United States
Environmental Protection
Agency

Risk Reduction
Engineering Laboratory
Cincinnati OH 45268

Center for Environmental
Research Information
Cincinnati OH 45268

Technology Transfer

EPA/625/6-88/018

EPA

Guide to Technical Resources for the Design of Land Disposal Facilities

EPA/625-6-88/018
December 1988

Guide to Technical Resources for the Design of Land Disposal Facilities

Risk Reduction Engineering Laboratory
and
Center for Environmental Research Information
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

Printed on Recycled Paper

Notice

This document has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication. Mention of trade names or commercial products does not constitute endorsement or recommendation for use.

Acknowledgments

This document was developed by EPA's Office of Research and Development under the direction of Clarence A. Clemons, Center for Environmental Research Information and Robert E. Landreth, Risk Reduction Engineering Laboratory. The document was prepared by A.T. Kearney, Inc., under contract no. 68-01-7374. Special thanks are given to Dr. Robert M. Koerner of Drexel University and Dr. David Daniel of the University of Texas, and Dr. Gregory N. Richardson of S&ME/Westinghouse for their input in reviewing and commenting on the document.

Table of Contents

Table of Contents (continued)

Table of Contents (continued)

Table of Contents (continued)

Table of Contents (continued)

List of Exhibits

CHAPTER 1.0
Introduction

Subtitle C of the Resource Conservation and Recovery Act (RCRA) of 1976, as amended by the Hazardous and Solid Waste Amendments (HSWA) of 1984, establishes requirements for landfills and surface impoundments to ensure that land disposal of hazardous waste in such units is conducted in a manner protective of human health and the environment. Performance standards and minimum technology requirements have been promulgated by EPA at Title 40 Code of Federal Regulations (CFR) Part 264 for new and existing units. These standards and requirements are implemented through permits issued by authorized states or EPA in accordance with the regulations of 40 CFR Part 122 through 124. Specific information requirements for RCRA permit applications have been promulgated at 40 CFR Part 270.

1.1 Purpose

EPA has issued numerous technical documents intended to assist preparers and reviewers of permit applications for hazardous waste land disposal facilities, including RCRA Technical Guidance Documents, Permit Guidance Manuals, and Technical Resource Documents. Some of these documents provide extensive detail on a select number of specific technical subjects related to the design and operation of land disposal facilities, while others provide broad guidance on RCRA permitting issues. The objective of these documents is to facilitate the expeditious preparation and processing of RCRA permit applications and to achieve consistency in permitting decisions.

EPA is concerned, however, that permit applicants and reviewers may not be familiar with all of the technical documents which the Agency has issued and may not be taking advantage of the information which they offer. EPA recognizes a need to provide a concise directory of information resources which are available and to suggest how these resources may be effectively used in the RCRA permitting process.

The purpose of this Guide is to direct permit applicants and permit application reviewers to the EPA documents which may be helpful in answering specific technical questions which often arise during permit application preparation and processing. Non-EPA technical literature has also been included in this Guide as appropriate. It should be noted that the list of non-EPA documents for any one subject may not be all inclusive. Other literature, i.e., books, may be as appropriate. The Guide does not provide detailed guidance on each regulatory standard for RCRA permit applicants but rather provides an overview of technical considerations.

1.2 Scope

Since this is a Guide to other information sources, it contains very limited primary information itself. To maximize its usefulness as a Guide, emphasis has been placed on brevity and conciseness. While the Guide generally does not discuss in detail the information provided in the primary sources, it directs the reader to the locations within these sources where specific technical subjects are addressed. (The references, as shown in the text, generally refer to the paragraph in which they appear. Where references appear within a paragraph, they refer to the text that immediately precedes them.)

The topics included in this Guide are limited to key performance standards and minimum technology requirements specified in 40 CFR Part 264 for hazardous waste landfills and surface impoundments. Each topic is addressed in an individual chapter as follows:

- Foundations (Chapter 2.0)
- Dike Integrity and Slope Stability (Chapter 3.0)
- Liner Systems (Chapter 4.0)
- Cover Systems (Chapter 5.0)
- Run-on/Run-off Controls (Chapter 6.0)

While the subjects addressed in this Guide are those which frequently arise in preparing and reviewing permit applications, the information and references provided in each chapter may also be useful in designing and operating other types of land disposal units (i.e., waste piles and land treatment units) and land disposal facilities for non-hazardous wastes.

1.3 Use

It is important to note that environmental performance standards in the regulations are qualitative objectives designed to protect human health and the environment and to guide the evaluation of permit applications. There may be a variety of technical approaches to designing, constructing and operating landfills and surface impoundments to accomplish these objectives.

Therefore, permit applications must provide a demonstration that the design, construction and operation of the specific land disposal units covered will meet these objectives. Such demonstrations must be made on a facility-specific basis. The general process for these demonstrations is summarized as follows:

1. Identify and justify the specific technical parameters which are important to attainment of the performance standard;

2. Identify and justify the methodologies used in determining whether such technical parameters are within an acceptable range of values. This could include, for example, test procedures, mathematical calculations, and/or references to commonly accepted engineering standards or EPA guidance; and

3. Demonstrate that each technical parameter falls within an acceptable range using the methodologies selected.

This Guide is organized in accordance with this process. The first section in each chapter provides a brief summary of the existing regulations (as of May 31, 1988) under 40 CFR Part 264 (the performance standards and minimum technology requirements) and 40 CFR Part 270 (RCRA permit application informational requirements) which correspond to the technical area covered in the chapter. The first section of each chapter also presents major technical parameters which are commonly considered in evaluating permit applications with respect to each performance standard. In subsequent sections of each chapter, the reader is referred to those technical documents which can be helpful in selecting evaluation methodologies and in determining acceptable ranges for these parameters. Each of the chapters are meant to be free-standing documents, hence, there may be some duplication of information from chapter to chapter. This duplication is necessary in order for the individual chapters to flow properly.

1.4 Update

This document is intended to be practical and informative. It is requested that Guide users submit ideas and/or suggestions to EPA, at the following address, regarding ways this document can be improved, including additional information sources they have found to be helpful in the preparation and review of permit applications:

Risk Reduction Engineering Laboratory
Office of Research and Development
U. S. Environmental Protection Agency
Cincinnati, Ohio 45268

Requirements for liners and leachate collection and removal systems are currently being revised. On March 28, 1986, EPA proposed a rule implementing minimum technology requirements of the Hazardous and Solid Waste Amendments of 1984 (HSWA) for double liner systems and leachate collection systems. Following proposal of these regulations, EPA collected data characterizing and comparing the performance of compacted soil bottom liners and composite (soil/flexible membrane liner) bottom liners. The data indicated that the use of a flexible membrane liner improved the performance of a composite bottom liner over that of a compacted soil liner, with respect to leachate collection efficiency, leak detection capability, and leakage both into and out of the bottom liner.

On April 17, 1987, EPA made available the background document presenting the data on bottom liner performance and draft minimum technology guidance documents on single- and double-liner systems. EPA proposed stronger regulations for double liners and leak detection systems on May 29, 1987.

When these regulations are promulgated, it may be necessary to update this Guide accordingly. Chapter 4.0, which addresses low-permeability soil liners, flexible membrane liners, and leachate collection/leak detection systems is most likely to be affected by these regulatory changes.

Chapter 2.0
Foundations

Proper subsoil foundation design of a land disposal system is critical because liner system components, especially leachate collection pipes and sumps, can be easily damaged by stresses caused by foundation movement. Permit applications must include a comprehensive evaluation of subsoil foundation conditions, followed by a demonstration that the foundation design will minimize the effects of foundation movements on the rest of a unit's components.

2.1 Regulations and Performance Standards

The regulations governing foundations provide a performance standard rather than a design standard. The performance standards in 40 CFR 264.221(a)(2) for surface impoundments and 40 CFR 264.301(a)(1)(ii) for landfills state that foundations must be "capable of providing support to the liner and resistance to pressure gradients above and below the liner to prevent failure of the liner due to settlement, compression or uplift."

Foundations for hazardous waste land disposal facilities should provide structurally stable subgrades for the overlying facility components. The foundations also should provide satisfactory contact with the overlying liner or other system components. In addition, the foundation should resist settlement, compression and uplift resulting from internal or external pressures, thereby preventing distortion or rupture of overlying facility components (Reference 1, p. 12)

In addition, likely seismic activities at the location must be confirmed. The jurisdictions where the seismic location standard is applicable are designated at 40 CFR 264.18(a). However, regardless of whether the facility is located in one of these jurisdictions, it is advisable to design foundations capable of withstanding maximum likely earthquake events.

The foundation analysis presented in a permit application should assess the potential for, and present calculated estimates of, settlement, compression, consolidation, shear failure, uplifts, liquefaction of the foundation soil, and the potential for hydraulic and gas pressures on the foundation. Typically, the analysis should provide geologic data, geotechnical data, hydrogeologic data, and seismic setting information. The following sections will describe the type of information and analyses needed to evaluate the foundation. The references that are cited describe how to evaluate the analyses.

Exhibit 2-1 summarizes the types of information and technical parameters commonly included in RCRA permit applications for landfills and surface impoundments to demonstrate that the foundation performance standard is met.

The steps normally taken to prepare such a demonstration are as follows:

- Preparation of a final design of the units, including design drawings showing their location on the site, their depth, configuration and dimensions, and their position relative to existing and final grade;
- Performance of a location-specific site investigation;
- Laboratory analyses of soil samples obtained during the site investigation;
- Analysis, as appropriate, of settlement potential, bearing capacity, hydrostatic or gas uplift pressures, liquefaction potential, and subsidence and sinkhole potential; and
- A Construction Quality Assurance Plan that identifies the level of inspection and testing necessary to construct the foundation to the specifications used in the design.

The following sections discuss these steps in detail, with specific instructions on how to evaluate the information provided in a permit application.

2.2 Site Investigation

Adequate site investigations are necessary to ensure that the foundation design is developed to accommodate expected site conditions. Site investigations are designed to establish the in-situ subsurface properties, site hydrogeologic

Exhibit 2-1. Types of Information Used to Demonstrate That the Performance Standard for Foundations Is Met

Information	Typical Parameters
Description of Foundations	Description of: • General foundation design • Foundation materials • Include geological and construction drawings indicating bearing elevations
Subsurface Exploration Data	Detailed engineering characteristics of: • Subsurface soil • Bedrock • Hydrogeologic conditions
Subsurface Exploration Data	Engineering characteristics of foundation materials verified through procedures including: • Historical data • Test borings • Test pits or trenches • In situ tests • Geophysical exploration methods
Laboratory Testing Data	Test results for: • Index testing • Hydraulic conductivity • Shear strength • Compressibility
Engineering Analyses	Engineering analyses using data obtained through subsurface explorations and laboratory testing including, as appropriate: • Settlement potential • Bearing capacity • Stability of cut or constructed slopes • Potential for excess hydrostatic or gas pressure • Seismic conditions • Subsidence potential • Sinkhole potential • Liquefaction potential
Analysis of Settlement Potential	Estimates of total and differential settlement, including: • Elastic settlement • Primary consolidation • Secondary compression
Analysis of Bearing Capacity	Analysis of allowable bearing capacity and comparison of required bearing capacity based on actual loading

(continued)

Exhibit 2-1. Continued

Information	Typical Parameters
Analysis of Stability of Landfill Slopes	Analyses of static and dynamic cases for: • Excavated slopes • Embankment slopes • Slopes including liners and/or cover • Drained and/or undrained conditions
Analysis of Potential for Hydrostatic Pressures	Estimates of potential for bottom heave or blow-out due to unequal hydrostatic or gas pressures.

characteristics and the area seismic potential, all of which are critical to facility design (Reference 1, p.13).

2.2.1 Foundation Description

Foundation design procedures are site specific and very often are an iterative procedure. A typical preliminary foundation description should include:

- the geographic setting;
- the geologic setting;
- ground-water conditions;
- soil and rock properties;
- surface-water drainage conditions;
- seismic conditions; and
- basis of information.

Site plans should include the unit locations within the site; the unit depths, configurations, and dimensions; and whether the unit will be completed below or above grade. It is particularly important that the investigation borings, test pits, and other procedures be performed as near as possible to the units, if not within their boundaries. Some other critical elements of the foundation design that need to be addressed prior to completion of the site investigation are the foundation design alternatives, the foundation grade, the loads exerted by the unit or the foundation, and the preliminary settlement tolerances.

2.2.2 Subsurface Exploration Programs

Subsurface exploration programs are conducted to determine a site's in-situ subsurface properties, as well as its geology and hydrogeology. The in-situ subsurface properties and hydrogeologic characteristics have a significant influence on the bearing capacity settlement potential, slope stability, and uplift potential for the site. The site's subsurface geology may impact the settlement and seismic potential at the site and exert an influence on the site's hydrogeologic characteristics.

Reference 2, p. 5-3 and Reference 13 provide a strong guidance for planning subsurface exploration programs. The following list provides details of the elements of a subsurface exploration program:

- Relate the site geology to the regional geological setting;
- Provide analysis of the engineering properties of representative subsurface samples;
- Establish in-place subsurface characteristics that include depth to bedrock, and the presence of features that can act as failure planes or hydrogeologic pathways;
- Identify bedrock characteristics such as lithology, orientation, extent of weathering, fractures, joints and solution cavities; and
- Establish the hydrogeologic site characteristics such as depth of the water table; horizontal and vertical flow components; hydrogeologic pathways; seasonal variability; and the location, use, and type of aquifers present.

Subsurface exploration programs can utilize both indirect and direct methods. Indirect investigation methods include geophysical techniques (e.g., electrical survey methods, ground-penetrating radar and seismic refraction). These methods do not require drilling or excavation. The selection of the proper geophysical techniques is dependent on the geologic settings. (Reference 2, pp. 5-4 and 5-5). While geophysical procedures can provide large amounts of data at a lower cost, they require careful interpretation which must be done by qualified experts only. Furthermore, geophysical data must be verified by direct procedures such as borings or test pits.

Direct investigation methods include drilling boreholes and wells and excavating pits and trenches. Direct methods allow the site geologic conditions to be observed and measured. Typically, boring logs should provide descriptions of the soil strata and rock formations encountered, as well as the depth at which they occur. In addition, the boring logs should provide standard penetration test results for soils and rock quality designation results for rock core runs. The boring logs should also record the intervals for, and the results of, any field hydraulic conductivity testing conducted in the borings.

Direct methods allow the investigator to obtain samples of subsurface material for laboratory testing of engineering properties. Soil samples can be obtained either by split spoon or thin-walled tube. Split spoon samples are disturbed and are of limited value other than for identification and water content. The thin-walled tube sample provides an undisturbed sample that can be used for a wide variety of laboratory tests; however, its use is limited to certain soil types and conditions.

The scope of the subsurface exploration program will vary depending upon the complexity of the subsurface geology, seasonal variability in site conditions and the amount of site information available. Typically, the investigator should drill an adequate number of borings across the site to characterize the underlying deposits and bedrock conditions and to establish a reasonably accurate subsurface cross-section. Depth of borings is highly dependent on site-specific conditions. However, typically, the borings should extend below the anticipated site base grade, or below the water table, whichever is deeper. A sufficient number of water table observation wells and piezometers should be installed to define both the horizontal and vertical ground-water flow directions. When subsurface heterogeneities are encountered that could lead to seepage or loss in strength in the foundation, additional subsurface exploration is sometimes necessary to identify and determine the extent of these features (Reference 2, p. 5-6). Typically the hydrogeologic conditions are identified as part of the ground-water monitoring program.

2.2.3 Laboratory Testing Data

Laboratory testing for foundations may include the following (Reference 2, Chapter 3):

- Atterberg Limits,
- Grain size distribution,
- Shrink/swell potential,
- Cation exchange capacity,
- Mineralogy,
- Shear strength,
- Dispersity,
- Compressibility,
- Consolidation properties,
- Density and water content, and
- Hydraulic conductivity tests.

Soil index properties are simplified tests that provide indirect information about the engineering properties of soils beyond what can be gained from visual observations. Although the correlation between index properties and engineering properties is not perfect, it is generally adequate for QC purposes (Reference 2). Index property tests commonly used to screen soils are described below.

Atterberg Limits include tests to establish the liquid limits and the plastic limit of a soil (Reference 8). These tests are commonly used along with grain-size distribution, for monitoring changes in soil type. A significant change in Atterberg limits usually reflects a change in important engineering properties, such as the relationship among water content, density, compactive effort, and hydraulic conductivity.

Grain-size analysis is another important screening test for changes in soil composition. The percentage of silt and clay-size particles and the overall particle size distribution of a soil affects its engineering properties, especially hydraulic conductivity and strength. Rough estimates of grain size may be obtained through manual estimates (Reference 3; ASTM D 2488) and may be sufficient for screening. A 200-mesh sieve may be used to separate coarse (sand and gravel) and fine (soil and clay) particles. More detailed grain-size distributions may be obtained by sieving the coarse fraction and by using several settling methods (hydrometer, decantation, or pipette) for the fine fraction (Reference 3; ASTM D 422). Again, it is important to monitor carefully for soil-type changes as backfill material is being placed (Reference 2).

Soil index properties, Atterberg limits and grain-size distribution, are simplified tests that provide indirect information about the engineering properties of soils beyond what can be gained from visual observations. Although the correlation between index properties and engineering properties is not perfect, it is generally adequate for construction QC purposes (Reference 1). These two index property tests commonly used to screen soils are described below.

2.2.4 Seismic Conditions

Seismic analysis is particularly critical when there is a high potential for liquefaction to occur, such as in seismically-active areas underlain by loose, saturated sands and silts. Many regions in the country that have experienced earthquake activity should have information on the frequency and magnitude of earthquakes. There may also be established local standards for the design of structures. Generally, earth structures can be designed to withstand the vertical and horizontal accelerations experienced during such design earthquakes. A more detailed discussion of methods for evaluating site seismic parameters is presented in Chapter 3.0 of this Guide.

2.3 Design

Foundations are designed to provide structural support and to control settlement. Foundations must also be designed to withstand hydrostatic and gas pressures.

2.3.1 Waste and Structure

The engineering analysis for foundations is based on subsurface conditions; however, the results of these analyses are based on loading conditions. In order to perform the appropriate engineering analysis to demonstrate the adequacy of the foundation, the permit application should provide an accurate estimate of the loadings (including both structure and waste), plans showing the structure's shape and size, the expected waste characteristics and volumes, and the foundation elevations.

2.3.2 Settlement and Compression

The performance standards require that the foundation be capable of preventing failure of the liner system due to settlement and compression. Therefore, it is important that the permit application provides an analysis estimating total and differential settlement/compression expected due to the maximum design loadings. The results of this analysis are then used to evaluate the ability of the liner system and leachate collection and recovery systems to maintain their integrity under the expected stresses.

A settlement analysis will provide an estimate of maximum settlement. This maximum settlement can be used to aid in estimating the differential settlement and distortion of a land disposal unit. Allowable settlement is typically expressed as a function of total settlement, rather than differential settlement, because the latter is much more difficult to predict. However, the differential settlement is a more serious threat to the integrity of the structure than total settlement (Reference 4 and Reference 10).

Total settlements of a few inches or less are usually not a problem for soil liner foundations, since most are sufficiently thick and flexible to withstand some differential settlement of the foundation. As long as the topography is fairly uniform and significant subsurface heterogeneities are not present, differential settlement should be minimal. Foundation settlement analyses based on the site's subsurface conditions (determined during site investigation) should be conducted during the design of the facility. These analyses should take into account the loadings of all facility components on the foundations, including footings for pile-type structures such as leachate collection risers, which, if improperly designed, can be forced into or through the liner. Compensated foundation, which implies that the weight of soil extracted from the site balances the weight of fill material, also can be used as part of the design to minimize subgrade settlement. In addition, the expected differential settlement should be compared to the design slope of the leachate collection system to ensure the latter's adequacy is maintained (Reference 2, p. 5-14).

Landfill design calculations should include estimates of the expected settlement, even if it is expected to be small. Small amounts of settlement, even a few inches, can cause serious damage to leachate collection piping or sumps. The ability to predict the extent of settlement depends upon the type of process anticipated to cause settlement. There are several settlement processes, each of which should be considered in a land-based unit design. These are discussed in the following paragraphs and include:

- Primary consolidation,
- Secondary compression, and
- Elastic Compression.

Primary consolidation, which is typically a reduction in void ratio due to removal of pore fluids by mechanical loading, generally occurs in saturated fine-grained soils according to the consolidation theory developed for soil. Basically, the theory states that the rate and amount of compression is equal to the rate and amount of pore fluids squeezed out of the soil (Reference 5). The classic Terzaghi theory for one-dimensional consolidation of a soil is discussed in Reference 6, at page 17.

Primary consolidation of soils by lowering of the water table has been identified as an additional cause of ground subsidence in some locations. The effect of lowering the water table in a soil is to surcharge the soil particles by increasing the effective stress (the vertical stress minus the pore water pressure) through a decrease in pore pressure (Reference 6, p. 17).

Secondary compression is the gradual settlement from creep under essentially constant load. It depends upon the applied load and the chemical and physical nature of the soil particles. Secondary compression is more irregular and less predictable than primary consolidation and may be significant in settlement of plastic clay soils, heterogeneous fill materials, organic materials, and other compressible materials. Although secondary compression occurs at the same time as primary consolidation, secondary compression is usually taken into account at later times in the loading history of the fill when primary consolidation is complete and the applied stress is transferred from the pore fluids to the soil skeleton (Reference 2, p. 5-15 and Reference 11).

Elastic compression occurs when the volume of solids is reduced. The effect of elastic compression of mineral soils is minimal; however, it may be a major concern with organic soils, soluble materials, and materials subject to chemical attack. This type of compression is highly irregular and is influenced by a number of environmental factors that make it difficult, if not impossible, to predict and are, therefore, not typically considered in geotechnical practice (Reference 2, p. 5-15).

Both theoretical and empirical approaches are utilized for predicting settlement. The theoretical methods are based on elastic theory and summation of strains. Empirical or semi-empirical methods include performing load tests and penetration tests in the field. Theoretical methods should be used only in conjunction with empirical methods that provide field verification (Reference 4, Chapters 8 and 14, and Reference 6, Chapter 9:3).

The elastic theory applies to soil only in a very approximate way. Soil itself is not elastic; however, elastic theory provides a convenient means to estimate stresses induced within a soil mass by applied loads. Knowing these stresses allows the engineer to compute the strains, and by adding up the strains along any vertical line, the settlement of the surface can be computed (Reference 4, Chapters 8 and 14).

2.3.3 Seepage and Hydrostatic Pressures

Foundations should be designed to control seepage and hydrostatic pressures. Heterogeneities such as large cracks, sand lenses, or sand seams in the foundation soil offer pathways for leachate migration in the event of a release through the liner and could cause piping failures. In addition, soft spots in the foundation soils due to seepage can cause differential settlement possibly causing cracks in the liner above and damaging any leachate collection or detection system installed. Cracks can also be caused by hydrostatic pressure where the latter exceeds the confining pressure of the foundation and liner (Reference 2, p. 5-15).

Solutions to these problems include various systems that are available to lower the hydraulic head at the facility. These systems include pumping wells, slurry walls and trenching. Other methods to control foundation seepage include grouting cracks and fissures in the foundation soil with bentonite and designing compacted clay cut-off seals to be emplaced in areas of the foundation where lenses or seams of permeable soil occur (Reference 2, p. 5-16).

2.3.4 Bearing Capacity

For waste disposal units, the major issue of concern for foundations is differential settlement. However, for structures such as tank foundations, leachate risers, etc., an additional area of concern is bearing capacity failure (Reference 6, Chapters 9:2 and 9:3).

The basic criterion for foundation design is that settlement must not exceed some permissible value. This value varies, dependent on the structure and the tolerance for movement without disruption of the unit's integrity. To ensure that the basic criterion is met, the bearing capacity must be established for the foundation soil. The bearing capacity of a soil, often termed its stability, is the ability of the soil to carry a load without failure within the soil mass. The load carrying capacity of soil varies not only with its strength, but often with the magnitude and distribution of the load. Reference 7, Chapter 10, provides information regarding the evaluation of bearing capacities and typical ranges of key parameters. After the bearing capacity is determined, the settlement under the expected load conditions should be estimated and compared to the permissible value. The foundation design should be such that the actual bearing stress is less than the bearing capacity by an appropriate factor of safety (Reference 4, Chapter 14; Reference 6, Chapter 9:2 and Reference 12).

Several types of structural foundation failures can occur that are highly site specific. These failures depend on subsurface conditions and loading type and conditions. The various types of foundation failures that can occur are discussed in Chapter 9 of Reference 7, Chapter 9:2 of Reference 6 and Chapter 14 of Reference 4. In addition, for cases where the foundation consists of soft soils, special care must be taken to ensure that local shear failures do not occur due to equipment movement during placement of the liner system or the waste.

Many large metropolitan areas have records of allowable design soil pressures that have been successful and also those that have failed. These are called presumptive bearing pressures because they are based on past performance that the soil can support such a pressure without experiencing a bearing capacity failure or excessive settlement (Reference 6, Chapter 9:5).

These values are not defendable for design work and are not to be considered a performance standard. For example, the sand pressures usually provided are highly dependent on footing size; the soft-to-firm clays need an analysis based on site-specific conditions and soil properties; and the stiff-to-hard clay assumes no fissures. However, the presumptive values do provide a general guidance for the reviewer of typical values to be expected. In short, they provide a reference point, but not a hard and fast design criterion. (Reference 7, Chapter 10:5)

2.4 Excavations

Most hazardous waste units are constructed below existing grade. Therefore, most sites must be excavated to final foundation grade. If the total depth of excavation exceeds approximately ten feet, a slope stability analysis of the excavated slopes should be made. Slope stability analyses are discussed in Chapter 3.0, and generally the same procedures apply to cut slopes, except that lower factors of safety are acceptable.

