261), and
- thickness (per ASTM D5199).

Figure 7-10. Geonets Being Temporarily Stored at the Job Site.

Additional conformance tests, such as compression (per ASTM D1621) and transmissivity (per ASTM D4716), may also be stipulated. The thermally bonded geotextiles cannot be stripped off the geonet and retain the original properties of either the geotextiles or the geonet. The bonding process changes the properties of each of the components. If the geotextiles and geonet must be approved individually, unbonded samples of each must be sent to the job site along with the geocomposite product for the required testing.

7.4.2.6 Placement

The placement of geonets in the field is similar to that described for geotextiles. (Refer to Section 7.2.2.6 for a complete description.)

7.4.3 Joining of Geonets

Geonets are generally joined together by providing a stipulated overlap distance and using plastic fasteners or polymer braid to tie adjacent ribs together at minimum intervals (Figure 7-11).

Recommended items for a specification or CQA document on the joining of geonets include the following:

1. Adjacent roll *edges* of geonets should be overlapped a minimum distance. This is typically 75 to 100 mm (3–4 in.).
2. The roll *ends* of geonets should be overlapped 150 to 200 mm (6–8 in.) because flow is usually in the machine direction.
3. All overlaps should be joined by tying with plastic fasteners or polymeric braid. Metallic ties or fasteners are not allowed.

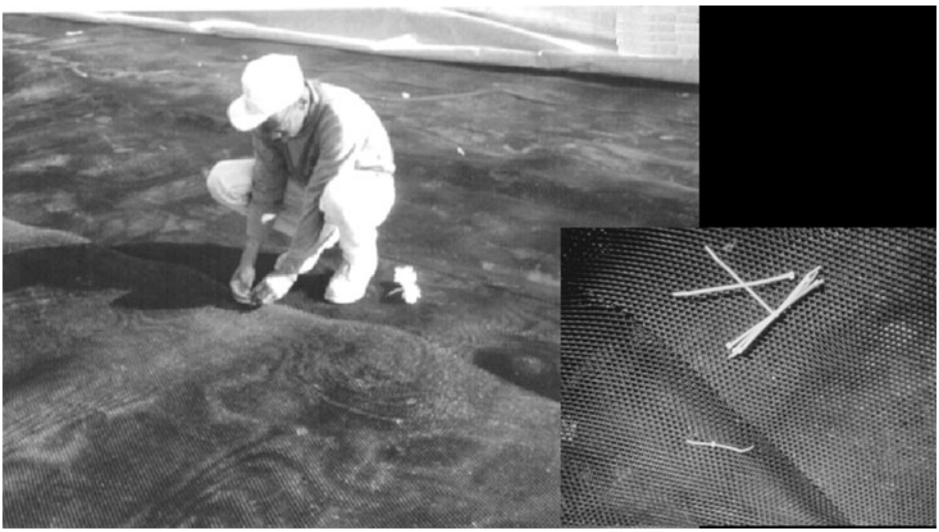


Figure 7-11. Photographs of Geonet Joining by Using Plastic Fasteners.

4. The tying devices should be white or yellow, as contrasted to the black geonet, for ease of visual inspection.
5. The tying interval should be specified. Typical tie intervals are generally every 1.5 m (5.0 ft) along the edges and every 0.15 m (6.0 in.) along the ends and in anchor trenches.
6. If possible, horizontal seams should be avoided on sideslopes. This requirement suggests that the length of the geonet should be at least as long as the sideslope, and that a minimum should be run out at the bottom of the facility. If horizontal seams are necessary for long slopes, they should be close to the bottom of the slope and should be staggered from one roll to the adjacent roll.
7. The machine direction for *biplanar* geonets is along the diagonal of the diamond shaped pattern. This direction should be aligned directly along the slope, although such alignment is generally not critical from a flow, or transmissivity, perspective. Conversely, *triplanar* geonets have their major flow, or transmissivity, direction parallel to the main central ribs. This direction must be carefully aligned along the maximum slope direction in the field.
8. In difficult areas, such as corners of sideslopes, double layers of geonets are sometimes used. This requirement should be stipulated in the plans and specifications.
9. If double geonets are used, they should be layered on top of one another such that interlocking does not occur.
10. If double geonets are used, roll edges and ends should be staggered so that the joints do not lie above one another.
11. Holes or tears in the geonet should be repaired by placing a geonet patch extending a minimum of 300 mm (12 in.) beyond the edges of the hole or tear. The patch should be tied to the underlying geonet at 150 mm (6.0 in.) spacings.
12. Holes or tears along more than 50% of the width of the geonet on sideslopes should require the entire length of geonet to be removed and replaced.

7.4.4 Geonet–Geotextile Composites

Geonets are always covered with either a geomembrane or a geotextile (i.e., they are never covered directly by soil because the soil particles would fill the apertures of the geonet, rendering it useless). Many geonets have a geotextile bonded to one or both surfaces. The bonding is done by heating the geonet (using a hot wedge, a hot knife, or a flame) and laminating the geotextile into it in such a way that the melted geonet surface engages the geotextile fibers. These products are then referred to as geonet composites, geocomposites, or drainage geocomposites. In this document, however, geocomposites can also refer to many different types of drainage core structures. Clearly, covered geonets are included in this group. However, geocomposites also consist of fluted, nubbed, and cuspated cores, covered with geotextiles or geomembranes or both, and will be described separately in Section 7.5. Still further, some European manufacturers refer to the entire group of geosynthetic drainage materials as "geospacers."

Regarding a specification or CQA document for geonet–geotextile or geotextile–geonet–geotextile drainage composites, some recommendations are offered:

1. The geotextiles covering the geonet surface should be bonded in such a way that neither component is compromised to the point where proper functioning is impeded. Thus, adequate, but not excessive, bonding of the geotextiles to the geonet is necessary. Specified ply adhesion strength (per ASTM 7005 and GRI-GC7) is generally 175 N/m (1.0 lb/in.) or slightly lower if full bonding can be assured (Figure 7-12).
2. Bonding is generally done by heating the surface of the geonet and laminating the geotextile into this slightly melted surface, achieving a mechanical interlock. As such, the geotextile cannot be compromised to the point where burn-through occurs. Also, the geonet core cannot be melted to the point where its thickness is decreased. The transmissivity under load test (ASTM D4716) should be performed on the intended geocomposite product.
3. If bonding is by adhesives, the type of adhesive must be identified, including its water solubility and organic content. Excessive adhesive cannot be used because it could fill up some of the geonet's void space. The transmissivity under load test (ASTM D4716) should be performed on the proposed geocomposite product. The geotextile's permittivity should be evaluated using ASTM D4491.
4. If the shear strength of the geotextile to the geonet is of concern, an adapted form of an interface shear test (e.g., ASTM D5321) can be performed with the

Figure 7-12. Ply Adhesion Test of a Geotextile Thermally Bonded to a Geonet Core.

geotextile firmly attached to a wooden substrate or other satisfactory arrangement. More generally, however, a ply adhesion test is used (per GRI-GC8; Figure 7-12 shows such a test in progress).

5. For factory-fabricated geocomposites with geotextiles placed on both sides of a geonet, the geonet must be free from all dirt, dust, and accumulated debris before lamination occurs.
6. For field-placed geocomposites, the geonet should be free of all soil, dust, and accumulated debris before subsequent covering. In extreme cases, this procedure may require washing the geonet to accumulate the particulate material at the low end (sump) area, where it is subsequently removed by hand.
7. The machine direction for *biplanar* geonet composites is along the diagonal of the diamond-shaped pattern. This direction should be aligned parallel to the slope gradient, although such alignment is generally not critical from a flow, or transmissivity, perspective. Conversely, *triplanar* geonet composites have their major flow, or transmissivity, direction parallel to the main central ribs. This direction must be carefully aligned along the maximum slope gradient in the field.
8. The overlapping of the end of one geocomposite roll to the beginning of the next geocomposite roll must be done by a shingled overlap of minimum specified distance. This should be at least 150 to 200 mm (6–8 in.). The geotextiles in the overlapped region must have been stripped back off the geonet cores and either cut away or overlapped above the joined ends. Thus, liquid flow from the upgradient geonet core can continue into the downgradient geonet core without having to cross over one or two intermediate geotextiles. Figure 7-13 gives recommended joining details for both factory and field-cut ends.
9. When placing geosynthetic clay liners (GCLs) above geocomposites, cleanliness is particularly important in assuring that fugitive bentonite clay particles do not find their way into the geonet.
10. Placement of a covering geomembrane should not shift the geotextile or geocomposite out of position or damage the underlying geonet.
11. An overlying geomembrane or geotextile should not be deployed such that excess tensile stress is mobilized in the geocomposite.

7.5 Other Types of Drainage Geocomposites

Geocomposite drainage systems consist of a polymer drainage core protected by a geotextile acting as both a filter and a separator to the adjacent material. Thus a geonet, with a geotextile attached to one surface or to both surfaces as described in Section 7.4.4, is indeed a drainage geocomposite. There are, however, many other types of drainage cores, which are the subject of this section. For the drainage geocomposites discussed in this section, the geotextile filter is always attached to the drainage core and the core can take a wide variety of nongeonet shapes and configurations. In some cases, the geotextile is only on one side of the core (the side oriented toward the inflowing liquid); in other cases, it is wrapped completely around the drainage core.

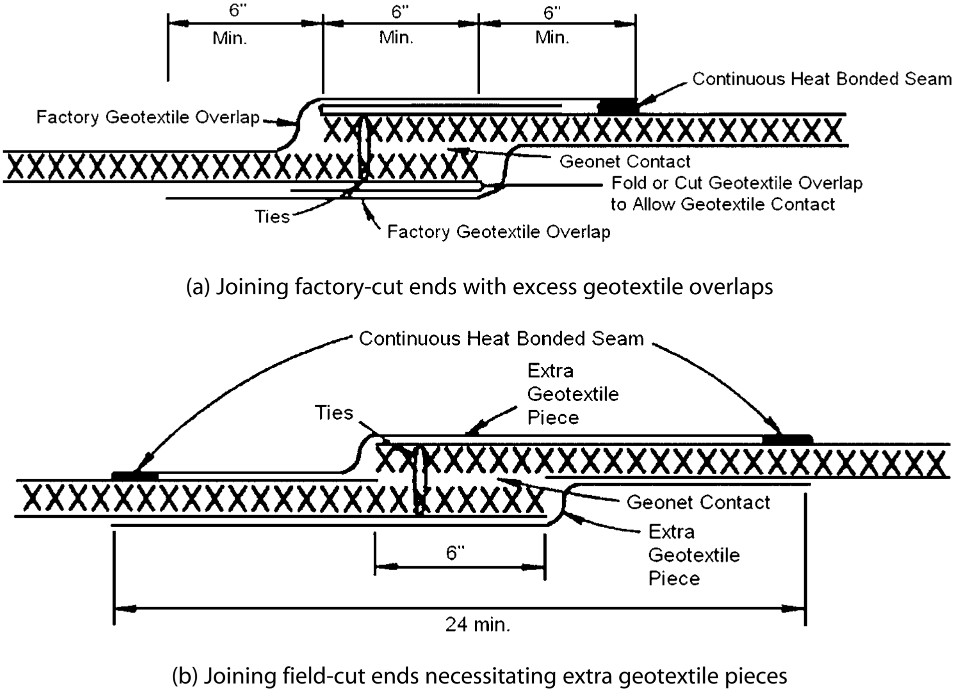


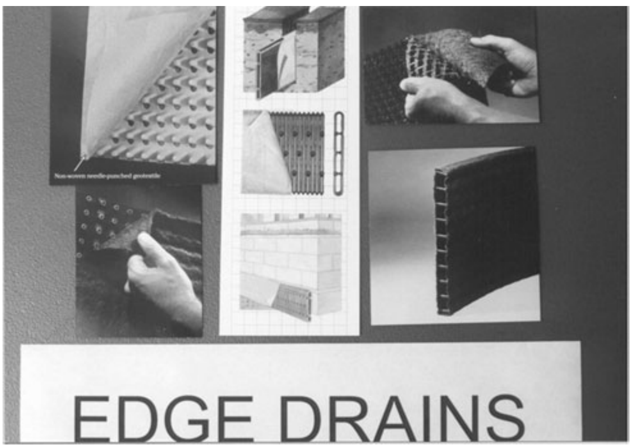
Figure 7-13. Joining of Drainage Geocomposites Such That No Geotextile Exists within the Drainage Core Overlap Area.

Source: Adapted from U.S. Army Corps of Engineers documents. Courtesy of David Jaros.

There are three different types of drainage geocomposites referred to in this section: sheet drains, wick drains (also called prefabricated vertical drains, or PVDs), and edge drains. Typical variations are shown in Figure 7-14. For drainage systems associated with waste containment facilities, sheet drains (Figure 7-14(a)) are sometimes used as surface water drains in cover systems of closed landfills and waste piles (refer back to Figure 7-1 for these locations). Infiltration water that moves within the cover soil enters the sheet drain and flows gravitationally to the edge of the site (or cell), where it is generally collected by a perforated pipe or edge drain. Pipes will be discussed separately in Chapter 9. The other possible use for sheet drains is for primary leachate collection systems in landfills. If the required flow rate in a landfill is too great for a geonet, the greater drainage capacity of the type of cores in this section is sometimes required. Of course, when used in this application, the drainage geocomposite must resist the compressive and shear stresses imposed by the waste and it must be chemically resistant to the leachate, but these are all design considerations. Finally, such composite sheet drains can be used to deal with the introduction of liquids in wet landfilling, and the following type of drains as well. Wick (or PVD) drains (Figure 7-14(b)) in waste containment have been used as vertical drains within a solid-waste landfill to promote

(a) Geocomposite sheet drains

(b) Geocomposite wick (or PVD) drains



(c) Geocomposite edge drains

Figure 7-14. Various Types of Drainage Geocomposites.

leachate communication between individual lifts. The edge drains (Figure 7-14(c)) have potential applicability around the perimeter of a closed landfill facility to accumulate the surface water coming from a cap or closure system. A variety of perimeter drains could use such geocomposite edge drains.

Of the different types of drainage geocomposites shown in Figure 7-14, only sheet drains will be described because they have the greatest applicability in waste containment systems.

7.5.1 Manufacturing of Drainage Composites

The manufacture of the drainage core of a geocomposite sheet drain is often accomplished by taking the desired type of polymer sheet and then vacuum-forming dimples, protrusions, or cuspations, which give rise to the thickness of the drainage core. The polymer sheets of drainage geocomposites have been made from a wide variety of polymers. Commercial products that are currently available consist of the following:

- polystyrene,
- nylon,
- polyvinyl chloride,
- polypropylene,
- polyethylene, or
- coextruded polyethylene–polystyrene–polyethylene.

With coextrusion, there exists a variety of possibilities in addition to those listed above. Recognize, however, that stiff entangled webs, coarse fiber mattings, various filament mattings, and other variations of geocomposite cores are also possible.

To arrive at the proper type and thickness of polymer sheet, a geocomposite core usually goes through a vacuum-forming step. In this method, a vacuum draws portions of the polymer sheet into cusps at prescribed locations. Depending on the particular product, the protrusions are at 12 to 25 mm (0.5–1.0 in.) centers and are of a controlled depth and shape. In many of the systems, the protrusions are tapered for ease in manufacturing during release of the vacuum and for a convenient male-to-female coupling of the edges or ends of the product in the field. Alternatively, a different type of processing can produce an entangled web of stiff nylon or polypropylene filaments. This process results in a three-dimensional web of various thickness, which is the actual drainage core. This latter concept can be extended to coarse fiber nettings and various filament mats.

Drainage geocomposites are usually manufactured in continuous rolls of widths varying from 2.0 to 5.0 m (6.6–16.4 ft). Some of the stiff cuspated or nubbed products are produced in panel form.

The geotextile, which acts as both a filter to allow liquid into the drainage core and as a separator to keep soil out of the core by spanning from cusp to cusp, is attached to the core as a secondary operation. Quite often an adhesive is placed on the tops of the cusps to adhere the geotextile to the core. Alternatively, heat bonding might be used. A variety of geotextiles can be used, and the site-specific design will dictate the actual selection. As far as the MQA/CQA of the geotextile, it is the same as was described in Section 7.2.

Several items should be included in a specification or MQA document for drainage geocomposite cores:

1. There should be verification and certification that the actual geocomposite core properties meet the manufacturer's specification for that particular type and style.
2. QC certificates should include at a minimum polymer composition, thickness of sheet (per ASTM D5199), height of raised cusps, spacing of cusps, compressive strength behavior (both strength and deformation values at core failure, per ASTM D1621), and transmissivity using site-specific conditions (per ASTM D4716).
3. For drainage systems consisting of coarse fibers, entangled webs or filament mattings, the thickness under load per ASTM D5199 and transmissivity under load per ASTM D4716 are the main tests to be conducted.
4. Values for each property should meet or exceed the manufacturer's listed values or the project specification values, whichever are higher.
5. A statement indicating if, and to what extent, regrind polymer was added during manufacturing should be included. No amount of reclaimed or postconsumer polymer should be allowed.
6. The frequency of performing each of the preceding tests should be covered in the MQA plan or manufacturer's quality manual, which should be implemented and followed.

Additionally, several items should be included in a specification or MQA document for the geotextile–drainage core geocomposite.

1. The type of geotextiles should be identified and properly evaluated. (See Section 7.2 for these details).
2. For wick (or PVD) drains and edge drains (Figs. 7-14(b) and (c)), the geotextile completely surrounds the drainage core and, generally, no fixity is required. For sheet drains (Figure 7-14(a)), this is not the case.
3. Excessive geotextile approximately 150 mm (6.0 in.) should extend beyond the edges of the geocomposite core. The purpose of the extended geotextile width is to ensure complete core coverage for adjacent field overlaps. This requirement applies to both geotextiles if the composite is double-sided with geotextiles.
4. The geotextile covering of a drainage core should be bonded in such a way that neither component is compromised to the point where proper functioning is impeded. Thus, adequate but not excessive bonding of the geotextiles to the drainage core is necessary.
5. If bonding is by heating, the geotextile strength cannot be compromised to the point where failure or holes could occur. Conversely, the drainage core cannot be compromised insofar as its thickness or strength is concerned. The

transmissivity under load test (ASTM D4716) should be performed on the proposed geocomposite product. The GRI-GC8 guide might be considered in this regard.

6. If bonding is by adhesives, the type of adhesive must be identified, including its water solubility and organic content. Excessive adhesive cannot be used because it could fill up some of the drainage core's void space. The transmissivity under load test (ASTM D4716) should be performed on the intended geocomposite product. The geotextile's permittivity could be evaluated using ASTM D4491.
7. If the shear strength of the geotextiles to the core is of concern, an adapted form of an interface shear test (e.g., ASTM D5321) can be performed with a wooden substrate or other satisfactory arrangement. Alternatively, a ply adhesion test may be adequate (ASTM D6636 or GRI-GC7), which can be suitably modified for geotextile-to-core adhesion.
8. For factory-fabricated geocomposites with geotextiles placed on both sides of the drainage core, the core must be free from all dirt, dust, and accumulated debris before covering.
9. Because a geotextile is the outer surface of a drainage geocomposite, the roll must be protected against UV degradation. A plastic (usually polyethylene) wrap protection or bag is necessary in this regard.

7.5.2 Handling of Drainage Geocomposites

A number of activities occur between the manufacture of drainage geocomposites and their final positioning at the waste facility. These activities involve packaging, storage at the manufacturing facility, shipping, storage at the site, acceptance and conformance testing, and final placement at the facility. Each of these topics will be described, although most will be by reference to the appropriate geotextile or geonet composite section.

7.5.2.1 Packaging

Usually a manufacturer will not attach the geotextile to the core until an order is received and shipment is imminent. Thus, warehousing is not a major issue. The cores are either rolled onto themselves or are laid flat if they are in panel form. When an order is received, the geotextile is bonded to the core, the rolls are banded together with polymer straps, and, if panels, they are banded in a similar manner.

7.5.2.2 Storage at the Manufacturing Facility

Storage of the drainage cores at the manufacturing facility is usually not a major issue. The cores are generally stored indoors and are thus protected from atmospheric conditions.

7.5.2.3 Shipping

Shipment of drainage geocomposites (with the geotextile attached) is quite simple because of the light weight of these geosynthetics compared to other types. The text in Section 7.2.2.3 should be used, however, because accidental damage can always occur.

7.5.2.4 Storage at the Site

The storage of drainage geocomposites at the project site is similar to that described for geotextiles (recall Section 7.2.2.4).

7.5.2.5 Acceptance and Conformance Testing

The acceptance and conformance testing of the geotextile portion of a drainage geocomposite is the same as that described in Section 7.2.2.5. The acceptance and conformance testing of the core portion of a drainage geocomposite is project-specific with the exception of the conformance tests themselves, which are different. The recommended conformance tests for geocomposite drainage cores are the following;

- thickness of sheet or of the geocomposite (per ASTM D5199),
- thickness of raised cusps (per ASTM D1621), and
- spacing of raised cusps (per ASTM D1621).

Optional conformance tests such as compression (per ASTM D1621) and transmissivity (per ASTM D4716), may also be stipulated. The frequency of conformance tests of the drainage core must be stipulated. In general, one test per 5,000 m^2 (50,000 ft^2) should be the minimum test frequency.

7.5.2.6 Placement

The placement of drainage geocomposites in the field is similar to that described for geotextiles. (Refer to Section 7.2.2.6 for details.)

7.5.3 Joining of Drainage Geocomposites

Drainage geocomposites are usually joined together by folding back the geotextile from the lower core and overlapping or inserting it into the bottom void space of the upper core (recall Figure 7-13). Shingling of the upgradient core over the downgradient core is necessary so that the overlap contains no intermediate geotextiles. The geotextile must be refolded or added over the connection area, ensuring complete coverage of the core surface.

Recommended items for a specification or CQA document on the joining of drainage geocomposites include the following:

1. Adjacent edges of drainage cores should be overlapped for at least two rows of cusps.
2. The ends of drainage cores (in the direction of flow) should be overlapped for at least four rows of cusps.
3. The geotextiles covering the joined cores must provide a complete seal against backfill soil entering into the core.