2.5 Quality Assurance

Once the design of the foundation has been completed in accordance with acceptable standards, the foundation's construction can begin, as designed and specified, following strict quality assurance procedures. The primary quality assurance issues for foundations are to assure the adequacy of the subgrade and, if necessary, the compacted fill through density testing. A brief discussion is provided below on EPA's guidance for construction quality assurance procedures, as provided in Reference 1. This is followed by a discussion of field density testing that can be performed to establish adequate subgrade and compaction (Reference 2, p. 3-26).

2.5.1 General Quality Assurance Procedures

Reference 1 provides technical guidance regarding quality assurance procedures during the preconstruction, construction and post-construction procedures. In addition, Reference 9 provides a summary of items for inspection of old or new concrete. During the preconstruction phase, it is especially important for all construction quality assurance personnel and the construction contractors to review site investigation information to familiarize themselves with the expected site conditions upon which the facility designs were based. This will help ensure their ability to identify any unexpected site conditions encountered during construction (Reference 1, pp. 12-15).

2.5.2 Materials

Soil and rock underlying the facility must possess adequate strength to support the expected loading. If tests on samples of the materials examined in the laboratory and on-site bearing tests show inadequate properties, the site design and construction plans should include specifications that provide for preparation of an adequate foundation. If appropriate, samples of materials from potential borrow areas to be used to construct the foundation should be analyzed to determine their acceptability for the specified design. This information is used to identify desirable materials and reject undesirable materials. The principal concern is to verify that the specified materials of any load-bearing foundation are specified in enough detail to compare with the characteristics shown to be required by the engineering analysis. (Reference 2, pp. 1-4).

2.5.3 *Subgrade Requirements*

Visual observation of the subgrade is necessary to assure that the foundation is constructed as designed. The site engineer needs to ensure that all soft, organic and otherwise undesirable materials are removed. This can be done by proof rolling with heavy equipment to detect soft areas. As outlined in Reference 1, various tests are available to verify the condition of the foundation subgrade.

In addition, the site engineer should inspect soil and rock surfaces for rock joints, clay fractures and depressions. These features should be adequately filled. If sand seams are encountered, they should be removed and refilled with compacted material (Reference 1, p. 13).

2.5.4 *Compaction Requirements*

If required, selection of backfill material that can be compacted to the required density and permeability involves a series of laboratory tests of the engineering properties of the candidate materials. One such engineering characteristic is the water content/density relationship which is established for the material by compacting samples of the material at various water contents with a set compactive effort (Reference 2; pp. 3-21 to 3-25). A standard method has been adopted and described in ASTM standard test method D698 (Reference 8; ASTM D698).

Based on the compaction test data, compaction specifications should indicate the minimum percent of maximum density and the water content relative to the optimum water content at which the soil should be compacted. Soils have different characteristics at water contents above, at, or below the optimum water content. For instance, clays compacted on the wet side of the optimum water content are less permeable than those compacted on the dry side. On the other hand, clays compacted dry of optimum are stronger and have a higher stress-strain module than do clays compacted wet of optimum (Reference 2, pp. 3-21 to 3-25).

2.5.5 *Concrete Requirements*

In some cases, a hazardous waste management unit may be placed on a concrete surface. The concrete might be new (poured recently with the intent of being the FML supporting surface) or old (an older structure that is being retrofitted with an FML) (Reference 9, p. 4-17). Old concrete must be checked very carefully, because it is more likely to have cracks, surface chipping, and a rougher surface. Old concrete is also more likely to chip and crack when drilling is required to set items like FML anchor bolts. The effects of surface irregularities may be minimized by the use of various coating materials or by covering with a geotextile (Reference 9, pp. 4-17).

New concrete must be allowed to age in order to obtain the strength needed to set items like FML anchor bolts. In addition, any wax-type curing compound used must be removed prior to FML placement, since sealing compounds (adhesive, cements, and caulks) will not adhere to this type of surface. If surface voids exist, they must be eliminated by sacking with cement grout (Reference 9, p. 4-19).

2.5.6 *Placement*

During placement of soil materials, the soil is spread uniformly as specified. The loose lift thickness of the soil should be measured systematically over the entire site, with a marked staff or shovel blade, and survey levels should be made every few lifts for verification and documentation of fill thickness. Following spreading, the backfill material is disked or tilled to break up large soil aggregates and to homogenize the material (Reference 2, pp. a-6 to a-9).

2.5.7 *Compaction Equipment*

The principal types of compacting equipment are the smooth wheel roller, the rubber-tired roller, the sheepsfoot roller, and the vibratory compactor. The latter would be the most effective piece of equipment for compacting coarse-grain, cohesionless soils. However, vibratory rollers are the least effective compactors for cohesive soils. Rubber-tired rollers with high tire pressures and sheepsfoot rollers are effective for cohesive soils. Sheepsfoot rollers are particularly effective at bonding of lifts during compaction of cohesive soils. Reference 6 provides a detailed discussion of compactive equipment and methods.

2.5.8 *Field Density Testing*

Two traditional methods are used for measuring density in the field. In one type of test, a small hole is dug in the compacted fill and the excavated material is saved and weighed. The volume of the hole is measured by filling it with sand or liquid with a device that measures the amount of material required to fill the hole. The sand cone and rubber balloon methods are examples of this type of test. Another technique is to drive a hollow cylinder into the fill, remove a core, trim it to a known volume, and then determine its weight. This drive-cylinder method and the sand cone and rubber balloon methods take time because the sample must be oven dried before the dry density can be determined (Reference 2, p. 3-26).

Nuclear probes (Reference 8, ASTM D-2922 and D307) offer a faster and more convenient method for measuring field density and water content than the traditional methods and are presently widely used for earthwork compaction quality control. Nuclear gauges are designed to give very rapid measurements of density and moisture content. The operation of nuclear gauges is discussed in Reference 2, p. 3-29. In order to compensate for the soil compositions that may affect the neutron response, it is customary to calibrate the nuclear density gauge against oven dried water content measurements by the appropriate laboratory test method (Reference 8; ASTM D2216).

2.6 References

1. Technical Guidance Document: *Construction Quality Assurance for Hazardous Waste Land Disposal Facilities*; Hazardous Waste Engineering Research Laboratory, Office of Research and Development, U. S. Environmental Protection Agency, Cincinnati, Ohio 45268; EPA Contract No. 68-02-3952, Task 32; October 1986.

2. Draft Technical Resource Document: *Design, Construction, and Evaluation of Clay Liners for Waste Management Facilities*; Hazardous Waste Engineering Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency; Cincinnati, Ohio 45268; EPA/530 - SW-86-007, March, 1986.

3. *Permit Applicant's Guidance Manual for Hazardous Waste Land Treatment, Storage and Disposal Facilities* (Volumes 1 and 2), Office of Solid Waste and Emergency Response, U.S. Environmental Protection Agency, Washington, D.C. 20460; SW-970, January 1984.

4. *Soil Mechanics* by T. William Lambe and Robert V. Whitman, Massachusetts Institute of Technology, John Wiley & Sons, Inc., New York, New York, 1969.

5. Anderson, D.: 1982, *In-place Closure of Hazardous Waste Surface Impoundments* (draft), Chapter 5, "Evaluating Stabilized Waste Residuals," EPA-68-83-2943.

6. Technical Guidance Document: *Prediction/Mitigation of Subsidence Damage to Hazardous Waste Landfill Covers* by P.A.Gilbert and W.L.Murphy of the U. S. Army Engineer Waterways Experiment Station at Vicksburg, MS, for the Hazardous Waste Engineering Research Laboratory, Office of Research and Development, U. S. Environmental Protection Agency, Cincinnati, OH 45268; Interagency Agreement No.DW21930680-01-0, July, 1987.

7. *Introductory Soil Mechanics and Foundations* by G.B. Sowers & G.F. Sowers, Third Edition, MacMillan Publishing Co., Inc., New York, New York, 1970.

8. *ASTM 1988, The American Society for Testing and Materials 1985 Annual Book of ASTM Standards*, Volume 4.08, "Soil and Rock; Building Stones," Philadelphia, Pennsylvania.

9. Wright, T.D., W.M. Held, J.R. Marsh, and L.R. Hovater: 1987 *Manual of Procedures and Criteria for Inspecting the Installation of Flexible Membrane Liners in Hazardous Waste Facilities* prepared for Hazardous Waste Engineering Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, Cincinnati, Ohio 45268, EPA-68-03-3247.

10. Wahls, H.E., "Tolerable Settlement of Buildings," *Journal of Geotechnical Engineering*, ASCE, Vol. 107, No. GT11, 1981, pp.1489-1504

11. Mesri, G., "Coefficient of Secondary Compression," *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 99, No. SM1, 1973, pp.123-137.

12. *Foundation Engineering Handbook*, by H.F.Winterkorn and H.Y.Fang, 1975, Van Nostrand Reinhold, New York, 751 pgs

13. "Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes" by M. Hvorslev, Reprint of Research Project Report for the Committee on Sampling and Testing, Soil Mechanics and Foundation Division, American Society of Civil Engineers; 1965; 521 pgs. (Recently Updated).

Chapter 3.0
Dike Integrity and Slope Stability

Most landfills and surface impoundments are constructed above natural grade through the use of earthen dikes, excavated below grade slopes constructed around the unit, or some combination of dikes and excavation, depending on site topography. Surface impoundments are often designed to achieve some balance of cut-and-fill, with the excavated soils used to construct the dikes. Landfill cells are excavated below grade in order to provide operating cover materials and to allow for restoration of the site after filling.

These excavated slopes and earthen dikes are vulnerable to stability failures via several mechanisms that will be discussed in this chapter. Slope and dike failures at hazardous waste management units are potentially very serious; a surface impoundment failure can allow the sudden release of large amounts of hazardous waste to ground water and surface waters, and a landfill slope failure can seriously damage the liner system, allowing releases of waste and leachate to surrounding soils and ground water.

For these reasons, earthen dikes must be carefully designed and excavated slopes must be carefully evaluated to assure that they are sufficiently stable to withstand the loading and hydraulic conditions to which they will be subjected during the unit's construction, operation and post-closure periods. This chapter will discuss the regulatory requirements that apply to slope stability issues and will describe how to design and evaluate dikes and slopes for stability.

One of the apparent differences between landfill and a surface impoundment unit is that of solid vs. liquid wastes. From the viewpoint of stability, however, there is no real difference; the forces on a slope exerted by liquids are modeled in a manner identical to those of solids. Another issue related to the impoundment of liquids is that of seepage through the dikes, causing piping or hydrostatic uplift pressures; however, this seepage condition is modeled in a manner identical to the condition of ground-water seepage at a cut slope. Since the failure mechanisms are similar for dikes and excavated slopes, these two configurations will be discussed concurrently.

3.1 The Regulations and Performance Standards

The regulations for surface impoundments, 40 CFR 264.221(g), (Reference 1), require simply that massive failure of dikes be prevented through adequate design, construction and maintenance. This is a performance standard only; the regulations do not contain design standards. For landfills, there are no specific slope stability regulations; however, the regulations at 40 CFR 264.301 require that a liner system in a landfill be placed upon a foundation or base that will prevent the failure of the liner.

In order to demonstrate that the entire liner system is placed upon a stable base, the stability of the slopes must be demonstrated.

The surface impoundment regulations (Reference 1, 40 CFR 270.17 and 264.226) also require that the structural integrity of each dike be certified by a qualified engineer. Exhibit 3-1 summarizes the types of information and technical parameters commonly used to demonstrate that the performance standard is met.

3.2 Design and Materials Selection

Slope stability failures usually occur in one of three major modes, depending upon the site soils, slope configuration, and hydraulic conditions (Reference 2). These three major failure modes are the following:

- rotation on a curved slip surface approximated by a circular arc.
- translation on a planar surface whose length is large compared to its depth below ground.
- displacement of a wedge-shaped mass along one or more planes of weakness in the slope.

Exhibit 3-2 illustrates basic concepts of translational and rotational failures, and Reference 2, Chapter 7 shows more examples of these potential slope failures in natural and in cut and fill slopes.

Slope failures occur when sliding forces from the weight of the soil mass itself and external forces

Exhibit 3-1. Information Typically Submitted to Demonstrate Satisfaction of Performance Standards for Dike Integrity

Information	Typical Parameters
Description of Dike Design	Data and/or drawings specifying: • Design layout of dikes • Design layout of components • Materials of construction • Elevations of critical points
Demonstration of Stability	Capability to withstand expected static and dynamic loadings and the effects of erosion
Demonstration of Erosion and Piping Protection	Demonstration of minimization of erosion and prevention of failure considering the erosion potential of: • Rainfall • Surface water runon and runoff • Contact between impounded wastes and dikes • Potential leakage and piping through dikes • Potential leakage and piping along conduits or structures through dikes
Analysis of Subsurface Conditions	Engineering characteristics of foundations and soil dike materials through testing and subsurface explorations, such as: • Test borings • Test pits or trenches • In situ tests • Geophysical methods • Strength and consolidation tests on foundation materials • Permeability
Stability Analyses	Description and results of stability analyses for the following conditions, as appropriate: • Foundation soil bearing failure or settlement • Failure in dike slopes • Failure of impoundment cut slopes • Build-up of hydrostatic pressure due to failure of drainage system, dike cover, and liner • Rapid drawdown
Construction Specifications	Procedures for dike construction
Engineer Certification	Certification of integrity of dike designs and construction

including waste pressures exceed the resisting forces from the strength of the soil and from any reinforcing structures. Slope stability analysis consists of a comparison of these resisting forces or moments to the sliding forces or moments, to obtain a factor of safety, (FS). Generally, the FS takes the following form (Reference 3, Section 12-1):

$$FS = \frac{\text{Sum of resisting momemts}}{\text{Sum of sliding moments}}$$

When a stability analysis is performed, a slope is analyzed for one or more of several potential modes of failure, including rotational, translational and wedge, as appropriate. A safety factor is obtained for each mode, and the lowest FS is the most critical.

In addition to the three major failure modes, dikes and excavated slopes are also vulnerable to failure due to differential settlement, seismic effects including liquefaction, and seepage-inducted piping failure. Safety factors are determined in a manner similar to the three modes. These failure modes will be discussed in greater detail in Section 3.2.3.

Exhibit 3-3 lists the EPA-recommended minimum factors of safety for slope stability analyses. If a dike or excavated slope design analysis yields lower safety factors, then steps should be taken to reduce the sliding forces or increase the resisting forces, or the slope should be redesigned to produce a safer structure.

In order to evaluate an existing, conceptual, or final slope design, the designer or reviewer must consider the following factors:

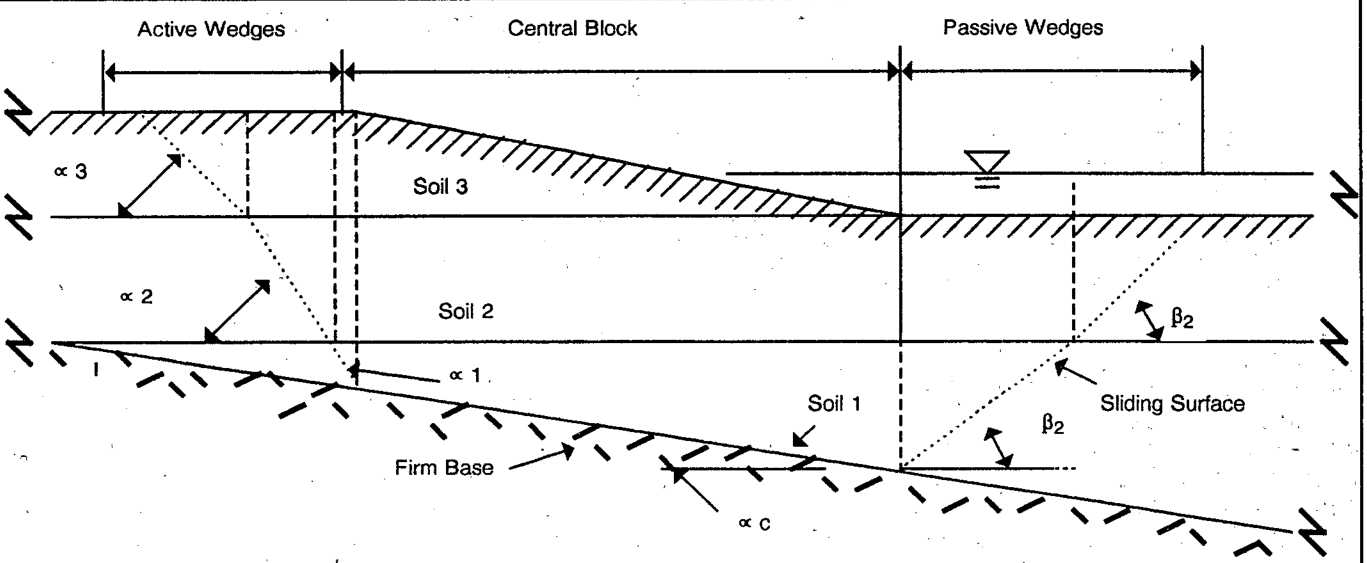
- the adequacy of the subsurface exploration program
- the stability of the dike slopes and foundation soils
- liquefaction potential of the soils in the dike and the foundation
- the expected behavior of the dike when subjected to seismic effects
- potential for seepage-induced piping failure
- differential settlements in the dike.

The following sections will discuss each of these factors, including the use of an EPA-developed computer model called GARDS, Geotechnical Analysis for the Review of Dike Stability (Reference 4).

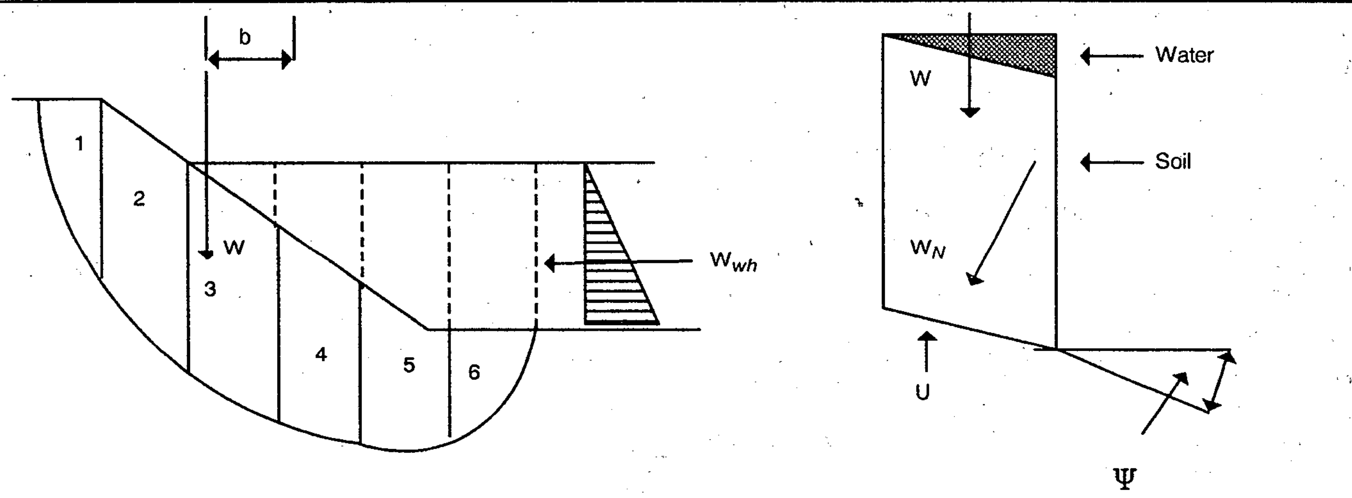
3.2.1 Subsurface Exploration Program

Site investigations are conducted to delineate a site's topography, subsurface geology and hydrogeology. They are necessary to evaluate the foundation for a constructed dike, to evaluate dike materials obtained from a borrow area, and to evaluate a slope excavated below ground. These investigations include

Exhibit 3-2. Conceptual slope failure modes.



Elements of the Translational (Wedge) Slope Stability Analysis
(Reference 4, p. 42)



a. Circular segment divided into slices

b. Forces acting on slice 3

Method of Slices for Circular Arc Analysis of Slopes in Soils Whose Strength Depends on Stress (Reference 3, p. 578)

field testing performed during drilling programs and laboratory testing performed on field samples. Of particular importance in some circumstances are laboratory strength tests performed on soil samples to determine the strength of the foundation and embankment soils under the expected conditions of saturation and consolidation. Site investigations include field exploration procedures such as remote sensing techniques, geophysical methods, test pits and trenches, and borings. The field exploration is followed by laboratory analysis of soil samples obtained during the field program. The field and laboratory data is then used to obtain a detailed characterization of the site with respect to the engineering properties of the soils and rock. These engineering properties provide the input data for evaluation of the stability of slopes. (See Chapter 2 of this guide for additional discussion on field investigations).

Exhibit 3-3. Recommended Minimum Values of Factor of Safety for Slope Stability Analyses (Reference 4)

Consequences of Slope Failure	Uncertainty of Strength Measurements	
	Small[1]	Large[2]
No imminent danger to human life or major environmental impact if slope fails	1.25 (1.2)*	1.5 (1.3)
Imminent danger to human life or major environmental impact if slope fails	1.5 (1.3)	2.0 or greater (1.7 or greater)

1. The uncertainty of the strength measurements is smallest when the soil conditions are uniform and high quality strength test data provide a consistent, complete, and logical picture of the strength characteristics.
2. The uncertainty of the strength measurements is greatest when the soil conditions are complex and when available strength data do not provide a consistent, complete, or logical picture of the strength characteristics.

* Numbers without parentheses apply for static conditions and those within parentheses apply to seismic conditions.

The number of borings or test pits required to characterize the subsurface is dependent on its complexity; a site with fairly uniform geologic conditions across the site can be sufficiently characterized with fewer exploratory probes than a more complex site. In any case, the test pits or borings must be performed at locations within or very near to the actual unit. Reference 2, Chapter 2 provides a discussion of field exploration, testing and instrumentation methods used to characterize a site.

For embankments constructed of on-site borrow materials, the borrow area should also be investigated to verify that it contains an adequate volume of acceptable material. This investigation will be very similar to that performed for excavated units, with the notable exception that all laboratory testing, i.e., strength, hydraulic conductivity, should be performed on remolded samples because the soils will be excavated and recompacted.

Hydrogeologic investigations are also necessary to determine the elevation of the water table at the site (including seasonal variability) and to locate, identify, and delineate hydrologic pathways (e.g., fractures and sand seams) that can contribute to slope failure (Reference 5). The significance of hydrogeologic conditions concerning slope stability will be discussed further in Section 3.2.3.

Laboratory testing is conducted using representative soil samples. Testing, as appropriate to evaluate the embankments, the foundation area, and those areas under consideration as a source for borrow material, include Atterburg Limits (Plasticity Index, Liquid Limit), grain-size distributions, shrink/swell potential, shear strength, compressibility, consolidation properties, density and water content, moisture-density relationships, and laboratory hydraulic conductivity (Reference 5).

For slope stability analyses, the most critical soil parameter is that of shear strength. The shear strength of a soil is a measure of the amount of stress that is required to produce failure in plane of a cross section of the soil structure. The shear strength of a soil must be known before an earthen structure can be designed and built with assurance that the slopes will not fail (Reference 5). To adequately determine a soil's shear strength, the potential effect of pore water pressures from the expected site loading conditions must be considered during testing.

While laboratory soil strength testing data is highly desirable, these tests are limited to small size samples, and in many locations dikes are constructed using material which contains large particle sizes. Furthermore, in existing dikes, the type of material may make the obtaining of undisturbed soil samples near to, if not, impossible. Therefore, it is not uncommon in standard engineering practice to estimate or assume these parameters based on the best data available. While it is acceptable to do this, it must be done and evaluated by a qualified geotechnical engineer.

Slope stability is also dependent on hydraulic conditions in the slope. Potential hydrostatic or seepage forces from large hydraulic gradients should be identified and considered during the stability analyses. Ground-water levels and hydraulic analyses are used to determine the configuration of the steady-state piezometric surface through sections of the foundation and/or the dike structure. For sections involving a steep piezometric surface or an upstream static or flood pool, hydraulic analyses also determine (Reference 5):

- seepage quantity
- critical (highest) exit gradient
- potential for uplift of a clay liner due to excess pore pressures produced by a confined seepage condition

Hydraulic boundary conditions may reflect unconfined, steady state seepage conditions or confined seepage conditions involving an impermeable barrier (soil liner) and excess pore pressure on the barrier. The hydraulic conditions of a slope are determined using seepage analysis, as discussed in Reference 13, Chapter 10.

3.2.2 Design

The design plans for dikes and cut slopes should show the design layout, cross-sections showing the proposed grade and bearing elevations relative to the existing grade, and details of the dikes or cut slopes, including all slope angles and dimensions. Materials present at the cut slope or to be used to construct the dike must be adequately characterized or specified (Reference 5). This design configuration then must be evaluated for its stability under all potential hydraulic and loading conditions. If the stability analyses result in unacceptably low factors of safety, then the design must be modified to stabilize the slope. The revised design must then be evaluated to verify that it is sufficiently stable.

In addition, in a landfill or surface impoundment, often the cut slopes or dikes will not be identical around the entire perimeter of the unit. For this reason, it is important that the most critical slope or dike section be identified for analysis. Generally, the most critical section will be the steepest and/or the highest portion of the slope or dike. However, particularly in a cut slope, the in situ materials may vary enough that the more critical slope may be shallower or flatter, but may be composed of weaker soils or may be subject to significant pore pressures or seepage from high ground-water levels.

3.2.3 Stability Analyses

Slope stability analyses are performed for both excavated side slopes and above-ground embankments. Three analyses will typically be performed as appropriate to verify the structural integrity of a cut slope or dike; they are slope stability, settlement and liquefaction. Exhibit 3-4 indicates the minimum required soil parameter data usually needed to perform these analyses. Slope stability analysis requires the establishment of various site conditions including:

1) The soil shear strength conditions that represent actual site conditions (discussed in Section 3.2.1)

2) The steady state hydraulic boundary conditions occurring through the site's section (discussed in Section 3.2.1)

3) The seismic conditions established for the site area.

The slope stability is typically evaluated using either a rotational (slip circle) analysis and/or a translational (sliding block or wedge) analysis using a computer model. These analyses are run for both static and dynamic (seismic) conditions. The latter is typically performed using a coefficient that approximates seismic conditions established for the site area. For large dikes in areas of major earthquakes, a more rigorous method of dynamic analysis may be warranted. When appropriate, the liquefaction potential of the foundation or embankment is also determined using seismic conditions established for the site area.

Analyses to establish total and differential settlement are also performed to ensure that the estimated settlement will not adversely impact the integrity of the unit and its components. The analysis of potential settlement is discussed in Chapter 2.0.

The slope stability analysis uses data from the site investigation and soil testing to perform either of two conventional slope stability analyses. The first is a rotational (circular slip surface) analysis and the second is a translational (plane slip surface or wedge) analysis. The translational wedge analysis applies primarily to stratified sections, especially where stratum boundaries are inclined or where a stratum with low shear strength exists. Even so, a rotational stability analyses may yield a lower Factor of Safety for the section and should always be checked (Reference 4).

3.2.3.1 Rotational Slope Stability Analysis

A rotational slope stability analysis is typically performed using a method that divides the slope into discrete slices and sums all driving and resisting forces on each slice. For each trial arc, the section is subdivided into vertical slices, each having its base coincident with a portion of the trial arc. Slices are defined according to the section geometry such that the base of each slice comprises only one soil type. The driving and resisting forces acting on each slice are then used to compute driving and resisting moments about the center of rotation of a circular section of the slope. The overturning and resisting moments for each slice are then summed and the Factor of Safety is computed (Reference 4).

3.2.3.2 Translational Slope Stability Analysis

The major features of the translational analysis are the same as those for the rotational case except that the trial surface consists of straight line segments which form the base of one or more active (thrusting) wedges, a neutral or thrusting central block, and one or more passive (restraining) wedges. This analysis is based upon selection of a trial central block defined by the surface and subsurface soil layer geometry, followed by computation of the coordinates for the associated active and passive wedges (Reference 4).

3.2.3.3 Settlement Analysis

Settlement analysis is used to determine the compression of foundation soils due to stresses caused by the weight of an overlying dike. Required parameters for each soil include unit weight, initial void ratio, compression and recompression indices, and the overconsolidation ratio (Reference 4). A

Exhibit 3-4. Minimum Data Requirements for Stability Analysis Options (Reference 5)

Soil Parameter	Units	Stability Analysis Options: Rotational	Translational	Settlement	Liquefaction
1. Cohesion* (UU, CU, CD cases)	pounds/sq.ft. (psf)	X	X		X CD
2. Angle of internal friction* (UU, CU, C cases)	degrees	X	X		
3. Total (wet) unit weight	pounds/cu. ft. (pcf)	X	X	X	X
4. Clay content	percent (0 to 100)				X
5. Overconsolidation ratio	unitless (decimal)			X	
6. Initial void ratio	unitless (decimal)			X	
7. Compression index	unitless (decimal)			X	
8. Recompression index	unitless (decimal)			X	
9. Hydraulic conductivity** (permeability, k)	ft/yr				
10. Median grain size	mm				X
11. Plasticity index (PI)	percent (0 to 100)				X
12. Liquid limit (LL)	percent (0 to 100)				X
13. Standard penetration number (N)	unitless (integer)				X

* Required strength case dependent upon hydraulic boundary condition selected
** Used only in hydraulic analysis

detailed discussion of settlement analysis is provided in Chapter 2 of this Guide.

Settlements are calculated at the toes, crest points, and centerline of the dike. The consolidation of each soil is calculated for each layer and summed up for all soils to determine the total settlement at each point. Differential settlements are calculated between each toe and crest, toe and centerline, and crest and centerline on both sides of the dike. Recommended maximum differential settlements can be found in Reference 4.

3.2.3.4 Liquefaction Analysis

The liquefaction analysis determines the potential for liquefaction of the dike and foundation soils to occur during seismic events.

Factors which most influence liquefaction potential are: soil type, relative density, initial confining pressure, and the intensity and duration of earthquake motion (Reference 4). Reference 4 provides information on seismic risk zones of the U.S. and on the range of seismic parameters for source zones. Methods for estimating the potential for liquefaction are provided in the GARDS software package described in Section 3.2.3.5. Additional methods and charts for estimating the liquefaction potential can be found in References 14, 15, and 16, Chapter 11.

3.2.3.5 Geotechnical Analysis for Review of Dike Stability (GARDS)

A computer software package called Geotechnical Analysis for Review of Dike Stability (GARDS) has been developed by EPA's Risk Reduction Engineering Laboratory (RREL) to assist permit writers and designers in evaluating earth dike stability. GARDS details the basic technical concepts and operational procedures for the analysis of site hydraulic conditions, dike slope and foundation stability, dike settlement, and liquefaction potential of dike and foundation soils. It is designed to meet the expressed need for a geotechnical support tool to facilitate evaluation of existing and proposed earth dike structures at hazardous waste sites.

The GARDS software package is available from RREL, and a technical manual explaining its operation, Reference 4, is also available. Both the software and this support documentation contain text explanations and graphic examples designed to guide the user through the customary steps of earth dike analysis. User-friendliness is accomplished through the use of menu selection of available program options, including data check and simplified editing procedures, automatic internal check of input parameter values, cautionary statements regarding the recommended sequence of program options, and error diagnostic statements with interactive instructions for corrective action.

GARDS is designed to guide the reviewer through the customary steps of earth dike analysis considering slope stability, settlement, liquefaction, hydraulic flow and pressure conditions. GARDS includes an internal automatic search routine to determine the critical failure surface for both rotational (slip circle) and translational (wedge) stability analyses; an internal, automatic search routine to locate zones of greatest liquefaction potential and to compute total and differential settlements of foundation soils; and internal finite element hydraulic analysis to determine the steady state piezometric surface through the section, including the case of an impermeable barrier such as soil liner; the ability to model excess pore pressure conditions produced by confined steady flow and evaluate slope stability and any resulting uplift conditions; and the ability to determine the maximum exit gradient which defines the potential for piping failure (Reference 4).

The GARDS user must identify the various site conditions which need to be investigated and select the appropriate combination of options which best models those conditions. GARDS offers the user six idealized hydraulic boundary conditions, three stability options (slope stability, settlement, liquefaction), and three soil shear strength options: Unconsolidated Undrained (UU); Consolidated Undrained (CU); and Consolidated Drained (CD). A limited amount of guidance logic has been built into GARDS to assist the non-specialist user in making decisions regarding the available analysis options (Reference 4).

GARDS incorporates summary output block which allows the user to obtain a hard copy of the input data and the results of all analyses run for the dike section under study. The critical factors of safety, failure circle center coordinates, radius, and plane failure line segment coordinates are all highlighted in the output listing, along with the computed differential settlements, liquefaction potential, and critical exit gradient. If an analysis was not run, this is indicated in a summary table at the end of the output listing.

3.3 Materials/Specifications

Material and construction specifications should be provided as appropriate for all load supporting embankments.

3.3.1 Subgrade Requirements

The subgrade requirements for slope stability are the same as these addressed in Section 2.4.1 for foundation materials.

3.3.2 Borrow Materials

The native soil at the facility excavated during foundation excavations is the ideal backfill material from the standpoint of cost and convenience. However, if the native soil is not suitable, a suitable soil from a nearby borrow source should be utilized (Reference 2). At a minimum, material specifications should provide the range of acceptable materials. All materials should then be required to meet the minimum requirements of the national specifications as verified through specified field testing.

3.3.2.1 Selection

Once potential borrow sources have been identified, the site should be investigated (see Section 3.2.1) to determine the amount of suitable materials present at the site and the degree of spatial variability of material properties in the soil deposits. The investigation results can also be used to plan an efficient extraction procedure for the materials (Reference 7).

As discussed in Section 3.4, representative samples of the borrow material are subjected to laboratory compaction and hydraulic conductivity tests to establish the relationships among water content, density, compaction effort and permeability (Reference 7).

When suitable soils are not available at an economic distance from the facility, the engineer may recommend blending an additive, such as bentonite, to the native soil in order to achieve the proper material properties and performance (Reference 7).

3.3.2.2 Test Fill

Laboratory results and design assumptions need to be verified in the field. This verification can be accomplished through a test-fill program. The test-fill program allows the engineer to establish the material, equipment and construction procedures required to meet the design requirements for the fill materials that comprise the dike. The test-fill program is also a convenient tool for evaluating critical performance standards such as shear strength, density, and permeability (Reference 7).

Test fills, if used, should be constructed for each borrow source and whenever significant changes occur in the material, equipment, or procedures used, to construct the soil liner (Reference 7). Samples of the test fill should be obtained for testing to assure that the materials meet the minimum specifications.

3.4 Embankment Construction

Embankment construction for landfills or surface impoundments involves standard earthwork construction practices. Dike construction activities include fill placement and compaction, drainage system construction, and implementation of erosion control measures (Reference 7).

3.4.1 Compacted Fill Construction

Compacted fill may be part of the dike core, the dike shell, or may constitute the entire dike. Critical construction activities include emplacement, conditioning, and compaction. To insure that these activities are conducted properly, the following measures must be taken (Reference 7, p. 16):

- Placing loose lifts to the thickness established during the test fill program
- Removing or reducing clod size material to a maximum size as determined in the test fill
- Providing uniform compaction coverage using the type of equipment and number of passes specified in the test fill program
- Ensuring uniformity of backfill material
- Protecting the surface lifts from desiccation or frost action
- Scarifying between compacted lifts
- Ensuring adequate connection between lifts

3.4.2 Drainage Systems Installation

Installation procedures and equipment for dike drainage systems are similar to those for leachate collection systems. The observations and tests that are necessary to monitor the installation of drainage system components are discussed in Section 4.4.3 of this guide (Reference 7, p. 39).

3.4.3 Erosion Control Measures

Erosion control measures are applied to the outer slopes of embankments and may include benches and vegetative covers. The construction activities necessary for ensuring the quality of erosion control measures are the same as those for topsoil and vegetation subcomponents of cover systems discussed in Section 1. (Reference 7).

3.5 Quality Assurance/Quality Control (QA/QC)

Observation of the construction process is the most effective approach to QC, coupled with a well-defined testing program. Beyond the minimum specified test frequency and spacing, visual observations are used to identify problem areas and to call for more intensive testing to document and delineate any substandard backfill areas. Typical items to be on the lookout for include wet spots, large clods in backfill material, effects of exposure to frost, erosive effects of heavy rains and surface water runoff, poor bonding between lifts due to lack of scarification, and inclusion of undesirable foreign objects. Remedial actions (e.g., removal and reconstruction) are then ordered for the substandard areas so delineated. A qualified inspector should be on the site at all times during construction (Reference 5).

3.5.1 Compaction

During compaction of each lift, compactive effort and uniformity of compaction should be observed and recorded. Compactive effort is estimated by the number of passes or equipment of a known size and weight that will achieve the design specifications for the fill material (Reference 5). The compaction effect, the testing program and the fill's engineering properties are established by the test fill program.

Design specifications usually require achievement of a minimum percentage of the maximum density at a specified range of water contents (i.e. ASTM methods D698 or 1557). The specified density/water content corresponds to the density/water content at which the minimum specified soil properties can be achieved. This density/water content is then tested during quality control of the backfilling (Reference 5).

Specific tests to ensure that compaction results correspond to design standards include field density tests (nuclear, sand-cone and others), field water content measurements, laboratory compaction tests, and both field and laboratory permeability tests. The methods and QC measures for conducting these tests may be found in several documents (References 2, 8, 9). The main tools used for controlling the quality of compaction are field density and water content measurements, with supplementary laboratory compaction tests to monitor changes in soil material. Presently, nuclear probes are often used to measure field density and water content because of ease and quickness of testing. However, nuclear devices must be calibrated for each soil that is to be tested. In addition, if nuclear devices are used, other field density and moisture content measurements, such as sand cones and oven drying, should be made periodically to confirm nuclear results. Again, it is necessary to measure density, moisture, and compactive effort in the field to ensure that the required density and hydraulic conductivity is achieved during compaction (Reference 5).

Minimum test frequency and test spacing should be specified for all tests in the test plan (Reference 5).

Thin-walled tube or block samples may be taken for laboratory hydraulic conductivity tests (ASTM D 1587; Reference 9), or field hydraulic conductivity tests may be performed using techniques such as a sealed double-ring infiltrometer. Several design engineers recommend that sealed water content/density measurements and thin-walled tube samples for laboratory hydraulic conductivity tests be obtained from the lift underlying the lift that has just been compacted. Following thin-walled tube sampling or nuclear density measurement, the resulting hole is filled with backfill material and hand-tamped or is grouped with bentonite (Reference 5).

Upon completion of the dike, QC personnel should check that it is rolled smooth to seal the surface so that precipitation and/or leachate can run freely to the leachate collection sump. The completed dike should be surveyed to ensure that thickness, slope, and surface topography are as required by the design specifications. Seals around objects penetrating the slope and dikes (e.g., leak detection system stand pipes) also should be checked for integrity (Reference 5).

3.5.2 Backfill Material Inspection

Inspection of the backfill material can be largely visual; however, QC personnel conducting this inspection must be experienced with visual-manual soil classification techniques (Reference 9; ASTM D 2488). Changes in color or texture may indicate a change in soil type or soil water content. The soil also may be inspected for roots, stumps, and large rocks. In addition, as a check of visual observations, samples of the soil usually are taken and tested to ensure that the soil's index properties are within the range stated in the specification. The number of index tests to be conducted depends on site-specific conditions (i.e., soil type and heterogeneity) and the experience of the QC personnel. Usually a minimum number of tests per cubic yard of material is specified, with additional tests required by the inspector if visual observations suggest a change in soil type (Reference 5).

When bentonite/soil mixtures are specified, incoming bentonite should be inspected to ensure that its quality is as specified. For all bentonite shipments, certification of compliance with material specification should be obtained from the manufacturer or supplier. In addition, the quality of the arriving bentonite should be tested frequently for dry fineness, pH, and viscosity and fluid loss of a slurry made from the bentonite. Dry fineness is the percentage passing a 200-mesh sieve. It is necessary to control dry fineness to ensure proper mixing of the bentonite (Reference 5). Slurry viscosity, slurry fluid loss, and pH are standard tests specified by the American Petroleum Institute (Reference 10).

3.6 References

1. *Code of Federal Regulations* Title 40, Parts 264 and 270, July 1, 1987.

2. U.S. Department of the Navy, May 1982, *Engineering Design Manual NAVFAC DM-7-1,* Naval Facilities Command, Washington, D.C.

3. Sowers G.F. 1979, *Soil Mechanics and Foundations: Geotechnical Engineering,* The MacMillan Company, New York.

4. Technical Manual: *Geotechnical Analysis for Review of Dike Stability (GARDS)* developed by R. M. McCandless, Dr. A. Bodoczi, and P. R. Cluxton of the University of Cincinnati, for the Hazardous Waste Engineering Laboratory, Office of Research and Development, U. S. Environmental Protection Agency; Cincinnati, Ohio 45268; EPA Contract No. 68-03-3183, Task 19, March 1986.

5. Draft Technical Resource Document: *Design, Construction, and Evaluation of Clay Liners for Waste Management Facilities*; Hazardous Waste Engineering Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency; Cincinnati, Ohio 45268; EPA/530-SW-86-007, March 1986.

6 *Permit Applicant's Guidance Manual for Hazardous Waste Land Treatment, Storage and Disposal Facilities* (Volumes 1 and 2), Office of Solid Waste and Emergency Response, U.S. Environmental Protection Agency; Washington, D.C. 20460; SW-970, January 1984.

7. Technical Guidance Document: *Construction Quality Assurance for Hazardous Waste Land Disposal Facilities*; Hazardous Waste Engineering Research Laboratory, Office of Research and Development, U. S. Environmental Protection Agency, Cincinnati, Ohio 45268; EPA Contract No. 68-02-3952, Task 32; October 1986.

8. U.S. Department of the Army, 1970, *Laboratory Soils Testing* EM-1110-2-1906, Office of the Chief of Engineers, Washington, D.C.

9. ASTM, 1985, *The American Society for Testing and Materials 1985 Annual Book of ASTM Standards* Volume 4.08, "Soil and Rock; Building Stones," Philadelphia, Pennsylvania.

10. API 1982, American Petroleum Institute, *API Recommended Practice: Standard Procedure for Testing Drilling Fluids* API RP13B, American Petroleum Institute, Dallas, Texas

11. Boutwell, G.P. Jr., R. B. Adams, and D.A. Brown, 1980. Hazardous Waste Disposal in Louisiana. Geotechnical Aspects of Waste Disposal. American Society of Chemical Engineers, A Two Day Seminar.

12. *An Engineering Manual for the Evaluation of Stability of Dikes,* Permit Writer's Training Program, U.S. EPA, 1983.

13. *Groundwater*. Freeze and Cherry, Prentice-Hall, Englewood Cliffs, New Jersey, 1979.

14. Seed, H. Bolton, et. al., "Evaluation of Liquefaction Potential Using Field Performance Data"; In: *Journal of Geotechnical Engineering*; Vol 109, No. 3, March 1983.

15. Seed and Idriss; "Simplified Procedure for Evaluating Soil Liquefaction Potential"; In: *Journal of the Soil Mechanics and Foundations Division ASCE* Vol. 9, No. 7, September, 1971, p. 1,249-1,273.

16. Das, Braja M.; *Fundamentals of Soil Dynamics*; Elsevier Science Publishing Co., Inc., 1983.

Chapter 4.0
Liner Systems

Current regulations (40 CFR Part 264) require that hazardous waste land disposal facilities be constructed with liner systems that prevent any migration of wastes out of the unit. A single liner system required for certain units identified in Section 4.1, consists of one liner and one leachate collection system as shown in Exhibit 4-1. A double liner system required for new units, also identified in Section 4.1, includes two liners, primary and secondary, with a primary leachate collection system, above the primary (top) liner and a secondary leak detection/leachate collection system between the two liners, as shown on Exhibit 4-2. The term "liner system" includes the liner(s), leak detection/leachate collection system(s), and any special additional structural components such as filter layers or reinforcement. The major components of both single and double liner systems are the following:

- Low-permeability soil liners,
- Flexible membrane liners (FML), and
- Leachate Collection and Removal Systems (LCRS).

This chapter will discuss the regulatory performance requirements for liner systems and will provide criteria for the design and construction of liner systems and the review of liner system designs and construction plans on a component-by-component basis. This discussion will begin with low-permeability soil liners, usually the lower-most system component, and end with leachate collection and removal systems. Each component will be discussed with respect to its purpose, design configuration and design calculations, material specifications, and construction specifications, with an additional discussion of quality control issues specific to that component.

4.1 The Regulations and Performance Standards

There are two "sets" of regulations and standards that apply to permitted landfills and surface impoundments. The first is in 40 CFR 264.221(a) and 264.301(a) (Reference 1) and applies only to portions of existing units that are not covered by waste at the time of permit issuance. For landfills, this regulation requires only a single liner and a leachate collection and removal system above the liner. This system would include only the synthetic or composite liner and the LCRS shown on Exhibit 4-1 (Reference 6).

For a surface impoundment, the corresponding regulation in 40 CFR 264.221(a) requires only a single liner (synthetic, soil, or composite) and no LCRS. Waste piles are required by 40 CFR 264.251 to have only a single liner with a LCRS above it, similar to landfills.

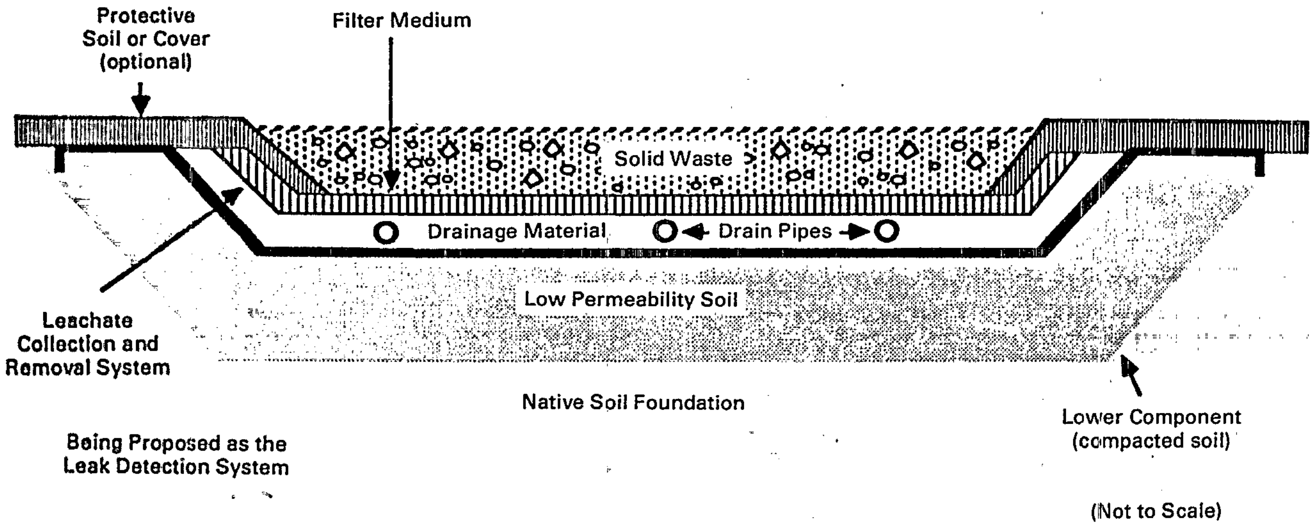
The other "set" of regulations and standards were mandated by Section 3004(o) of HSWA, are codified in 40 CFR 264.221(c) and 264.301(c) (Reference 1), and apply to:

- Each new landfill or surface impoundment,
- Each new landfill or surface impoundment at an existing facility,
- Each replacement of an existing landfill or surface impoundment unit, and
- Each lateral expansion of an existing landfill or surface impoundment unit.

These regulations, containing the minimum technological requirements (MTR) mandated by HSWA Section 3004 (o), require two or more liners and a leachate collection system between the liners in a surface impoundment, and two or more liners and two leachate collection systems in a landfill. The basic landfill design is that shown in Exhibit 4-2 (Reference 7). The basic surface impoundment design is similar to the landfill, without the uppermost LCRS. This chapter will discuss the design and construction of the three major liner system components identified earlier and will provide references containing detailed information to design and to evaluate these systems.

There is some inconsistency in the regulations and the guidance concerning terminology of LCRS, with the lower system occasionally referred to as a leak detection or leachate detection system. This document will use the term "primary LCRS" to refer to the system above the top liner in a landfill (leachate collection zone), and "secondary LCRS" to designate the system between the liners (leak detection/leachate collection zone).

Exhibit 4.1. Schematic of a Single Clay Liner System for a Landfill

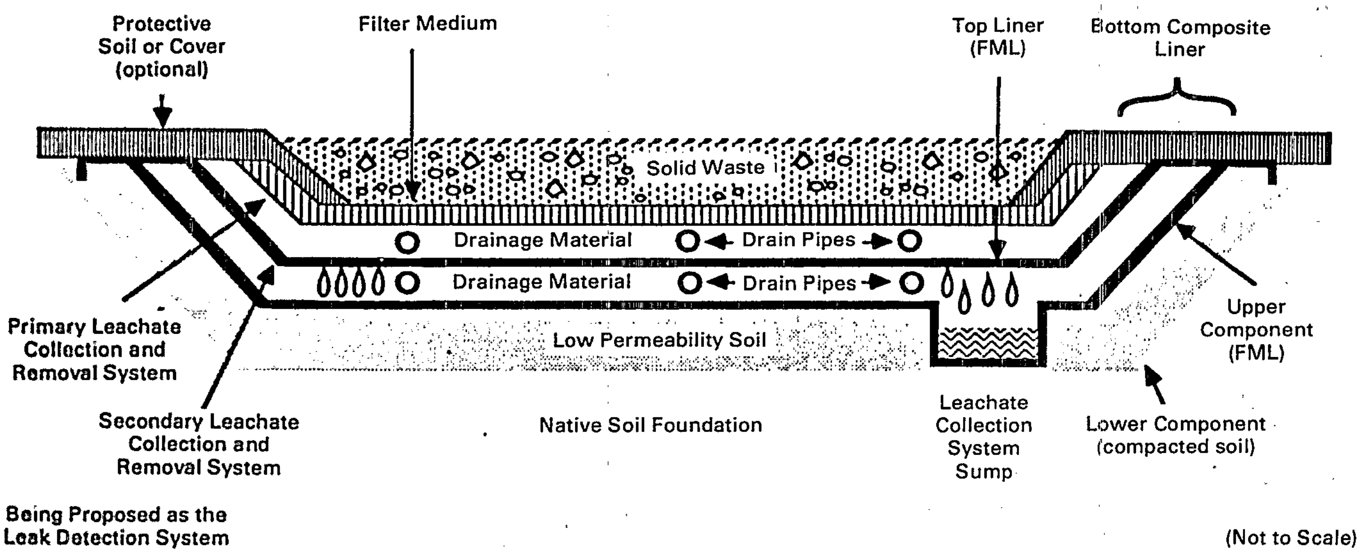


4.1.1 Low-Permeability Soil Liners

For soil liners, the regulations in 40 CFR 264.221, 264.251, and 264.301 (Reference 1) apply to surface impoundments, waste piles, and landfills, respectively.