4. Horizontal seams should not be allowed on sideslopes. This process requires that the drainage geocomposite be provided in rolls that are at least as long as the sideslope.
5. Holes or tears in drainage cores are repaired by placing a patch of the same type of material over the damaged area. The patch should extend at least four cusps beyond the edges of the hole or tear.
6. Holes or tears of more than 50% of the width of the drainage core on sideslopes should require the entire length of the drainage core to be removed and replaced.
7. Holes or tears in the geotextile covering the drainage core should be repaired as described in Section 7.2.3.3.

7.5.4 Covering

Drainage geocomposites, with an attached geotextile, are covered with soil, waste, or in some cases a geomembrane. Regarding a specification or CQA document, some comments should be included:

1. The core of the drainage geocomposite should be free of soil, dust, and accumulated debris before backfilling or covering with a geomembrane. In extreme cases, this process may require washing the core to accumulate the particulate material at the low end (sump) area for removal.
2. Placement of the backfilling soil, waste, or geomembrane should not shift the position of the drainage geocomposite or damage the underlying drainage geocomposite, geotextile, or core.
3. When using soil or waste as backfill on sideslopes, the work progress should begin at the toe of the slope and work upward.

7.6 References

AASHTO M288. "Standard specification for geotextile specification for highway applications."

ASTM D792. "Test method for specific gravity (relative density) and density of plastics by displacement."

ASTM D1238. "Test method for flow rates of thermoplastics by extrusion plastometer."

ASTM D1248. "Specification for polyethylene plastics molding and extrusion materials."

ASTM D1505. "Test method for density of plastics by the density-gradient technique."

ASTM D1603. "Test method for carbon black in olefin plastics."

ASTM D1621. "Test method for compressive properties of rigid cellular plastics."

ASTM D4218. "Test method for carbon black content in polyethylene compounds by the muffle-furnace technique."

ASTM D4354. "Guide for sampling of geosynthetics for testing."

ASTM D4355. "Test method for deterioration of geotextiles from exposure to ultraviolet light and water (xenon-arc type apparatus)."

ASTM D4491. "Test methods for water permeability of geotextiles by permittivity."

ASTM D4533. "Test method for index trapezoidal tearing strength of geotextiles."

ASTM D4632. "Test method for grab breaking load and elongation of geotextiles."

ASTM D4716. "Test method for determining the (in-plane) flow rate per unit width and hydraulic transmissivity of a geosynthetic using a constant head."

ASTM D4751. "Test method for determining apparent opening size of a geotextile."

ASTM D4759. "Guide for determining the specification conformance of geosynthetics."

ASTM D4833. "Test method for index puncture resistance of geotextiles, geomembranes and related products."

ASTM D4873. "Guide for identification, storage and handling of geosynthetics."

ASTM D4884. "Test method for seam strength of sewn geotextiles."

ASTM D5199. "Test method for measuring nominal thickness of geotextiles and geomembranes."

ASTM D5261. "Test method for measuring mass per unit area of geotextiles."

ASTM D5321. "Test methods for determining the coefficient of soil and geosynthetic or geosynthetic and geosynthetic friction by the direct shear method."

ASTM D5494. "Test methods for the determination of pyramidal puncture resistance of unprotected and protected geomembranes."

ASTM D6241. "Test method for the static puncture strength of geotextiles and geotextile related products using a 50-mm probe."

ASTM D6636. "Test method for determination of ply adhesion strength of reinforced geomembranes."

ASTM D7005. "Test method for determining the bond strength (ply adhesion) of geocomposites."

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GRI-GC7. "Test Method for determination of adhesion and bond strength of geocomposites."

GRI-GC8. "Standard guide for determination of the allowable flow rate of a drainage geocomposite."

GRI-GT12. "Specification for test methods and properties for nonwoven geotextiles used as protection (or cushioning) materials."

Industrial Fabrics Association International (IFAI). (1990). "A design primer: Geotextiles and related materials," Industrial Fabrics Association International, St. Paul, Minn.

Koerner, R. M. (2005). *Designing with geosynthetics*, fifth ed., Prentice Hall Publishing Co., Englewood Cliffs, N.J.

Koerner, R. M., Wilson-Fahmy, R. F., and Narejo, D. (1996). "Puncture protection of geomembranes, Part III: Examples," *Geosynthetics Int.*, 3(5), 655–676.

Shercliff, D. A. (1996). "Optimization and testing of liner protection geotextiles used in landfills," *Proc. EuroGeo1.* M. B. DeGroot, G. Den Hoedt, and R. J. Termatt, Editors, A. A. Balkema, Rotterdam, Netherlands, 823–828.

CHAPTER 8

Vertical Cutoff Walls

8.1 Introduction

Vertical cutoff walls are commonly used to restrict horizontal movement of liquids and gases around waste-disposal facilities or site remediation projects. Examples of the uses of vertical cutoff walls include the following:

1. Controlling groundwater seepage into an excavated disposal cell to maintain stable sideslopes or to limit the amount of water that must be pumped from the excavation during construction (Figure 8-1(a)).
2. Controlling horizontal groundwater flow into buried wastes at older waste disposal sites that do not contain a liner (Figure 8-1(b)).
3. Providing a seal into an aquitard (low-permeability stratum), thus encapsulating the waste to limit inward movement of clean groundwater in areas where groundwater is being pumped out and treated (Figure 8-1(c)).
4. Providing a long-term barrier to impede contaminant transport (Figure 8-1(d)).

8.2 Types of Vertical Cutoff Walls

The principal types of vertical cutoff walls are sheet pile walls, geomembrane walls, and slurry trench cutoff walls. Other techniques, such as grouting and deep soil mixing, are also possible but have been used less commonly for waste containment applications.

8.2.1 Sheet Pile Walls

Steel sheet piling has been frequently used in conventional civil engineering construction for reducing groundwater flow in the subsurface. Sheet pile walls have a long history of use for dewatering applications, particularly where the sheet pile wall is also used as a structural wall.

Sheet pile walls also have been used on several occasions to cut off horizontal seepage through permeable strata that underlie dams (Sherard et al. 1963). However, for environmental applications, steel sheet piling has been used rarely until

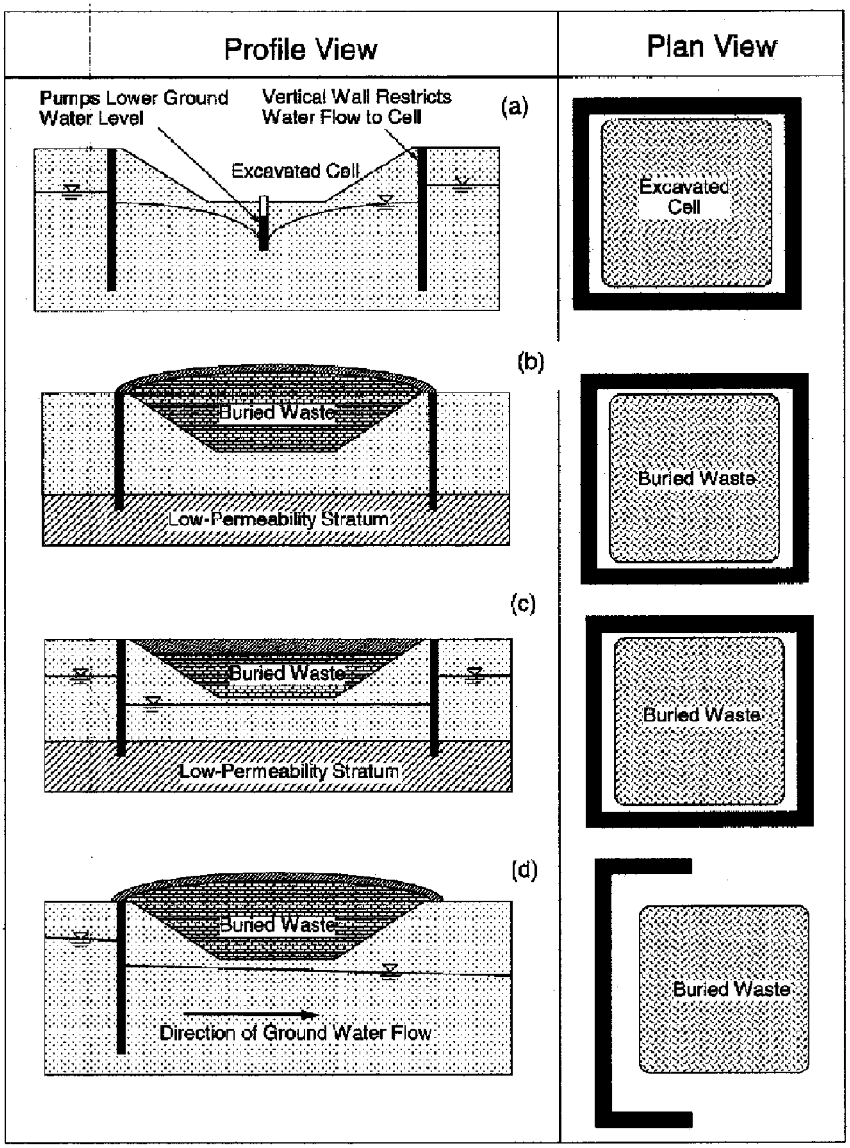


Figure 8-1. Examples of Vertical Barriers.

Note: (a) Dewatering of cell; (b) containment for landfill; (c) limiting flow of fresh water into pump-and-treat zone for groundwater remediation; and (d) hydraulic control.

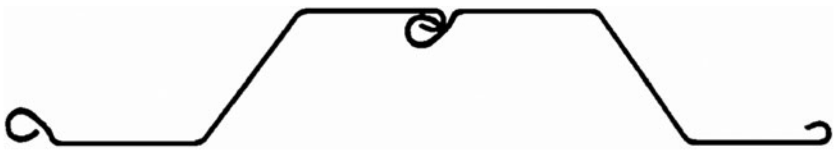
recently because traditional steel sheet piling is subjected to leakage through the interlocks. Also, the issue of corrosion in a chemically aggressive environment has been a concern with the use of steel sheet piling for long-term environmental containment applications (Rumer and Mitchell 1995). Steel sheet piles measure approximately 0.5 m (18 in.) wide, and interlocks join individual sheets together.

Lengths are essentially unlimited, but sheet piles are rarely longer than about 10 to 15 m (30–45 ft).

Plastic sheet piles are a relatively recent development and are used on a limited basis for vertical cutoff walls. Plastic sheet piles are different from geomembrane panels, which are discussed later. Plastic sheet piles tend to be relatively thick-walled (wall thickness >3 mm or 1/8 in.) and rigid; geomembrane panels tend to have less thickness (<2.5 mm or 0.1 in.), greater width, and lower rigidity.

Sheet pile walls are installed by driving or vibrating interlocking steel sheet piles into the ground. With plastic sheet piles, special installation devices may be needed (e.g., a steel driving plate to which the plastic sheet piles are attached). To promote a seal, a cord of material that expands when hydrated and attains low permeability may be inserted in the interlock. Other schemes have been devised and will continue to be developed for attaining a watertight seal in the interlock.

An important recent development is the Waterloo Barrier steel sheet piling (Smyth and Cherry 1997), which consists of 7.5-mm-thick cold-formed steel sheet piling with a modified sealable interlock that can be driven to depths up to 20 mm. As shown in Figure 8-2, a sealable cavity is incorporated into the interlock between adjacent sheet piles when the sheet is manufactured. After driving the sheet piles, the entire length of each cavity is cleaned using pressurized water or air, and the low-permeability sealant is introduced from bottom to top in the cavity. The types of sealants available include clay-based grouts, epoxy polymers, urethane polymers, and miscellaneous sealants, such as vinyl esters, polysulfides, swelling gaskets, and bituminous grouts (Smyth and Cherry 1997). In the case of using organic



(a) Standard sheet pile plan view

(b) Sealable cavity detail

Figure 8-2. The Waterloo Barrier Steel Sheet Piling System.

Source: Smyth et al. 1997, with permission from National Academies Press.

polymer as a sealant, an in situ test result showed the bulk hydraulic conductivity ranged from 10^{-8} to 10^{-10} cm/s (Smyth et al. 1997).

Sheet piles can be installed rapidly compared to slurry trench methods, and standard construction equipment can be used to drive the sheet piles. No soil is excavated, which makes sheet piles particularly attractive when constructing a cutoff wall in contaminated or potentially contaminated soil by reducing health and safety concerns (Rumer and Mitchell 1995).

Sheet pile walls have historically suffered from problems with leakage through interlocks, although much of the older experience may not be applicable to modern sheet piles with expanding material located in the interlock. Leakage through sheet pile interlocks depends primarily on the average width of openings in the interlocking connections, the percentage of the interlocks that leak, and the quality and integrity of any sealant placed in the interlock. Concerning problems with pile corrosion, the lifetime of sheet piles depends on groundwater characteristics such as pH. If the pH of the groundwater is in the range of 5.8 to 7.8, sheet piles can last beyond the service life of most applications. However, a pH as low as 2.3 can shorten the lifetime to less than seven years (U.S. Army Corps of Engineers 1996). Because of these problems, sheet pile cutoffs have not been used for waste containment facilities as extensively as some other types of vertical cutoff walls.

8.2.2 Geomembrane Walls

Geomembrane walls represent a relatively new type of vertical barrier that began to be used in 1980. The geomembrane wall consists of a series of geomembrane panels joined with special interlocks or installed as a single unit. If the geomembrane panels contain interlocks, a water-expanding cord is used to seal the interlock. Geomembranes used for vertical barriers are typically made from high-density polyethylene (HDPE) because of its strength, excellent chemical resistance, and durability, but other types of geomembranes can be used according to site- and project-specific requirements.

Geomembrane wall technology has its roots in Europe, where slurry trench cutoff walls that are backfilled with cement–bentonite have been commonly used for several decades. One of the problems with cement–bentonite backfill, as we discuss later, is that it is somewhat difficult to make the hydraulic conductivity of the cement–bentonite backfill less than or equal to 1×10^{-7} cm/s, which is often required of regulatory agencies in the United States. To overcome this potential problem and to improve the overall containment provided by the vertical cutoff wall, a geomembrane may be inserted into the cement–bentonite backfill material.

Early installation methods for geomembranes as vertical barrier walls used the slurry-supported excavation along with the inserted geomembrane displacing the slurry in a progressive manner. Since these applications, typical installation methods have been developed and used on a regular basis, as summarized in Table 8-1. Details of each method are presented in Chapter 5 of Rumer and Mitchell (1995). In addition to the five typical installation methods, there are recently developed techniques. Bocchino and Burson (1997) introduced the "one-pass deep trench-

Table 8-1. Installation Methods for Geomembrane Vertical Barrier Walls

Method Number	*Method or Technique*	*Geomembrane Configuration*	*Trench Support*	*Typical Trench Width (mm)*	*Typical Trench Depth (m)*	*Typical Backfill Type*
1	Trenching machine	Continuous	None	300–600	1.5–4.5	Sand or native soil
2	Vibrated insertion plate	Panels	None	100–150	1.5–6.0	Native soil
3	Slurry supported	Panels	Slurry	600–900	No limit	Soil–bentonite or cement–bentonite backfill
4	Segmented trench box	Panels or continuous	None	900–1200	3.0–9.0	Sand or native soil
5	Vibrating beam	Panels	Slurry	150–220	No limit	Cement–bentonite backfill

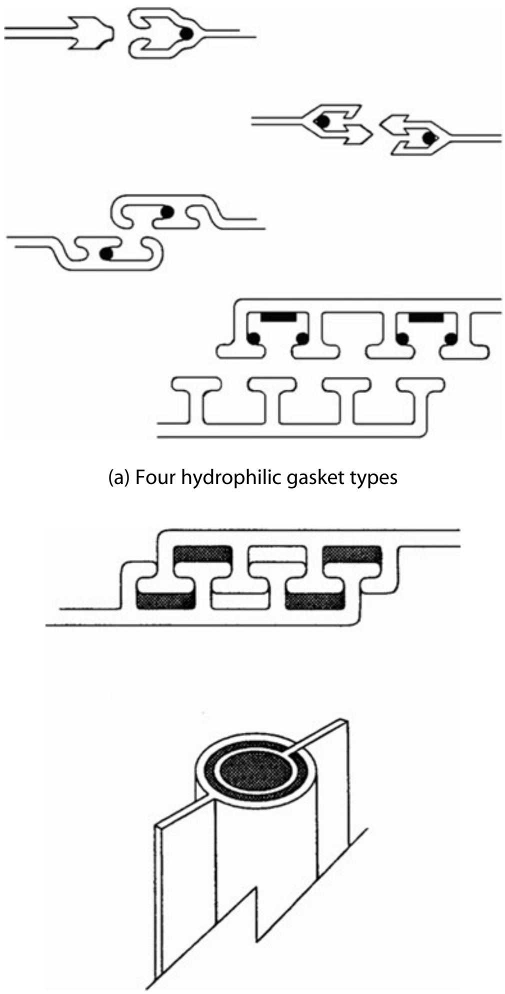
Source: Rumer and Mitchell 1995.

ing system." The trencher digs the trench and immediately inserts a geomembrane panel. Although panel widths exceeding 2.4 m can be installed, narrower panels are more convenient to accommodate changed conditions encountered during excavation. Rawl (1997) described the "polywall" technique, also a one-step operation, in which a 400-mm-wide trench is excavated with the cutters oriented vertically. A continuous HDPE geomembrane barrier is housed in an installation box that is pulled through the ground behind the cutter assembly. The geomembrane is unrolled vertically from fabricated rolls that are equipped with male–female joints on the beginnings and ends of the rolls. The maximum depth of the barrier wall is limited to approximately 10 m.

Several types of interlocks are available to connect geomembrane panels or rolls together. These types include (a) hydrophilic gasket types; (b) grouted chemical or tube types; and (c) continuous seaming by welding (Rumer and Mitchell 1995). Figure 8-3(a) shows four hydrophilic gasket types of interlock configurations. The gaskets are either circular or rectangular in cross section and are mostly made from rubber (chloroprene or neoprene) formulated with a hydrophilic polymer. The gaskets swell in water five to eight times the original volume. Figure 8-3(b) displays grouted interlocks, which can be made using various slurries or grouts that are pumped and flow down channels or tubes (Rumer and Mitchell 1995).

8.2.3 Walls Constructed with Slurry Techniques

Walls constructed by slurry techniques (sometimes called "slurry trench cutoff walls") are described by Xanthakos (1979), D'Appolonia (1980), U.S. EPA (1984),



(a) Four hydrophilic gasket types

(b) Grouted chemical types

Figure 8-3. Various Types of Geomembrane Panel Interlocks.

Source: Rumer and Mitchell 1995.

Ryan (1987), Evans (1993), and Rumer and Mitchell (1995). With this technique, an excavation is made to the desired depth using a backhoe or clamshell bucket. The trench is filled with a clay–water suspension ("mud" or "slurry"), which maintains stability of sidewalls via hydrostatic pressure. As the trench is advanced, the slurry tends to flow into the surrounding soil. Clay particles are filtered out, forming a thin skin of relatively impermeable material along the wall of the trench called a "filter cake." The filter cake has low hydraulic conductivity and allows the pressure from the slurry to maintain stable walls on the trench (Figure 8-4). However, the level of slurry must generally be higher than the surrounding groundwater table to maintain stability. If the water table is at or above the sur-

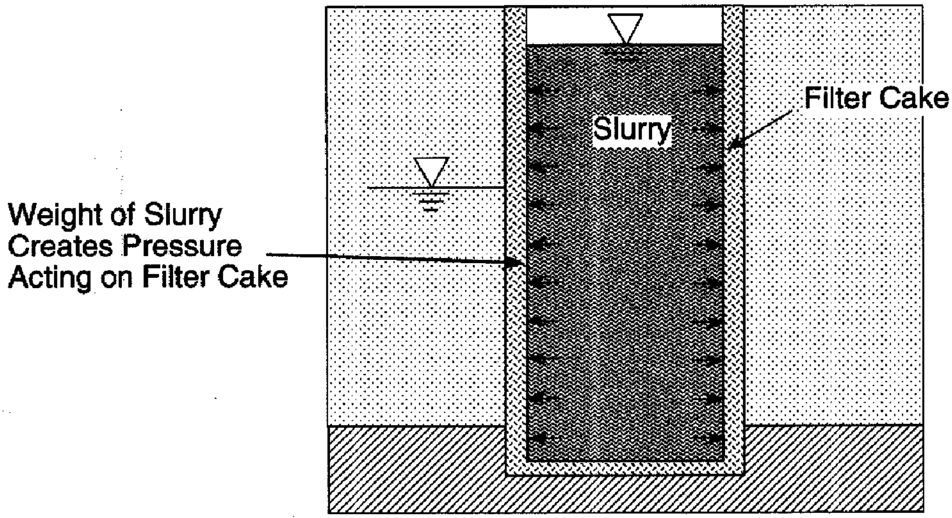


Figure 8-4. Hydrostatic Pressure from the Slurry Maintains the Stable Walls of the Trench.

face, a dike may be constructed to raise the surface elevation along the alignment of the slurry trench cutoff wall (Figure 8-5).

In most cases, sodium bentonite is the clay used in the slurry. A problem with bentonite is that it does not gel properly in highly saline water or in some heavily contaminated groundwaters. In such cases, an alternative clay mineral, such as attapulgite, or other special materials may be used to maintain a viscous slurry.

The slurry trench must either be backfilled or the slurry itself must harden into a stable material; otherwise clay will settle out of suspension, the slurry will cease to support the walls of the trench, and the walls may eventually collapse. If the slurry is allowed to harden in place, the slurry is usually a cement–bentonite (CB) mixture. If the slurry trench is backfilled, the backfill is usually a soil–bentonite (SB) mixture, although plastic concrete may also be used (Evans 1993). Occasionally, the SB mixture may also contain Portland cement, which is intended to strengthen the soil–cement–bentonite backfill.

In the United States, slurry trenches backfilled with SB have been the most commonly used type of vertical cutoff trenches for waste containment applications. In Europe, the CB method of construction has been used more commonly. The reason for the different practices in the United States and Europe stems at least in part from the fact that abundant supplies of high-quality sodium bentonite are readily available in the United States but not in Europe. Also, in most situations, SB backfill will have a somewhat lower hydraulic conductivity than cured CB slurry, and in the United States, regulations have tended to drive the requirements for hydraulic conductivity to lower values than those in Europe.

The construction sequence for a soil–bentonite backfilled trench is shown schematically in Figure 8-6.

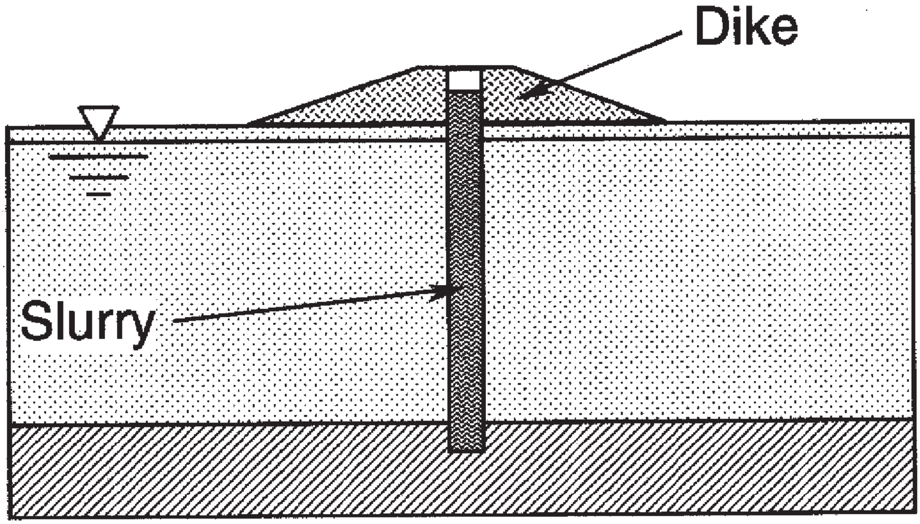


Figure 8-5. Construction of a Dike to Raise the Ground Surface for the Construction of a Slurry Trench.

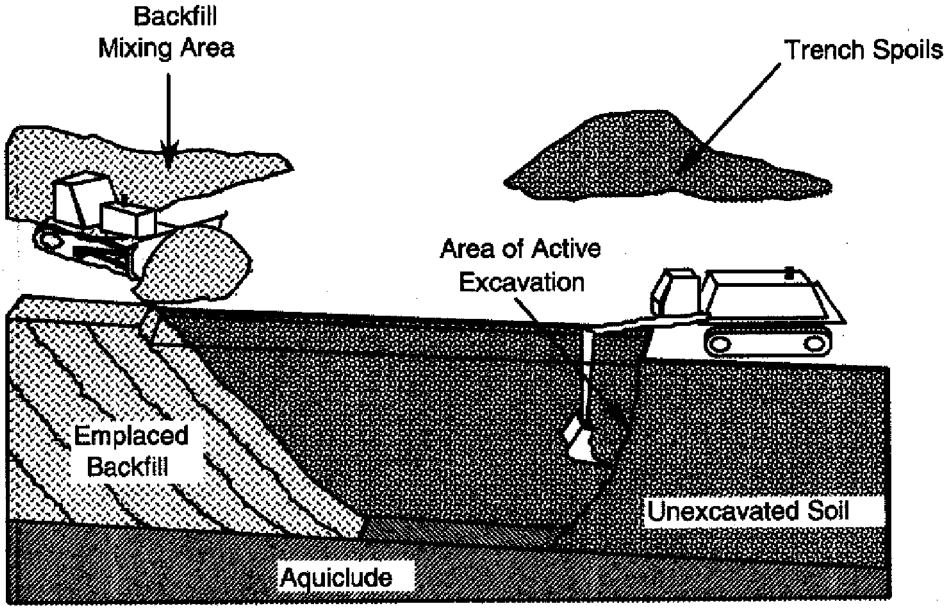


Figure 8-6. Diagram of the Construction Process for a Soil–Bentonite Backfilled Slurry Trench Cutoff Wall.

The main reasons that slurry trench cutoff walls are so commonly used for vertical cutoff walls are the following:

1. The depth of the trench may be checked to confirm penetration to the desired depth, and excavated materials may be examined to confirm penetration into a particular stratum.
2. The backfill can be checked before placement to make sure that its properties are as desired and specified.
3. The wall is relatively thick (compared to a sheet pile wall or a geomembrane wall).
4. There are no joints between panels or construction segments with the most common type of slurry trench cutoff wall construction.

In general, in comparison to sheet pile walls, deep-soil mixed walls, and grouted walls, there is more opportunity with a slurry trench cutoff wall to check the condition of the wall and confirm that the wall has been constructed as designed. In contrast, it is much more difficult to confirm that a sheet pile wall has been installed without damage, that grout has fully penetrated all of the desired pore spaces in the soil, or that deep mixing has taken place as desired.

With respect to design of backfilled slurry trenches, there are five essential design concerns (U.S. Army Corps of Engineers 2004):

1. *Piping.* The backfill should be designed to prevent possible blowout or piping of the backfill into the surrounding foundation material because of the hydraulic gradient through the cutoff wall. The gradational characteristics of the backfill, and possible addition of any cementitious materials (if any), are the key CQC/CQA parameters to ensure that design objectives are realized during construction to minimize the risk of backfill material being washed into the surrounding formation.
2. *Hydraulic conductivity.* The overall hydraulic resistance of the barrier wall is a function of the thickness and hydraulic conductivity of both the backfill material and the filter cake. It is customary in design to be conservative and to ignore the filter cake, thus relying solely on the backfill in the design process. The gradation of the amended soil, the amount of quality of bentonite added, the thoroughness of mixing, the quality of water in or added to the backfill during mixing, and any additional amendments to the backfill can all influence the hydraulic conductivity of the backfill. A testing program in which some or all of these parameters are varied and tested in a laboratory is a normal part of the process of selecting the final mix design.
3. *Shear strength.* Shear strength is not usually a major concern in design of vertical barrier walls, but in some cases it can be. If shear strength is a concern, shear tests are usually incorporated into the laboratory testing program used to establish the mix design.
4. *Compressibility.* Slurry wall backfill materials are almost always compressible and subject to settlement over time as a result of consolidation. Compressibility

is usually not one of the design parameters. Consolidation tests can be included in the laboratory testing program that establishes the mix design if compressibility is a concern.

5. *Chemical compatibility.* Because chemical contaminants commonly associated with hazardous waste sites may increase the permeability of an SB backfill, a compatibility testing program should be undertaken before construction. If the trench is to be excavated through contaminated material, the designer should perform compatibility testing using two potential backfill materials: soils to be excavated from the trench and an uncontaminated borrow source. Compatibility testing can take from two to six months to complete and, therefore, should be completed during the design phase of the project. A recommended compatibility testing program (U.S. Army Corps of Engineers 2004) consists of the following:

   A. Free swell (ASTM D5890) tests of several bentonites using contaminated site groundwater and site mixing water that will be used during construction to determine acceptable bentonites for use on the project.
   B. Mix design optimization tests to determine the most economical mix of soils, dry bentonite, and bentonite slurry to produce the required design characteristics. These tests may consist of short (48–72 h) hydraulic conductivity tests, varying the amount of dry bentonite added (0%, 2%, and 4%) and if necessary the amount of additional fines added (0%, 10%, 20%), using site mixing water as the permeating liquid.
   C. Long-term flexible wall hydraulic conductivity testing of at least three SB backfill samples following procedures in ASTM D7100: the optimum mix design with site mixing water only as the permeant (control); the optimum mix design with contaminated site groundwater as a permeant liquid (after one pore volume of site mixing water permeant to ensure a good test setup); and a bentonite content 2% greater than the optimum determined in Step B with contaminated site groundwater as the permeant liquid (after one pore volume of site mixing water permeant). It is recommended that a minimum of three pore volumes of groundwater permeant pass through the SB backfill samples. This testing typically takes at least two months. To approximate field conditions in the laboratory, it is important to obtain contaminated groundwater and mixing water from the site. The site mixing water used during compatibility testing shall be the water used to make the bentonite slurry during construction.

8.3 Construction of Slurry Trench Cutoff Walls

The major construction activities involved in building a slurry cutoff wall are preconstruction planning and mobilization, preparation of the site, slurry mixing and hydration, excavation of soil, backfill preparation, placement of backfill, cleanup of the site, and demobilization. These activities are described briefly in the paragraphs that follow.

8.3.1 Mobilization

The first major construction activity is to make an assessment of the site and to mobilize for construction. The contractor locates the slurry trench cutoff wall in the field with appropriate surveys. The contractor determines the equipment, amounts of materials, and facilities that may be required. Plans are made for mobilizing personnel and moving equipment to the site.

A preconstruction meeting among the designer, contractor, and CQA engineer is recommended. In this meeting, materials, construction procedures, procedures for MQA of the bentonite and CQA of all aspects of the project, and corrective actions are discussed (see Chapter 2).

8.3.2 Site Preparation

Construction begins with preparation of the site. Obstacles are removed, necessary relocations of utilities are made, and the surface is prepared. One of the requirements of slurry trench construction is that the level of slurry in the trench be greater than the level of groundwater. If the groundwater table is high, it may be necessary to construct a dike to ensure that the level of slurry in the trench is above the groundwater level (Figure 8-4). There may be grade restrictions in the construction specifications that will require some regrading of the surface or construction of dikes in low-lying areas. The site preparation work will typically also include preparation of working surfaces for mixing materials. Special techniques may be required for excavation around utility lines.

8.3.3 Slurry Preparation and Properties

Before excavation begins, as well as during excavation, the slurry must be prepared. The slurry usually consists of a mixture of bentonite with water, but sometimes other clays such as attapulgite are used. If the clay is bentonite, the specifications should stipulate the criteria to be met (e.g., filtrate loss) and the testing technique by which the parameter is to be determined. The criteria can vary considerably from project to project.

The clay may be mixed with water in either a batch or flash mixing operation. In the batch system, specified quantities of water and bentonite are added in a tank and mixed at high speeds with a pump, paddle mixer, or other device that provides adequate high-speed colloidal shear mixing. Water and clay are mixed until hydration is complete and the desired properties of the slurry have been achieved. Bentonite is difficult to mix thoroughly, but with high-speed, high-shear mixers used properly, complete mixing is usually achieved in a few minutes. The size of batch mixers varies, but a typical batch mixer will produce several cubic meters of mixed slurry at a time.

Flash mixing is achieved with a Venturi mixer. With this system, bentonite is fed at a predetermined rate into a metered water stream that is forced through a nozzle at a constant rate. The slurry is subjected to high shear mixing for only a fraction of a second. The problem with this technique is that complete hydration does not take place in the short period of mixing. After the clay is mixed with water, the resulting slurry is tested to make sure the density and viscosity are within the requirements set forth in the CQA plan.

The mixed slurry may be pumped directly to the trench or to a holding pond or tank. If the slurry is stored in a tank or pond, CQA personnel should check the properties of the slurry periodically to make sure that the properties have not changed due to thixotropic processes or sedimentation of material from the slurry. The specifications for the project should stipulate mixing or circulation requirements for slurry that is stored after mixing.

The properties of the slurry used to maintain the stability of the trench are important. The following pertains to a bentonite slurry that will ultimately be displaced by soil–bentonite or other backfill; requirements for cement–bentonite slurry are discussed in Section 8.3.6. The slurry must be sufficiently dense and viscous to maintain stability of the trench. However, the slurry must not be too dense or viscous; otherwise, it will be difficult to displace the slurry when backfill is placed. The specifications should set limits on these parameters as well as specify the test method. Standards of the American Petroleum Institute (API), along with those of ASTM, are often cited for slurry test methods. Construction specifications normally set limits on the properties of the slurry. Typically about 4% to 8% bentonite by weight is added to fresh water to form slurry that has a specific gravity of about 1.05 to 1.15. The slurry should be dense enough to support the walls of the trench but not so dense and viscous that the slurry cannot be displaced by the backfill. It is recommended that the slurry density not exceed 13.4 kN/m^3 (85 lb/ft^3), per Evans et al. (2004). During excavation of the trench, additional fines may become suspended in the slurry, and the specific gravity is likely to be greater than the value of the freshly mixed slurry. The slurry should not contain excessive sand, that is, no more than 10% to 15% by weight (Evans et al. 2004). The specific gravity of the slurry during excavation is typically on the order of 1.10 to 1.25.

The density of the slurry is measured with the procedures outlined in ASTM D4380. A known volume of slurry is poured into a special "mud balance," which contains a cup on one end of a balance. The weight is determined and the density is calculated from the known volume of the cup.

The viscosity of the slurry is usually measured with a Marsh funnel (ASTM D6910). To determine the Marsh viscosity, 946 mL (1 qt) of slurry is poured into the funnel. The number of seconds required to discharge the slurry into a cup is measured. Water has a Marsh viscosity of about 26 s at 23 °C. Freshly hydrated bentonite slurry should have a Marsh viscosity in the range of about 40 to 50 s. During excavation, the Marsh viscosity typically increases to as high as about 65 s. If the viscosity becomes too large, the thick slurry must be replaced, treated (e.g., to remove sand), or diluted with additional fresh slurry.

Another rheological characteristic of the slurry can be evaluated by the filter loss test (API 13B-1 1990). Details of the filter loss test will be explained in a subsequent section. It is recommended that filter loss value for the slurry should be less than 20 cm^3.

The sand content of the slurry may also be specified. Although sand is not added to fresh slurry, the slurry may pick up sand in the trench during the construction process. The sand content by volume is measured with ASTM D4381. A special glass measuring tube is used for the test. The slurry is poured onto a No. 200 sieve (0.075-mm openings), which is repeatedly washed until the water running through the sieve is clear. The sand is washed into the special glass measuring tube, and the sand content (volumetric) is read directly from graduation marks. Other criteria may be established for the slurry. Limits may also be set on pH (which should be in the range of 7 to 10) (ASTM D4972), gel strength, and other parameters, depending on the specific application.

The primary responsibility for monitoring the properties of the slurry rests with the CQC team. The properties of the slurry directly affect construction operations but may also affect the final quality of the slurry trench cutoff wall. For example, if the slurry is too dense or viscous, the slurry may not be properly displaced by backfill. On the other hand, if the slurry is too thin and lacks adequate bentonite, the stability of the trench may be compromised. The CQA inspectors may periodically perform tests on the slurry, but these tests are usually conducted primarily to verify test results from the CQC team. CQA personnel should be especially watchful to make sure that (1) the slurry has a sufficiently high viscosity and density (if not, the trench walls may collapse); (2) the level of the slurry is maintained near the top of the trench and above the water table (usually the slurry level must be at least 1 m above the groundwater table to maintain a stable trench); (3) the slurry does not become too viscous or dense (otherwise backfill will not properly displace the slurry); and (4) the slurry does not contain excessive sand.

8.3.4 Excavation of the Slurry Trench

The slurry trench is excavated with a backhoe or a clamshell bucket. Long-stick backhoes can dig to depths of approximately 20 to 25 m (60–80 ft). For slurry trenches that can be excavated with a backhoe, the backhoe is almost always the most economical means of excavation. For trenches that are too deep to be excavated with a backhoe, a clamshell bucket is usually used. The trench may be excavated first with a backhoe to the maximum depth of excavation that is achievable with the backhoe and to further depths with a clamshell bucket. Special chopping, chiseling, or other equipment may be used as necessary. The width of the excavation tool is usually equal to the width of the trench and is typically 0.6 to 1.2 m (2–4 ft).

In most instances, the slurry trench cutoff wall is keyed into a stratum of relatively low hydraulic conductivity. In some instances, the vertical cutoff wall may be relatively shallow. For example, if a floating nonaqueous-phase liquid such as gasoline is to be contained, the slurry trench cutoff wall may need to extend only a short distance below the water table surface, depending on the site-specific circumstances. CQC/CQA personnel should monitor the depth of excavation of the slurry trench by lowering a weight attached to a measuring tape to the bottom of the slurry-filled trench. Personnel should also log excavated materials to verify the types of materials present and to ensure specified penetration into a low-permeability layer. Monitoring normally involves examining soils that are excavated. Additional equipment such as airlifts may be needed to remove sandy materials from the bottom of the trench before backfilling.

8.3.5 Soil–Bentonite (SB) Backfill

Soil is mixed with the bentonite–water slurry to form soil–bentonite (SB) backfill. If the soil is too coarse, additional fines can be added. Dry, powdered bentonite may also be added, although it is difficult to ensure that the dry bentonite is uniformly distributed. In special applications in which the properties of the bentonite are degraded by the groundwater, other types of clay may be used (e.g., attapulgite) to form a mineral–soil backfill. If possible, soil excavated from the trench is used for the soil component of SB backfill. However, if excavated soil is excessively contaminated or does not have the proper gradation, excavated soil may be hauled off for treatment and disposal.

Two parameters concerning the backfill are important: (1) the presence of extremely coarse material (i.e., coarse gravel and cobbles) and (2) the presence of fine material. Coarse gravel is defined as material with particle sizes between 19 and 75 mm (ASTM D2487). Cobbles are materials with particle sizes greater than 75 mm. Fine material is material passing through the No. 200 sieve, which has openings of 0.075 mm. Cobbles will tend to settle and segregate in the backfill; coarse gravel may also segregate, but the degree of segregation depends on site-specific conditions. In some cases, the backfill may have to be screened to remove pieces that exceed the maximum size allowed in the specifications. The hydraulic conductivity of the backfill is affected by the percentage of fines present (D'Appolonia 1980; Ryan 1987; Evans 1993). Often, a minimum percentage of fines is specified. Ideally, the backfill material should contain at least 10% to 30% fines to achieve low hydraulic conductivity ($<10^{-7}$ cm/s).

The bentonite may be added in two ways: (1) soil is mixed with the bentonite slurry, usually with a bulldozer, to form a viscous SB material and (2) additional dry powdered bentonite may be added to the soil–bentonite slurry mixture. Dry, powdered bentonite may or may not be needed. D'Appolonia (1980) and Ryan (1987) discuss many of the details of SB backfill design.

When SB backfill is used, a more or less continuous process of excavation, preparation of backfill, and backfilling is used. To initiate the process, backfill is placed by lowering it to the bottom of the trench (e.g., with a clamshell bucket) or placing it below the slurry surface with a tremie pipe (similar to a long funnel) until the backfill rises above the surface of the slurry trench at the starting point of the trench. Additional SB backfill is then typically pushed into the trench with a bulldozer. The viscous backfill sloughs downward and displaces the slurry in the trench. As an alternative method to initiate backfilling, a separate trench that is not part of the final slurry trench cutoff wall, called a lead-in trench, may be excavated at a point outside the limits of the final slurry trench and backfilled with the process just described to achieve full backfill at the point of initiation of the desired slurry trench.

After the trench has been backfilled, low hydraulic conductivity is achieved via two mechanisms: (1) the SB backfill itself has low hydraulic conductivity (typical design value is $\leq 10^{-7}$ cm/s) and (2) the filter cake enhances the overall function of the wall as a barrier (D'Appolonia 1980). Designers do not usually count on the filter cake as a component of the barrier; it is viewed as a possible source of added impermeability that enhances the reliability of the wall.

The compatibility of the backfill material with the groundwater at a site should be assessed before construction. However, CQA personnel should be watchful for groundwater conditions that may differ from those assumed in the compatibility-testing program. CQA personnel should familiarize themselves with the compatibility-testing program. Substances that are particularly aggressive to clay backfills include non-water-soluble organic chemicals, high and low pH liquids, and highly saline water. If there is any question about groundwater conditions in relation to the conditions covered in the compatibility-testing program, the CQA engineer or design engineer should be consulted.

Improper backfilling of slurry trench cutoff walls can produce defects (Figure 8-7). More details are given by Evans (1993). CQA personnel should watch out for

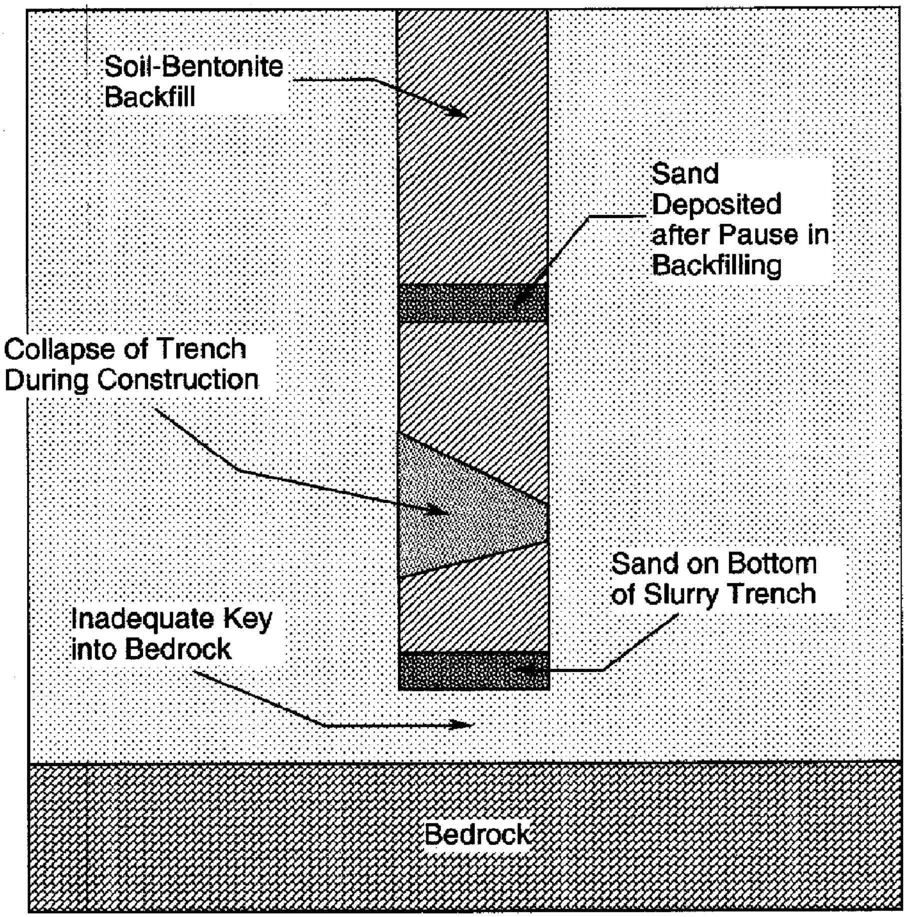


Figure 8-7. Examples of Problems Caused by Improper Backfilling of Slurry Trenches.

accumulation of sandy materials during pauses in construction (e.g., during shutdowns or overnight); an airlift can be used to remove or resuspend the sand, if necessary.