For surface impoundments and waste piles that will be closed by removal, a soil liner must prevent migration of wastes out of the unit, that is, through the liner, during the active life of the facility. For surface impoundments that will be closed in place

Exhibit 4-2. Schematic of a Double Liner and Leak Detection System for a Landfill



and for landfills, the regulations require that liners be constructed of materials that prevent wastes from migrating into the liner; this effectively requires a synthetic primary liner (flexible membrane liner), since soil liners cannot prevent migration into the soil. For those units required to have double liners, the secondary liner must meet the above performance standard, that of preventing migration through the liner. The criteria by which a soil liner is determined to meet this standard are discussed in Section 4.2.1.

4.1.2 Flexible Membrane Liners

The requirements of 40 CFR 264.221(a) for surface impoundments, 264.251(a) for waste piles, and 264.301(a) for landfills state that "the liner must be designed, constructed, and installed to prevent any migration of wastes out of the impoundment to the adjacent subsurface soil or ground water or surface water at any time during the active life (including the closure period)" of the surface impoundment, waste pile, or landfill. The liner must be constructed of materials that have appropriate chemical properties and sufficient strength and thickness to prevent failure due to pressure gradients (including static head and external hydrogeologic forces), physical contact with the waste or leachate to which they are exposed, climatic conditions, the stress of installation, and the stress of daily operation." (Reference 1). In short, the regulations require FMLs to have physical strength, low permeability, and chemical compatibility with the waste contained by the liner. EPA regulations and guidance refer to synthetic liners as FMLs (flexible membrane liners). The manufacturers of these materials, as well as a large number of engineers, refer to these materials as geomembranes.

On March 28, 1986, the EPA proposed a rule implementing minimum technology requirements (MTR) for double liner systems and leachate collection systems. These regulations implement the statutory requirements of HSWA (Reference 2). Following proposal of the MTR regulations, the EPA collected data characterizing and comparing the performance of compacted soil bottom liners and composite (soil/FML) bottom liners. This data was evaluated with respect to leachate collection efficiency, leak detection capability, and leakage, both into and out of the bottom liner; and the data indicated that the use of an FML improved the performance of a composite bottom liner over that of a compacted soil liner with respect to all three parameters. (Reference 3). On April 17, 1987, the EPA made available the background document presenting the data on bottom liner performance (Reference 4) and the draft minimum technology guidance documents on single and double liner systems [References 5, 6, 7 and 3].

4.1.3 Leachate Collection and Removal Systems

The leachate collection and removal system (LCRS) regulations for single-lined waste piles and landfills specifically require that the system be designed and operated to ensure that the leachate depth over the liner does not exceed 30 cm (one foot). The system must also be chemically resistant to wastes and leachate, sufficiently strong to withstand landfill loadings, and protected from clogging through the scheduled closure of the unit [Reference 1, 40 CFR 264.251(a) and 264.301(a)].

The regulations concerning double liner systems for new surface impoundments [40 CFR 264.221(c)] and landfills [40 CFR 264.301(c)] require only that the liner and leachate collection systems protect human health and the environment. However, minimum technology guidance for double liners (Reference 7, pages 4-5 and 43-44) provides specific design criteria for primary and secondary LCRS. These criteria include the following:

- the primary system should be capable of maintaining a leachate head of less than 30 cm (one foot) (Reference 7, p. 1);
- each system should have at least a 12-inch thick granular drainage layer that is chemically resistant to the waste and leachate, with a hydraulic conductivity not less than 1×10^{-2} cm/sec, or an equivalent synthetic drainage material (geonet), and with a minimum bottom slope of two percent;
- the primary system should have a granular or synthetic fabric filter (geotextile) above the drainage layer to prevent clogging;
- both systems should have a drainage system of pipes to efficiently collect leachate; the pipes should have sufficient strength and chemical resistance to perform under landfill loadings.

Section 4.2.3 of this Guide will discuss how an LCRS is designed to meet the regulations and guidance, and how a design can be evaluated to determine if it meets these requirements.

4.2 Design and Materials Selection

4.2.1 Low-Permeability Soil Liners

Low-permeability soil liner design is site- and material-specific. Prior to design, many fundamental yet important criteria should be considered, such as:

- in-place permeability of the liner;
- liner stability against slope failure, settlement and bottom heave;
- liner - waste compatibility; and
- long-term integrity of the liner.

Important criteria to consider when reviewing a design for a soil liner include liner site and material selection; liner thickness; strength and bearing capacity; slope stability and controls for liner failure. Exhibit 4-3

Exhibit 4-3. Types of Information Used to Demonstrate that the Performance Standard for Soil Liners is Met

Information	Typical Parameters
Description of Soil Liner	Description of: • Classification • Thickness • Hydraulic conductivity • Source of soil • Any recompacting and amendments (in-place soil) • Location of borrow area and any amendments (borrow material)
Material Testing Data	Test results for: • Index tests • Hydraulic conductivity • Strength • Consolidation • Shrink-swell properties • Potential for dispersion/piping of soil due to liquid flow through liner
Liner Compatibility Data	Results of hydraulic conductivity testing of liner material with representative leachate.
Liner Thickness	For units with only one FML, demonstration that the soil liner's thickness is sufficient to retard liquid flow-through during operating life and post-closure period.
Construction Specifications	Procedures for liner installation, including: • Method of compaction • Degree of compaction, water content to be achieved • Lift thickness • Methods to alter water content of soil • Scarification requirement between lifts • Method of amending soil, if applicable
Construction Quality Assurance	Description of QA program and testing procedures.

summarizes the types of design information and technical parameters commonly used to demonstrate that the performance standards are met, if low-permeability soil liners are used at a facility. These design considerations are important throughout the installation and construction phases of a clay liner. The following sections provide information on design criteria.

4.2.1.1 Site and Material Selection

A site investigation should be conducted prior to design, and the following factors should be considered:

- Site geology,
- Topography (especially drainage patterns),
- Analyses of soil properties,
- Field and laboratory hydraulic conductivity,
- Bedrock characteristics,
- Hydrology, and
- Climate.

All these factors are important to the design of the soil liner. The site will require a foundation designed to control settlement and seepage and to provide structural support for the liner. If satisfactory contact between the liner and the natural foundation is achieved, settlement and cracking will be minimized. Engineering designs and test methods for site foundation construction are discussed in Chapter 2 of this Guide.

Soil liners must meet the following requirements:

- A field hydraulic conductivity of 1×10^{-7} cm/sec when compacted;
- Sufficient strength after compaction to support itself and the overlying materials without failure; and
- Compatibility with hazardous wastes or leachates to be contained at the site.

Soil liner material may originate at the site or may be hauled in from a nearby borrow site if the native soil is not suitable. If suitable soils are not available on site or from a borrow site, it may be necessary to introduce a soil additive such as bentonite to increase performance potential of the selected material. Although additives such as bentonite are known to decrease hydraulic conductivity, it is important to test additives under actual field conditions as with any potential soil liner material (Reference 8, pp. 9-25).

Because physical properties differ from one soil to the next, testing procedures are necessary to assist in the selection of liner material. Once potential soil sources have been identified, it is necessary to begin testing to eliminate undesirable soils or to determine whether the source requires an amendment. Many procedures have been standardized for soil testing by organizations such as the American Society of Testing and Materials (ASTM) and by individuals currently researching clay soils for use in soil liner construction (Reference 9; Sec. 2.3.4 and Reference 10).

Representative samples of the proposed material must be subjected to laboratory testing. This will establish the *properties* of the material with respect to water content, density, compactive effort and hydraulic conductivity. As will be discussed in Section 4.3.1.2.3 of this chapter, clay soils exhibit characteristic changes when compacted. Therefore, all analyses of a potential material must be performed on a compacted sample. Exhibit 4-4 provides a listing of pertinent soil tests and methods (Reference 9, p. 84).

Exhibit 4-4. Methods for Testing Low-Permeability Soil Liners (Reference 9)

Parameter to be Analyzed	Methods	Test Method Reference
Soil type	Visual-manual procedure	ASTM D2488
	Particle size analysis	ASTM D422
	Atterberg limits	ASTM D4318
	Soil classification	ASTM D2487
Moisture content	Oven-dry method	ASTM D2216
	Nuclear method	ASTM D3017
	Calcium carbide (speedy)	AASHTO T217
In-place density	Nuclear methods	ASTM D2922
	Sand cone	ASTM D1556
	Rubber balloon	ASTM D2167
	Drive cylinder	ASTM D2937
Moisture-density relations	Standard effort	ASTM D698
	Modified effort	ASTM D1557
Strength	Unconfined compressive strength	ASTM D2166
	Triaxial compression	ASTM D2850
Cohesive soil consistency (field)	Penetration tests	ASTM D3441
	Field vane shear test	ASTM D2573
	Hand penetrometer	Horslev, 1943
Hydraulic conductivity (laboratory)	Fixed-wall double ring permeameter	EPA, 1983 SW-870 Anderson et. al., 1984
	Flexible wall permeameter	Daniel et al., 1985 SW-846 Method 9100 (EPA, 1984)
Hydraulic Conductivity (field)	Sealed Double-Ring Infiltrometer	Day and Daniel, 1985
	Sai-Anderson-Gill double-ring Infiltrometer	Anderson et al., 1984

Compatibility testing of liner soils to wastes or waste leachates should be conducted as part of the material selection process. Compatibility testing procedures and the problems associated with them will be discussed in Section 4.3.1.5 of this chapter.

4.2.1.2 Thickness

Transit time prediction methods can be used to determine the required thickness of a soil liner. These methods are used to determine the rate at which hazardous constituents will eventually pass through the liner if the overlying FML is ever breached. This transit time equation is instrumental in determining liner thickness for new liners and in evaluating performance of existing liner systems (Reference 11, pp. 161-165 and Reference 8, pp. 9-25). Transit time prediction methods are seriously flawed unless they accurately account for channel or bypass flow within the soil liner as a whole. For this reason, laboratory and field hydraulic conductivity values must be determined under careful quality control, so that they reflect the actual soil liner performance. The regulations (Reference 1, 40 CFR 264.301) state that a soil layer 3-feet thick, with a hydraulic conductivity of 1×10^{-7} is assumed to satisfy the regulatory standard discussed in 4.1.1.

Liners are designed to be of uniform thickness over the entire facility. Thicker areas may be encountered wherever there may be recessed areas for leachate collection pipes or collection sumps. Some engineers suggest extra thickness and compactive effort for the edges of the sidewalls to adequately tie them together with the liner itself. In smaller facilities, a soil liner may be designed for installation over the entire area, but in larger or multi-cell facilities, liners are designed in segments. If this is the case, it will be necessary to specify in the design a beveled or step-cut joint between segments to ensure the segments properly adhere together (Reference 12, pp. 5-27).

4.2.1.3 Hydraulic Conductivity

The coefficient of permeability or hydraulic conductivity expresses the ease with which water passes through a soil. Hydraulic conductivity is the most critical criterion for a potential soil liner. The hydraulic conductivity of a soil is dependent on the viscosity and density of the permeant liquid and on the size, shape and area of the conduits through which the permeant liquid flows. Darcy's Law is the basis for the equation of hydraulic conductivity. The hydraulic conductivity of a partially saturated soil will be less than the hydraulic conductivity of the same soil when saturated. This is due to air in the voids (pore space, cracks etc.) reducing the flow cross section and blocking the smaller voids completely (Reference 13, pp.59-63). Clay minerals also affect the hydraulic conductivity of a soil because they are small and, therefore, have smaller pores creating a lower hydraulic conductivity value. (Reference 14, p. 155). Most importantly, clod remnants and improper scarification have the greater affect on soil hydraulic conductivity. This is due to remnants creating improper moisture conditions and large voids. Poor scarification may cause shifting in lifts.

When designing a soil liner, field hydraulic conductivity is the most important factor to consider. Therefore, conductivity tests must be conducted on the proposed soil to evaluate liner performance. Test fills are the most accurate method of determining hydraulic conductivity because laboratory values generally are lower than those measured in test fills or actual liners. Therefore, it is essential that field hydraulic conductivity testing be conducted on a test fill area or on the liner itself (Reference 9, pp. 19-24).

4.2.1.4 Strength and Bearing Capacity

Another important criterion to consider when designing a soil liner is the strength and bearing capacity of the liner material. Analysis of these parameters will determine the stability of the liner material. Simply, the bearing capacity of a soil is its ability to withstand overburden pressure without the soil failing from within. More detailed discussions of bearing capacity and strength can be found in Chapter 2 of this Guide; Reference 15, Chapter 4, and in Reference 12, Chapter 3.

4.2.1.5 Slope Stability

The strength of a soil also controls its resistance to sliding. Failure of a liner slope can result in slippage of the compacted soil liner along the excavated slope. This would result in a breach of the liner and allow potential pollutants to escape into the environment. Therefore, analysis of slope stability must be considered in the design of a soil liner. More detail on slope stability analysis is provided in Chapter 3 of this Guide. In addition, a detailed discussion of slope stability can be found in Reference 12, Chapter 5 and Reference 13, Chapter 4.

4.2.2 Flexible Membrane Liners (FMLs)

The design of a lined hazardous waste management unit requires consideration of more than the performance requirements of the FML; it also requires careful design of the foundation supporting the FML (see Chapter 2 for a discussion of foundation design). The foundation provides support for the liner system, including the FMLs and the leachate collection and removal systems. If the foundation is not structurally stable, the liner system may deform, thus restricting or preventing its proper performance. Severe deformation of the liner system may result in failure in any of its components (References 10 and 11). Additional information on foundation design and slope stability is found in Chapters 2 and 3 of this Guide, as well as in References 16, 17, and 18.

4.2.2.1 Performance Requirements of the FML

The performance requirements of an FML used in a hazardous waste management unit include low permeability, chemical compatibility, mechanical compatibility, and durability. The land disposal unit designer must specify the necessary criteria for each of these properties based on engineering requirements, performance requirements of the unit, applicable regulatory requirements, and the specific site conditions. Exhibit 4-5 summarizes the types of design information and technical parameters used to demonstrate that the FML performance standards are met. In addition, the FML design must be compatible with the present technology used in the installation of FMLs (Reference 19, Chapter 7).

Exhibit 4-5. Types of Information Used to Demonstrate that the Performance Standards for Flexible Membrane Liners are Met

Information	Typical Parameters
Description of Flexible Membrane Liner (FML)	Descriptions of: • Type of FML • Material of construction • Thickness • Brand Name • Manufacturer • Detailed material specifications
Liner Compatibility Data	Results of liner/waste compatibility testing demonstrating strength and performance adequate after exposure to representative waste and to both primary and secondary leachates
Liner Strength	Demonstration that liner and seams will have sufficient strength to support expected loads/stresses after exposure to waste and leachates
Adequacy of Liner Bedding	Demonstration that sufficient bedding will be provided above and below FMLs to prevent rupture during installation and operation.
Construction Specifications	Procedures for placement of FMLs including: • Inspection of liner bed for protrusions • Placement procedures • Liner seam bonding techniques • Procedures of protection of liner before and during covering • Placement of protective layers

4.2.2.2 Selection of the FML

The performance requirements determined by a designer/engineer serve as the basis for the selection of an FML for a given facility. Based upon the designed use of the unit, the designer must make decisions on the composition, thickness, and construction (fabric-reinforced or unreinforced) of an FML. Composition of the liner is based primarily on chemical compatibility. Mechanical compatibility and sometimes permeability determine the thickness of the FML sheeting. It should be noted that liner performance does not correlate directly with any one property (e.g., tensile strength) and that specifications that appear in specific technical resource documents such as Reference 19 should not be used alone as the basis for selection of an FML.

FMLs are based on polymeric materials, particularly plastics and synthetic rubbers. There are four general types of polymeric materials used in the manufacture of FML sheeting (Reference 19, Chapter 4):

- Thermoplastics and resins, such as PVC and EVA; and
- Semicrystalline plastics, such as polyethylenes.

The various polymers are used to make a variety of liners that can be classified by production process and reinforcement. Hazardous waste management unit liners are normally constructed using factory-manufactured sheeting.

4.2.2.2.1 Polymers Used in FMLs

A variety of polymers have been used in FMLs to line waste management facilities. This section will discuss those polymers that are currently used in the manufacture of FML sheeting. Exhibit 4-6 lists the polymers currently used in lining materials (Reference 19, Table 4-4).

The polymers used in FMLs have different physical and chemical properties, and they also differ in methods of installation and seaming, costs, and chemical compatibility with wastes. Composition and properties of compounds within a generic polymeric type can also differ considerably since polymers are usually not used alone in a product. Polymers are usually compounded with a variety of ingredients to improve properties and to reduce compound cost. The other ingredients mixed with the polymer include fillers, plasticizers or oils, antidegradants, and curatives. Some compounds used in FMLs use a blend, or alloy, of two or more polymers to improve specific properties (Reference 19, Chapter 4).

Reference 19, Chapter 4 provides detailed information about the composition and properties of each of these polymers.

4.2.2.2.2 Seaming of FML Sheeting

The construction of a continuous watertight FML is critical to the containment of hazardous waste and is heavily dependent on the construction of the seams bonding the sheeting together. The seams are the most likely source of failure in an FML. Seams are manufactured both in the factory and the field. Sheeting manufactured in relatively narrow widths (less than 90 inches) is seamed together to fabricate panels. These factory seams are made in a controlled environment and are generally of high quality. Both fabricated panels and sheeting of wider widths (21 to 64 feet) are seamed on site during the installation of the FML. The quality of field seams is difficult to maintain since the installer must deal with changing weather conditions, including temperature, wind, and precipitation, as well as construction site conditions, which include unclean work areas and work on slopes. Constant inspection under a construction quality assurance plan is necessary to ensure the integrity of field seams (see Section 4.5 of this chapter) (Reference 19, Chapter 4).

Several bonding systems are available for the construction of factory and field seams in FMLs. Bonding systems include solvent methods, heat seals, heat guns, dielectric seaming, extrusion welding, and hot wedge techniques. The selection of a bonding system for a particular FML is dependent primarily on the polymer making up the sheeting.

To ensure the integrity of seams, a given FML should be seamed using the bonding system recommended by the FML manufacturer (Reference 19, Chapter 4). Additional information on the applicability and performance of bonding systems is presented in Reference 19, Chapter 4.

At the present time, none of the available bonding systems can be used to repair leaks and damage in FMLs that are covered by wastes. Repairs can be made, however, in FMLs exposed to weather if the polymer has not degraded and the bonding surfaces are clean and dry (Reference 19, Chapter 4).

4.2.2.2.3 Properties and Characteristics of FMLs

The principal characteristics of concern regarding FML sheeting include:

- Low permeability to waste constituents,
- Strength or mechanical compatibility of the sheeting,
- Chemical compatibility with the contained waste, and
- Durability for the lifetime of the facility.

These characteristics are assessed through laboratory and pilot-scale testing of the various properties of FML sheeting (Reference 19, Chapter 4). The sheetings used for FMLs were developed by the rubber, plastics, and textile industries, and, consequently, three different groups of standard test methods were developed for testing these materials, since each industry developed inherently different products. The analyses and tests that are performed on FML sheeting measure its inherent analytical properties, physical properties, permeability characteristics, environmental and aging properties, and performance properties (Reference 19, Chapter 4). Testing is essential to the designer/engineer who uses the data to determine whether a specific FML sheeting will meet the design requirements of the waste facility. The selection of an FML should be

Exhibit 4-6. Polymers Currently Used in FMLs for Waste Management Facilities

Polymer	Type of compound used in liners: Thermoplastic	Type of compound used in liners: Cross-linked	Fabric reinforcement: With	Fabric reinforcement: Without
Chlorinated polyethylene (CPE)	Yes	Yes	Yes	Yes
Chlorosulfonated polyethylene (CSPE)	Yes	Yes	Yes	No
Elasticized polyvinyl chloride (PVC-E)	Yes	No	Yes	No
Polyester elastomer (PEL)	Yes	No	Yes	Yes
Polyethylene (LDPE, LLDPE, HDPE)	Yes	No	No	Yes
Polyvinyl chloride (PVC)	Yes	No	Yes	Yes

based on actual test data characterizing the various sheetings under consideration for lining a waste facility (References 16 and 21). These tests are discussed in detail in Reference 13, Chapter 4 and in References 19, 30 and 31.

4.2.2.2.4 Permeability

The primary function of a liner system in a hazardous waste management unit is to minimize and control the flow of hazardous waste from the unit to the environment, particularly the ground water. A properly designed FML has a low permeability to the waste contained within the liner, allowing it to perform its primary function in accordance with the MTR guidance. However, the permeability of an FML made of a particular polymer may change upon exposure to waste or leachate, depending on the composition of the waste contained by the FML. This property is the chemical resistance, or compatibility, of a particular polymer to specific chemicals. Since different plastics and rubbers exhibit various degrees of compatibility with different chemicals, a number of materials are used to manufacture FML sheeting. The material is selected based on exposure during its intended use. An FML that is compatible with a specific waste displays a low permeability toward that waste and will minimize its flow through the FML to the environment. Additional factors affecting the rate of transmission through the FML are the thickness of the FML sheeting and concentration of the chemical species (Reference 19, Chapter 5, and Reference 21).

4.2.2.2.5 Mechanical Compatibility

An FML must be mechanically compatible with the designed use of the lined facility in order to maintain its integrity during and after exposure to short-term and long-term mechanical stresses. Short-term mechanical stresses can include equipment traffic during the installation of a liner system, as well as thermal expansion and shrinkage of the FML during operation of the unit. Long-term mechanical stresses usually result from the placement of waste on top of the liner system or from differential settlement of the subgrade (Reference 19, Chapter 6).

Mechanical compatibility requires adequate friction between the components of a liner system, particularly the soil subgrade and the FML, to ensure that slippage or sloughing does not occur on the slopes of the unit. Specifically, the foundation slopes and the subgrade materials must be considered in design equations in order to evaluate:

- The ability of an FML to support its own weight on the side slopes,
- The ability of an FML to withstand down-dragging during and after filling,
- The best anchorage configuration for the FML, and
- The stability of a soil cover on top of an FML.

Mechanical compatibility requirements may affect the choice of FML material, including polymer type, fabric reinforcement, and thickness (Reference 13, Chapter 7 and Reference 21).

4.2.2.2.6 Durability

An FML must exhibit durability, that is, it must be able to maintain its integrity and performance characteristics over the operational life and the post-closure care period of the unit. The service life of an FML is dependent on the intrinsic durability of the FML material and on the conditions under which it is exposed. Exposure conditions can vary greatly within a given facility and an FML must resist the combined effects of chemical, physical, and biological stresses (References 6, 7, and 13, Chapter 7).

4.2.2.2.7 Chemical Compatibility

Chemical compatibility of FMLs and waste liquids or leachates is a critical factor in the service life of a liner system. Chemical compatibility requires that the mechanical properties of the FML remain essentially unchanged after the FML is exposed to the waste. If the seams between the sheets are made with materials other than the sheet parent products, they also must be compatible with the waste liquid. Incompatibility is due primarily to the absorption of waste constituents by the FML, the extraction of components of the FML compound by wastes or leachates, or reactions between FML constituents and wastes or leachates (Reference 13, Chapter 7).

Incompatibility may result in a failure of the FML material or of seams and consequent leakage of waste or leachates to the ground water. Due to the serious consequences resulting from incompatibility, an evaluation is required prior to permitting to determine the effects that waste will have on the FML proposed for installation at a facility.

Elevated exposure temperatures are believed, in general, to be an effective means of accelerating the effects of immersion on polymeric products and serve as the basis for Method 9090 immersion testing (Reference 29). However, elevated temperature acceleration is effective only for specific conditions. In some cases, elevated temperatures may change a polymeric product in ways that do not correlate with service at a lower temperature (Reference 13, Chapter 5). Limited data on field-exposed FMLs are available, and there has been an attempt to correlate the available information with data on FMLs exposed in simulated service environments (Reference 13, Chapter 6).

Evaluation of data obtained from compatibility testing is best performed by specialists knowledgeable in flexible membrane liners, FML testing, and EPA Method 9090. EPA has developed a computer advisory system, named the Flexible Liner Evaluation Expert (FLEX), that serves as a tool to assist in interpretation of data from Method 9090 tests. The following discussion provides details on both EPA Method 9090 and FLEX (Reference 36).

4.2.2.2.7.1 EPA Method 9090, Compatibility Test for Wastes and FMLs

EPA Method 9090 is used to assess the compatibility of a candidate FML with the specific waste liquid or leachate to be contained. This test simulates the conditions that the FML may encounter in service and assesses what effects, if any, the exposure to waste liquid has on the FML. The test involves immersion of a candidate FML for a minimum of 120 days at 23°C and 50°C in a representative sample of waste liquid. Physical and analytical tests are performed on unexposed FML sheeting to establish baseline data and on samples exposed to waste liquid for 30, 60, 90, and 120 days (Reference 29). The test procedure involves several steps, including (Reference 13, Chapter 5):

- Selection of representative samples of the waste liquid or leachate and the FML;
- Preparation of the exposure cells for operation during the 120-day exposure period;
- Exposure of the FML samples to the waste liquid or leachate in the simulated service environment;
- Physical and analytical testing of the unexposed and exposed FML samples; and
- Analysis of test data for trends during the 120-day exposure period.

The FML samples selected for exposure and testing should be free of flaws and defects in order that the test specimens prepared from them are, as nearly as possible, influenced only by exposure to the waste liquid in the simulated service environment. Maintenance of constant exposure conditions is important since variation in any of them may influence the quality of the data collected from testing the exposed samples. Factors critical to the performance of Method 9090 include (Reference 29):

- Use of exposure cells made of materials that are not reactive with the waste liquid (e.g., a stainless steel exposure cell may not be reactive with an organic leachate, but corrodes in the presence of an inorganic salt brine);
- Constant temperature of the waste liquid;
- Stirring of the waste liquid to prevent stratification of phases (unless the system being modeled allows stratification);
- Exchange of the waste liquid in the exposure cells monthly or more frequently, to maintain constant concentrations of constituents that may be reduced due to their uptake by the FML samples or volatilization into a headspace in the exposure cell; and
- Maintenance of zero headspace in sealed exposure cells to reduce the potential loss of volatile organic constituents from the waste liquid.

Method 9090 suggests testing semicrystalline FMLs (e.g., HDPE) for environmental stress-cracking since ethylene plastics are susceptible to this type of failure (Reference 29). Additional testing, not prescribed by Method 9090, may be appropriate in certain cases. Analytical testing may be performed on the unexposed FML samples to yield a fingerprint, which is a body of data that describes and identifies a specific FML (Reference 33). Fingerprinting is discussed in Section 4.2.2.2.8.

The data collected from Method 9090 testing are analyzed for any changes or trends over the four-month exposure period. The data should be compiled and analyzed as the testing proceeds, with a final analysis following the completion of testing. Interpretation of Method 9090 data involves a holistic assessment of all changes and trends in the data. The physical property values are assessed with respect to volatiles and extractables data, as well as dimensions and weight, to determine whether uptake of waste constituents or leaching of FML constituents occurred in the FML and how these two mechanisms affect physical properties. The effects of exposing an FML to a simulated or actual service environment may be one or more of three basic types (Reference 13, Chapter 5):

- Degradation of the polymer making up the FML;

- Extraction of plasticizers or other compounding ingredients in the FML; and/or
- Swelling of the FML due to absorption of organics and water.

Based on the evaluation of the total effect of the waste liquid or leachate on the FML in a simulated service environment, a judgment may be made on the long-term performance, serviceability, and durability of the FML in actual service.

4.2.2.2.7.2 FLEX - Flexible Membrane Liner Advisory Expert System

In an effort to simplify and standardize the analysis of Method 9090 data, the EPA developed FLEX, an acronym for Flexible Liner Evaluation Expert, which is a computer program designed to assist the reviewer in the data analysis process. The program is not a substitute for review of Method 9090 test results by a trained professional, but rather a screening tool to be used by those familiar with flexible membrane liners and their testing, especially Method 9090 compatibility testing (Reference 36, pp. 1-2). FLEX checks the Method 9090 test results for compliance with applicable National Sanitation Foundation (NSF) standards and requirements suggested by FML specialists. Limits have been set on the acceptable statistical variation of test values for a specific parameter. These limits have been determined through interviews with liner manufacturers and testing experts. The rules which are used to check the test results are listed in Appendix A of Reference 36.

FLEX Version 2.0, used to evaluate Method 9090 test results for HDPE, CSPE, and PVC liners, is available for use at EPA Regional offices. Version 2.0 is being distributed on a limited basis for the purpose of field testing and to obtain feedback concerning the system's performance and content (Reference 36, p. 15).

Details on the hardware requirements, contents of the FLEX System, procedures for installing the system, menus and data sheets are provided in the FLEX User's Guide (Reference 36). Operation of the system is straightforward and relatively simple since each screen in the program provides instructions to the user. The data required by FLEX is presented in Exhibit 4-7 alongside the types of data provided by Method 9090.

As a screening tool, FLEX pinpoints inconsistencies in the test data and test results according to a series of programmed decision-making statements and then recommends that the liner is substandard or incompatible if inconsistencies are found. However, the recommendations provided by FLEX should not be considered absolute, especially if the system finds no problems with the data. In this capacity as a screening tool, FLEX can save time, reduce oversights, and enhance the consistency of Method 9090 test reviews (Reference 36, p. 2).

4.2.2.2.8 Fingerprinting of FMLs

The fingerprint of an FML is the sum total of its analytical properties, which establish a body of data that can identify a particular FML. A fingerprint performed on an FML at the time of its installation can be used to (Reference 13, Chapter 4):

- Assess the quality of the specific sheeting being installed,
- Assure the designer/owner/operator of the facility that the sheeting being installed is equivalent to the sheeting tested in compatibility studies,
- Establish a baseline for assessing the effects of service exposure on the FML.

4.2.2.2.9 Effects of Exposure on FMLs

Until recently, information on the effects of exposure on FMLs was limited to laboratory studies of simulated service environments. Accelerated testing data are now being correlated with data collected on the actual field performance of FMLs. Case studies of field investigations and discussions of failure mechanisms are presented in Reference 13, Chapter 6.

Chemical compatibility plays the greatest role in the durability and service life of an FML in a hazardous waste management unit. Swelling of the FML, in particular, has the most serious effect on FMLs at hazardous waste facilities and can cause loss in strength, elongation, creep and flow, and loss in puncture resistance. Extraction of plasticizers from FML sheeting can result in embrittlement and shrinkage and possibly breakage of the FML. Dissolved organic constituents in the waste liquid or leachate, even in very low concentrations, can preferentially combine with organic liner materials and may cause the FML to degrade and possibly fail (Reference 13, Chapter 6).

4.2.2.3 Effect of FML Selection on Design

FMLs made from different polymers or rubbers have different physical characteristics that can affect the design of a liner system. The coefficient of friction between the FML and the foundation slopes is specific to a particular FML sheeting; this factor, as well as adequate anchorage of the FML, can determine whether the installed FML will slip down the foundation slopes. The use of an FML with a relatively low friction angle, such as HDPE, can affect the design of the anchor trench, or other anchorage, used to secure the FML at the top of the slope (Reference 13, Chapter 7 and Reference 21).

The coefficient of thermal expansion of the FML sheet can affect its installation and its service performance.

Exhibit 4-7. Data Requirements of FLEX

Data provided by Parameter	Data required Method 9090	Data points provided by FLEX	Minimum number required by Method 9090	FLEX
Width	in.	mm	3	3
Length	in.	mm	3	3
Thickness	in.	mm	3	3
Weight	lb.	grams	1	3
Tensile strength at yield	psi	psi	3M, 3T*	3
Tensile strength at break	psi	psi	3M, 3T	3
Elongation at yield	percent	percent	3M, 3T	3
Elongation at break	percent	percent	3M, 3T	3
Tear Resistance	lb/in. of width	psi	3M, 3T	3
Puncture Resistance	lb.	psi	2	3
Modulus of Elasticity	psi	psi	2M, 2T	3

*M = machine direction of test; T = transverse direction of test.

The FML should lie flat on the underlying soil, but shrinkage and expansion of the sheeting due to changes in temperature may result in excessive wrinkling or tautness in the FML. Wrinkles on the FML surface may affect drainage in the leachate collection and removal systems. Tautness of the FML may affect its ability to resist puncture and localized stresses on the seams. In addition to thermal expansion and contraction of the FML, residual stresses from manufacture remain in some FMLs and can cause shrinkage when the FML is heated by sunlight. The design of the FML may need to include provisions to deal with the dimensional changes resulting from thermal expansion and contraction (Reference 13, Chapter 7).

4.2.2.4 FML Layout

Upon selection of an FML, the designer must create a layout plan for the sheeting or panels used to construct the FML. The layout plan is a scale drawing showing where each sheet or prefabricated panel is placed within the FML; it also indicates the location of each seam in the FML. The FML layout must take into account the site conditions, as well as the width of the rolls in which the FML is manufactured or the dimensions of any prefabricated panels used in the FML. Since an FML is an engineered structure, a well-designed layout avoids using horizontal seams on slopes and seams at the toe of slopes, because these seams may be subjected to excessive stresses (Reference 13, Chapter 7).

4.2.2.5 Appurtenances and Penetrations

Various ancillary components are necessary for the proper operation of a lined system. These components can be categorized as penetrations or appurtenances. Appurtenances include any adjoining structures to the liner system, such as sumps; splash pads at pipe outfalls; anchorage systems; inlet, outlet, overflow, or underflow pipes; gas vents; level-indicating devices; emergency spill systems; pipe supports; aeration systems; and protective soil covers. Penetrations are made through the FML to accommodate pipes, vents, level-indicating devices, and pipe racks. However, to reduce the potential for leaks, penetrations should be avoided when possible, since they create additional locations where leakage may occur. When penetrations are necessary, the seal between the appurtenance and the FML must be liquid-tight (References 6, 7, and 13). Additional information on appurtenances and penetrations may be found in References 16, 17, and 18.

4.2.3 Leachate Collection and Removal Systems (LCRS)

Each leachate collection and removal system, whether above (primary) or between (secondary) the liners, consists of the following components shown on Exhibit 4-2 (Reference 7; Sections I - III):

- A low-permeability base which is either a soil liner, composite liner, or flexible membrane liner (FML);
- A high-permeability drainage layer constructed of either natural granular materials (sand and gravel) or synthetic drainage material (geonet), which is placed directly on the primary and/or secondary liner, or its protective bedding layer;
- Perforated leachate collection pipes within the high-permeability drainage layer to collect leachate and carry it rapidly to the sump;
- A protective filter material surrounding the pipes, if necessary, to prevent physical clogging of the pipes or perforations;
- A leachate collection sump or sumps, where leachate can be removed;

- A protective filter layer over the high-permeability drainage material which prevents physical clogging of the material; and
- A final protective layer of material that provides a wearing surface for traffic and landfill operations.

The design features of each of these components and operation of the entire LCRS will be described in the following sections. Exhibit 4-8 summarizes the types of design information and technical parameters used to demonstrate that the LCRS performance standards are met. The primary LCRS acts as a leachate collection system to remove leachate from the landfill or waste pile before it can leak through the primary liner component. This system also aids in reducing leakage by removing or reducing the hydraulic head of leachate that exists within the system. The secondary LCRS acts as a leak detection system by collecting and removing any leachate which leaks through the top or primary liner component of the liner system.

4.2.3.1 Layout of System Components

The design of an LCRS for a hazardous waste land disposal unit begins with a layout of the system components within the unit. This layout should be presented in plan view, cross-section, and detail drawings of the unit. The drawings should show dimensions and slopes of the unit design features and all the components of the LCRS.

The system components should be shown on the plan and cross-section drawings and should clearly show the lateral and vertical extent of the primary and secondary liners. The drawings should show the elevations of the tops of the liner system components at critical points, including the toes of the sidewalls, the boundaries of any sub-areas of the unit that drain to different sumps, and the inlet and low point elevations of the sumps. This information is essential to evaluate the ability of the system to drain leachate toward the collection sumps. The recommended bottom liner slope is two percent at all points in each system (Reference 7, Section I.A). This slope is necessary for effective leachate drainage through the entire operating and post-closure period; therefore, these slopes must be maintained under operational and post-closure loadings. The settlement estimates performed as discussed in Chapter 2 must be evaluated to ensure that the slopes will be two percent throughout the period of operation of the LCRS. It may be necessary to initially design the slopes steeper than two percent to allow for settlement.

The high-permeability drainage layer is placed directly over the liner or their protective bedding layers. Often the selection of a drainage material is based on the on-site availability of natural granular materials. Since hauling costs are high for sand and gravel, a facility may elect to use geonets or synthetic drainage materials as an alternative. These materials are described in Section 4.3.3. Frequently, geonets are substituted for granular materials on steep sidewalls in order to provide a layer that is more stable with respect to sliding than a granular layer.

Each leachate collection system, primary and secondary, must have a separate sump. This configuration allows for the measurement of any leakage through the primary liner into the secondary LCRS. Therefore, each landfill cell must have at least two LCRS sumps, more if the cell is divided into leachate collection sub-areas using several different collection points for each system. The drawings should clearly show separate sumps for each of the systems, with separate methods for removing

Exhibit 4-8. Information Typically Submitted to Demonstrate Satisfaction of LCRS Requirements

Information	Typical Parameters
Materials of Construction	Detailed material specifications for: • Drainage layer material • Filter fabric or filter layer • Piping • Sumps
Descriptions of LCRS Operation and Design	Description of: • Design of system and timely removal of leachate • Design of system and timely detection of leakage through liners • Removal of liquid from system
Drainage Capacity of Synthetic Materials	Demonstration that synthetic drainage material has drainage capacity equal to or greater than a 12-inch granular layer with hydraulic conductivity of 1×10^{-2} cm/sec.
Demonstration of Grading and Drainage	Design details including: • Slopes • Contour plan • Layout and spacing of piping • Sumps, pumps, etc. • Fate of collected leachate • For slopes of < 2%, demonstration of equivalency • Sizing of pipes and perforations • Separate primary and secondary leachate collection sumps
Calculation of Maximum Leachate Head	Demonstration that leachate depth over top of primary liner will not exceed one foot. Include: • Calculations • Justification of assumed parameters • Numerical techniques used
Compatibility of LCRS	Test results demonstrating compatibility with wastes and leachates for all system components and materials.

leachate from each sump. Access to the sumps is usually provided by either a solid pipe laid in a shallow trench along the sidewalls or a vertical manhole that is constructed as the cell is filled.

The following sections of this Chapter will discuss how the system is designed using sound engineering practices and how design is evaluated to assure that it will meet the performance requirements.

4.2.3.2 Sidewalls

Most hazardous waste landfills are constructed by excavation of a cell, pit, or trench, followed by filling and final restoration of the site to a condition similar to its original topography. For this reason, liner systems are placed on excavated sidewalls. The issue of slope stability is discussed in Chapter 3; however, in addition to the stability of the slope itself, the stability of the individual liner components on the slope must also be considered.

Reference 16, Section 4.3.5.2, provides a method for calculating the factor of safety (FS) against sliding for soils placed on a sloped FML surface. It considers the slope angle and the friction angle between the FML and its cover soil. Generally the slope angle is known and is specified on the design drawings. A minimum factor of safety is then selected. For a landfill, the FS can be as low as 1.1 to 1.2 because the slope will be unsupported (i.e., no waste will be filled against it) for only a short time, and any failures that do occur can be repaired fairly easily. Surface impoundment slopes should be designed with an FS above 1.25, because the slope will be subject to loading and unloading cycles as the level of waste in the impoundment varies, and because a failure would be more likely to cause immediate releases to the environment.

From the slope angle and the FS, a minimum allowable friction angle is determined, and the various components of the liner system are selected based on this minimum friction angle. If the design evaluation results in an unacceptably low FS, then either the sidewall slope or the materials must be changed to produce an adequate design.

4.2.3.3 Grading and Drainage

In order for leachate to be effectively collected and removed, both the primary and secondary systems must be sloped to drain toward their respective collection sumps. Minimum technology guidance (Reference 1, p. 6) requires that the leachate depth over the top liner not exceed 30 cm (one foot). The regulations, 40 CFR 264.301(a), contain this same requirement for a single lined system.

4.2.3.3.1 Granular Drainage Layers

EPA SW-869 (Reference 20, Section 3.2) provides a method for calculating the maximum leachate depth over a liner for granular drainage systems materials, using an equation developed by the author. (It should be noted, however, that this equation contains an error; the variable "n," for drainage layer porosity, should not be included in the equation.) The leachate head in the layer is a function of the liquid impingement rate, bottom slope, pipe spacing, and drainage layer hydraulic conductivity. The impingement rate is estimated using a complex liquid routing procedure discussed in Reference 6-26; care must be taken to use input data to this procedure that will yield a conservative result. If the maximum leachate depth exceeds 30 cm for the system, except for short term occurrences, the design should be modified to improve its efficiency.

4.2.3.3.2 Geosynthetic Drainage Layer-Geonets

Geonets may be substituted for the granular layers in either of the LCRS on the bottoms and sidewalls of the landfill cells. Geonets may be used if they are equivalent to the granular design, including chemical compatibility, flow under load, clogging resistance, and protection of the FML (Reference 21, p. III-2).

A geonet used to replace the granular drainage layer must provide the minimum flow capacity equivalent to 12 inches of material with a minimum hydraulic conductivity of 1 x 10^{-2} cm/sec (transmissivity of 3 x 10^{-5} m^2/sec) and the capacity required to maintain the liquid levels over the liner at less than one foot. Reference 4, pages III-2 through III-6 provides an explanation of the calculation used to compute the capacity of a geonet. This transmissivity in geosynthetics is strongly affected by large compressive loads on the system exerted by overlying wastes. Current research has shown that the transmissivity of a geonet can be reduced by as much as an order of magnitude during the first 30 days of service under load if a soil is immediately adjacent to it. Therefore, it is very important that the laboratory transmissivity test be performed under conditions, loads, and configurations that closely replicate the actual field conditions and that the transmissivity value used in the LCRS design calculations be selected based upon those loaded conditions (Reference 21; p. III-5).

4.2.3.3.3 Piping

The design of piping systems requires the consideration of pipe flow capacity and structural strength. As indicated above, the spacing of leachate collection pipes can be determined based on the maximum allowable leachate head on the liner. This maximum head calculation assumes that liquids can drain away freely through the piping systems; therefore, the pipes must be sized to carry the expected flow.

The leachate piping configuration shown on the facility design drawings, as discussed in Section 4.2.3.1, should be evaluated for its ability to maintain the maximum leachate head and for its ability to carry the expected flows. Appendix V of Reference 13 provides an algorithm to estimate the required flow per 1000 feet of collection pipe in gallons per minute based on the impingement rate (percolation) discussed in Section 4.2.3.3.1 and the pipe spacing as shown on the facility design drawings. Figure V-2 of the same reference can then be used to verify that the pipe sizes are adequate.

4.2.3.3.4 The HELP Model

EPA has developed a computer program called the Hydrologic Evaluation of Landfill Performance (HELP) which is a quasi-two-dimensional hydrologic model of water movement across, into, through, and out of landfills. The model uses climatologic, soil, and landfill design data and incorporates a solution technique which accounts for the effects of surface storage, run-off, infiltration, percolation, evapotranspiration, soil moisture storage, and lateral drainage. The program estimates run-off drainage and leachate expected to result from a wide variety of landfill conditions, including open, partially open, and closed landfill cells. Most importantly, in consideration of this topic, the model can be used to estimate the buildup of leachate above the bottom liner of the landfill. The HELP program can be used to estimate the depth of leachate above the bottom liner for a variety of landfill designs, time averages, and storm events. The results may be used to compare designs or to design leachate drainage and collection systems. References 22 and 23, a users guide and model documentation, should be obtained before attempting to run the HELP model.

4.2.3.4 System Compatibility

The failure of any one of the LCRS components can lead to failure of the entire system; therefore, it is essential that every component be demonstrated to be chemically compatible with the expected leachate from the landfill or surface impoundment. Synthetic components of LCRS may be subjected to compatibility testing similar to that for FMLs. Geotextiles, geonets, and plastic pipe and fittings may be exposed in accordance with Method 9090, but the testing protocols are different for these products due to their different physical properties. References 34 and 35 suggest general procedures for compatibility testing of pipe, geotextiles, geonets, and earthen materials. EPA Technical Resource Document SW-870 also provides some general information on these products and appropriate test methods (Reference 13, Chapter 4).

4.2.3.5 System Strength

All components of the LCRS must have sufficient strength to support the weight of the overlying waste, cover system, and post-closure loadings, as well as stresses from operating equipment and from the weight of the components themselves. They are also vulnerable to sliding under their own weight and the weight of equipment operating on the slopes. The components that are most vulnerable to strength failures are the drainage layers and piping. LCRS piping can fail by excessive deflection leading to buckling or collapsing.

4.2.3.5.1 Stability of Drainage Layers

If the drainage layer of the LCRS is constructed of granular soil materials, i.e., sand and gravel, then it must be demonstrated that this granular drainage layer has sufficient bearing strength to support expected loads. This demonstration will be very similar to that required for the foundations and low-permeability soil liner in Chapters 2.0 and 4.0, with the exception that the test selected must be appropriate for drained cohesionless soils rather than clays (cohesive soils).

The landfill design should provide calculations demonstrating that the selected granular drainage materials will be stable on the steepest slope (i.e., the most critical) in the design. The calculations and the assumptions should be shown, especially the friction angle between the geomembrane and soil, and if possible, supported by laboratory and/or field testing data.

The friction factors against sliding for geotextiles, geonets, and FMLs can often be demonstrated using manufacturers data, since these materials do not exhibit the range of characteristics that soil materials do. It is important that the sliding stability calculations accurately represent the actual design configuration and the specified materials.

4.2.3.5.2 Pipe Structural Strength

Pipes installed at the base of a landfill can be subjected to high loading of waste. The evaluation of a design should consider both the maximum depth of fill over the piping and the loading exerted by landfill equipment on a pipe with very little cover. The pipe must be selected based upon the most critical of these loadings.

Leachate collection pipes beneath landfills are generally installed in one of two configurations:

- a trench condition, where the pipe is placed in a shallow trench excavated into the underlying soil liner or foundation soil, and does not project above the top of the trench and/or
- a positive projecting condition, where the pipe is placed directly upon a lower liner system component and projects above it.

Loads on the pipe in the trench condition are caused by both the waste fill and the trench backfill. These two loads are computed separately and then added to obtain the total vertical pressure acting on the top of the pipe. The total vertical load on the pipe itself is reduced somewhat by the trench backfill; the amount of the load reduction is determined by load coefficients which are a function of the type of backfill. A detailed discussion of these load coefficients is found in Reference 13, Appendix V. For the projecting condition the vertical pressure on the pipe is assumed to be equal to the unit weight of the refuse fill multiplied by the height of the fill above the pipe.

In the early phases of landfilling the piping system is subject to concentrated and impact loadings from trucks and landfill equipment. Since the pipe at this point may be covered with only a foot or so of granular drainage material, wheel and impact loads are transmitted directly to the pipes. These loads may be calculated using a method found in Reference 24, Chapter 9, Section C. This traffic load should be compared to the static load from the waste, and the pipe selected based upon the larger of the two loads.

These loadings can cause pipe failure either by excessive deflection followed by collapse or by buckling. The deflection under the landfill loadings is calculated using the Iowa formula (Reference 24, Chapter 9, Section E.1.; and Reference 13, Appendix V). This deflection should be compared to manufacturers data on allowable deflection for the size and material of pipe specified. The capacity of a buried flexible pipe to support the landfill loads may be limited by buckling. Buckling is a function of the flexibility of the pipe and the type of backfill soil. Specific buckling information for the sizes and pipe materials proposed for use in the collection system should be obtained from the pipe manufacturer.

Pipes are slotted or perforated to allow flow of leachate into the collection system. These perforations reduce the effective length of the pipe available to carry loads and to resist deflection. See Reference 13, Appendix V for a discussion of how to allow for perforations in pipe strength calculations.

The piping system design must be evaluated for its ability to support all the loads to which it is subjected under all of its potential failure modes.

4.2.3.6 Prevention of Clogging

The piping system must be protected from physical clogging by the granular drainage materials. This is most effectively accomplished by careful sizing of pipe perforations and by surrounding the pipe with a filter medium, either a graded granular filter or a geotextile. In addition, clogging of the pipes and drainage layers of the LCRS can occur through several other mechanisms, including chemical and biological clogging. A detailed discussion of these mechanisms is found in Reference 25, Section 9.2.2.3.4.

To prevent physical clogging of leachate drainage layers and piping by soil sediment deposits, filter and drainage layer size gradations should be designed using criteria established by the Army Corps of Engineers (Reference 25; p. 9-90). Drainage layers should be designed to have adequate hydraulic conductivity; and granular drainage media should be washed before installation to minimize fines. Drain pipe should be slotted or perforated with a minimum inside diameter of six inches to allow for cleaning.

Two criteria are suggested for use in design of drainage and filter layers for drain systems. The first criterion is for the control of clogging by piping of small soil particles into the filter layer and the drain pipe system, while the second criterion is meant to guarantee sufficient permeability to prevent the buildup of large seepage forces and hydrostatic pressure in filters and drainage layers. Calculations for these criteria are explained in Reference 25, Section 9.

When geotextiles (filter fabrics) are used in place of graded filters, the protective filter may be only about one mm in thickness. Caution should be exercised to ensure that no holes, tears, or gaps are permitted to form in the fabric. The advantages to using geotextiles in place of granular filters are cost, uniformity, and ease of installation. With increases in costs of graded aggregate and its installation, geotextiles are competitive with graded filters. One of the most important advantages to geotextiles is quality control during construction. The properties of geotextiles will remain practically constant independent of construction practices, whereas graded filters can become segregated during placement. These geotextiles must be designed and References 16 and 21 provide guidance on how to design those systems.