Some slurry trench cutoff walls fully encircle an area. As the slurry trench reaches the point of initiation of the slurry trench cutoff wall, closure is accomplished by excavating into the previously backfilled wall.

8.3.6 Cement–Bentonite (CB) Cutoff Walls

A cement–bentonite (CB) cutoff wall is constructed with a cement–bentonite–water mixture that hardens and attains low hydraulic conductivity. The slurry trench is excavated, and excavated soils are hauled away. Then the trench is backfilled in one of two ways. In one method, the slurry used to maintain a stable trench during construction is CB rather than just bentonite–water, and the slurry is left in place to harden. An alternative technique is to construct the slurry trench with a bentonite–water slurry in discrete diaphragm cells and to displace the bentonite–water slurry with CB in each cell. The development of the CB cutoff wall technology is discussed in detail by Jefferis (1997).

The CB mixture cures with time and hardens to the consistency of a medium to stiff clay (CB backfill is not nearly as strong as structural concrete). A typical CB slurry is based on a weight of 75% to 80% water, 15% to 20% cement, 5% bentonite, and a small amount of viscosity-reducing material ("super plasticizer"). In Europe, the CB often contains other ingredients (e.g., slag and fly ash), which help to reduce both the porosity and the hydraulic conductivity of the CB product. After hardening, CB walls generally have strength between about 15 and 35 N/cm^2. Without special design mixes and fillers, CB backfill is usually more permeable than SB backfill. Hydraulic conductivity of CB backfill is often on the order of 10^{-6} to 10^{-7} cm/s, which is up to or more than an order of magnitude greater than typical SB cutoff walls.

The CB cutoff wall is constructed using procedures almost identical to those used in building structural diaphragm walls. In Europe, CB backfilled slurry trench cutoff walls are much more common than in the United States, at least partly because the diaphragm wall construction capability is more broadly available in Europe and because high-grade sodium bentonite (which is critical for soil–bentonite backfilled walls) is not readily available in Europe. Tallard (1997) describes an example of an alternative cement–bentonite material containing attapulgite clay and blast furnace slag. Evans and Dawson (1999) show that mixing cement–bentonite with slag lowers hydraulic conductivity significantly.

The CB cutoff wall is best suited for sites contaminated with hydrocarbons, but other contaminants may be contained if appropriate chemical compatibility tests are performed. The CB cutoff wall provides some advantages over the SB cutoff wall. The CB cutoff wall need not mix soil with bentonite, which makes construction more convenient when space for mixing is limited. The CB cutoff wall can be installed in soils with questionable stability because of the relatively quick setting times of the slurry (U.S. Army Corps of Engineers 1996).

8.3.7 Geomembranes in Slurry Trench Cutoff Walls

Geomembranes may be used to form a vertical cutoff wall. The geomembrane may be installed in several ways:

1. The geomembrane may be inserted in a trench filled with CB slurry to provide a composite CB–geomembrane barrier (Manassero and Pasqualini 1992). The geomembrane is typically mounted to a frame, and the frame is lowered into the slurry. The base of the geomembrane contains a weight such that when the geomembrane is released from the frame, the frame can be removed without the geomembrane floating to the top. CQA personnel should be particularly watchful to ensure that the geomembrane is properly weighted and does not float out of position. Interlocks between geomembrane panels (Figure 8-3) provide a seal between panels. The panels are typically relatively wide (on the order of 3 to 7 m) to minimize the number of interlocks and to speed installation. The width of a panel may be controlled by the width of excavated sections of CB-filled panels.
2. Stiff geomembranes (e.g., unplasticized PVC) may be driven directly into the CB backfill or into the native ground. Panels of geomembrane with widths on the order of 0.5 to 1 m are driven individually or are attached to a guide or insertion plate, which is driven or vibrated into the subsurface. If the panels are driven into a CB backfill material, the panels should be driven before the backfill sets up. Interlocks between geomembrane panels (Figure 8-3) provide a seal between panels. This methodology is essentially the same as that of a sheet pile wall.

Although use of geomembranes in slurry trench cutoff walls is relatively new, the technology is gaining popularity. The promise of a practically impermeable vertical barrier and the excellent chemical resistance are compelling advantages. Development of more efficient construction procedures should make this type of cutoff wall increasingly attractive.

8.3.8 Other Backfills

Structural concrete could be used as a backfill, but if concrete is used, the material usually contains bentonite and is termed *plastic concrete* (Evans 1993). Plastic concrete is a mixture of cement, bentonite, water, and aggregate. Plastic concrete is different from structural concrete because it contains bentonite and is different from SB backfill because plastic concrete contains aggregate. Other ingredients (e.g., fly ash) may be incorporated into the plastic concrete. Construction is typically with the panel method (Figure 8-8). Hydraulic conductivity of the backfill can be $<10^{-8}$ cm/s. The high cost of plastic concrete limits its use.

A relatively new type of backfill is termed soil–cement–bentonite (SCB). The SCB wall uses native soils (not aggregates, as with plastic concrete). Placement is in a continuous trench, rather than using the panel method.

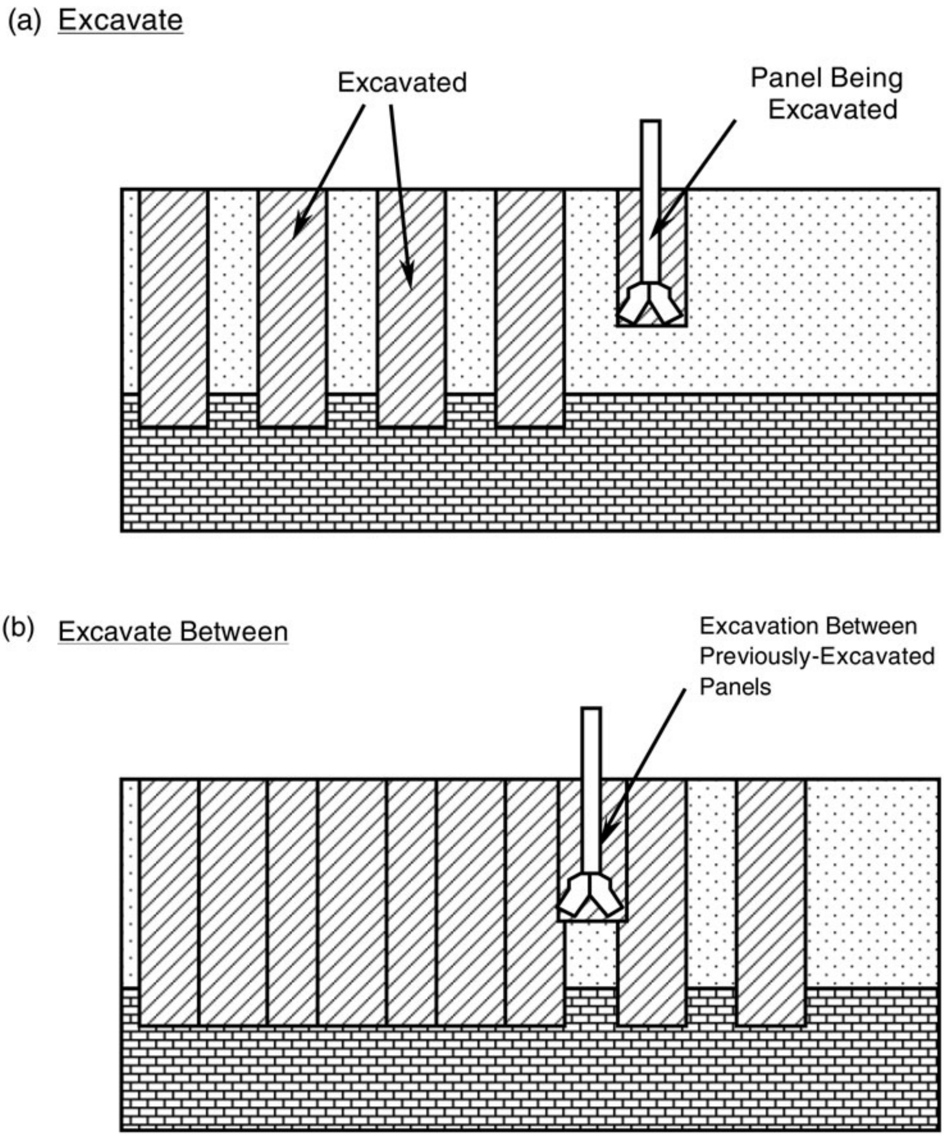


Figure 8-8. Schematic Diagram Showing Two Methods for the Construction of Diaphragm Walls.

8.3.9 Caps

A cutoff wall cap represents the final surface cap on top of the slurry trench cutoff wall. The cap may be designed to minimize infiltration, withstand traffic loadings, or serve other purposes. Freeze–thaw cycles do not significantly degrade soil–bentonite backfill (Zimmie et al. 1997), but wet–dry cycles can damage wet soil–bentonite backfill. The potential for desiccation cracking is so large that the installation of caps is necessary. CQA personnel should inspect the cap as well as the wall itself to ensure that the cap conforms to specification.

8.4 Other Types of Cutoff Walls

Evans (1993) discusses other types of cutoff walls. These include vibrating beam cutoff walls, deep soil mixed walls, and other types of cutoff walls. These walls are not discussed in detail here because these types of walls have been used much less frequently than the other types.

A type of wall that is increasing rapidly in popularity is a "permeable reactive barrier." Such walls are trenches filled with a reagent material that will react with contaminants in groundwater that flows horizontally through the wall and reduce the concentration of contaminant through sorption, biodegradation, or similar processes. This type of barrier has not yet reached the point of standardization of construction methodology and, hence, is not discussed further here.

8.5 Hydraulic Conductivity

One of the most important aspects of creating a vertical barrier is verification of low hydraulic conductivity. The design maximum hydraulic conductivity varies, depending on the application, but is typically less than 1×10^{-7} cm/s.

Evans (1994) and Daniel and Choi (1999) review methods for determining hydraulic conductivity of vertical barriers. The methods fall into the following three basic categories: (1) laboratory tests on reconstituted samples (often called "wet samples") recovered from the barrier at the time of construction and cured in the laboratory; (2) laboratory hydraulic conductivity tests on relatively "undisturbed samples" taken from the constructed barrier; and (3) in situ tests on the constructed barrier. The primary methods of hydraulic conductivity testing are discussed in the succeeding sections. The relative advantages and disadvantages are summarized in Table 8-2.

8.5.1 Laboratory Tests on Reconstituted Samples

By far the most commonly used type of hydraulic conductivity test for vertical barriers is the laboratory test on a reconstituted sample of the barrier material. For soil–bentonite backfilled slurry walls, the sample is usually a "grab" sample removed from the backfill mixing area just before placement in the slurry-filled trench. For barrier walls constructed in situ (e.g., deep soil mixed walls), the sample is removed from the actual barrier near the surface and before the backfill has had time to set up or to cure.

There are three ways to measure the hydraulic conductivity on reconstituted samples of barrier walls: (1) place the reconstituted sample in a filter-press device, apply an air pressure, and measure the apparent hydraulic conductivity (API 13B-1; ASTM D5891); (2) place the backfill in a rigid-wall permeameter, apply a confining stress (e.g., in a consolidation-cell permeameter), and then measure hydraulic conductivity after the backfill has consolidated; and (3) fabricate a cylindrical

Table 8-2. Advantages and Disadvantages of Hydraulic Conductivity Testing Methods for Vertical Barriers

Category	*Method*	*Advantage*	*Disadvantage*
Laboratory tests on reconstituted samples	Test in fluid loss apparatus	• Inexpensive • Minimal sample-handling issues	• Void ratio of test material highly variable • No control over stresses or saturation • Reconstituted sample may not be representative of constructed barrier material
	Consolidation-cell test	• Compressive stress controlled • Easy to form a test specimen by consolidation test material from a loose or nearly fluid state • Convenient for permeation with contaminated liquids	• Equipment relatively complex and expensive • No control over saturation • Reconstituted sample may not be representative of constructed barrier material
	Flexible-wall permeameter	• Industry standard of testing low-permeability materials • Full control over stresses • Sample completely saturated	• Difficult to form a test specimen from a soft material • Equipment relatively complex and expensive • Reconstituted sample may not be representative of constructed barrier material
Laboratory tests on undisturbed samples	Test in sampling tube	• Inexpensive	• Potential sidewall leakage • Lack of control over stresses • Lack of control over saturation • Lack of flexibility over direction of fluid flow (vertical or horizontal) • Sample almost certain to be disturbed to some extent
	Flexible-wall permeameter	• Industry standard of testing low-permeability materials • Full control over stresses • Sample completely saturated	• Soft sample may be difficult to handle • Difficult to test at very low effective stress • Sample almost certain to be disturbed to some extent

Table 8-2. Continued

Category	*Method*	*Advantage*	*Disadvantage*
Laboratory tests on undisturbed samples (continued)	Flexible-wall permeameter (continued)	• No restriction on size of samples • Can trim sample to permeate in any direction	
In situ tests	Piezocone	• Additional information besides hydraulic conductivity is collected • In situ barrier material is tested	• Permeated volume very small • Experience very limited
	Single Well ("Slug") Test	• Large volume of material tested • In situ barrier material is tested	• Borehole may be smeared • Proximity of well screen to edge of barrier unknown • Methods for calculating hydraulic conductivity is not well developed for thin, compressible vertical barriers

Source: Daniel and Choi 1999, ASCE

test specimen of the barrier material and then place the test specimen in a permeameter (e.g., a flexible-wall permeameter conforming to ASTM D5084).

The simplest and least expensive method for measuring hydraulic conductivity is the API fluid loss test. The fluid loss test (API 13B-1; ASTM D5891) is routinely performed on bentonitic slurries and provides an excellent indicator of the quality of bentonite as a gelling agent. Figure 8-9 shows a schematic diagram of the fluid loss testing equipment adapted to hydraulic conductivity testing of soil–bentonite backfill. The soil–bentonite backfill is placed in the device, an air pressure is applied, and a "filter cake" forms near the interface with the underlying filter sand. After some consolidation has occurred, the rate of flow through the material is measured, and hydraulic conductivity is calculated (Heslin et al. 1997; Filz et al. 2001). It may be best to use a layer of sand above the test material to provide confinement (Barvenik and Ayres 1987; Heslin et al. 1997).

One of the problems with the fluid loss test is the fact that the interpretation of the test results is complicated because during the test the soil–bentonite specimen is consolidated by seepage forces, which produce a variation in effective consolidation stress from the top of the specimen and to the bottom. The effective stress is zero at the top of the specimen and is approximately equal to the applied air pressure at the bottom (i.e., the void ratio is lower near the bottom of the material). To logically interpret results of the fluid loss test, Filz et al. (2001) apply

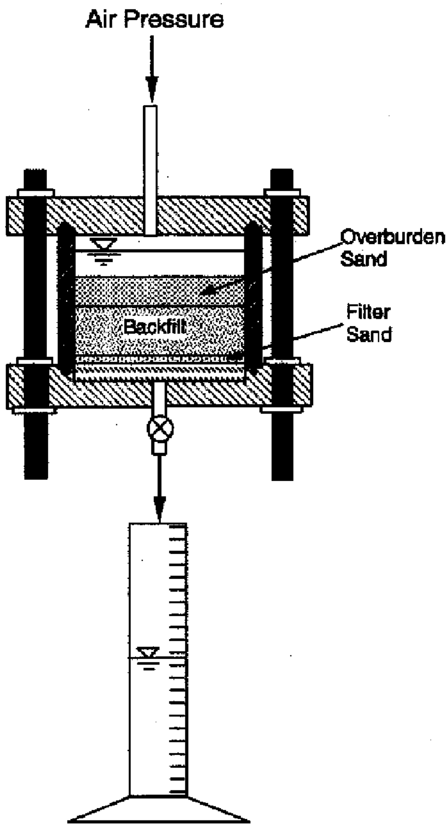


Figure 8-9. Schematic Diagram of the Fluid Loss Apparatus for Hydraulic Conductivity Testing.

the seepage consolidation theory (Fox and Baxter 1997) to analysis of test results and show good agreement with the result of tests performed in a consolidometer and a flexible-wall permeameter. Although the test method appears relatively crude, it is used fairly frequently, and experience suggests that the method seems to provide hydraulic conductivities in the same general range as other testing methods (Barvenik and Ayres 1987; Heslin et al. 1997).

Another method of laboratory hydraulic conductivity testing on reconstituted samples involves placing the barrier material in a consolidation-cell permeameter, tamping or "rodding" the material to remove air voids, consolidating the material, and then permeating it. This process provides a convenient method for forming a consolidated test specimen in the same rigid-wall cell that will be used for permeation.

An additional method of testing reconstituted samples is to form a test specimen in a cylinder (e.g., by tamping the material with a rod into a compaction mold), transferring the test specimen to a flexible-wall permeameter, and then measuring the hydraulic conductivity in the flexible-wall cell, per ASTM D5084. This method is probably the best of the three test methods on reconstituted samples in the sense that the flexible-wall method provides the greatest degree of control and industry acceptance. The problem, however, is that many barrier materials are extremely soft and difficult to shape into a cylindrical specimen that will stand under its own weight. Backfills with cement can be cured before transfer to the flexible-wall permeameter, although it is best to transfer the test specimen to the permeameter and subject the specimen to the effective confining stress that simulates in situ conditions as quickly as possible.

The fundamental disadvantage of all test methods involving reconstituted samples is that the material being tested may be quite different from the in situ material. "Rodding" the backfill material into a mold may or may not produce a material with similar pore-size distribution and hydraulic conductivity as the actual in situ backfill material. Despite this problem, however, laboratory tests on reconstituted samples (so-called "wet" tests) are the industry standard method used today for evaluating hydraulic conductivity of vertical barriers in the United States because the other methods (discussed below) have major problems and uncertainties associated with them and because tests on reconstituted samples are far faster and simpler than the alternative procedures.

8.5.2 Laboratory Tests on Undisturbed Samples

Laboratory tests on "undisturbed" samples of material recovered from vertical barriers are performed in essentially the same manner as tests on undisturbed samples recovered from soil strata. Although hydraulic conductivity tests can be performed on the sample contained inside the thin-walled sampling tube, such tests are rare. Rather, the normal procedure is to extrude a sample from the sampling tube, trim a cylindrical test specimen, and permeate the specimen in a flexible-wall permeameter, per ASTM D5084.

The process of measuring hydraulic conductivity of "undisturbed" samples is fraught with potential difficulty. First, there is a high probability that the barrier material will be damaged to some extent by the sampling process. This problem is particularly true for backfills containing a small amount of cement, which have a fragile structure and are easily cracked (Yang et al. 1993). Furthermore, the test specimen must be handled carefully when it is set up in the permeameter; soft samples (e.g., soil–bentonite backfill in slurry trenches) can be extremely difficult to handle. Time must be allowed for consolidation of the backfill to occur before the material is sampled, which creates highly undesirable delay between construction and hydraulic conductivity verification. In addition, the material must be consolidated in the permeameter to a predetermined effective stress, but experience indicates that because of arching, the actual in situ vertical effective stress may be far lower than the value computed from geostatic conditions (Evans et al. 1995). If the specimen is consolidated to the geostatic vertical effective stress or higher, the measured hydraulic conductivity will be too low. In addition, if the constructed barrier wall is not perfectly vertical, if the borehole used to obtain a relatively undisturbed sample is not vertical, or if the borehole is drilled off center from the barrier wall, the "undisturbed" sample may be taken from outside the vertical barrier. Finally, if the vertical barrier does contain an occasional defect ("window"), the probability of sampling from that particular zone is small. For example, assume that windows 0.5 m by 0.5 m occur in a 20-m-deep vertical barrier at a spacing of 50 m between windows. The "windows" occupy an area that is only 0.025% of the total area, yielding a probability of encountering the window in any single sample of approximately one in 4,000. Thus, even if a relatively undisturbed sample is obtained from within the boundaries and consolidated to the proper vertical effective stress, there is no assurance that the more permeable zones within the barrier have been sampled or tested.

8.5.3 In Situ Hydraulic Conductivity Tests

In situ hydraulic conductivity tests on vertical barriers offer the opportunity to permeate the actual backfill material. Two kinds of in situ tests have been used: the piezocone test (ASTM D5778; ASTM D6067) and the slug test (ASTM D4044; ASTM D5912).

Manassero (1994) describes the possible usage of the piezocone penetration tests to provide a continuous assessment of hydraulic conductivity for a cement–bentonite barrier. The assessment procedure uses an empirical relation between hydraulic conductivity and three piezocone penetration parameters: the pore pressure increment, the total point resistance, and the sleeve friction.

Use of the piezocone in a hardened CB backfill that is relatively stiff, hard, and brittle may not give a reasonable estimation of hydraulic conductivity because the insertion of the piezocone could cause cracking. Hydraulic conductivity measured from the piezocone pore pressure dissipation test in the standard mix slurry was found to be several orders of magnitude larger than laboratory and other in situ measurements (Tedd et al. 1995). Ratnam et al. (2001) determined hydraulic conductivity of CB backfill with the aid of the Cambridge self-boring pressuremeter, which minimizes installation disturbance and preserves the structure of the CB cutoff wall.

There are several advantages of the piezocone test in measuring hydraulic conductivity of vertical barriers. The method is fast and cost effective. A continuous log of hydraulic conductivity versus depth can be obtained. A disadvantage is that the piezocone permeates only a tiny volume of material relative to other in situ test methods. This method may, on insertion into the barrier, create sufficient disturbance (or even cracks) to alter hydraulic conductivity.

The more commonly used in situ hydraulic conductivity test in the United States (although still rarely used) is a single-well, falling-head or rising-head test, commonly termed a "slug test." A slug test is initiated by causing an instantaneous change in the water level in a borehole through the sudden introduction or removal of a known volume of water ("slug"). A rate of water rise or drop in a borehole after withdrawing or adding a known volume of water "slug" is measured and used to determine hydraulic conductivity in the slug test. The recovery of the water level with time is analyzed as a graph of head versus time history.