When drainage pipe systems are embedded in filter and drainage layers, no unplugged ends should be allowed, and the filter materials in contact with the pipes must be coarse enough to be excluded from joints, holes, or slots. Specifications for the drainage layer materials should be checked against pipe specifications to be sure that the piping system will not become clogged by the granular drainage layer particles.

Chemical clogging can occur when ionic species are dissolved in the leachate and then precipitate in the piping of the LCRS. This type of clogging can be controlled by providing a sufficiently steep slope in the system to allow flow velocities high enough for self-cleansing. These flow velocities are dependent on the diameter of the precipitate particles and on their specific gravity in the relationship given in Reference 24. Generally, flow velocities should be in the range of one to two feet per second to allow for self-cleansing of the piping.

Biological clogging due to algae and bacterial growth is a serious problem in sanitary landfills, but is less critical in most hazardous waste landfills because of the types of waste received. There are no universally effective methods of preventing biological growth in the systems. If organic materials will be present in the landfill and there will be a potential for biological clogging, then, the system design should include features to allow cleaning of the piping system, which includes the following components:

- Minimum of six-inch diameter pipes to facilitate cleaning;
- Manholes located at major pipe intersections or bends to allow for access, inspections, and cleaning; and
- Valves, ports, or other appurtenances to introduce biocides and/or cleaning solutions.

4.3 Materials/Specifications

Following design development, the designer/engineer must prepare a materials and construction specification document for the proposed facility. The specification document includes plans, technical specifications, and drawings for the proposed facility and is used in the bid package for construction cost estimates, as well as for the construction of the facility. The specification document should also be submitted with the Part B application for the proposed hazardous waste management unit.

4.3.1 Low-Permeability Soil Liners

Low-permeability soils often used to construct liners include clay, silty clay and sandy clay soils, all of which exhibit low permeability values under compaction. The following sections provide information about low-permeability soils.

4.3.1.1 Sources

In order to reduce costs, in situ soils are often utilized as a material for containment of hazardous constituents. However, in some cases this may not be possible. In the event that a suitable soil cannot be located at the site, borrow material may be brought in from a location off site. If the hydraulic conductivity value after compaction of in situ or borrow material is too high to be acceptable, it may be necessary to amend these soils to make them less permeable and more suitable. A soil amendment such as bentonite may be mixed with a more permeable soil to create a suitable soil to use as liner material. Bentonite, when added by as little as two or three percent, has reduced permeability of some soils after compaction by two to three orders of magnitude (Reference 12, pp. 5-10); however, reduced permeability enhanced by bentonite is dependent on the specific properties of the soil it is added to.

4.3.1.2 Soil Properties

Generally, the more clay a soil contains the lower the hydraulic conductivity. Clay particles are less than 0.002 mm in diameter and clay soils consist of at least 40 percent clay particles (Reference 21, p. 237). Clay minerals are made of hydrous silicates (aluminum, magnesium and iron), which have an affinity for water. Properties of clay soils are influenced by their clay mineral content. The greater the clay mineral content in a soil the greater the influence on the behavior of the soil. Detailed information on clay mineralogy and its effect on compacted soils is provided in Reference 12.

4.3.1.3 Test Methods

Many test methods have been developed to predict the performance of a soil, most of which have been standardized. Moisture/density relationships, shrink-swell potential, effective porosity, and most importantly, hydraulic conductivity, are important parameters to be measured (Reference 12, pp. 1-4). The most important test for soil liners, field hydraulic conductivity, has not yet been standardized.

Moisture/density relationships must be established when evaluating liner material that is compacted. In clay soils this refers to the optimum moisture content that results in maximum dry density after compaction. The optimum moisture content of a soil provides enough water to permit soil grains to distort and reposition themselves when compacted, but not so much water that the void spaces are filled. Dry density simply describes the weight or mass per unit volume of the compacted soil. This moisture/density relationship is usually established through a curve developed from a standard proctor analysis. A detailed description of this analysis is provided in Reference 10, Test Method, p. 698.

Volume changes in soils are caused by changes in moisture content. Shrinkage is caused by capillary tension which compresses soil structure. Swelling is caused by an increase in moisture content. Swelling is influenced by a number of factors, such as soil elasticity, clay mineral affinity, cation exchange capacity of the clay particles, and expansion of air trapped in soil voids (Reference 12; Section 2). The shrink-swell potential of clays has been correlated to Atterberg limits (Reference 10, Test Method, p. 4,318), another standardized test for evaluating soils.

Effective porosity is that fraction of the total volume through which flow can occur. Therefore the higher the effective porosity for a given hydraulic conductivity, the longer time it will take waste constituents to move through a soil liner. A detailed discussion of effective porosity is provided in Reference 2, pp. 3-5 and Reference 27, pp. 7-46.

4.3.1.4 Hydraulic Conductivity

Darcy's Law is the basis for the equation of permeability. (Reference 27, pp. 7-35).

Various laboratory methods are available for determining the hydraulic conductivity of a soil compacted at optimum water content. These are listed in Exhibit 4-3. Permeability values determined in the laboratory are generally poor indicators of field performance; however, they may provide some information on the liner material. Several studies have been conducted which indicate field hydraulic conductivity is by far a better indicator than laboratory methods. Therefore, it is essential that field permeability testing be conducted on all potential liner material. Extensive field studies have been conducted to evaluate field permeability (Reference 28, Chapter 4 and Reference 14, Chapter 19). A detailed discussion on hydraulic conductivity, as related to compacted soil liners, is provided in Reference 12, Section 3.8 and in Reference 32.

Other tests to conduct when evaluating soil liner material include:

- Particle Size Distribution,
- Bulk Density,
- Particle Density,
- Liquid and Plastic Limit and Plasticity Index,
- Compactive Effort,
- Strength and Bearing Capacity, and
- Compressibility.

Further discussion of these tests and related methods is provided in Chapter 2.0 of this Guide and Reference 12, Chapter 3. Exhibit 4-4 provides references for most pertinent tests used in evaluating clay liner material.

4.3.1.5 Liner-Leachate Compatibility

Compatibility testing is essential to determine if chemical components in permeant fluids may affect the hydraulic conductivity of a soil. Several studies have been conducted by researchers on theories to predict soil liner-leachate interactions. These studies are summarized in Reference 12, Section 4.2.

Hydraulic conductivity may be affected by chemicals in the following manner (Reference 13; Section 4.2 and Reference 28; Section 5.5):

- Alterations in the double layer surrounding clay particles in soil fabric,
- Dissolution of soil fabric by strong acids and bases, and
- Precipitation of solids in soil pore space.

Soil pore blockage from microorganism activity is a biological factor which often affects hydraulic conductivity values of soils.

Currently there is no standard testing procedure for soil liner-leachate compatibility; however, EPA Method 9100 provided in Reference 29, and the hydraulic conductivity test in Reference 27 (pp. 7-105 to 7-124) provide a means for evaluating soil liner-leachate compatibility under laboratory conditions.

Soil liner-leachate compatibility testing provides a basis for evaluating the performance of a soil liner material after it is exposed to the leachate that it is designed to contain. Of interest is the representativeness of the permeant liquid to actual leachate at the site, and most of all, the outcome of the analysis. It is critically important that the permeant liquid used in the compatibility test should be representative of the worst-case leachate that is to be contained at the facility.

4.3.1.6 Mechanisms of Soil Liner Failure

Liner failure may be caused by various mechanisms. These include:

- Desiccation cracking,
- Slope instability,
- Settlement,
- Piping and dissolution,
- Penetration,
- Erosion,
- Earthquakes,
- Hydraulic uplift (heaving),
- Design or construction errors,
- Exposure to organic chemicals,
- Design and construction errors,
- Incomplete remolding of clods, and
- Inadequate scarification between lifts.

A detailed discussion of these liner failure mechanisms is provided in Reference 12, Chapter 7.

4.3.2 Flexible Membrane Liners

The materials specifications document must include detailed standards to assure that adequate FMLs are selected and that inferior FML materials are not substituted at any time during liner system construction.

These minimum standards are selected based upon the conditions identified during the unit design, including loads and stresses and chemical compatibility. The specifications should be carefully compared to the design calculations and the liner/leachate compatibility data to verify that the selected specified FML will not fail during installation or during its service life.

The specifications should list minimum mechanical, analytical, and environmental and aging properties, and the FML selected should be of the type that possessed all these necessary minimum properties. A detailed discussion of material standards for FMLs is provided in Appendix VIII of Reference 13.

4.3.3 Leachate Collection and Removal Systems (LCRS)

After the design calculations have demonstrated that the LCRS will perform as required, then the material specifications must be carefully prepared to ensure that the system components will be constructed of materials that are equivalent to those assumed in the calculations. For example, if the pipe strength calculations discussed in Section 4.2.3.5.2, above, evaluate a pipe of a specified material and Standard Dimension Ratio (SDR), then the construction specifications must require that same pipe. The SDR of a pipe is the ratio of pipe diameter to wall thickness. The following sections will discuss the types of information necessary to prevent the substitution of inferior materials in the LCRS.

4.3.3.1 Granular Drainage Layer Materials

For a granular drainage layer the most critical specification is the particle size distribution. In order for the materials to drain properly, they must have a saturated hydraulic conductivity of 1 x 10^{-2} cm/sec (Reference 7). Therefore, the material specifications should specify the range of particle sizes that has been demonstrated, as discussed in Section 4.2.3.3.1 to provide adequate flow. Generally, sands and gravels with a group designation of GW, GP, SW, or SP on the Unified Soil Classification Chart (Reference 13, Appendix I) will satisfy the permeability requirements. These specifications should be called out on the design drawings and in the material specifications, usually under the heading "drainage layer material" or "select fill." They must also match the assumptions used in the leachate system drainage calculations, Section 4.2.3.3.

4.3.3.2 Geonets

For geonets the most critical specifications are concerned with the materials' ability to transmit fluids under loadings; therefore, the specifications must include a minimum transmissivity under the expected landfill loads. This transmissivity will have been demonstrated by the testing and calculations discussed in Section 4.2.3.3.2. The specifications for thickness and type of material should be called out on the drawings and in the materials section of the specifications, and they must match each other and the design calculations.

4.3.3.3 Granular Filter Layers

The specifications required for granular filter layers surrounding perforated pipes and protecting the primary (upper) drainage layer from clogging include a detailed particle size distribution, selected as discussed in Section 4.2.3.6. The layers and their specifications should be shown on all drawings and described, with ranges of particle sizes, in the materials section of the specifications. Again, the criteria on the drawings and in the specifications must match each other and must match the assumptions of the design calculations in Section 4.2.3.6.

4.3.3.4 Geotextiles

Since the primary function of a geotextile is to prevent the migration of soil fines into the drainage layer and of drainage layer fines into the leachate pipes, while allowing the passage of leachate, the most critical specifications are those of permeability and retention. The permeability of the geotextile should be at least ten times the permeability of the soil it is retaining. The retention ability for loose soils is evaluated based upon the average particle size of the soil and the apparent opening size (AOS) of the geotextile. The maximum apparent opening size, sometimes called equivalent opening size (EOS), is determined for the soil to be retained, and a geotextile is then selected that meets the specification. The material specifications should contain a range of AOS values for the geotextile, and these AOS values should match those used in the design calculations (Reference 21; p. III-6).

4.3.3.5 Piping

The selected pipe materials should be indicated on the plans and drawings as well as in the specifications, and these should match each other as well as the assumptions in the design calculations. The specifications should include:

- Type of piping material,
- Diameter and wall thickness,
- Size and distribution of slots or perforations,
- Type of coatings used in the pipe manufacturing, and
- Type of pipe bedding material used to support the pipes.

4.3.3.6 Sumps and Pumps

Often sumps are constructed of concrete or other materials, although they may be simply extensions of the leachate collection system layer or pipe trenches. The drawings should clearly show the dimensions and materials of construction for each sump. If the sumps are constructed of materials not used elsewhere, then there must be specifications for all the materials.

Concrete strength is indicated by its specified compressive strength at a given time which is a function of the ratio of water to cement; generally the lower the water-to-cement ratio, the higher the compressive strength. The specifications will require a minimum strength. Detailed concrete standards are contained in ASTM specifications (Reference 37) and in standards of the American Concrete Institute (References 38, 39, 40).

If the sumps are constructed of reinforced concrete, specifications for the reinforcing must be indicated. These include the ASTM bar numbers (Reference 41) and their configuration within the concrete. If concrete strength calculations are included in the design calculations, the specifications should be compared to the design assumptions to assure that the sump will be constructed as designed. This is particularly important if the sump is a large concrete structure supporting a tall vertical access manhole through the landfill, because the sump may be supporting a large load over a small area.

Pumps used to remove leachate from the sumps should be sized to ensure removal of leachate at the expected rate of generation and must have a sufficient operating head to lift the leachate the required height, from the sump to the access port. Often portable vacuum pumps that can be moved in sequence from one leachate sump to another are used. The type of pump specified should be small enough to be lowered through the leachate sump access pipes. The specifications may include pump details if the pumps will be installed at the time of construction; otherwise this information will be included in the landfill operating equipment.

4.4 Construction/Installation

4.4.1 Low-Permeability Soil Liners

The following discussion provides a summary of soil liner construction and installation activities. Prior to construction of the actual liner system, a pilot construction test should be conducted. Construction activities should commence upon completion and approval of the test fill analysis. These activities include compaction, scarification, placement on side slopes and final preparation for FML placement. A detailed discussion of construction and installation activities is provided in Reference 12, Chapter 5.2 and Reference 13, Chapter 5.0.

4.4.1.1 Soil Liner Test Fill

A soil liner test fill is a small-scale study conducted to determine whether the design specification, liner material, equipment and construction procedures will result in an acceptable low-permeability soil liner (Reference 12, Chapter 5.2) a schematic of a test fill is shown in Exhibit 4-9. A test fill will minimize potential costs and dangers of construction of an unacceptable soil liner, as well as provide a quality assurance measure for design specifications. Procedures and equipment which are intended for use with the full-scale facility should be utilized during the construction of the test fill. Field hydraulic conductivity analysis should be conducted on the test fill prior to construction of the liner to verify that laboratory hydraulic conductivities have been achieved (Reference 9, Section 2.3.4.1.2).

4.4.1.2 Compaction

Soil liners are installed in a series of compacted lifts of specified thickness. Lift thickness is dependent on the soil characteristics, compaction equipment, and the required compactive effort. Soil liner lifts must be thin enough to allow adequate compactive effort to reach the lower portions of the lift. Thinner lifts provide more assurance that even compaction is occurring between lifts; however, thinner lifts require more lifts to achieve the proper soil liner thickness. Some engineers think that adequate compaction of thicker lifts (5-10 inches) is possible if heavy enough compaction equipment is used (Reference 12, pp. 5-54 to 5-58).

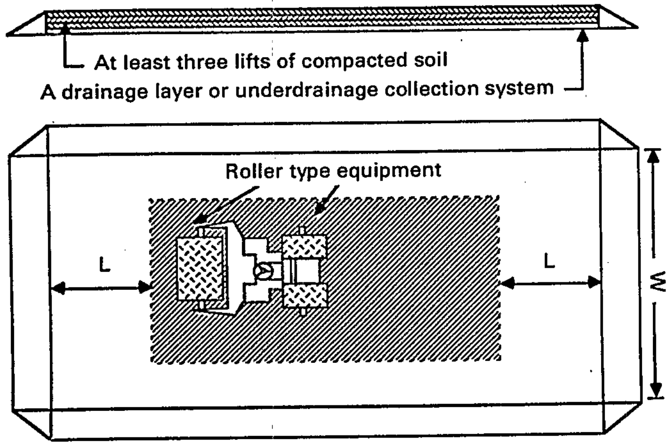
Placement of the soil liner material and compaction activities may begin once the foundation preparations are completed. Clods (soil aggregates) in the soil liner material, if kept to smaller sizes, will facilitate in a more uniform water content (Reference 12, p. 5-48). Opinions differ among design engineers on maximum clod sizes for soil liners. Some engineers suggest one to three inches in diameter, no larger than one-half the lift thickness and no longer than the lift thickness (Reference 12, p. 5-51). The main objective is to remold all clods in the compaction process to keep the permeability values consistent throughout the soil liner. It has been demonstrated in field studies that it is the macropores between clod remnants that can result in an unacceptably high field hydraulic conductivity.

It is essential that the optimum water content derived for the proper compaction of the material be maintained in order to obtain the specified hydraulic conductivity. Several EPA studies have found that compacting a soil at or below optimum water content often results in an unacceptably high field hydraulic conductivity. These studies and numerous reports from test fills indicate that a lower hydraulic conductivity is obtained by compacting the soil at 2-4% above optimum water content. Environmental conditions such as rainfall or extreme temperatures should be considered when initiating construction activities. Adverse weather conditions, such as dry and hot spells can cause desiccation cracks. Freezing can cause shrinkage cracks. (Reference 9, p. 25).

Standard compaction procedures are normally employed when constructing soil liners. The following factors affect the quality of the compaction:

- lift thickness and number,
- full scale or segmented placement,
- number of equipment passes,
- scarification between lifts, and

Exhibit 4-9. Schematic of a Test Fill



L = Distance required for construction equipment to reach normal running speed
W = Distance at least four times wider than the widest piece of construction equipment
▨ = Area to be used for testing

- material water content.

It is necessary to control all of these factors to achieve the desired compaction in the liner. A detailed discussion of current construction and installation procedures, as well as available equipment is provided in Reference 12, pp. 5-54 to 5-65 and Reference 13, Chapter 5.

Soil/bentonite mixtures generally require central plant mixing by means of a pugmill, cement mixer, or other mixing type equipment where water is added during the process. Not only is water content monitored constantly, but bentonite content and particle size distribution are of concern and must be measured during the mixing and placement of the material. Spreading of the soil/bentonite mixture may be accomplished similar to spreading for natural soil liners by using tracks, scrapers, graders, dozers or a continuous asphalt paving machine. After mixing and spreading the soil/bentonite liner, compaction by means of vibrating smooth-wheeled rollers or vibratory-plate compactors may take place. A discussion of specific procedures for construction of soil/bentonite liners is provided in Reference 12, pp. 5-67 to 5-76 and Reference 13, p.238.

4.4.1.3 Placement on Side Slopes

Placement of soil liner material and compaction on side slopes is dependent on the angle of the slope. Gradual inclines from the bottom of the liner enable continuous placement of the lifts. This provides better continuity between the bottom and sidewalls of the soil liner. However, when steep slopes are encountered lifts may have to be placed and compacted horizontally. This is due to operational difficulties of the heavy compaction equipment on steeper slopes. When sidewalls are compacted horizontally, it is essential to tie in the edges with the bottom of the soil liner to prevent areas of weakness in the soil liner. Information concerning sidewall placement and compaction is provided in Reference 13, p. 219 and Reference 12, p. 521.

4.4.1.4 Final Preparation for FML Placement

Post-installation activities begin upon completion of placement and compaction of the soil liner material. Quality control inspection should be conducted prior to placing the FML. It is essential to inspect the soil liner to ensure that design specifications (thickness, water content, sidewall slope, etc.) have been met and that the integrity of seals around leachate collection sumps or pipes are secure. Once these inspections are conducted and complete, the FML may be installed on top of the soil liner. If any amount of time will pass before the FML is installed, a plastic cover should be placed on the soil liner to prevent desiccation, erosion, or freezing (Reference 12, pp. 5-78).

4.4.2 Flexible Membrane Liners

4.4.2.1 Materials and Construction Specification Document for FMLs

The specification document for a proposed hazardous waste facility must address the FML components of the unit, including the bottom liner, the top liner, and the cover. The technical specification for FMLs must include detailed information concerning material properties, shipping and storage of the FML sheeting or panels, installation of the FML, and quality assurance/quality control by the manufacturer, fabricator (if panels are constructed), and the installer. Installation procedures addressed by the technical specification include FML layout, deployment of the FML at the construction site, seaming of the FML sheeting or panels, and sealing of the FML to appurtenances, both adjoining and penetrating the liner. The performance of inspection activities, including both nondestructive and destructive testing of the sheets and seams, during installation of the FML should be addressed in the technical specification or in a separate construction quality assurance document (Reference 13, Chapter 8).

4.4.2.2 Construction Procedures

Construction procedures must be incorporated in the design plans and specifications for a construction quality assurance program must also be established. The design specifications should describe the requirements for labeling, shipping, and on-site storage of the FML sheeting or panels (Reference 13, Chapter 8). The FML sheeting is shipped in rolls or panels from the suppliers, manufacturers, or fabricators to the construction site. Each roll or panel must be labeled according to its position on the FML layout plan so that installation of the FML does not turn into a jigsaw puzzle. The sheeting must be inspected upon delivery to the construction site to determine whether any damage has occurred during shipping. Proper storage of the rolls or panels prior to installation is critical to the final performance of the FML. Some FML materials may be sensitive to ultraviolet exposure and should not be stored in the direct sunlight prior to installation and placement of soil cover. Others, such as CSPE and CPE, are sensitive to moisture and heat and can partially crosslink or block (stick together) under improper storage conditions before installation. Adhesives or welding materials should also be stored appropriately (Reference 42).

Deployment, or placement, of the FML panels or rolls should be described in the FML layout plan. Rolls of sheeting can be deployed by manually unrolling the sheet, or it can be unrolled using a front-end loader or a tractor. Panels are usually folded on pallets and require a large crew of workers to manually unfold and place the FML material. Placement of the FML goes hand-in-hand with the seaming process; only the amount of sheeting that can be seamed during a shift or work day should be deployed at any one time (Reference 42). On windy days, sand bags may be used to hold the sheeting in place before and after seaming in order to prevent the wind from whipping and damaging the FML.

The seaming process is critical to the success or failure of an FML installation. For this reason, several conditions that may affect seam integrity must be monitored and controlled during the seaming process; control includes postponement of seaming until conditions improve. The factors affecting the seaming process include (Reference 42):

- Ambient temperature at which the seams are made,
- Relative humidity,
- Amount of wind,
- Effect that clouds have on the FML temperature,
- Water content of the subsurface beneath the FML,
- Supporting surface on which the seam is bonded,
- Skill of the seaming crew,
- Quality and consistency of the adhesive or welding material,
- Proper preparation of the liner surfaces to be joined, and
- Cleanliness of the seam interface, i.e., the amount of airborne dust and debris present.

The bonding system used to seam the FML is dependent on the polymer making up the sheeting. Thermal methods of seaming require cleanliness of the bonding surfaces, heat, pressure, and dwell time to produce high-quality seams. The requirements for adhesive systems are the same as thermal systems, except that the adhesive takes the place of the heat. Sealing the FML to appurtenances and penetrating structures should be performed in accordance with detailed drawings included in the design plans and specifications. As with the FML, the materials used to seal the FML to appurtenances must be mechanically and chemically compatible for use at the facility (Reference 13, Chapter 8).

The FML must be anchored at its perimeter to prevent sloughing or slipping down the slopes of the containment. An anchor trench is generally used to secure the FML along the berm of an embankment; other anchorage systems may be employed at appurtenances. In any case, the FML must be anchored according to the detailed drawings provided in the design plans and specifications (Reference 13, Chapter 8).

FMLs that are subject to damage from exposure to weather and work activities should be covered with a layer of soil as soon as possible after quality assurance activities associated with the FML are completed. When possible, the soil should be placed without driving construction vehicles directly on the FML. Trucks carrying soil may back over the FML as they dump the soil in front of the path of the rear wheels. This or a similar procedure should be used to avoid damage to the FML caused by stones embedded in the tires.

The FML used in a final cover system is installed following the construction and filling of the facility. The design and construction of the cover liner system is detailed in the facility closure plan. Installation of this FML is subject to the same procedural and quality assurance requirements as the FMLs lining the containment. Additional information on cover systems is provided in Chapter 5 of this Guide.

4.4.3 Leachate Collection and Removal Systems

After the LCRS design is completed and specifications prepared, the construction will proceed in the following general sequence:

- Sump construction and.pump installation,
- Piping installation,
- Placement of granular materials or geonet, and
- Granular filter layer or geotextile placement.

4.4.3.1 Sump Construction and Pump Installation

Since the sumps will be the lowest points in the LCRS, they will be constructed first. Generally the components of a double-lined system will be sloped toward the sumps, and the sump areas will be formed to receive the sump structures. The drawings should clearly show the dimensions of the sump areas and the elevations of the low points of the inlets and of the sumps themselves.

If the sump is concrete, the form work should be constructed, reinforcing bars or wire installed, and the concrete poured and allowed to cure. If precast concrete structures are used, such as manhole sections, these are placed before the drainage layers and piping are installed. Again, it is critical to maintain careful control of all elevations to assure that the system will drain properly.

Pumps that will remain in the sump need not be installed until after the LCRS is completed. As indicated in Section 4.3.3.6., leachate generally is removed using.pumps that are portable and are not left in the sumps between uses.

4.4.3.2 Piping Installation

The leachate collection piping is generally installed after each liner is placed, but before the drainage layer is constructed. The construction drawings and specifications should clearly indicate the type of bedding to be used under the pipes and the dimensions of the trenches, if the pipes are placed in trenches. The specifications should indicate how the pipe lengths are joined. The drawings should show how the pipes are placed with respect to the perforations; in order to maintain the lowest possible leachate head, there should be perforations near the pipe invert, but not directly at the invert. The pipe invert itself should be solid, in order to allow for efficient pipe flow at low volumes. Geotextiles or granular filter layers should be placed around the pipes in the same manner as on the top of the drainage layer, as discussed in Section 4.4.3.

4.4.3.3 Placement of Granular Materials

Granular materials are generally placed using conventional earthmoving equipment, including trucks, scrapers, bulldozers, and front-end loaders. If the materials are being placed over an FML or geotextile, then they should be placed without driving vehicles directly over the FML or geotextile.