The slug test has been used routinely by hydrogeologists to evaluate hydraulic conductivity of aquifers and aquitards (Hyder et al. 1994; Butler 1998). Four analytic and semianalytic methods are commonly used for analyzing the results of a slug test: (1) the Hvorslev method (Hvorslev 1951); (2) the Cooper et al. method (Cooper et al. 1967; Papadopulos et al. 1973); (3) the Bouwer and Rice method (Bouwer and Rice 1976; Bouwer 1989); and (4) the Kansas Geological Survey method (Hyder et al. 1994).

There are three fundamental problems in the interpretation of data from slug tests: (1) Available slug test analysis methods are applicable to porous media that extend infinitely in the horizontal direction (and not for a wall whose boundaries are often much less than 1 m from the well); (2) the distance from the well to the edge of the wall is usually not known or even knowable (Figure 8-10); and (3) most methods of data analysis assume that the porous medium is incompressible and barrier materials such as SB backfill are highly permeable.

Teeter and Clemence (1986) suggested that one can take into account a cutoff wall boundary by using a flow net solution. Choi and Daniel (2006a, 2006b) present much more detailed methodologies that enable one to account for the boundaries in the system and the compressibility of the barrier wall material.

8.6 Specific CQA Requirements

No standard types of tests or frequencies of testing have been adapted in the industry for construction of vertical cutoff walls. Recommendations from this section

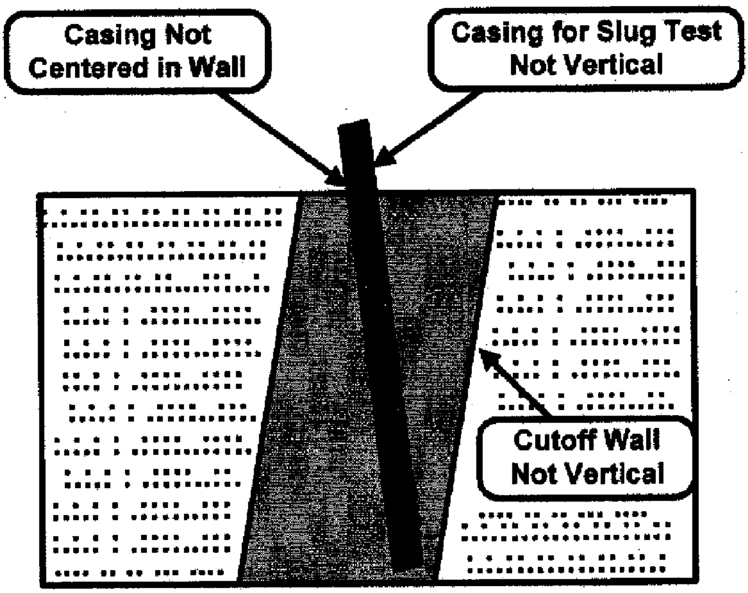


Figure 8-10. Potential Problems with Centering a Screened Well in a Vertical Barrier.

Source: Choi and Daniel 2006a, ASCE.

were taken largely from the author's experience, recommendations provided by Evans (personal communication), and U.S. Army Corps of Engineers (2004). Comprehensive overviews for CQA requirements are found in Tamaro and Poletto (1992), Rumer and Mitchell (1995), and Poletta and Good (1997). Case histories described by Evans et al. (2004) are quite useful.

8.6.1 Slurry

8.6.1.1 Water

The water used to form the slurry can influence its quality. Acceptable slurries can generally be achieved with most water sources. Potable water is almost always suitable. Fresh, reasonably nonturbid water from a lake or similar source will often prove suitable, as well. Problems can be encountered when the water is salty or brackish because bentonite tends not to gel well in such liquids. Seawater is almost never suitable unless special gelling agents are used to form the slurry.

If the specifications require any testing of water parameters (such as pH, hardness, total dissolved solids, or volatile organic compounds), one set of tests would usually be required per water source used. More commonly, however, no specific tests are required, and it is left to CQC and CQA personnel to ensure that suitably fresh water is used. If unsuitable water is used, the result is usually poor gelling of the bentonite, which will likely result in failure to meet specifications with regard to slurry parameters (see Section 8.6.1.3).

8.6.1.2 Bentonite

The raw bentonite (or other clay) that is used to make the slurry may have specific requirements that must be met. If so, tests should be performed to verify those properties. There are no standard tests or frequency of tests for the bentonite. The reader may wish to consult Section 3.7.5 for a general discussion of tests and testing frequencies for bentonite–soil liners and Section 5.2.1 relative to GCLs.

The most commonly used test in the industry to measure bentonite quality is the fluid loss test (ASTM D5891), which is also called the "filtrate loss test" and "filter press test" and is described by API 13B-1 (2003). With this test, a slurry is mixed, placed in a cell, and forced through filter paper under pressure. A bentonite filter cake forms on the surface of the filter paper, and the fluid flow rate gradually declines over time. The better the quality of the bentonite, the better the seal formed by the filter cake. With the test, the amount of water that passes through the filter cake in a fixed time is collected and measured; this is referred to as the "fluid loss" or "filtrate loss."

With GCLs, a commonly specified maximum filtrate loss is 18 mL. In the slurry wall industry, a maximum value of 12.5 mL is often recommended (U.S. Army Corps of Engineers 2004).

8.6.1.3 Bentonite Slurry As-Mixed

The bentonite slurry must contain enough bentonite to form a viscous slurry, but the slurry must not be so dense or viscous that it cannot be displaced adequately by the soil–bentonite backfill. Bentonite is mixed with water in various types of mixers. The best mixers are high-shear colloidal mixers that are designed specifically for mixing these types of materials. Once the bentonite and water are mixed, the slurry can be stored to allow hydration or used immediately. Often, allowing some hydration time can be helpful in allowing time for the thixotropic bentonite to gel. With relatively ineffective mixers, allowing hydration time may be essential. In any case, a hydration time of eight hours or more is sometimes recommended to ensure that the bentonite is fully hydrated.

The specific slurry tests that are often performed are as follows:

1. *Viscosity.* The Marsh funnel viscosity test, per API 13B-1 and ASTM D6910, is usually used as a simple way to measure viscosity. The suggested minimum viscosity is 40 s, as shown in Table 8-3.
2. *Density.* A special cup and balance facilitate rapid measurement of the bulk density of the slurry. The minimum density is sometimes set at 1.025 g/cm^3 (U.S. Army Corps of Engineers 2004), though larger specified minimum values are common. To ensure that the density is not so great as to make the slurry difficult to displace by the backfill, a maximum density of 1.15 g/cm^3 is suggested during the mixing phase. The slurry is likely to become more dense in the trench as suspend solids (e.g., fine sand) become mixed in with the slurry.
3. *Fluid loss.* The actual slurry may be tested for fluid loss, and the suggested maximum fluid loss is 20 mL (Table 8-3).
4. *pH.* The pH is an issue in some environments because the properties of bentonite can change in highly acidic or basic environments. Natural bentonites, when suspended in pure water, are slightly basic. A pH range of 6.5 to 10 should yield conditions that will promote favorable properties of bentonite.

8.6.1.4 Bentonite Slurry in the Trench

The recommended tests for bentonite slurry in the trench are similar to those for as-mixed slurry (Table 8-1) but should include two added constraints that limit the suspended sediments in the slurry: (1) the maximum sand content is monitored, and (2) a maximum total density is specified. The concern is that if too much fine sand or other soils are suspended in the slurry in the trench, the slurry may not be fully displaced by backfill and pockets of slurry may be entrapped in the final wall (Evans et al. 2004). If the sand content is less than 10% to 15% and the total density no more than about 13.4 kN/m^3 (85 pcf), the slurry should be capable of being displaced by backfill. A rule of thumb is that the total density of the backfill should be at least 2.4 kN/m^3 (15 pcf) greater than the density of the slurry being displaced.

8.6.2 Backfill

Several tests on backfill are often specified more for documentation than control purposes, such as grain-size distribution and water content. Sometimes a minimum

Table 8-3. Recommended Tests for Soil–Bentonite Backfill Slurry Trenches

Component	*Parameter*	*Typical Specified Value*	*Typical Testing Method*	*Recommended Minimum Testing Frequency*
Water for slurry	pH, hardness, total suspended solids, total dissolved solids, other as appropriate	—	—	1 per water source
Bentonite	Fluid loss	≤12.5 mL	ASTM D5891	1 per truck or rail car
	Free swell	≥25 mL	ASTM D5890	
Mixed slurry	Marsh viscosity	>40 s	API 13B-1 or ASTM D6910	2 per eight-hour shift during mixing, and once per batch just before placement in trench
	Total density	10.06–11.32 kN/m^3 (62.4–72 lb/ft^3)	API 13B-1 or ASTM D4380	
	Fluid loss	<20 cm^3	API 13B-1	
	pH	6.5 –10	API 13B-1	
In-trench slurry	Total density	10.6–13.36 kN/m^3 (64–85 lb/ft^3) and at least 2.36 kN/m^3 (15 lb/ft^3) less than total density of backfill	API 13B-1 or ASTM D4380 (with one sample approximately 0.6 m below top of slurry and one sample 0.6 m above bottom of trench)	2 per eight-hour shift, near beginning and end of shift, during backfill placement or other in-trench work
	Marsh viscosity	>40 s	API 13B-1 or ASTM D6910	
	pH	6.5–10.0	API 13B-1	
	Sand content	≤10–15%	API 13B-1 or ASTM D4381	
Soil–bentonite backfill material	Grain-size distribution	—	ASTM D422	1 per 1,000 m^3 of backfill
	Moisture content	—	ASTM D2216	
	Fines content	—	ASTM D1140	
	Atterberg limits	—	ASTM D4318	
	Slump	100–150 mm (4–6 in.)	ASTM C143	
	Total density	At least 2.36 kN/m^3 (15 lb/ft^3) greater than density of slurry	API 13B-1 or ASTM D4380 or ASTM D698 (variance)	
	Hydraulic conductivity	$<1 \times 10^{-6}$–1×10^{-7} cm/s	ASTM D5084	

"fines content" (i.e., percent of soil on a dry-weight basis finer than the No. 200 sieve, which has openings of 0.075 mm) is specified because if the soil has too few fines, this can be a warning that low hydraulic conductivity may be difficult to achieve without additional bentonite. A soil with less than 20% fines may be a candidate for extra bentonite. Atterberg limits are commonly measured, and occasionally a minimum may be specified, although there are not industry standard minimum values. The total density is an important control parameter and, as mentioned above, should be at least 2.4 kN/m^3 (15 pcf) greater than the density of the slurry being displaced. The density may be measured by placing backfill in a standard 101.6-mm-diameter mold and rodding the material 10 times, per ASTM D698. Additional backfill is then added to fill the mold. Alternatively, a mud balance per API 13B-1 may be used.

The methods for sampling and performing hydraulic conductivity tests were discussed in Section 8.5. Usually, samples of backfill are taken just before placement in the trench and are used to prepare a test specimen in the laboratory, e.g., by "rodding" the wet backfill into a mold. In fairly rare situations, relatively undisturbed samples are recovered from the completed wall. A key issue is the effective confining stress used to test for hydraulic conductivity testing. The U.S. Army Corps of Engineers (2004) recommends using a value representative of the in situ effective confining stress of the upper quarter to one-half of the wall depth.

8.6.3 Other Measurements and Observations

Monitoring of the depth of the trenching and key-in to an impermeable stratum before backfilling is often the major component of CQA. The depth is usually measured by lowering a weight attached to a tape measure to the bottom of the trench. Repeated soundings of this type are made to determine the depth profile. The soil that is excavated from the trench should be continuously logged by CQA personnel to verify that subsurface conditions are similar to those that were anticipated. The CQA personnel should look for evidence of instability in the walls of the trench (e.g., sloughing at the surface next to the trench or development of tension cracks). If the trench is to extend into a particular stratum (e.g., an aquitard), CQA personnel should verify that adequate penetration has occurred. A minimum key depth of 0.9 m (3 ft) is commonly specified. The recommended procedure is to measure the depth of the trench once the excavator has encountered the aquitard and to measure the depth again after adequate penetration is thought to have been made into the aquitard.

CQA personnel should be careful to check for sedimentation in the slurry-filled trench after periods when the slurry has not been agitated (e.g., after an overnight work stoppage). Figure 8-7 illustrates some of the defects ("windows") that can occur in soil–bentonite backfilled slurry trenches. Tachavises and Benson (1997) and Lee and Benson (2000) discuss the effect of hydraulic imperfections in a vertical cutoff wall and conclude that the hydraulic conductivity of the defect is a key factor. The overall hydraulic conductivity of a barrier wall can be orders of magnitude higher than design specifications when the wall contains small permeable

Table 8-4. CQA/CQC Evaluation for SB Slurry Barrier Walls against Acceptable Industry Practice

Category	*Less Than Acceptable*	*Acceptable*	*Better Than Acceptable*
Specialty contractor experience	<4	4–6 comparable projects	>6
Trench excavation methods	No inspection	Periodic inspections	Constant inspection
Trench width, verticality, and continuity	No inspection	Periodic inspections	Measured
Trench sounding (slope and bottom)	>6 m	3–6 m	<3 m
Trench bottom cleaning	None	Yes	Yes
Trench key confirmation	No sampling	Sampling every 20 ft	Sampling <20 ft
Slurry mixing	<	Agitation >12 h hydration	>
Slurry viscosity testing	<2	2 per shift	>2
Slurry viscosity	<40	40+ seconds (Marsh funnel)	40–50 seconds (Marsh funnel)
Slurry sand content testing	<2	2 per shift	>2
Slurry sand content	>15%	<15%	<<15%
Backfill slump testing	<	1 per 400–600 cycles	>
Backfill slump	<3″ or >6″	Most tests 3″–6″	All tests 3″–6″
Backfill gradation testing	<1	1 per 400–600 cy	>1
Backfill permeability testing	<1	1 per 400–600 cy	>1
Backfill target permeability	>	1×10^{-7} cm/s	<
Backfill mixing and placement	Loosely controlled	Controlled mixing and placement	Central mixing and guided placement
Capping confirmation	None	Cap confirmed	>
Barrier continuity	Interrupted	Continuous	Continuous and confirmed
Postconstruction barrier sampling and testing	None	Minimal	Regular and documented
As-built records	None	Construction completion report	Report, drawings, and test results
Groundwater head monitoring	None	Monitored fluctuation	Periodic and across barrier
Final barrier alignment survey	None	Surveyed	Surveyed and monumented

Table 8-4. Continued

Category	*Less Than Acceptable*	*Acceptable*	*Better Than Acceptable*
Barrier construction specification	None	Barrier	Barrier and CQA plan
CQA/CQC program and testing specification	None	Designer specified	Independent duplicate QA
Groundwater chemistry monitoring	None	Minimal	Periodic and across barrier

Source: Data are from U.S. EPA 1998.

defects. The trench can be "cleaned" from sediment by agitation with a back pump or airlift pump.

The amount of bentonite in the backfill has a critical effect on hydraulic conductivity. Bentonite content is difficult to measure accurately and therefore is not commonly measured. The methylene blue test (Alther 1983) is probably the most common test. This is essentially a titration test that measures the amount of methylene blue (which is strongly sorbed by bentonite) that is absorbed by the mixture. However, silt and clay lines also sorb methylene blue, which makes test results difficult to interpret. In laboratory tests, the amount of bentonite can be carefully controlled to develop the recommended mix design. Backfill for a vertical cutoff wall typically contains about 4% to 6% bentonite by weight.

The specific CQA/CQC programs are essential for the successful implementation of the design and the performance of the barrier wall. Experience in the installation of barrier walls at hazardous waste sites has been obtained for the past 20 years and has established typical industry practice for performing CQA/CQC programs. Table 8-4 presents a summary of industry practice for CQA/CQC programs, which are evaluated from 36 SB slurry wall construction sites. This table shows the acceptable industry practice in CQA/CQC programs for each site (U.S. EPA 1998).

8.7 Postconstruction Tests for Continuity

At the present time, no testing procedures are available to determine the continuity of a completed vertical cutoff wall.

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ASTM D698. "Standard test method for laboratory composition characteristics of soil using standard effort (12, 400 ft-lb/ft^3 (600 kNm/m^3))."

ASTM D1140. "Standard test method for amount of material in soil finer than the No. 200 (75-μm) sieve."

ASTM D2216. "Standard test method for laboratory determination of water (moisture) content of soil and rock by mass."

ASTM D2487. "Standard classification of soils for engineering purposes (Unified Soil Classification System)."

ASTM D4044. "Standard test method for (field procedure) for instantaneous change in head (slug) tests for determining hydraulic properties of aquifers."

ASTM D4318. "Standard test method for liquid limit, plastic limit, and plasticity index of soils."

ASTM D4380. "Standard test method for density of bentonitic slurries."

ASTM D4381. "Standard test method for sand content by volume of bentonite slurries."

ASTM D4972. "Standard test method for pH of soils."

ASTM D5084. "Standard test methods for measurement of hydraulic conductivity of saturated porous materials using a flexible wall permeameter."

ASTM D5778. "Standard test method for performing electronic friction cone and piezocone penetration testing of soils."

ASTM D5890. "Standard test method for swell index of clay mineral component of geosynthetic clay liners."

ASTM D5891. "Standard test method for fluid loss of clay component of geosynthetic clay liners."

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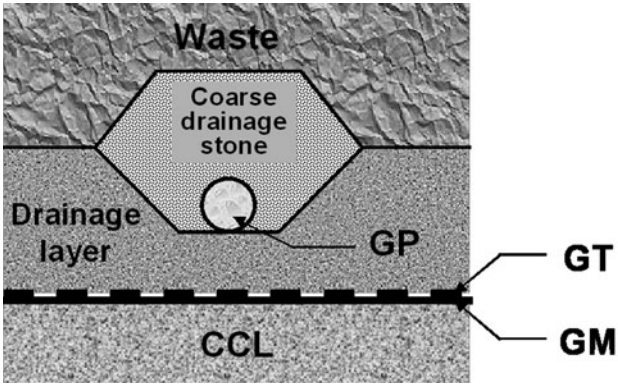
Ancillary Materials, Appurtenances, and Other Details

This chapter is devoted to ancillary materials used within waste containment facilities, various appurtenances that are necessary for proper functioning of the system, and other important details. Ancillary materials such as plastic pipe for leachate transmission, sumps for collection of leachate, manholes, and pipe risers for removal of leachate will be covered in this chapter. Appurtenances, such as penetrations made through various barrier materials, will also be covered. Other important details requiring careful inspection, such as anchor trenches, internal dikes and berms, and access ramps, will also be addressed. Finally, two important topics are presented to conclude the book. They are geosynthetic reinforcement materials and erosion-control materials.

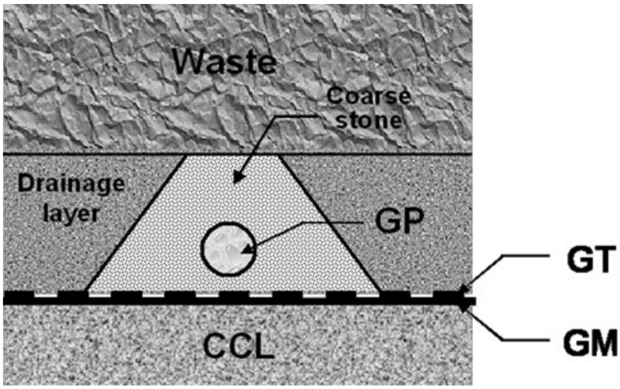
9.1 Plastic Pipe (Also Known as "Geopipe")

Whenever the primary or secondary leachate collection system at the bottom of a waste containment facility is a natural soil material, such as sand or gravel, a perforated piping system must be located within it to rapidly transmit the leachate to a sump and removal system. (When using extremely large gravel, pipes might not be necessary, but this situation is uncommon.) Figure 9-1 illustrates various cross sections of such a pipe system in different configurations, all of which are located above the uppermost geomembrane or protection geotextile of the primary liner system. The choice is a design issue, and the plans and specifications must be clear and detailed regarding the configuration and its dimensions.

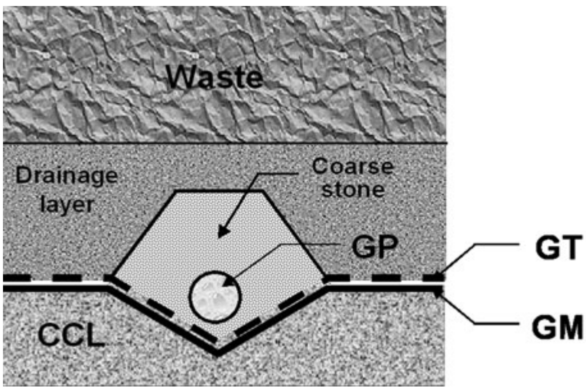
The pipes are sometimes placed in a manifold configuration with feeder lines framing into a larger main header line, thus covering the entire footprint of the landfill unit or cell (Figure 9-2). The entire pipe network flows gravitationally to a low point, where the sump and removal system, consisting of a manhole, penetration, or pipe riser, is located. The diagonal feeder pipes, if included, are always perforated to allow the leachate to enter them. The central header line may or may not be perforated, depending on the site-specific design. However, a large variety of schemes are possible, and it is clearly a design issue that must be unequivocally presented in the plans and specifications.



(a) Shallow trench in the drainage layer



(b) Embankment type within the drainage layer



(c) V trench in the liner system

Figure 9-1. Three Configurations of Leachate Removal Systems Containing Perforated Geopipe Systems.

Note: CCL, compacted clay liner; GM, geomembrane; GP, geopipe; and GT, geotextile.

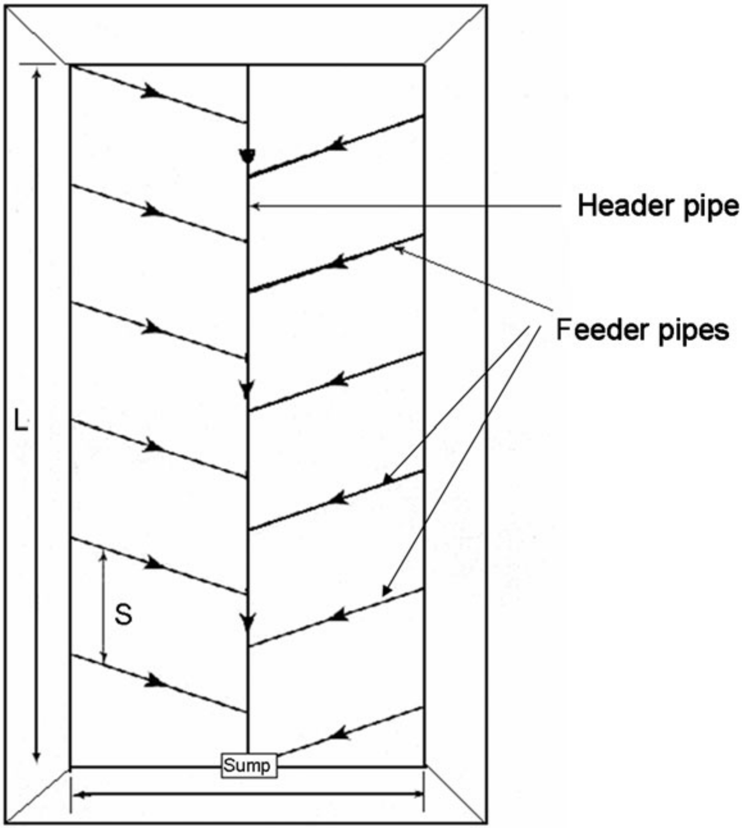


Figure 9-2. Plan View of a Possible Removal Pipe Scheme in a Primary Leachate Collection and Removal System.

Leachate collection and transmission lines in most waste containment facilities are plastic pipe (sometimes called "geopipe"); polyvinyl chloride (PVC) and high-density polyethylene (HDPE) are the two major material types in current use. Furthermore, there are two types of HDPE pipe in current use: solid-wall and corrugated types. Each of these three types of plastic pipes will be described.

9.1.1 Polyvinyl Chloride (PVC) Pipe

Polyvinyl chloride (PVC) pipe (Figure 9-3) has been used in waste containment systems for leachate collection and removal in a number of locations and configurations. The pipes are usually perforated to a particular site-specific design. The pipes are often supplied in 6.1-m (20-ft) lengths, which are joined by chemical fusion, thermal fusion, bell and spigot ends, or couplings. The PVC material typically consists of resin, fillers, carbon black or pigment, and additives. PVC pipe does not contain any plasticizers.

Figure 9-3. PVC Pipe for Use in a Landfill Leachate Collection System.

Regarding a specification or an MQA document for PVC pipe and fittings, the following items should be considered:

1. The basic resin should be made from PVC as defined in ASTM D1755. Details are contained therein.
2. Other materials in the formulation, such as fillers, carbon black or pigment, and additives, should be stipulated and certified as to the extent of their prior use in plastic pipe as leachate removal systems.
3. Clean rework material, generated from the manufacturer's own pipe or fitting production, may be used by the same manufacturer, providing that the rework material meets the above requirements.
4. Postconsumer material should not be used in any amount whatsoever.
5. Pipe tolerances and properties must meet the applicable standards for the particular grade required by the plans and specifications. For PVC pipe specified as Schedule 40, 80, and 120, the appropriate specification is ASTM D1785. For PVC pipe in the standard dimension ratio (SDR) series, the applicable specification is ASTM D2241.
6. Both of the above referenced ASTM standards have sections on product marking and identification that should be followed, as well as requiring the manufacturer to provide a certification statement stating that the applicable standard has been followed.
7. PVC pipe fittings should be in accordance with ASTM D3034. This standard includes comments on solvent cement and elastomeric gasket joints, as well as a section on product marking and certification.

9.1.2 High-Density Polyethylene (HDPE) Smooth-Wall Pipe

High-density polyethylene (HDPE) smooth-wall pipe has been used in waste containment systems for leachate collection and removal in a number of locations and configurations. The pipe is generally perforated to a particular site-specific design. The pipes are often supplied in 6.1-m (20-ft) lengths, which are generally joined together with butt-end thermal fusion using a hot plate, as is done in the natural gas pipe construction industry. The HDPE material itself consists of 97% to 98% resin, approximately 2% carbon black, and up to 1% additives. Figure 9-4 illustrates the use of nonperforated HDPE smooth-wall pipe as sideslope risers extending from sumps up into the removal shed.

The following items should be considered regarding the contract specification or MQA document on HDPE solid-wall pipe and fittings:

1. The basic material should be made of HDPE resin and should conform to the requirements of ASTM D1248. Details are contained therein.
2. QC tests on the resin are typically density (either ASTM D1505 or D792) and melt flow index (ASTM D1238). Other in-house QC tests should be encouraged and followed by the manufacturer.
3. Typical densities for HDPE pipe resins are 0.950 to 0.960 g/cm^3. This resin is a Type III HDPE resin, according to ASTM D1248, and has a higher density than the HDPE resin used in geomembranes and geonets.

Figure 9-4. HDPE Smooth-Wall Pipe Risers Used as Primary (Two Redundant Pipes) and Secondary (in Background) Removal Systems Extending from Sump Area to Removal Shed.

4. Carbon black is usually added as a concentrate, or master batch, which contains processing stabilizers and antioxidants as well. The type and amount of carbon black, as well as the type of carrier resin, should be stated and certified by the manufacturer.
5. The manufacturer should state the amount of additives used. If certification is required, it would typically not specify the type of additives because they are usually proprietary but should state that the additive package has successfully been used in the past as leachate removal systems.
6. Pipe tolerances and properties must meet the applicable standards for the particular grade required by the plans and specifications. For HDPE in the SDR series, the applicable specification is ASTM D3350. Furthermore, the particular cell limits within this specification must be identified accordingly.
7. HDPE solid-wall pipe is generally joined by thermal fusion. This method has been fully developed by the gas pipe industry. Other possible options may be designated, if approved by the designer or QCA organization (e.g., bell and spigot, screw connections, or HDPE sleeves).

9.1.3 High-Density Polyethylene (HDPE) Corrugated Pipe

Corrugated high-density polyethylene (HDPE), also called "profiled" pipe, has been used in waste containment systems for leachate collection and removal in a number of locations and configurations. The pipe can be slotted in the valleys created by the configurations, depending on the site-specific design. The inside can be smooth lined or not, depending on the site-specific design. The pipes are often supplied in 6.1-m (20-ft) lengths, which are joined together by couplings made by the same manufacturer as the pipe itself. This stipulation is important because the couplings are generally not interchangeable among different pipe manufacturer's products. The HDPE material itself consists of 97% to 98% resin, approximately 2% carbon black, and up to 1% additives. Figure 9-5 illustrates HDPE corrugated pipe, connections, and fittings.

Regarding the contract specification or MQA document on HDPE corrugated pipe and fittings, the following items should be considered:

1. The basic material should be made of HDPE resin and should conform to the requirements of ASTM D1248. Details are contained therein.
2. QC tests are typically density (ASTM D1505 or D792) and melt flow index (D1238). Other in-house QC tests are to be encouraged and followed by the manufacturer.
3. Typical densities for HDPE pipe resins are 0.950 to 0.960 g/cm^3. This resin is a Type III HDPE resin, according to ASTM D1248, and has higher density than the HDPE resin used in geomembranes and geonets.
4. Carbon black is usually added as a concentrate, or master batch, which contains processing stabilizers and antioxidants as well. The type and amount of carbon black, as well as the type of carrier resin, should be stated and certified by the manufacturer.

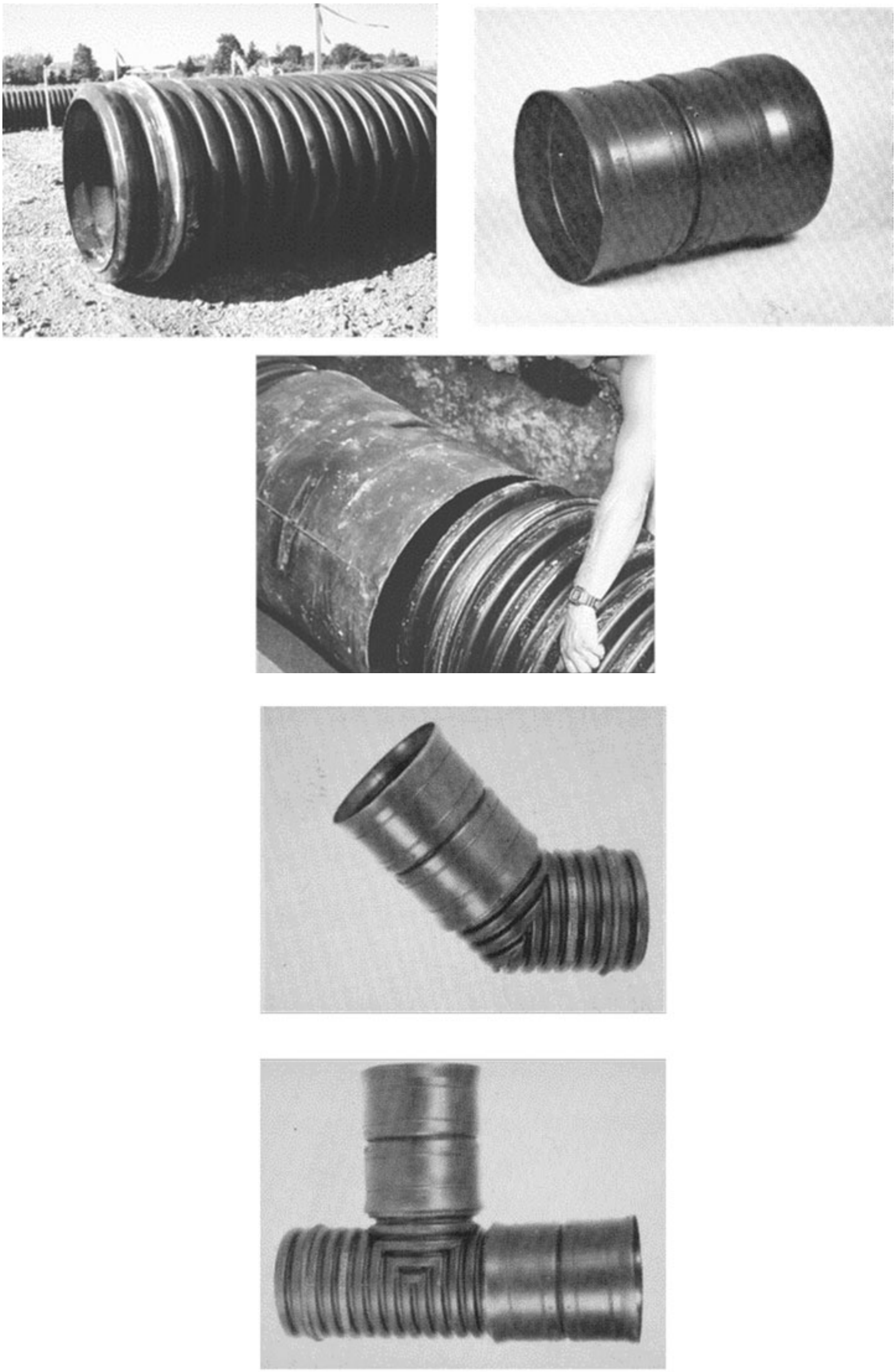


Figure 9-5. HDPE Corrugated Pipe, Connections, and Fittings Used in Solid-Waste Facilities.

5. The manufacturer should state the amount of additives used. If certification is required, it would typically not state the type of additive because they are usually proprietary but should state that the additive package has successfully been used as leachate removal systems.
6. The applicable material specification is ASTM D3350, which gives required pipe properties on the basis of cell limits. This specification should be used in conjunction with AASHTO M294 for corrugated HDPE pipe in the 300- to 900-mm (12–36-in.) diameter range or AASHTO M252 for corrugated HDPE pipe in the 75- to 250-mm (3–10-in.) diameter range.

9.1.4 Handling of Plastic Pipe

As with all other geosynthetic materials, a number of activities occur between the manufacturing of the pipe and its final positioning in a waste facility. These activities include packaging, storage at the manufacturing facility, shipping, storage at the site, conformance testing and acceptance, and placement.

9.1.4.1 Packaging

Both PVC pipe and HDPE pipe are manufactured in lengths of approximately 6.1 m (20 ft) with varying wall thicknesses and configurations. They are sometimes bundled together with plastic straps for bulk handling and shipment. The packaging is such that either forklifts or cranes using slings can be used for handling and movement. As the diameter and wall thickness increases, however, this may not be the case, and above 610-mm (24-in.) diameter, the pipes are generally handled individually.

9.1.4.2 Storage at the Manufacturing Facility

Plastic pipe can be stored at the manufacturing facility for relatively long periods with respect to other geosynthetics. However, if stored outdoors for more than 12 months, a temporary enclosure or covering should be used to protect the pipe from UV exposure and high temperatures. Indoors, there is no defined storage time limitation. Pipe fittings are usually stored in a container or plastic net.

9.1.4.3 Shipping

Plastic pipe and fillings are shipped from the manufacturer's or their representative's storage facility to the job site via common carrier. Ships, railroads, and trucks have all been used, depending on the locations of the origin and final destination. The usual carrier from within the United States is truck. When using flatbed trucks, the pipe is usually loaded by means of a forklift or a crane with slings wrapped around the entire unit. When the truck bed is closed (i.e., an enclosed trailer) the units are usually loaded by forklift. Pipes bigger than 610 mm (24 in.) in diameter are handled individually.

9.1.4.4 Storage at the Site

Off-loading of plastic pipe and fittings at the site and temporary storage are necessary follow-up tasks that must be done in an acceptable manner.

Items to be considered for the contract specification or CQA document are the following:

1. Handling of plastic pipe and fittings should be done in a competent manner such that damage does not occur to the pipe.
2. The location of field storage should not be in areas where water can accumulate. The pipe and fittings should be on level ground and oriented so as not to form a dam, creating the ponding of water.
3. The pipe sections should not be stacked more than five high. Furthermore, they should be stacked in such a way that access for conformance testing is possible.
4. Outdoor storage of plastic pipe and fittings should not be longer than 12 months. For storage periods longer than 12 months, a temporary covering should be placed over them or they should be moved to within an enclosed facility.

9.1.5 Conformance Testing and Acceptance

On delivery of the plastic pipe and fittings to the project site and temporary storage thereof, the CQA engineer should see that conformance test samples are obtained. These samples are then sent to the CQA laboratory for testing to ensure that the material supplied conforms to the project plans and specifications.

Items to consider for the contract specification or CQA document in this regard are the following:

1. The pipe should be evaluated according to its proper ASTM standard:
   (a) for PVC Schedule 40, 80, and 120, use ASTM D1785;
   (b) for PVC SDR series, use ASTM D2241;
   (c) for PVC pipe fittings, use ASTM D3034;
   (d) for HDPE SDR series, use ASTM D1248 or ASTM F714; and
   (e) for HDPE corrugated pipe and fittings, use AASHTO M294 or M252.
2. The conformance test samples should make use of the same identification system as the appropriate ASTM standard, if one is available.
3. A lot should be defined as a group of consecutively numbered pipe sections from the same manufacturing line. Other definitions are also possible (such as the total amount of pipe to be used on a specific project) and should be clearly stated in the CQA documents.
4. Sampling should be done according to the contract specification and CQA documents. Unless otherwise stated, sampling should be based on one sample per lot, but at a minimum of one sample per 300 m (1000 ft) of pipe.
5. Conformance tests at the CQA laboratory should include the following:
   (a) for PVC pipe and fittings, physical dimensions according to ASTM D2122, density according to ASTM D792, plate bearing test according to ASTM D2412, and impact resistance according to ASTM D2444;.
   (b) for HDPE solid-wall and corrugated pipe, physical dimensions according to ASTM D2122, density according to ASTM D792 or D1505, plate bearing

test according to ASTM D2412, and impact resistance according to ASTM D2444;

(c) for HDPE corrugated pipe in the 300 to 900 mm (12–36 in.) range, see AASHTO M294, and in the 75 to 250 mm (3–10 in.) range, see AASHTO M252.

6. Conformance test results should be sent to the CQA engineer before deployment of any pipe from the lot under review.
7. The CQA engineer should review the results and should report any nonconformance to the project manager.
8. The resolution of failing conformance tests should be clearly stipulated in the specifications or CQA documents.

9.1.6 Placement

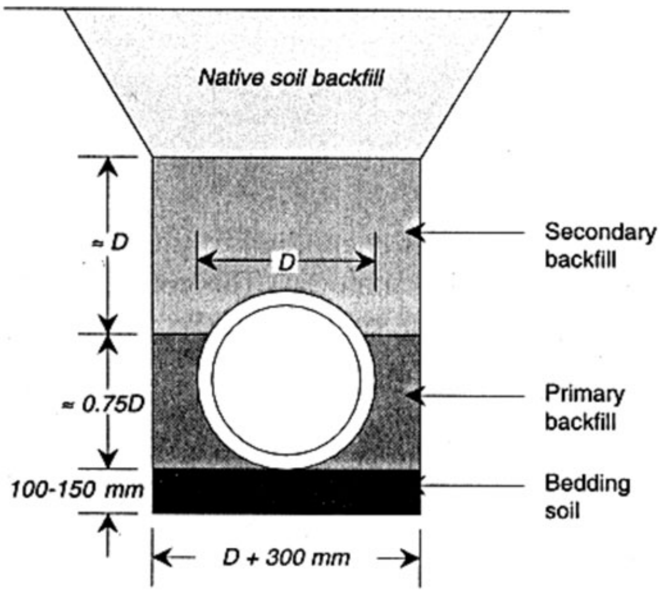
Plastic pipe is usually placed in a prepared trench or within other prepared subgrade materials. If the pipe is to be placed on or near a geomembrane, as in the leachate collection systems shown in Figure 9-1(a) and (c), the drainage sand or gravel should generally be placed first. There may be a requirement to lightly compact sand to 90% relative density, according to ASTM D4254. Shallow excavations of slightly greater dimension than the diameter of the pipe are then made, and the pipe is placed in these excavations. For a configuration as shown in Figure 9-1(b), the pipe system and surrounding gravel are placed first, and backfill is placed subsequently.

Where plastic pipe is placed at other locations adjacent to the containment facility and the soil is cohesive, compaction is critical if high stresses are to be encountered. Compaction control is necessary (e.g., 95% of standard Proctor compaction), and ASTM D698 is recommended to prevent subsidence of the pipe while it is in service.

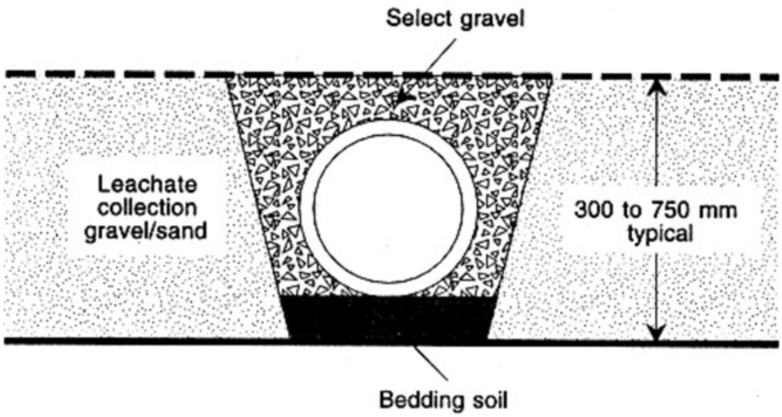
The importance of the density of the material beneath and adjacent to a plastic pipe insofar as its load-carrying capability is concerned cannot be overstated. Figure 9-6 shows the usual configuration and soil backfill terminology related to the various materials and their locations.

Regarding a specification or CQA document for plastic pipe placement, ASTM D2321 should be referenced. For waste containment facilities, the following should be considered:

1. The soil beneath, around, and above the pipe shall be Class IA, IB, or II, according to ASTM D2321.
2. The backfill soil should extend a minimum of one pipe diameter above the pipe, or 300 mm (12 in.), whichever is smaller.
3. Other conditions should be taken directly according to ASTM D2321.
4. Pipe fittings should be placed in accordance with the specification or CQA document. If not addressed, the specific pipe manufacturer's recommendations should be used.



(a) An excavated trench



(b) A leachate collection situation (see also Figure 9-1)

Figure 9-6. Two Buried-Pipe-Trench Cross-Sectional Schemes Showing Soil Backfill Terminology and Approximate Dimensions.

9.2 Sumps, Manholes, and Risers

Leachate that moves over the primary geomembrane at the bottom of landfills and waste piles flows gravitationally to a low point in the facility or cell, where it is collected in a sump. Three general variations exist. The first is a traditional sump (made either in situ or factory-prefabricated) with a pipe extension rising vertically through the waste and penetrating the final cover. The second is a sump (made either in situ or factory-prefabricated) with a gravity pipeline penetrating the liner system and extending out of the facility. The third is a shallow sump area formed in the liner itself with a pipe riser coming up the sideslope, where it eventually penetrates the final cover. These three variations are shown schematically in Figure 9-7. Each type of system will be briefly described.

Many existing landfills have been constructed with primary leachate collection and removal sumps and manholes constructed to the site-specific plans and specifications, as shown in Figure 9-7(a). The vertical riser is either a concrete or plastic pipe placed in 3-m (10-ft) sections. It is extended as the waste is placed, and eventually it must penetrate the final cover. Leachate is removed from this manhole using a submersible pump that is permanently located in the sump.