Coarse granular drainage materials, unlike low-permeability soils, can be placed dry and need not be heavily compacted. However, in order to assure that settlement following placement will be minimized, the granular material may be compacted with a vibratory roller after placement and rough grading. The elevations of the top of the drainage layer should be carefully surveyed after final grading to assure that the layer is of adequate thickness.

4.4.3.4 Placement of Geonets

Geonets are often used on the sidewalls of hazardous waste disposal facilities because of their stability and ease of installation. They should be placed with the top ends in a secure anchor trench and the strongest longitudinal length extending down the slope. They should be installed with a minimum of joints or seams on the slope. The geonets need not be seamed to each other on the slopes, only carefully butted or overlapped and tied. They should be placed in a loose condition, not stretched or in a configuration where they are bearing their own weight in tension. The construction specifications should contain instructions to the contractor to follow the procedures described above and the special installation instructions provided with the geonet by the manufacturer. All geonets not covered by an FML need to be protected by a filter geotextile to prevent clogging.

As with all components of the LCRS, the design drawings and specifications must not contradict one another and must match the assumptions used in the design calculations.

4.4.3.5 Granular Filter Layer Placement

Granular filter layers as described in Sections 4.2.3.6. and 4.3.3.1. are provided to allow passage of liquids while preventing the passage of fine soil particles into the drainage layer or into the LCRS piping, thereby preventing physical clogging of the system.

The critical placement criterion for this component, once the materials are selected, is that of thickness. Generally the granular filter materials will be placed around perforated pipes by hand, forming an "envelope," the dimensions of which should be clearly shown on the drawings. This envelope can be placed at the same time as the granular drainage layer, but it is critical that it be complete, that is, that the filter envelope protect all areas of the pipe where the clogging potential exists. The plans and specifications should indicate the extent of the envelope and the construction inspector should observe the placement of the envelope, to assure that it is complete.

A granular filter layer placed above the primary LCRS must also be placed to the required thickness and extent shown on the drawings. It is generally placed using the same earthmoving equipment as the granular drainage layer. The final thickness should be checked by surveying its elevations to assure that they are the specified distance above the top of the drainage layer.

This filter is the uppermost layer in the LCRS; however, most landfill designs include a buffer layer, 12 inches thick or more, to protect the filter layer and drainage layer from damage due to traffic. This final layer can be fairly general fill, as long as it is no finer than the soil used to design the filter layer discussed in Section 4.2.3.6.

4.4.3.6 Geotextile Placement

One of the advantages of geotextiles is their light weight and ease of placement. The geotextiles are brought to the site, unrolled, and held down with sandbags until they are covered with a protective layer. They are often overlapped, not seamed, however on slopes or in other configurations they may be sewn.

As in granular filter layers, it is critical that the design drawings be very clear in their designation of the extent of geotextile placement, such that no potential route of pipe or drainage layer clogging is left unprotected. If geotextiles are used on a slope, they should be secured in an anchor trench similar to those for FMLs or geonets.

Because geotextiles are vulnerable to damage from sunlight, wind, or traffic, they should also be covered with a layer of general fill, as soon as possible, that is no finer than that used to select the geotextile.

4.5 Quality Assurance

A specific Construction Quality Assurance plan (CQAP) should be developed for each construction project that specifies the type, amount, sequence, and frequency of inspections and testing for each component. The CQAP should include example reporting sheets, inspection logs, photographic logs, problem and corrective measures reports, and any other standardized forms to be used during construction. There should be a place in the CQAP documentation to record each of the observations discussed in the following sections.

4.5.1 Low-Permeability Soil Liners

Liner failures at some facilities have been attributed to poor construction quality assurance and quality control. Therefore, quality assurance has become an essential step in design and construction of hazardous waste facilities. Detailed information is provided in Reference 9 for construction quality assurance of low-permeability soils at pages 18-27. EPA considers quality assurance the highest priority for all facility components.

Sampling and testing of the soil liner during all phases of construction is necessary to ensure quality control. Testing provides verification of visual inspections. Field density and water content are two critical parameters which must be tested frequently during construction activities. Field and laboratory determinations should be made for these parameters and for hydraulic conductivity. Specific tests and methods are provided in Exhibit 4-4 and in References 9, 12 and 13. A specific sampling and test plan should be determined prior to all construction activities (Reference 9, p. 18).

4.5.2 Flexible Membrane Liners

Preconstruction quality control activities for FMLs include inspection of the raw materials, manufacturing operations, fabrication operations, and final product quality; observations related to transportation, handling, and storage of the membrane; inspection of foundation preparation; and evaluation of the personnel and equipment to be used to install the FML. Construction activities include inspection of FML placement, seaming of the FML, installation of anchors and seals, and placement of an upper bedding layer, or protective cover. Postconstruction activity includes checking for leaks in the installed FML (Reference 9).

The quality of the FML seams and the seaming process must be estimated from the results of inspecting representative samples of the total material installed in a lined facility. The quality of all materials is assessed under a 100-percent inspection program.

Nondestructive tests on seams are performed in the field on an in-place FML and retain the integrity of the FML seams or sheet being tested. Non-destructive test methods are listed in Exhibit 4-10 (Reference 43).

Destructive tests on seams are performed in either the field or laboratory. The intent is to determine the strength characteristics of a seam sample by stressing the bond until either the seam or the FML sheeting fails. Destructive testing of factory and field seam samples involves determining seam strength in both shear and peel modes, which is performed on a tensile testing machine (References 13 and 43).

If the test results for a seam sample do not pass the acceptance/rejection criteria, then samples must be cut from the same field seam on both sides of the rejected sample location. Samples are collected and tested until the areal limits of the low quality seam are

Exhibit 4-10. Non-Destructive Test Methods from FML Seams

Test Method	Detects
Probe test	Leak paths and unbonded edges of seams.
Air lance	Leak paths and unbonded edges of seams.
Vacuum box	Leak paths in seams or pinholes in sheets.
Ultrasonic pulse echo	Major voids or defective areas in the seam.
Ultrasonic impedance plane	Leak paths and unbonded factory field seams.
Electrical spark test	Voids, pinholes, or unbonded areas primarily in HDPE welds; it can also be used to test solvent bonds.
Pressurized dual seam	Leak paths and unbonded edges of double-wedge, thermally welded seams where an air chamber exists between the parallel bonds of the dual seam.
Electrical resistivity	Holes, seam unbonds, and improper penetration seals in FML installation.
Hydrostatic test	Any leaks in the FML including pinholes, tears, seam unbonds, and faulty attachments to penetrations.

defined. Corrective measures must be undertaken to repair the length of seam that has not passed the acceptance/rejection criteria. In many cases, this involves seaming a cap over the length of rejected seam (References 9, 13, and 43).

4.5.3 *Leachate Collection and Removal Systems*

Construction quality assurance (CQA) guidance for leachate collection systems is discussed in Section 2.3.6 of Reference 9. With the exception of granular drainage and filter layer materials which should have soils laboratory testing, most of the CQA activities involve observations and field testing.

4.5.3.1 Inspections

Prior to construction, all materials should be inspected to confirm that they conform to the design criteria, plans, and specifications. These include, as appropriate:

- Geonets;
- Geotextiles;
- Pipe size, materials, and perforations;
- Granular material gradation and quality;
- Prefabricated structures (sumps, manholes);
- Mechanical, electrical, and monitoring equipment; and
- Concrete forms and reinforcement.

In addition, the LCRS foundation (FML or low-permeability soil liner) should have been inspected and surveyed upon its completion to ensure that it has proper grading and is free of debris and liquids.

During construction, the following activities, as appropriate, should be observed and documented:

- Pipe bedding placement including quality, thickness, and areal coverage;
- Pipe network installation including location, configuration, grades, joints, filter layer placement, and final flushing if necessary;
- Granular drainage layer placement including thickness, coverage, compaction, and protection from clogging by runoff;
- Geonet placement including layout, overlap, seaming, weather conditions, and protection from clogging by runoff;
- Granular filter layer placement including material quality and thickness;
- Geotextile layer placement including coverage, overlap and seaming;
- Sumps and structure installation; and
- Mechanical and electrical equipment installation including testing.

4.5.3.2 Testing

In addition to field observations, actual field and laboratory testing should be performed to assure that the materials meet the specifications and that they will perform adequately after construction. These activities should also be documented in a manner similar to the field observations. They include the following:

- Geonet and geotextile layer sampling and testing;
- Granular drainage and filter layer sampling and testing for particle sizing; and
- Testing of pipes for leaks, obstructions and alignments.

Upon completion of construction, each component must be inspected to assure that it has not been damaged during its installation or during construction of another component, e.g., pipe crushing during placement of granular drainage layer. Any damage that does occur must be repaired, and these corrective measures must be documented in the CQA records.

4.6 References

1. *Code of Federal Regulations, Title 40, Parts 199 to 399,* July 1, 1987.

2. Federal Register. 1986. *Proposed Codification Rule for Minimum Technology Requirements for Double-Liner Systems.* Vol. 51, No. 60, March 28, 1986.

3. Federal Register. 1987. *Draft Minimum Technology Guidance Documents for Single and Double Liner Systems.* Vol. 52, No. 74, April 17, 1987.

4. EPA/OSW. 1987. *Background Document on Bottom Liner Performance in Double-Lined Landfills and Surface Impoundments.* EPA/530-SW-87-013. U.S. Environmental Protection Agency, Washington, D.C.

5. EPA/OSW. 1987. *Guidance on Implementation of the Minimum Technological Requirements of HSWA of 1984, Respecting Liners and Leachate Collection Systems.* EPA/530-SW-87-012, May 24, 1985. U.S. Environmental Protection Agency, Washington, D.C.

6. EPA/OSW. 1987. *Draft Minimum Technology Guidance on Single Liner Systems for Landfills, Surface Impoundments, and Waste Piles--Design, Construction, and Operation.* EPA/530-SW-85-013, May 24, 1985. U.S. Environmental Protection Agency, Washington, D.C.

7. EPA/OSW. 1987. *Draft Minimum Technology Guidance on Double Liner Systems for Landfills and Surface Impoundments–Design, Construction, and Operation.* EPA/530-SW-87-014, May 24, 1985. U.S. Environmental Protection Agency, Washington, D.C.

8. Grube, W.E., Jr., M.H. Roulier, and J.G. Hermmann. *Implications of Current Soil Liner Permeability Research Results.* In: The 13th Annual Research Symposium, Land Disposal Remedial Action, Incineration, and Treatment of Hazardous Waste. EPA/600/9-87/015. pp. 9-25. July 1987.

9. Technical Guidance Document: *Construction Quality Assurance for Hazardous Waste Land Disposal Facilities*; Hazardous Waste Engineering Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency; Cincinnati, Ohio 45268; EPA Contract No. 68-02-3952, Task 32; October 1986.

10. *ASTM 1987, The American Society for Testing and Materials 1987 Annual Book of ASTM Standards*, Volume 4.08, Soil and Rock; Building Stones, Philadelphia, Pennsylvania.

11. Goode, D., *Evaluation of Simplified Technique for Predicting Moisture Breakthrough of Soil Liners*, National Conference on Management of Uncontrolled Hazardous Waste Sites, Washington, D.C., October 31 - November 2, 1983.

12. Technical Resource Document: *Design, Construction, and Evaluation of Clay Liners for Waste Management Facilities*; Hazardous Waste Engineering Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency; Cincinnati, Ohio 45268; EPA/530-SW-86-007F, September 1988.

13. Technical Resource Document: *Lining of Waste Impoundment and Disposal Facility*; Solid and Hazardous Waste Research Division, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency, Cincinnati, OH 45268; SW870; March 1983. (Revised version Reference 19)

14. Anderson D. and S. Jones, *Clay Barrier - Leachate Interaction*, National Conference on Management of Uncontrolled Hazardous Waste Sites, Washington, D.C., October 31 - November 2, 1983.

15. *Foundations and Earth Structures*, Naval Facilities Engineering Command, Design Manual 7.02, September 1986.

16. Koerner, R. M. 1986. *Designing With Geosynthetics*. Prentice-Hall, Englewood Cliffs, NJ.

17. EPA. 1982. *Landfill Design, Liner Systems, and Final Cover*. Draft RCRA Guidance Document, July 1982.

18. EPA. 1982. *Surface Impoundments, Liner Systems, Final Cover, and Freeboard Control.* Draft RCRA Guidance Document, July 1982.

19. *Lining of Waste Containment and Other Impoundment Facilities*. Draft Technical Resource Document. EPA-600/2-88-052, September 1988.

20. Moore, Charles A.; *Landfill and Surface Impoundment Performance Evaluation*; USEPA SW-869;1982.

21. Richardson and Koerner; *Geosynthetic Design Guidance for Hazardous Waste Landfill Cells and Surface Impoundments*; EPA-600/2-87-097; NTISPB88-131263; December 1987.

22. *The Hydraulic Evaluation of Landfill Performance (HELP) Model - Volume 1. User's Guide for Version 1*; USEPA; June 1984.

23. *Hydraulic Evaluation of Landfill Performance (HELP) Model - Volume 2. Documentation for Version 1*; USEPA, June 1984.

24. ASCE, WPCF; *Design and Construction of Sanitary and Storm Servers, ASCE Manual on Engineering Practice No. 37, WPCF Manual of Practice No. 9*; 1969

25. GCA Corporation; *Draft Permit Writers' Guidance Manual for Hazardous Waste Treatment, Storage, and Disposal Facility*, 1983.

26. Perrier, E.R. and Gibson, A.C., *Hydrologic Simulation on Solid Waste Disposal Sites*, U.S. Environmental Protection Agency, Report SW-868, Washington, D.C., 1980. PB 81-166-332.

27. Technical Manual: *Guide to the RCRA Land Disposal Permit Writers' Training Program, Volume 1*, Prepared by GCA Corporation, U.S. Environmental Protection Agency, Land Disposal Branch, Washington, D.C. 20460, EPA Contract No. 68-02-3168, September 1984.

28. Anderson, D.C., J.O. Sai, and A. Gill, *Permeability and Morphology of a Soil Liner Permeated by Acid, and Field Permeability Testing for Soil Liners*, Division of Solid and Hazardous Waste Research, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency; Cincinnati, Ohio 45268; EPA Contract No. 68-03-2943.

29. EPA SW-846, *Test Methods for Evaluating Solid Waste*, September 30, 1986. U.S. EPA, Washington, D.C.

30. American Society For Testing and Materials. Issued Annually. *Annual Book of ASTM Standards; Section 8, Vols. 8.01, 8.02, 8.03, and 8.04; Section 9, Vols. 9.01 and 9.02*. American Society for Testing and Materials, Philadelphia, PA.

31. U.S. General Services Administration. 1980. *Federal Test Method Standard 101C*. U.S. General Services Administration, Washington, D.C.

32. Daniel, D.E. and K.W. Brown, *Landfill Liners: How Well Do They Work and What is Their Future?* Chapter 19, in: *Remedial Action and Case Histories*.

33. Haxo, H. E. 1983. *Analysis and Fingerprinting of Unexposed and Exposed Polymeric Membrane Liners*. In: Proceedings of the Ninth Annual Research Symposium: Land Disposal, Incineration, and Treatment of Hazardous Waste. EPA-600/9-83-018, pp. 157-171, Sept. 1983. U.S. EPA, Cincinnati, OH.

34. Landreth, R. E. 1987. *The EPA Testing Program for Components of Treatment, Storage, and Disposal Facilities*. Presented at the Geosynthetics '87 Conference, February 24-26, 1987, New Orleans, LA.

35. EPA. 1986. *Supplementary Guidance on Determining Liner/Leachate Collection System Compatibility*. U.S. EPA Office of Solid Waste and Emergency Response, Policy Directive No. 9480.00-13, August 7, 1986.

36. EPA. 1987. *FLEX: Flexible Membrane Liner Advisory Expert System User's Guide, Version 2.0*. U.S. EPA, Cincinnati, OH.

37. *ASTM Annual Book of ASTM Standards, Volume 04.01 Cement; Lime; Gypsum, Volume 04.02 Concrete and Mineral Aggregates*, Philadelphia, PA.

38. American Concrete Institute *Building Code Requirements for Reinforced Concrete*, ACI 318-83, ACI, Detroit, MI.

39. American Concrete Institute *Concrete Sanitary Engineering Structures*, ACI 350 R-83, ACI, Detroit, MI.

40. American Concrete Institute *Building Code Requirements for Structural Plain Concrete*, ACI 318.1-83, ACI, Detroit, MI.

41. *ASTM; Annual Book of ASTM Standards*; Philadelphia, PA *Volumes 01.04 and 01.05 Steel*.

42. Haxo, H. E. 1986. *Quality Assurance of Geomembranes Used as Linings for Hazardous Waste Containment*. Geotextiles and Geomembranes, No. 3 (1986), pp. 225-247. Elsevier Applied Science Publishers Ltd., England.

43. SCS Engineers. 1987. *Manual of Procedures and Criteria for Inspecting the Installation of Flexible Membrane Liners in Hazardous Waste Facilities*. U.S. EPA, Hazardous Waste Engineering Research Laboratory, Cincinnati, OH.

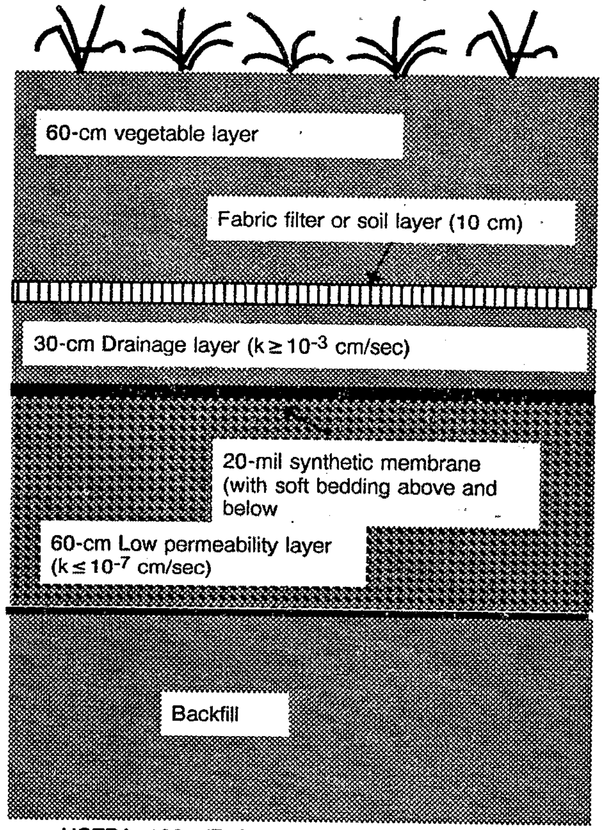
CHAPTER 5.0
Cover Systems

Land disposal unit covers are the final component in the design of a land disposal management system. As the protective outer layer placed on a landfill or a disposal impoundment after it has been filled, the cover should isolate the wastes from the environment. Specifically, the cover must be designed to minimize the infiltration of surface water, thus minimizing liquid migration and leachate formation. To achieve this performance standard, the owner/operator of a land disposal facility must design and construct a multi-layered cover system that can function with minimum maintenance. Generally, the system will include:

- an uppermost vegetated layer to prevent erosion and promote evapotranspiration;
- an underlying drainage layer to convey percolation out of the cover; and
- a moisture barrier to prevent infiltration.

Each of these layers is constructed either of natural soil materials or of synthetic materials including geomembranes (FMLs), geonets, and geotextiles. This chapter provides an overview of current regulatory performance standards and procedures for the design, materials selection, construction, maintenance, and quality control of cover systems.

Exhibit 5-1. Landfill Cover System Components



Source: USEPA, 1987 (Ref 1), p. 3

5.1 Regulations and Performance Standards

The performance standards set forth in 40 CFR 264.228 and 264.310 state that disposal impoundments and land disposal units must have covers that provide long term minimization of liquid migration. The covers must be able to function with minimum maintenance, promote drainage to minimize erosion and abrasion, accommodate settling and subsidence without losing integrity, and have a permeability no greater than the bottom lined system. The permitting standards of 40 CFR 270.21(e) require the permit applicant to provide detailed plans and an engineering report of the cover design as part of the closure plan for the facility. These plans and report must demonstrate through calculations and specifications that the cover will function as required. The multi-layer cover illustrated in Exhibit 5-1 has been provided as an example of a design that would meet minimal RCRA requirements as defined in current EPA guidance (Reference 1). The performance objectives for each layer will be discussed in Section 5.2.6 *Cover System Elements*. Actual geological settings are complex and can differ greatly; therefore, the minimal design shown cannot be directly applied to any one site. A technical analysis must be performed by a qualified design engineer registered in the state to guarantee that the performance standards for each cover system layer have been achieved in a particular geological setting

5.2 Design

To design a cover system, the characteristics of the site and the wastes in place must be accurately assessed. With this information, site and waste constraints, such as settlement and subsidence potential, may be calculated. Following this evaluation, the requirements for cover composition and configuration will be selected. Reference 1 provides technical guidance on the design process. Reference 2 describes a recommended 39-step approach to evaluating cover designs. The major design elements to consider will be discussed in the following section.

5.2.1 *Site Characterization*

Site characterization is of primary importance throughout the design process for a land disposal facility beginning with the initial siting of the facility. For cover design, site characterization directly impacts the criteria chosen for material selection and design to prevent erosion and to promote the establishment of hardy vegetation. The following discussion will introduce several key aspects of site characterization and their impacts on cover design.

5.2.1.1 Topography

Topography becomes a major factor in cover design when the landfill is sited in areas with hilly terrain or canyons and where surface impoundments may be below ground. In these environments, a considerable amount of surface water run-on should be expected, and the designer must be prepared to manage or prevent this surface run-on from traveling onto the cover. The designer should address the potential problem by performing the routine analyses of surface water flow for the surface water management requirements described in Chapter 6.0. It may be determined after analysis that traditional perimeter diversion systems will not be adequate and that other designs need to be considered. One option is to construct a central drainage system through the center of the landfill. For a flat site without natural positive drainage, the cover system must be designed to provide positive drainage of precipitation off the site to prevent ponding over the cover.

5.2.1.2 Precipitation

The intensity, duration, and frequency of storms must be determined to calculate the volume of surface water run-on or run-off that must be managed. The rate of infiltration (percolation) will directly impact the design of the drainage layer. An analysis of local precipitation patterns will also provide information on whether a potential for flooding exists. If there is flooding potential, the flood characteristics (e.g. stagnant backwater or scour potential due to flow) must be evaluated and measures designed to prevent damage to the cover or ponding on the cover. Local, site-specific precipitation data should be used whenever available for design calculations. Average annual precipitation maps developed by the U.S. Department of Agriculture and the National Weather Service are appropriate for use in review of designs. Section 2 of Reference 2 provides a more in-depth discussion of the review of precipitation data.

Precipitation data, particularly the annual distribution of rainfall, is also critical to the selection of the types of vegetation to be established on the cover. Since closure performance standards require that maintenance be minimized, vegetation should be selected that would adapt to the environment with a minimum amount of irrigation.

5.2.1.3 Other Climatological Data

In climates that experience freezing temperatures or drought, the upper surface layers of a land disposal unit cover may be damaged by the buckling or sliding of layers after thawing or by cracking during extended periods of drought. As a general rule, the geomembrane and the top of the low permeability soil layer, are the most susceptible to damage due to severe weather and should be placed below the depth of freezing or severe drying. Freezing also increases the amount of surface water run-off expected during winter months, as percolation through frozen ground is limited. This fact should be considered in run-off discharge calculations for drainage channel design. Freezing indexes illustrated as map contours have been developed by the U.S. Weather Bureau. Indices showing the severe drought regions of the country are available from regional Soil Conservation Service offices. As was true for precipitation data, the more site-specific data available, the more accurate the design calculations. Reference 1 discusses the influence climatology has on cover design in Section 3. Reference 2 addresses the review of climatological data in Section 3.

5.2.1.4 Soils

An assessment of the properties of the in-situ soils, while not a constraint in the design of the cover system, is important from a cost-effectiveness standpoint. Considerable savings could be gained if site material can be used as part of the intermediate or final cover system. Soil tests run under the direction of a qualified geotechnical engineer for land disposal unit siting and design are useful sources of information during the material selection process. A more detailed discussion of soil properties is provided in Section 5.3, Materials, of this chapter.

5.2.1.5 HELP Model

To assist the designer in determining the influence that site characterization factors will have on the performance of a cover system design, a computer model was developed called Hydrologic Evaluation of Landfill Performance (HELP). The HELP Model was designed by the U.S. Army Corps of Engineers Waterways Experiment Station (WES) for the U.S. EPA Municipal Environmental Research Laboratory. The Model is generally accepted for designing landfill cover layer systems and for comparing alternative cover and total landfill configuration designs. (Reference 3, Section 1). Use of the Model in design of leachate collection systems is discussed in Chapter 4.0.

The HELP program calculates daily, average and peak estimates of water movement across, into, through, and out of landfills. The input parameters for the model include soil properties, precipitation and other climatological data, vegetation type, and landfill design information. Default climatologic and soil data are available but should be verified as reasonable to expect in the particular site setting. Outputs from the model include precipitation, runoff, percolation through the base of each cover layer subprofile, evapotranspiration, and lateral drainage from each profile. The Model also calculates the maximum head on the barrier soil layer of each subprofile and the maximum and minimum soil moisture content of the evaporative zone. Data from the model are presented in a tabular report format and include the input parameters used and a summary of the simulation results. Results are presented in several tables of daily, monthly and annual totals for each year specified. A summary of the outputs is also produced, which includes average monthly totals, average annual totals and peak daily values for various simulation variables. (Reference 3, Sections 4 and 5)

Use of the HELP model should not be attempted without reading the User's Guide, Reference 3, or the Model Documentation, Reference 4, both prepared by the designers of the program.