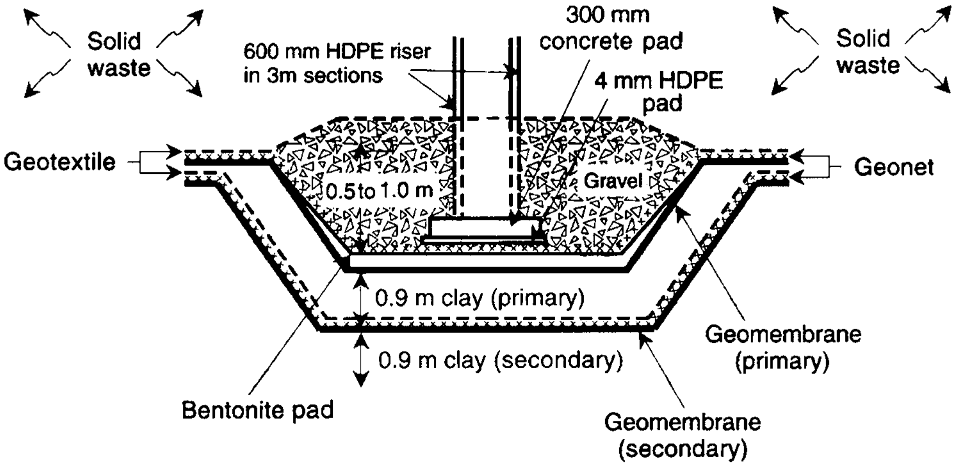
As shown in Figure 9-7(b), a sump can be located at the low point in the facility or cell from which leachate flows gravitationally out of the landfill area. In this case, the existing pipe must penetrate the liner, and in the case of double-lined facilities, both liners consist of geomembranes, perhaps GCLs and CCLs as well. Such penetrations and their backfilling are critical to the success of this type of leachate removal variation.

Quite different from the above deep sumps for primary leachate removal is a relatively large shallow area in the primary geomembrane into which the leachate collection pipe network flows. This low area creates a shallow sump, which is then filled with crushed stone and from which a pipe riser extends up the sideslope. The pipe riser is usually a solid-wall pipe with no perforations. When the facility is eventually filled with solid waste, the riser must penetrate the cover, as shown in Figure 9-7(c). (See also Figure 9-4, which illustrates this type of system). The leachate is withdrawn using a submersible pump that is lowered down the pipe riser on a sled and left in place except for maintenance or replacement.

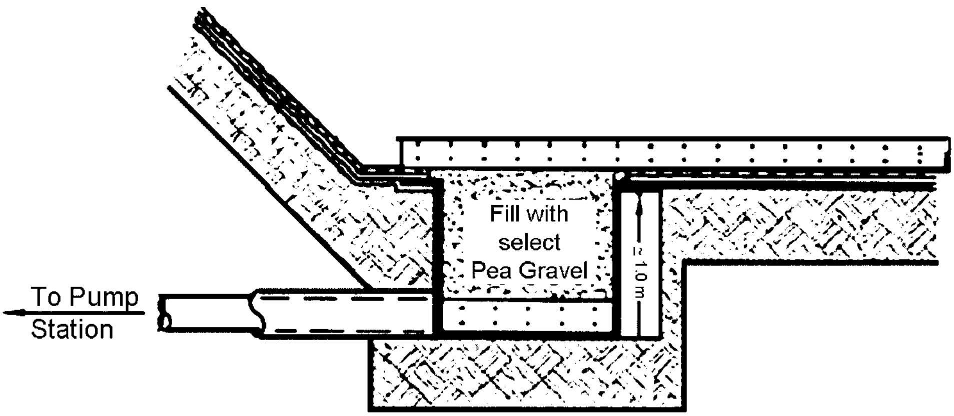
For each of these sump variations with double-lined facilities, a separate sump must be adjacent to those shown, but beneath the primary liner system, for leak detection and monitoring. This requirement becomes complicated for manholes and penetrations such as those illustrated in Figs. 9-7(a) and (b). Double-lined landfill systems favor sidewall risers, as illustrated in Figure 9-7(c). A small-diameter riser extends between the two liner systems, penetrates the primary geomembrane at the top of the slope, and extends into the removal and monitoring shed. As seen in Figure 9-8, this type of sidewall riser system is quite common.

Some specification and CQA document considerations for the various sump, manhole, and riser schemes just described are as follows. Note, however, that other possible design schemes are also available.

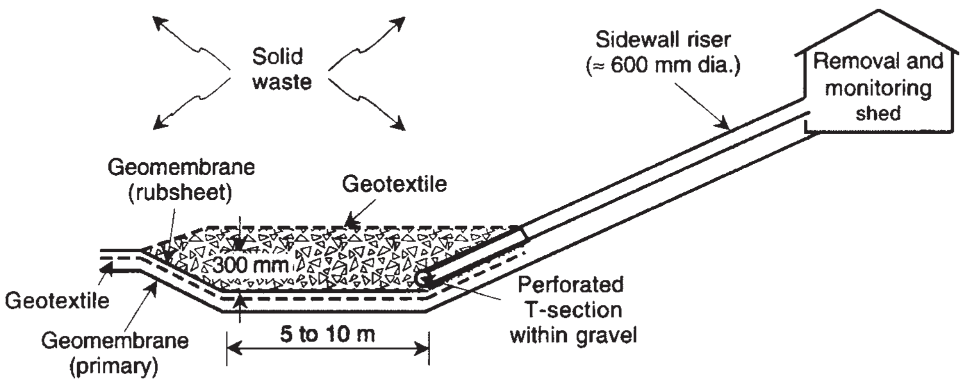
1. In situ fabrication of sumps requires a considerable amount of hand labor in the field. Seams for HDPE, LLDPE, and fPP geomembranes are extrusion fillet welded, whereas PVC and CSPE-R geomembranes are usually bodied chemical seams (U.S. EPA 1991). Careful visual inspection is necessary.



(a) A vertical sump riser through waste and cover



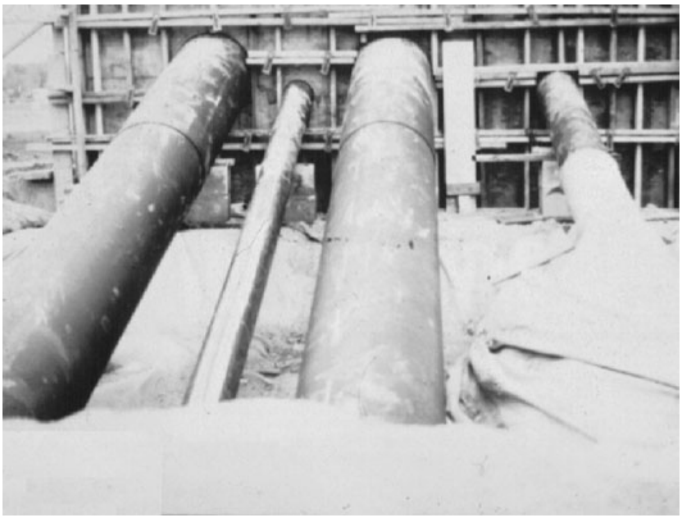
(b) A horizontal gravity-flow pipe through liner(s)



(c) Sidewall riser along slope through cover

Figure 9-7. Three Schemes for Leachate Removal from Landfill Sumps.

(a) A shallow sump area with redundant leachate removal pipes



(b) Large redundant sidewall riser pipes extending from leachate collection sump and a sidewall riser pipe (right side) extending from a leak detection sump. The small pipe between the larger pipes is for a header cleanout system.

Figure 9-8. Sidewall Riser Illustrations for Double-Lined Landfills.

2. Prefabricated factory sumps should be encouraged whenever possible. The excavation in which they are placed must be carefully inspected. Voids beneath and adjacent to the sumps must be carefully backfilled. The specification should reference ASTM D2321 with only backfill Types IA, IB, and II considered. Consideration should be given to "flowable" grout backfill materials. This requirement is particularly the case with horizontal gravity pipes, as shown in Figure 9-7(b).

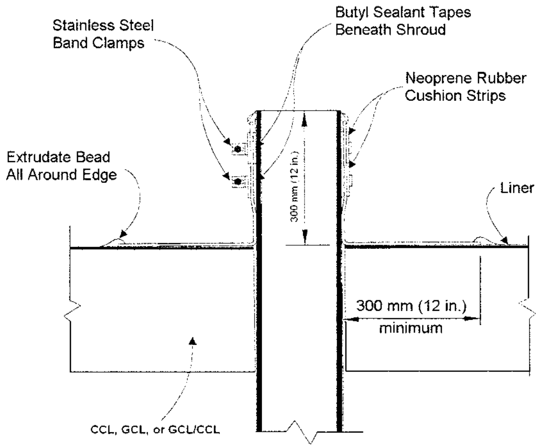
3. All of the removal pipes exiting the sumps shown in Figure 9-7 are solid-wall pipes. Generally they are HDPE with relatively low SDR-values (i.e., thick-walled pipes). The plans and specifications must be explicit in this regard.
4. Riser pipe joints and connections for primary and secondary leachate removal require special visual attention because neither destructive nor nondestructive tests can usually be accommodated.
5. The sump, manholes, and risers must be documented by the CQA engineer before acceptance and placement of solid waste.

9.3 Liner System Penetrations

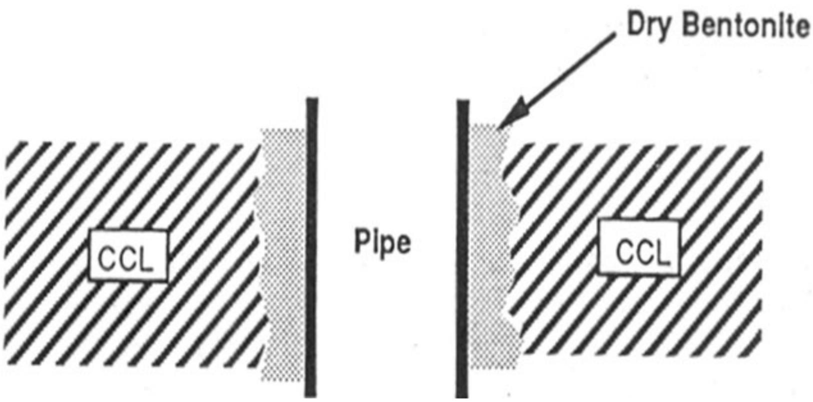
Although the intention of most designers of waste containment facilities is to avoid liner penetrations, leachate removal is inevitably required at some locations of the barrier system. Recall Figure 9-7, where the final cover or the liner must necessarily be penetrated for primary (and secondary) leachate removal. It should also be recognized that the penetrations will include geomembranes, compacted clay liners, and geosynthetic clay liners. Figure 9-9 illustrates some details of pipe penetrations through all three types of barrier materials.

The following recommendations are made for a specification or CQA document:

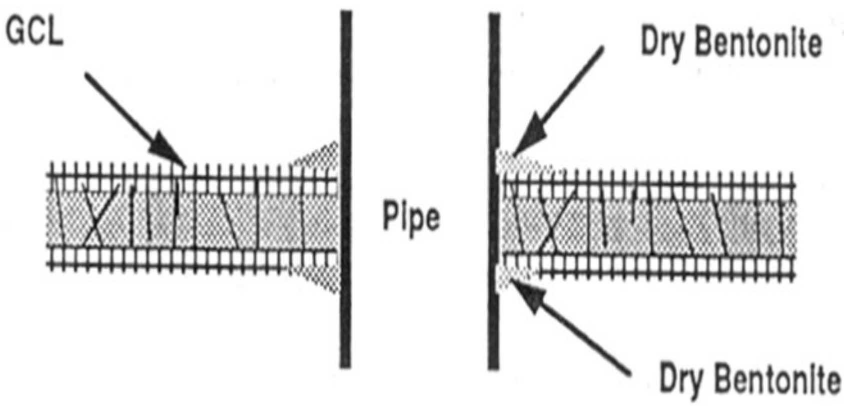
1. Geomembrane pipe boots are usually factory-fabricated to a size that tightly fits around the outside diameter of the penetrating pipes. Unique situations, however, will require field fabrication (e.g., when pipe penetration angles are unknown until final installation).
2. The skirt of the pipe boot, which flares away from the pipe penetration, should have at least 300 mm (12 in.) of geomembrane on all sides of the pipe.
3. The skirt of the pipe boot should be seamed to the base geomembrane by extrusion fillet or bodied solvent seaming, depending on the type of geomembrane (U.S. EPA 1991).
4. The nondestructive testing of the skirt of the pipe boot should be by vacuum chamber, air lance, or electric wire methods, depending on the type of geomembrane. (Refer to Table 4-7.)
5. The pipe boot should be of the same type of geomembrane as that of the liner through which the penetration is being made.
6. Pipe penetrations should be positioned with sufficient clearance to allow for proper welding and inspection.
7. Stainless steel pipe clamps used to attach pipe boots to the penetrating pipes should be of an adequate size to allow for a cushion of compressible "gasket" material (often neoprene rubber) to be placed between the inside surface of the clamp and that of the geomembrane portion of the pipe boot. Gasket material should be stipulated on the plans or specifications, and the entire assembly should be approved by the CQA personnel.
8. Location of pipe clamps should be as directed on the plans and specifications.



(a) Geomembrane penetration



(b) Compacted clay liner (CCL) penetration



(c) Geosynthetic clay liner (GCL) penetration

Figure 9-9. Pipe Penetrations through Three Types of Barrier Materials.

9. Pipe penetrations through compacted clay liners and geosynthetic clay liners should use an excess of hand-placed dry bentonite clay or a bentonite clay paste, as directed in the plans and specifications.
10. Backfilling of pipe penetrations should be delayed until backfilling of adjacent liner materials is complete. This is particularly the case for geomembrane pipe penetrations on sloping surfaces because adjacent material being backfilled often generates shear stresses, which cause deformations around the stationary penetrating pipe. If deformations occur, the connection must be adjusted or repaired to the approval of the CQA personnel.

9.4 Anchor Trenches

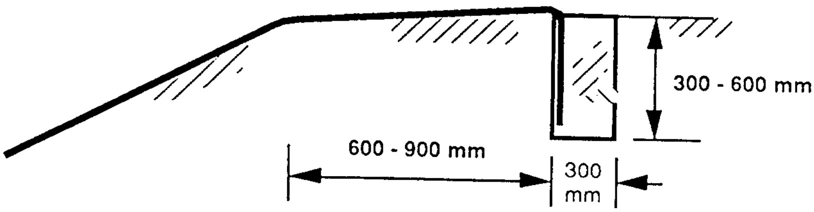
Generally, the geosynthetics used to line or cover a waste facility terminate in an anchor trench around the individual cell or around the entire site.

9.4.1 Geomembranes

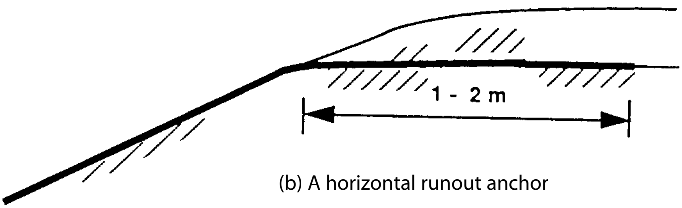
The termination of a geomembrane at the perimeter of the site is generally in an anchor trench. As shown in Figure 9-10, the variations are numerous. One aspect of the design that should be considered is rounding the edges, and particularly the corners, of anchor trenches. Thick, stiff geomembranes are difficult to conform to abrupt shape changes. Such details should be specifically addressed in the construction plans and specifications.

Some general items that should be addressed in the specification or CQA document regarding geomembrane termination in anchor trenches are as follows:

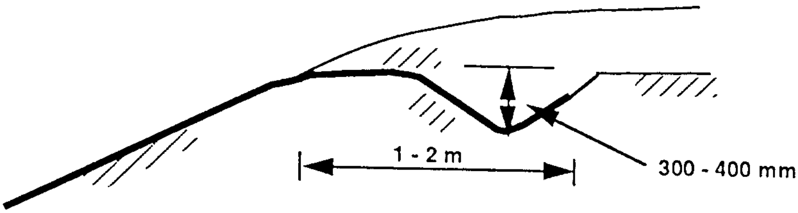
1. The seams of adjacent sheets of geomembranes should be continuous into the anchor trench to the full extent indicated in the plans and specifications.
2. Seaming of geomembranes within the anchor trench can be accomplished by temporarily supporting the adjacent sheets to be seamed on a wooden support platform so that horizontal seaming can be accomplished continuously to the end of the geomembrane sheets. The temporary support is removed after the seam is complete, and the geomembrane is then allowed to drop into the anchor trench.
3. Nondestructive tests can be performed while the seamed geomembrane is temporarily supported in the horizontal position.
4. Destructive seam samples can also be taken while the seamed geomembrane is temporarily supported in the horizontal position.
5. The anchor trench is generally backfilled after the geomembrane has been documented by the CQA engineer, but backfilling may be done later, depending on the particular site-specific conditions.
6. The anchor trench itself should be made with rounded edges and corners to avoid sharp bends in the geomembrane. Loose soil should not be allowed to underlie the geomembrane in the anchor trench.



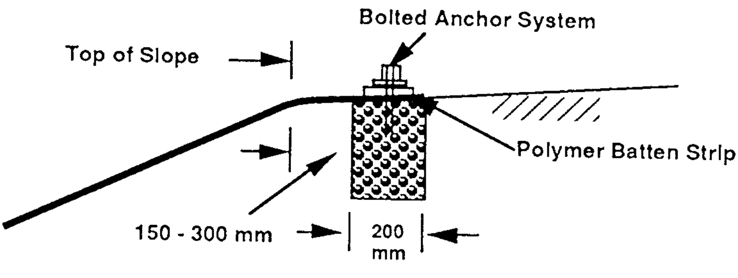
(a) A typical anchor trench



(b) A horizontal runout anchor



(c) A shallow V Anchor trench



(d) A concrete anchor block

Figure 9-10. Various Types of Geomembrane Anchor Trenches. Dimensions should be varied for site-specific conditions.

7. The anchor trench should be adequately sloped and drained to prevent ponding of water or softening of the adjacent soils while the trench is open.
8. Backfilling in the anchor trench should be accomplished with approved backfill soils placed at their required moisture content and compacted to the required density, per the plans and specifications.
9. The plans and specifications should provide detailed construction requirements for anchor trenches, regardless if soils or other backfill materials are used.

9.4.2 Other Geosynthetics

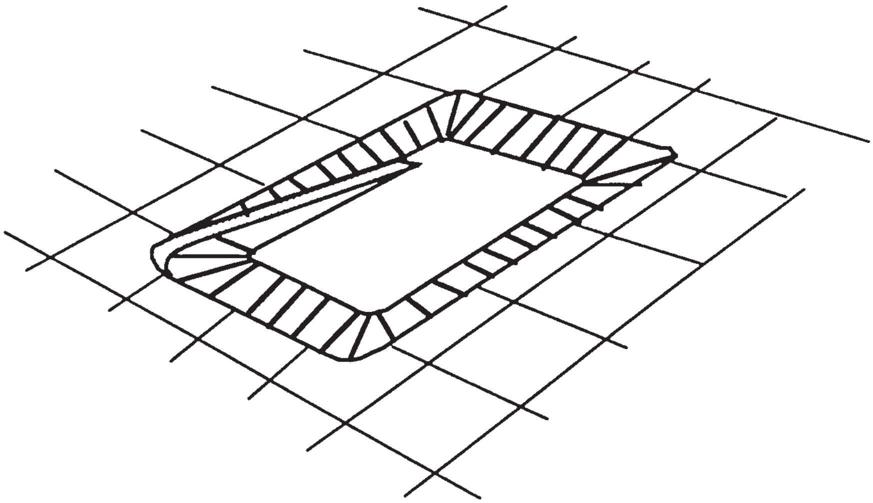
Because all geosynthetics, not only geomembranes, need adequate termination, some additional comments are offered in this regard for plans, specifications, or CQA documents.

1. Geotextiles, either beneath or above geomembranes, usually follow their associated geomembrane into the same anchor trenches.
2. Geonets, as well as geonet composites, may or may not terminate in the anchor trench. Water transmission from beyond the waste containment may be a concern when requiring termination of the geonet within the geomembrane's anchor trench or in a separate trench by itself. Thus termination of a geonet may be short of the associated geomembrane's anchor trench. This is obviously a design issue and must be clearly detailed in the contract plans and specifications.
3. When used by themselves, geosynthetic clay liners (GCLs) will generally terminate in an anchor trench of one of the types shown in Figure 9-10. When GCLs are with an associated geomembrane, as in a composite liner, each component will sometimes end in a separate anchor trench. These are design decisions and must be followed accordingly.
4. Double-liner systems will generally have separate anchor trenches for primary and secondary liner systems; however, this is also a design decision.
5. In all of the above cases, the plans and specifications should provide detailed dimensions and construction requirements for anchor trenches of all geosynthetic components.
6. The plans and specifications should also show details of how natural soil components (e.g., compacted clay liners and sand or gravel drainage layers) terminate with respect to one another and with respect to the associated geosynthetic components.

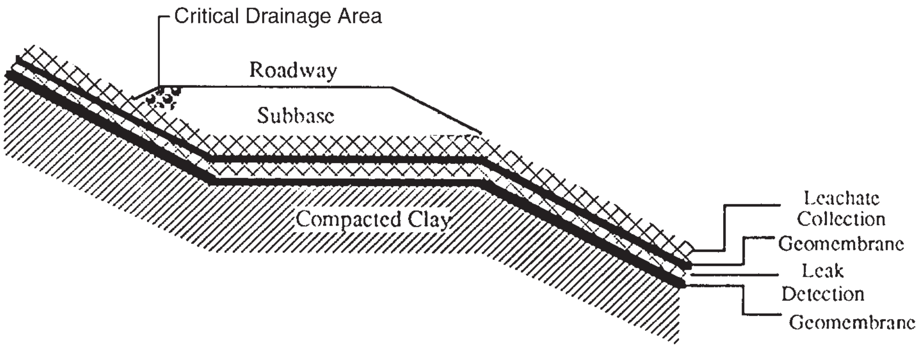
9.5 Access Ramps

Heavily loaded vehicles and construction equipment must enter the landfill facility during construction activities and during placement of the solid waste. Typical access ramps will be up to 5.5 m (18 ft.) wide and have grades up to 12%. The general geometry and possible cross section of an access ramp for a below-grade landfill is shown in Figure 9-11.

The traffic loads on such a ramp can be extremely large and generally involve some degree of dynamic force because of the constant braking action that drivers use when descending the steep grades. The entire liner cross section must extend uninterrupted from the lower slope to the upper slope, and in doing so must necessarily pass beneath the roadway base course. When working with a double-lined facility, this cross section can involve numerous geosynthetic and natural soil layers. Further complicating the access ramp design issue is the fact that drainage from the upper sideslopes must either communicate beneath the roadway base course layer or travel parallel to it and be directed accordingly. A reinforcing



(a) The geometry of a typical ramp



(b) A cross section of the ramp roadway

Figure 9-11. Typical Access Ramp Geometry and Cross Section.

materal (geotextile or geogrid) can be incorporated in the roadway base course material. This layer can serve several purposes: to protect long-term integrity of underlying systems, to minimize potential sliding failures, to minimize potential rutting, and to prevent bearing capacity failures. These are important design issues and must be clearly defined in the plans and specifications.

Regarding recommendations for the contract specifications or CQA document, the following items apply:

1. Many facilities will limit the number of vehicles on the access ramp at a given time. Such stipulations should be strictly enforced.
2. Vehicle speeds on access ramps should be strictly enforced.
3. Regular inspection should be required to observe if tension cracks open in the roadway base coarse soils. Cracks may indicate some degree of slippage of the soil and possible damage to the liner system.

4. Ponding of upper slope runoff water against the roadway profile should be observed for possible erosion effects and loss of base course material. If a drainage ditch or pipe system is indicated on the plans, it should be constructed as soon as possible after completion of the roadway subbase soils.
5. The roadway base course profile should be fully maintained for the active lifetime of the facility. Such base course material can be carefully removed as waste is placed in the facility.

9.6 Geosynthetic Reinforcement Materials

For landfill and waste pile final covers with slopes greater than 4 horizontal to 1 vertical (4H:1V), stability issues regarding downgradient sliding begin to be important. Additionally, the stability of primary leachate collection systems for landfill and waste pile liners with steep slopes is suspect at least until the solid waste within the landfill raises to a stabilizing level. Such issues, of course, must be considered during the design phase, and the contract plans and specifications must be clear on the method of reinforcement, if any. If reinforcement is necessary, it can be accomplished by using geogrids or high-strength geotextiles within the layer contributing to the instability to offset some, or even all, of the gravitational stresses. Refer to Figs. 9-12(a) and (b) for the general orientation of such reinforcement, which is sometimes called "veneer reinforcement."

The concept of using geogrid or geotextile reinforcement to support a liner or liner system when a new landfill is built above or adjacent to an existing landfill is sometimes practiced. The technique has been referred to as "piggybacking" when vertical or lateral expansions are involved (Figure 9-12(c)). The main focus of the reinforcement is to provide resistance against differential settlement that can occur in the existing landfill.

Geogrids will be described from both their manufacturing and reinforcement perspectives. Because separation and filtration geotextiles were described previously from a manufacturing standpoint, only reinforcement geotextiles will be discussed here.

9.6.1 Geogrids

Geogrids are reinforcement geosynthetics formed by intersecting and joining sets of longitudinal and transverse ribs with resulting open spaces called "apertures." Different classes of geogrids are currently available, see Figure 9-13(a). They are characterized as follows: (1) stiff, unitized geogrids made from polyethylene or polypropylene sheet material that is stretched into a postyield state during manufacturing; (2) flexible, textilelike geogrids made from high-tenacity polyester yarns that are joined at their intersections and coated with a polymer or bitumen, and (3) stiff, high-tenacity polyester or polypropylene straps that are laser or ultrasonically bonded at their junctions. Figure 9-13(b) shows geogrids being used as veneer reinforcement.

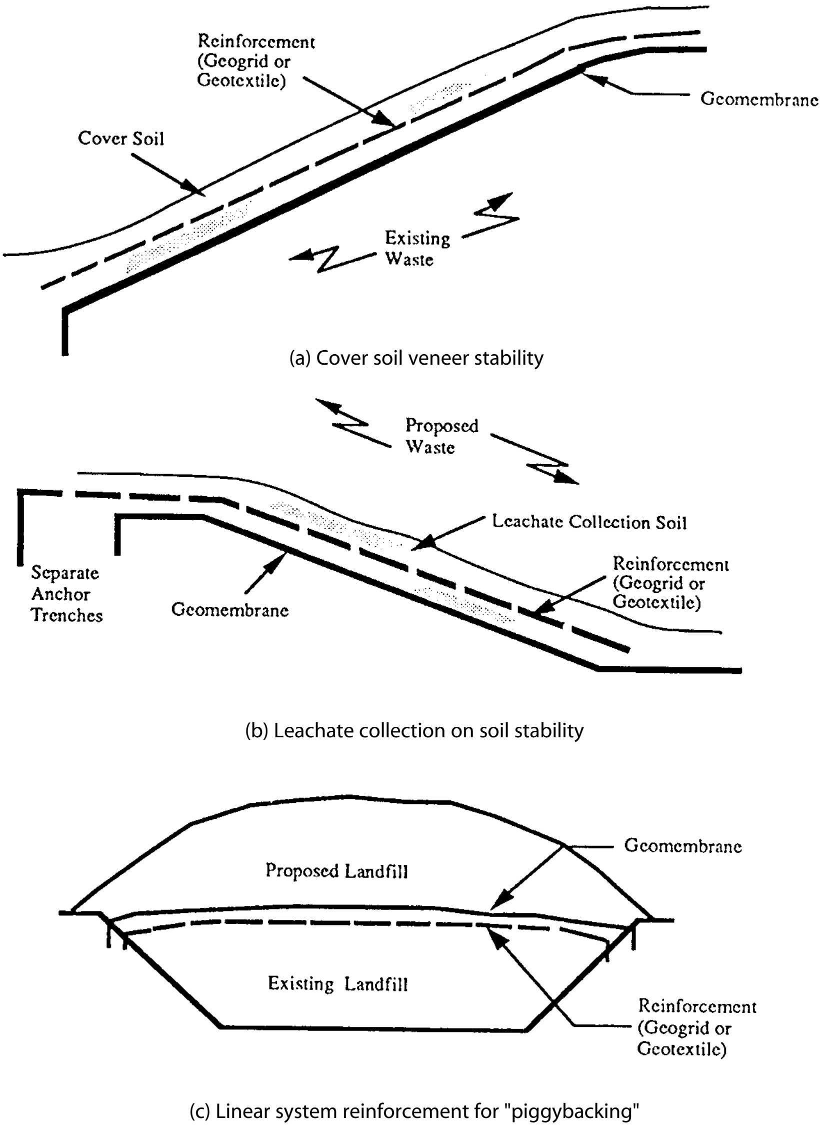
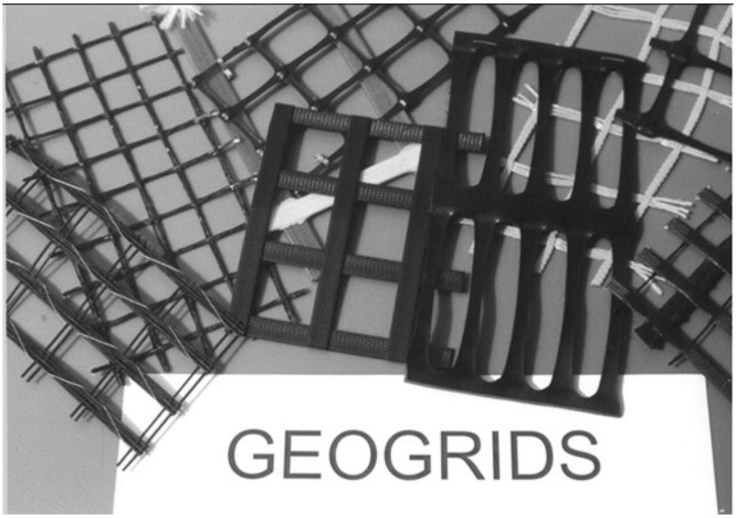


Figure 9-12. Geogrid or Geotextile Reinforcement.

Some recommended contract specification or CQA document items that should be addressed when using geogrids as reinforcement materials are as follows:

1. A manufacturer's certification should be provided that the geogrid meets the property criteria specified for project, per the plans and specifications.
2. CQA personnel should check that the geogrid delivered to the job site is the proper and intended material. This check is done by verifying the identification label and its coding and by identification of the product, its rib joining, thickness, and aperture size. If the geogrid has a primary strength direction, it must be indicated.



(a) Various types of geogrids

(b) Geogrids used as veneer reinforcement. Note that backfilling is proceeding from the bottom to the top of the slope.

Figure 9-13. Photographs of Geogrids Used as Soil (or Waste) Reinforcement Materials.

3. Conformance samples of the geogrid supplied to the job site should be obtained, per ASTM D4759. Typically, the outer wrap of the rolls is used for such sampling.
4. Conformance tests should be the following: aperture size by micrometer or caliper measurement, rib thickness and junction thickness (per ASTM D1777), and single- or multiple-rib tensile strength (per ASTM D6637). Additional

conformance tests that may be considered are polymer identification via thermal analysis methods and rib junction strength (per GRI-GG2).

5. Field placement of geogrids should be at the locations indicated on the contract plans and in the specifications. Details of overlapping or seaming should be included.
6. Geogrid placement (not backfilling) is usually from the top of the slope downward so that the geogrid is taut before soil backfilling proceeds.
7. The upper end of the geogrids must be terminated in an anchor trench. In this regard, the details shown in the contract plans should be fulfilled.
8. Soil backfilling should proceed from the bottom (or toe) of the slope upward, with a minimum backfill thickness of 22 cm (9.0 in.) of cover using light ground contact construction equipment of 40 kPa (6 lb/in.^{2}) contact pressure or less.
9. Connections of the ends of geogrid rolls on sideslopes should generally be avoided. If permitted, they should be located as close to the bottom of the slope as possible. Connections should be as approved by the CQA engineer. Test strips of connections should be requested for conformance tests in the CQA laboratory, following the ASTM D4884 (modified) test method.

9.6.2 Reinforcement Geotextiles

The manufacturing of geotextiles was described in Section 7.2, along with recommendations for MQC and MQA documents. Regarding CQC and CQA, the focus was on separation and filtration applications. Some specific recommendations regarding reinforcement geotextiles (which are usually heavy woven fabrics) for a specification or CQA document are as follows:

1. A manufacturer's certification should be provided that the geotextile meets the property criteria specified for use on the project via the plans and specifications.
2. CQA personnel should check that the geotextile delivered to the job site is the proper and intended material. This check is done by verifying the identification label and its coding and by visual identification of the product, its pattern and style, and other visual details.
3. Conformance samples of the geotextile supplied to the job site should be obtained, per ASTM D4759. Typically, the outer wrap of the rolls is used for such sampling.
4. Conformance tests should be the following: wide tensile strength (per ASTM D4595), trapezoidal tear strength (per ASTM D4533), and puncture strength (per ASTM D4833). Additional conformance tests that may be considered are polymer identification via thermogravimetric analysis and seam strength (per ASTM D4884).
5. Field placement of geotextiles should be at the locations indicated on the contract plans and in the specifications. Details of overlapping or seaming should be included.

6. Geotextile deployment is usually from the top of the slope downward, so that the geotextile is taut before soil backfilling proceeds.
7. The upper end of the geotextile must be terminated in an anchor trench. The details shown in the contract plans should be fulfilled.
8. Soil backfilling should proceed from the bottom of the slope upward, with a minimum backfill thickness of 220 mm (9 in.) of cover using light ground contact construction equipment of 40 kPa (6 lb/in.^{2}) contact pressure or less.
9. Seams in geotextiles on sideslopes are generally not allowed. If permitted, they should be located as close to the bottom of the slope as possible. Seams should be as approved by the CQA engineer. Test strips of seams should be requested for conformance tests in the CQA laboratory, following ASTM D4884.

9.7 Erosion-Control Materials

Often, on sloping solid-waste landfill covers, soil loss from rainfall (in the form of rill, gully, or sheet erosion) occurs in the topsoil and sometimes extends down into the cover soil. This problem requires continuous maintenance until the phenomenon is halted and long-term vegetative growth is established. Alternatively, the design may call for a temporary or permanent erosion-control material to be deployed within or on top of the topsoil layer. Additional concerns regarding erosion control are on perimeter trenches, drainage ditches, and other surface-water control structures associated with waste containment facilities. Listed below are a number of alternative erosion-control systems, ranging from traditional hand-distributed mulching materials to fully paved cover systems. They fall into two major groups: temporary degradable and permanent nondegradable; the latter has two subgroups:

Temporary Erosion-Control and Revegetation Mats (TERMs):

- mulches (hand- or machine-applied straw or hay);
- mulches (hydraulically applied wood fibers or recycled paper);
- jute meshes;
- fiber-filled containment meshes;
- woven geotextile erosion-control meshes; and
- fiber roving systems (continuous fiber systems).

Permanent Erosion-Control and Revegetation Mats (PERMs):

(a) Geosynthetic systems:
  - turf reinforcement and revegetation mats (TRMs);
  - erosion-control and revegetation mats (ECRMs);
  - geomatting systems; and
  - geocellular containment systems.

(b) Hard armor systems:
  - cobbles, with an underlying geotextile filter or separator;
  - rock riprap, with an underlying geotextile filter or separator;
  - articulated concrete block mattresses, with an underlying geotextile filter or separator;

- grout injected between geotextiles, forming mattresses; and
- partially or fully paved systems.

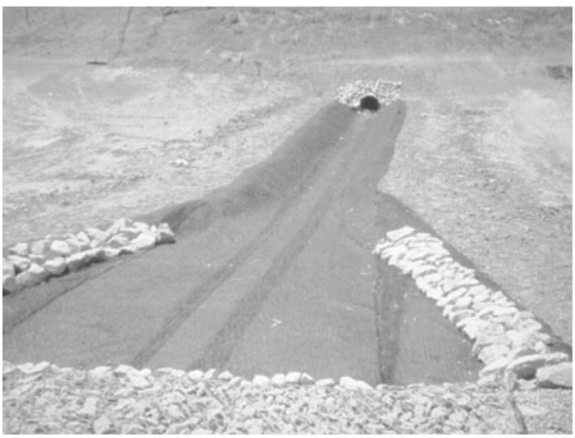
Regarding these three groups of erosion-control materials, temporary degradable systems are used to enhance the establishment of vegetation and then degrade, leaving the vegetation to provide the erosion protection required. Challenging sites that require protection above and beyond what vegetation can provide need to use a permanent and nondegradable system (e.g., high-flow channels or oversteepened slopes). Even further, steep let-down channels and waste slopes adjacent to running water might require some type of hard armor system. Figure 9-14 shows these three groups of erosion-control systems.

Some items that are recommended for contract specifications or CQA document for erosion-control systems are as follows:

1. The CQA personnel should check the erosion-control material on delivery to see that the proper materials have been received. Because these are UV-exposed materials, their durability under laboratory simulation conditions in a weatherometer is important. Various exposure devices are covered in ASTM D4355, G151, and G154.
2. Water- and UV-sensitive materials should be stored in dry conditions and protected from sunlight.
3. If the erosion-control material has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage, it should be rejected or suitably repaired to the satisfaction of the CQA personnel.
4. If the material is to be repaired, torn or punctured sections should be removed by cutting a section of the material out and replacing it with a section of undamaged material. The ends of the new section should overlap the damaged section by 30 cm (12 in.) and should be secured with proper anchors.
5. All ground surfaces should be prepared so that the material lies flat with complete intimate contact against the underlying soil.
6. Ground anchors, called "pins," should be at least 30 cm (12 in.) long with an attached oversized washer 50 mm (2.0 in.) in diameter, or "staples" made of No. 8 gauge U-shaped wire at least 20 cm (8.0 in.) long. For less severe temporary applications (e.g., TERMSs), one may consider 15 cm (6 in.) No. 11 gauge U-shaped wire staples.
7. Adjacent rolls of erosion-control material shall be overlapped a minimum of 75 mm (3.0 in.). Staples should secure the overlaps at 75-cm (2.5-ft.) intervals. The roll ends should overlap a minimum of 45 cm (18 in.) and be shingled downgradient. The end overlaps should be stapled at 45-cm (1.5-ft) intervals or closer, or as recommended by the manufacturer.
8. If required on the plans and specifications, the erosion-control material should be filled with topsoil and lightly raked or brushed into the mat to either fill it completely or to a maximum depth of 25 mm (1.0 in.).
9. For geosynthetic materials used in drainage ditches, the overlaps should always be shingled downgradient with overlap as recommended by the manufacturer or on the plans and specifications.

(a) Woven jute as a TERM

(b) Three-dimensional polymer mesh as a PERM (geosynthetic)



(c) Stone-filled gabions as a PERM (hard armor).

Figure 9-14. Three Erosion-Control Systems.

10. If required by the plans and specifications, the manufacturer of the erosion-control or drainage ditch material should provide a qualified and experienced representative on site to assist the installation contractor at the start of construction. After an acceptable routine is established, the representative should be available on an as-needed basis, at the CQA engineer's request.

9.8 References

AASHTO M252-90. "Specification for corrugated polyethylene drainage tubing."

AASHTO M294-90. "Specification for corrugated polyethylene pipe, 12- to 36-in. diameter."

ASTM D698. "Test method for moisture density relations of soils and soil/aggregate mixtures."

ASTM D792. "Test method for specific gravity and density of plastics by displacement."

ASTM D1238. "Test method for flow rates of thermoplastics by extrusion plastomer."

ASTM D1248. "Specification for polyethylene plastics and extrusion materials."

ASTM D1505. "Test method for density of plastics by the density–gradient technique."

ASTM D1755. "Specification for poly(vinyl chloride) (PVC) resins."

ASTM D1777. "Test method for measuring thickness of textile materials."

ASTM D1785. "Specification for poly(vinyl chloride) (PVC) plastic pipe, Schedules 40, 80, and 120."

ASTM D2122. "Test method for determining dimensions of thermoplastic pipe and fittings."

ASTM D2241. "Specification for poly(vinyl chloride) (PVC) pressure rated pipe (SDR series)."

ASTM D2321. "Practice for underground installation of thermoplastic pipe for sewers and other gravity-flow applications."

ASTM D2412. "Test method for external loading properties of plastic pipe by parallel plate loading."

ASTM D2444. "Test method for impact resistance of thermoplastic pipe and fittings by means of a tup (falling weight)."

ASTM D3034. "Specification for type PSM poly(vinyl chloride) (PVC) sewer pipe and fittings."

ASTM D3350. "Specification for polyethylene plastics pipe and fittings materials."

ASTM D4254. "Test method for maximum index density of soils and calculation of relative density."

ASTM D4355. "Test method for deterioration of geotextiles from exposure to ultraviolet light and water (xenon-arc type apparatus)."

ASTM D4533. "Test method for trapezoidal tearing strength of geotextiles."

ASTM D4595. "Test method for tensile properties of geotextiles by wide width strip method."

ASTM D4632. "Test method for breaking load and elongation of geotextiles (grab method)."

ASTM D4759. "Practice for determining the specification conformance of geosynthetics."

ASTM D4833. "Test method for index puncture resistance of geotextiles, geomembranes and related products."

ASTM D4884. "Test method for seam strength of sewn geotextiles."

ASTM D6637. "Test method for determining tensile properties of geogrids by the single or multi-rib tensile method."

ASTM F714. "Specification for polyethylene (PE) plastic pipe (SDR-PR) based on outside diameter."

ASTM G151. "Standard practice for exposing nonmetallic materials in accredited test devices that use laboratory light sources."

ASTM G154. "Standard practice for operating fluorescent light apparatus for UV exposure of nonmetallic materials."

GRI-GG2. "Test method for individual geogrid junction strength."

U.S. EPA (U.S. Environmental Protection Agency). (1991). "Inspection techniques for the fabrication of geomembrane field seams," U.S. Environmental Protection Agency, Cincinnati, Ohio, Technical Resource Document, EPA/530/SW–91/051.

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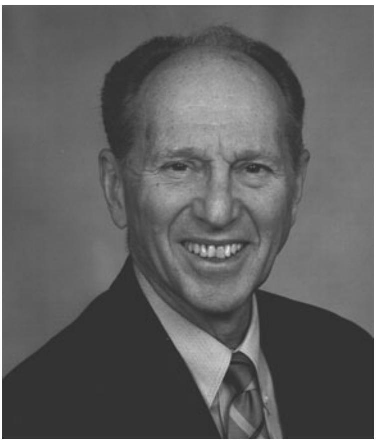
Note: Entries from figures are followed by *f*; tables are followed by *t*

About the Authors

David E. Daniel, Ph.D., P.E., is serving as the fourth president of University of Texas at Dallas. He received his B.S., M.S., and Ph.D. degrees in engineering from the University of Texas at Austin and served on the faculty there from 1980 to 1996. In 1996, he moved to the University of Illinois, finishing his service there as Dean of Engineering in 2005. Daniel's professional work has focused on environmental controls for contaminated land and groundwater. In addition to extensive teaching experience, he has published more than 100 technical articles and written or edited five books. He was awarded the Norman Medal in 1975 and the Croes Medal in 1984 and 2000, both of which are given by ASCE to recognize excellence in journal articles. In 2000, Daniel was elected to the National Academy of Engineering, and in 2005 and 2006 he served as chair of ASCE's External Review Panel, which was charged with a review of the government's work in identifying facts pertaining to the performance of New Orleans's levees during Hurricane Katrina.



Robert M. Koerner, Ph.D., P.E., is emeritus professor of civil engineering at Drexel University and founder and director of the Geosynthetic Research Institute/Geosynthetic Institute. He received his B.S. and M.S. degrees in civil engineering from Drexel and completed his Ph.D. in geotechnical engineering at Duke University. After 10 years as a practicing engineer, Koerner joined Drexel's engineering faculty in 1968 and has been teaching, writing, and consulting ever since. He was coauthor of the first book on geotextiles and is the author of a successful textbook on geosynthetics, now in its fifth edition. In addition, he is the author or coauthor of nearly 600 journal articles, proceedings papers, and books on the subject of geosynthetics and geotechnical engineering. As director of the Geosynthetic Institute, Koerner focuses on all aspects of waste disposal, particularly liner and cover containment systems. He delivered the 32nd Terzaghi Lecture in 1996 and received (with Daniel) ASCE's Croes Medal in 2000. In 1992, Koerner was named an Honorary Member of ASCE, and in 1998, he was elected to the National Academy of Engineering.

Daniel and Koerner are also the authors of *Final Covers for Solid Waste Landfills and Abandoned Dumps* (ASCE Press and Thomas Telford, 1997).