5.2.2 *Waste Characterization*

Cover settlement has been determined to be caused by primary consolidation and secondary compression of the waste mass, underlying natural soils, and from collapse of voids or cavities in the fill and around containers. Primary consolidation occurs when the void ratio of a soil or waste is decreased due to the expulsion of fluids from the voids under excess hydrostatic pore pressure. Secondary compression occurs by deformation of the skeletal structure of the mass and compression of gases in the voids. The collapse of voids or cavities is due to corrosion, oxidation, combustion, or biochemical decay of the landfilled materials. The designer should be aware of the distribution of void spaces and other physical conditions of the waste at the time of burial, the waste placement operations (e.g. lift thickness, compactive effort, etc.) and the chemically-related changes due to the composition of the wastes that may take place over a long period (Reference 1, p. 9, Reference 5, p. 2).

Wastes which enter the land disposal unit are either disposed of in bulk or in containers. Bulk wastes may exhibit the settlement characteristics of soils in that they continue to consolidate over time, but at a steadily decreasing rate depending upon the physical characteristics of the waste and the methods of waste placement. To assist in the settlement analysis, recent efforts have been made to determine the engineering properties of several types of wastes through laboratory testing. Results of these efforts are presented in Reference 5 in Section 2. The laboratory analyses, however, should never be used as more than a general guide to expected properties. The wastes reported to have been disposed of at a given facility should be evaluated to the extent possible for a site-specific determination.

Containerized wastes do not behave as predictably as bulk wastes. Consolidation of drummed wastes occurs at a considerable period of time after waste placement when drum deterioration occurs. An acceptable, accurate analytical method is not currently available for prediction of the time and extent of this later settlement due to container deterioration. However, the designer should address the potential for future subsidence due to the disposal of containers and qualitatively approximate the potential damage.

Another important characteristic of land disposal unit wastes which directly effects settlement is the percent of void space within the cell configuration of wastes. An estimate of the effects these void spaces will have on long term settlement is required. Often, sufficient attention is not given to filling the void spaces between containers within landfill cell lifts. When the lifts have not been properly backfilled, void spaces several rows deep may be left as channels for the downward migration, or piping, of backfill. Backfill piping can cause differential settlement and damage to the cover.

The chemical composition of the wastes must be carefully reviewed to determine gas generation potential. Diversions and vents may be required in the design to provide a release pathway for gases blocked by the cover from upward migration. If low concentrations of toxic components are expected, vents directly to the atmosphere may be adequate for dispersion in the air at acceptable levels. It may be necessary to provide on-line or contingency features for absorption filters or other means of reducing concentration of toxic components if the potential exists for the gas or volatile component to reach harmful concentrations (Reference 1, p. 13). Gases evolve from the decay or biodegradation of buried organic matter; thus, gas control (venting) is principally a concern at municipal waste, not hazardous waste, landfills.

5.2.3 *Settlement/Subsidence*

A potential threat to the integrity of the cover is uneven settlement of the wastes and fill that comprise the foundation of the cover. Recent guidance (Reference 5) has been published regarding the prediction/mitigation of subsidence damage to covers and will be briefly summarized in the following paragraphs.

Long-term settlement of hazardous waste land disposal units should be analyzed on the basis of the deformation of the waste layers and the deterioration of the waste containers. Settlement due to deformation of the waste layers is most likely to occur after closure of the land disposal unit and final placement of the cover. Therefore, this type of settlement has more potential to cause subsidence damage to the cover than consolidation settlement, much of which can occur or can be made to occur prior to closure. (Reference 5, p. 19)

Several models have been developed to analyze the process of differential settlement. Most equate the layered cover to a beam or column undergoing deflection due to various loading conditions. While these models are useful to designers in understanding the qualitative relationship between various land disposal unit characteristics and in identifying the constraining factors, accurate quantitative analytical methods have not been developed (Reference 5, Section 4).

If settlement is anticipated, several design options are available. For example, the cover thickness can be designed such that after displacement occurs, drainage of run-off is still adequate. Exhibit 5-2 illustrates this design compensation method. Another option is to increase the side slopes of the cover. (Reference 5, p. 71).

In summary, although settlement has the potential to seriously damage a land disposal unit cover, the analytical methods available to estimate the effects are still inexact and require additional experimentation and field observation. For now, the designer of land disposal unit covers should determine whether the potential for settlement exists due to the type of wastes and landfilling procedures used and design the cover to provide a tolerance for settlement effects (Reference 5, Section 5).

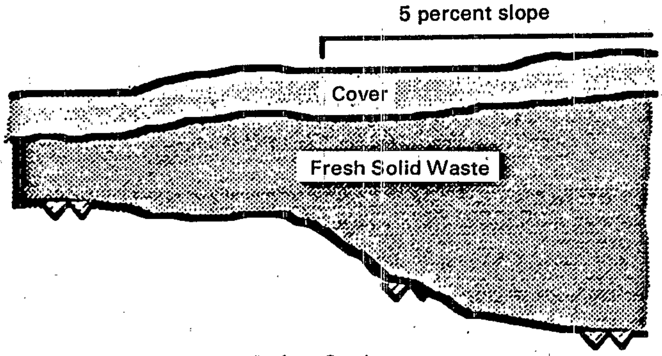
5.2.4 *Slope Stability*

One threat to the continued soundness of the cover is displacement due to the slope instability. Slope stability analyses should be performed to assess the potential for slope failure by various failure modes (e.g., rotational, sliding, wedge), as appropriate, based upon the slope configuration. To adequately perform the stability analyses, the strength properties of the cover system components, the waste, and the foundation soils must be known along with seepage conditions. A detailed discussion of slope stability can be found in Chapter 3 of this document.

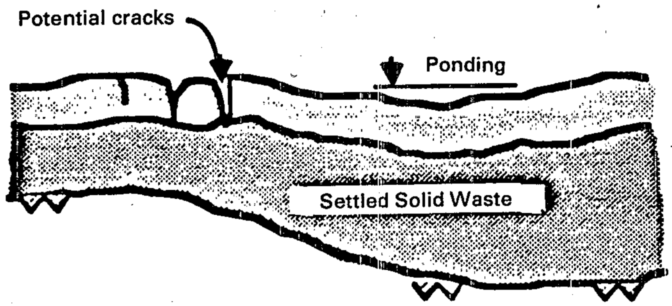
5.2.5 *Erosion Potential*

In addition to ensuring embankment slope stability, the designer should design the cover to minimize soil erosion. To assist the designer in predicting erosion potential of various design options, EPA recommends use of an empirical formula called the Universal Soil Loss Equation (USLE) which is used to calculate the average annual soil loss. The average annual soil loss is predicted based upon a number of factors including the geographical location, the length and steepness of slopes, the texture of the cover soil, and the vegetation established.

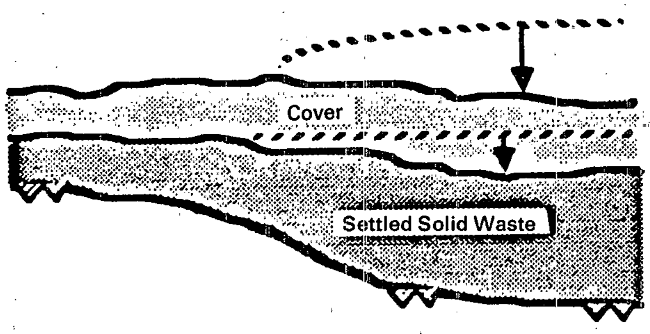
Exhibit 5-2. Thickened Cover for Tolerance of Settlement



a. Before Settlement



b. After Settlement



c. Thickening cover before and after settlement

Reprinted: USEPA, "Design of Cover Systems," 1987 (Ref. 1)

Reference 1 provides complete instructions for use of the USLE. Also provided in Reference 1 on pages 37 through 42 are various tables and figures from which input parameters can be selected. Cover design features that are used to prevent erosion include the establishment of vegetation and the construction of terraces or benches. Current EPA guidance suggests that the average annual soil loss not exceed 2.0 tons/acre in order to minimize the potential for gully development and future maintenance. (Reference 6, p. 29)

5.2.6 *Cover Systems Elements*

The requirements for the cover are performance-based; therefore, the elements incorporated into the design may vary depending on the type of materials available, the environment in which it is placed, and the type of wastes landfilled. However, the cover components that are shown in Exhibit 5-1 are found in most cover systems and will be described in the following section. Reference 1 was used as the main source of information for the following description of the cover system components.

Each of the elements must be shown on the cover design drawings. At a minimum, the drawings that should be included are:

- plan view of the landfill or disposal impoundment;
- cross-sections at several locations;
- details where cover components key into the liner system;
- details for cover connections between adjacent cells; and
- details of penetrations such as leachate removal manholes or gas vents (if required).

Drawings should clearly show the cover configurations, the dimensions (thickness, slope) of each element, and the finished elevations of the tops of the soil layers at critical points. These drawings should reflect the same dimensions and slopes used in the design calculations, especially those involving run-off controls, percolation estimates, and drainage layer performance.

5.2.6.1 Foundation, Backfill

The foundation or foundation layer serves several purposes. As the intermediate layer between the wastes and the cover, the backfill brings the landfill cell to final grade in preparation for cover placement and it serves as protection to the cover from the direct impact of settlement within the waste cell. Accordingly, the backfill will distribute the load and deformation imposed on the cover.

5.2.6.2 Low-Permeable Soil Layer

A two-foot thick low-permeable soil layer is recommended by EPA guidance in the design of cover systems. The low-permeable soil layer is commonly referred to as the secondary hydraulic barrier. (The geomembrane is considered the primary hydraulic barrier.) This soil layer is designed and constructed in a manner similar to that used for soil liners, with the following differences:

- Since the soil component of the cover is not exposed to waste leachate, liner-leachate compatibility is not considered.
- Since the cover is generally not subjected to large overburden loads, the issue of compression is less critical, unless post-closure land use will exert large loads.
- The soil cover, however, is subject to the loadings of construction equipment that is used during placement and compaction of the lifts, and during placement of the overlying drainage layer and topsoil. This type of loading is discussed in Chapter 4.0.
- The soil cover is also subject to loadings from settlement, as discussed in Section 5.2.5.1. The settlement potential should be evaluated and the cover and post-closure maintenance plan designed to compensate for future settlement.

For a further discussion of soil liners, refer to Chapter 4.0.

5.2.6.3 Geomembrane (Synthetic Membrane)

The primary hydraulic barrier is recommended by EPA guidance to be a geomembrane of at least 20 mil. thick. When used in cover design, the material properties of major concern include puncture resistance, burst strength, tensile strength, permeability, freezing effects, heating effects, and temperature cycling. For a complete discussion of geomembranes, refer to Chapter 4.0 (on FMLs).

5.2.6.4 Drainage Layer

The drainage layer provides a passageway for the rapid removal of rainfall infiltration from the center and for the horizontal migration of any gases that may have permeated from the wastes through the geomembrane and low-permeable soil layer. The drainage layer should be constructed of materials exhibiting a minimum hydraulic conductivity of 1×10^{-3} cm/sec and be at least 12 inches thick to accommodate infiltration from major storms. A slope of at least two percent after allowing for settlement is recommended in order to provide positive drainage through the layer. The most common materials used are narrowly graded granular soils such as sand or gravel. To enhance the effectiveness of the drainage layer, perforated pipes or vents can be used. Alternate drainage layer designs, e.g. use of geonets, are commonplace and acceptable. However, the permit applicant should demonstrate that an alternate design will perform as effectively as the recommended design. Section 5 of Reference 1 and Section E of Reference 6 provide guidance on the design of the drainage layer.

5.2.6.5 Filters

EPA guidance recommends that the drainage layer be overlain with a graduated granular soil or a geotextile, to prevent clogging of the porous layer with the overlying soil. The purpose of the filter is to block the downward migration of particles with the percolating water. The filter material, therefore, should have intermediate pore sizes which can provide a framework for particle bridging. A concern has been raised that filters may be permanently damaged by physical deterioration or biological or chemical deposits; however, insufficient field data is available on the service life of filters, fabric or soil. Detailed information regarding the design of filter materials is provided on pages 59 through 61 of Reference 1. In addition, the design and construction of both the drainage layer and the filters is very similar to the LCRS discussed in Chapter 4.0 of this Guide.

5.2.6.6 Vegetated Topsoil

The final cover component, the vegetated topsoil layer, protects the other cover layers from the effects of wind and water erosion. The topsoil layer is generally two feet thick to accommodate the roots of the vegetation. Final thickness design should consider the water needs of the vegetation and the water holding capacity of the soil. In addition, the combined thickness of the drainage layer and topsoil layer should be greater than the local frost depth to prevent frost damage to the geomembrane and low-permeable soil layer.

The major criteria considered for selection of the topsoil are soil type, nutrient and pH levels, climate, species selection, mulching, and seeding time. The most desirable soil to use is loam, a balanced mixture of clay, silt and sand. Loam is preferred because it is relatively easy to maintain in good condition, it provides a conducive environment for seed germination, and it is easily penetrated by roots.

The condition of a soil for purposes of topsoil selection is determined by its pH level, nutrient content, and other favorable physical properties such as workability, either in the native or amended state. Generally, it is recommended that the soils be maintained at a pH of 6.5. (Reference 2, p.46). Lime applications may be necessary to achieve this pH level. The primary nutrients of concern are nitrogen, phosphorus, potassium and organic matter. A table is provided on page 47 of Reference 2 that indicates the acceptable ranges of organic matter content in different soil types and an approximate range of other nutrient levels. Reference 7 also provides general guidelines for the application rates of the various nutrients to soils that are determined to be in "poor condition."

The major characteristics that should be considered during species selection, as recommended by Reference 2, include low growing and spreading from rhizomes or stolons, rapid germination and development, and resistance to fire, insects, and disease. Grasses and legumes are usually selected. Reference 2 provides a table on page 49 that lists the important characteristics for species selection and provides examples of grasses and legumes that exhibit those characteristics.

Planting or seeding is generally recommended for the fall or very early spring. It is important for the seedlings to establish a root system during a cool, moist period before enduring a winter freeze or summer drought. Recommended planting times are discussed in detail on page 50 of Reference 2.

Many good resources are available to assist the designer in the selection of the plant species, the seeding rates and seasons, and the fertilization applications required. In addition to the EPA references mentioned above, agronomists familiar with the local soils and climate should be consulted. These experts can be found at the local county agricultural extension or soil conservation service office.

Chapter 8 of Reference 1 provides a good discussion of the design and maintenance of the final vegetative layer. Reference 7 offers a more in-depth discussion of the standardized procedures for planting vegetation on completed disposal unit covers.

Sites in arid regions of the country tend to use a granular cover surface instead of a topsoil layer. This is done because conditions are such that the vegetation is very difficult to establish and keep growing due to low rainfall. In those cases, the organic and chemical make-up of the topsoil layer is not as important. The surface layer, usually constructed of gravel or rock, must be designed to prevent erosion of the surface.

5.2.6.7 Boundaries

Lateral boundaries of the outer edges of the cover system are considered critical elements to ensure the integrity of the entire cover. The cover boundary serves to block infiltration from entering the outer edges of the landfill, and to key the cover system into the liner system or to the site geological structure. In addition, the boundary of the cover system must provide a path for free drainage of water from the drainage layer. One simple edge feature could be a vertical low-permeable soil barrier wall that connects the hydraulic barrier in the cover to a low-permeable soil stratum in the site media. In many landfills, an acceptable boundary feature can be simply overlapping or welding of the cover geomembrane across the FML. Another suggested alternative, if practical, is to extend the outer edge of the cover beyond the outer edge of the landfill liner system and provide a separate anchor trench for the cover layers. A detailed discussion of the alternatives to consider for boundary design is included in Section 4 of Reference 1 on pages 72-75.

5.3 Materials

As discussed in the design sections, the materials generally used for covers include sand, gravel, soil, geotextiles, and geomembranes. Other materials that are used less frequently include geonets, fly ash, portland cement concrete, bituminous concrete, and seal coats. Due to the high costs of importing materials, the common practice is to use, modify or amend native soils or other available site material.

Adequate characterization of the soils through proper testing is essential. Material testing is required during design to ensure that available soils can achieve the cover layer performance criteria. Material testing is also critical during construction to confirm that the materials and modification procedures were sufficient to achieve the design specifications for the various cover materials. The testing done during construction will be discussed in Section 5.4.

Current EPA guidance, Reference 1, recommends the use of standard tests when specifying material characteristics and properties. Preference is given to tests sanctioned by the American Society for Testing and Materials (ASTM). Each layer of the cover system has specific performance goals, therefore, different tests are required. A number of tests, however, should be specified as a minimum for each layer to determine soil characteristics and performance properties. Soil characteristics tests include particle-size distribution, Atterberg limits, soil classification, and water content. Standard performance properties tested include compaction and permeability. Chapter 2.0 of this Guide and Section 3 of Reference 1 contain a detailed discussion and a table of the tests available for use in materials selection.

Reference 1 (pp. 26-28) also provides an extensive table ranking the suitability of various soils for some of the specialty cover functions such as trafficability, water percolation, gas migration, erosion control, reduction of freeze action, crack resistance, slope stability, and vegetation support. The table ranks soils from I through XIII for a general indication of the "best" through "poorest" material for each cover function. Reference 2 (p. 20) suggests that soils rated as IV or higher for a specific function require modification prior to being selected for that use.

The cover foundation is a non-select soil present at the site and is used to establish a uniform base layer for the cover system. If this base layer will be directly overlain by a geomembrane, rather than a low-permeable soil layer, then the soil must be fine and free of pebbles and clods that could puncture the geomembrane.

The specifications required for the low permeability soil layer and the geomembrane are identical to those required for liner systems, and are described in Chapter 4.0 of this Guide.

The drainage layer due to its function, requires a high permeability which can be determined from the standard tests described in Chapters 2.0 and 4.0 of this Guide. The filter layer must be properly graded based upon gradation of the drainage layer to prevent its finer particles from migrating below and clogging the drainage layer. Reference 2, p. 34 and Reference 1, p. 60 provide standard criteria to use to select properly graded filter media.

If a geotextile is used, an equivalent opening size (EOS) and percent open area are used as the criteria. Specific geotextile criteria are outlined on page 61 of Reference 1. Additional performance tests commonly specified for geotextiles can include, as appropriate, tensile strength, bursting strength, puncture strength, abrasion resistance, seam breaking, permeability, grab strength, and grab elongation. A table of the recommended standard methods and requirements for these tests is provided on page 167 of the example contract specifications in Reference 1.

The soil for the vegetated cover layer requires specific testing for pH and buffering capacity as well as nutrient content. The nutrients that should be tested for include, in order of importance, nitrogen, organic matter, phosphorus, and potassium.

5.4 Construction

The major tasks of construction of a cover are:

- excavation of borrow for cover materials
- material preparation (blending, amendment)
- material placement
- compaction
- geomembrane placement

Standard construction procedures used in building and roadway earthwork also are used for construction of covers. Crews must be made aware that it is critical that the integrity of the cover layer beneath the working surface be protected from damage at all times. This caution may preclude the use of certain earthwork equipment such as sheepsfoot rollers. Sheepsfoot rollers have steel feet designed to help interlock lifts in foundation construction. This feature could, however, penetrate the relatively thin barrier layers used for covers. Another limitation on selection of equipment is the need to use equipment whose weight is within the allowable load-bearing range of the cover components. Detailed guidance on construction techniques is available in Section 6 of

Reference 1. An outline of construction techniques to note during a review is shown on page 36 of Reference 2. The following sub-sections will highlight general construction issues.

5.4.1 Excavation

The primary concern during the excavation phase is that the materials excavated are of the desired quality. Sufficient test borings and laboratory testing should be conducted to determine if the material meets specification, and to determine the extent of available borrow.

5.4.2 Soil Material Preparation

Material preparation can be accomplished by blending or modification with additives. Soils can be modified with additives such as lime, bentonite, cement or asphalt. Soil amendment is typically performed to enable the use of in-situ soils rather than imported material. Prior to using the blended or modified soil mixtures, tests should be run to verify that the mixture will meet the design specifications.

5.4.3 Soil Placement

Soil or material placement is the most critical phase of the construction process. Soils must be spread evenly and, as discussed previously, specific minimum thicknesses must be achieved. Soil placement must be carefully supervised to avoid disturbance of the underlying layer of cover. As an extra precaution during construction over the top of the geomembrane layer, Reference 1 recommends the placement of a buffer layer of soil or a fine mesh geogrid above the geomembrane.

If the cover will be placed over a disposal impoundment, placement should not occur until waste solidification has been completed. Because solidification may take several days or weeks, this period should be noted in any construction schedules or accompanying specifications.

5.4.4 Soil Compaction

Compaction of the soil layers is performed to increase the strength of the soil through the process of densification. For low-permeable soil layers serving as moisture barriers, compaction is also necessary to achieve the desired low permeability, as discussed in Chapter 4.0 of this Guide. Compaction is generally required for all layers except the topsoil where compaction would prevent proper root growth.

When cover layers exceed six inches in thickness, it is recommended that placement occur in lifts. While different lift thicknesses have been studied, six-inch lifts are generally recommended as optimum.

5.4.5 Geomembrane Installation

Geomembrane installation involves several steps including: placement of strip panels, seaming the panels with appropriate bonding or heat treatment, sealing the membrane around penetrations, and covering the geomembrane with a bedding layer. Reference 1 cites seam failures and punctures or abrasions of the geomembrane during installation as the most common problems encountered during geomembrane installation. Refer to Chapter 4.0 of this Guide and Chapter 6 of Reference 1 for more complete discussions.

5.5 Maintenance

A maintenance program must be developed, per 40 CFR 264.310, to insure the continued integrity and effectiveness of the cover. Preventative maintenance work should be scheduled periodically for two to three years after cover installation to prevent loss of vegetation and gully development. Maintenance inspections should be regularly scheduled to provide early warning of more serious problems developing that would impact the cover's integrity such as cover subsidence, slope failure, leachate or upward gas migration, or deterioration of the drainage system. Exhibit 5-3 provides a brief overview of the elements of a typical maintenance program. Section 4 of Reference 7 and Section 10 of Reference 1 provide detailed guidance on development of a post-closure maintenance program.

5.6 Quality Control

The principal activities of a Quality Control (QC) program for the construction of the cover include:

- screening incoming materials
- construction and testing of test fills
- observation of construction procedures
- measurement of final cover layer thicknesses
- surveying of final grades

The QC inspector must become thoroughly familiar with the specifications to ensure that materials and installation procedures conform to contract standards. More detailed discussions of QC testing for soil and geomembranes are provided in Chapter 4.0 of this Guide and Section 2 of Reference 8. One of the most important tasks for the QC inspector is to make the construction crews sensitive to the relative "fragile" nature of the cover layer components. In many cases, these crews are more experienced in roadway or building excavation and are not aware that carelessness such as driving equipment over or leaving equipment on the geomembrane or filters may compromise the overall integrity of the cover.

Exhibit 5-3. Typical Elements of Maintenance Program

PREVENTATIVE MAINTENANCE (2 to 3 years)

Cover System Component	Frequency	Task
Vegetation	twice per year annual	mowing (weed and brush) fertilization
Topsoil	as needed	soil reconditioning (supplemental fertilization, aeration)

PROBLEM IDENTIFICATION/CORRECTION

Cover System Component	Problem	Repair
Cover System	gully development	backfill to original grade with stone of narrow size range regrade cover replant vegetation
	subsidence	backfill with additional cover soil (care should be taken to maintain continuity of low permeable soil layer, geomembrane and drainage layer)
	slope instability	reconstruct cover flatten slopes add toe berm along base of slope
	gas migration that causes cracking	upgrade or install gas venting system install perimeter vents
Run-off Control System	erosion, siltation	placement of stone riprap or concrete modify channel alignment and/or gradients

5.7 References

1. U.S. EPA, "Design, Construction, and Maintenance of Cover Systems for Hazardous Waste, An Engineering Guidance Document," U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS. PB 87-191656. May 1987..

2. U.S EPA, "Evaluating Cover Systems for Solid and Hazardous Waste," Office of Solid Waste and Emergency Response, Washington, D.C. SW 867, September 1982

3. U.S. EPA, "The Hydraulic Evaluation of Landfill Performance (HELP) Model-Volume 1. User's Guide for Version 1." June 1984.

4. U.S. EPA, "The Hydraulic Evaluation of Landfill Performance (HELP) Model-Volume Z. Documentation for Version 1." June 1984.

5. U.S. EPA, "Prediction/Mitigation of Subsidence Damage to Hazardous Waste Landfill Covers," Hazardous Waste Engineering Research Laboratory. EPA 600-2-87-025. PB 87-175378.

6. U.S. EPA, "Landfill Design, Liner Systems and Final Cover," draft RCRA Guidance Document. July 1982

7. U.S. EPA, "Standardized Procedures for Planting Vegetation on Completed Sanitary Landfills," Municipal Environmental Research Laboratory, Cincinnati, Ohio. Grant No. CR-807673. July 1982.

8. U.S. EPA, "Construction Quality Assurance for Hazardous Waste Land Disposal Facilities," Office of Solid Waste and Emergency Response, Washington, D.C. EPA/530-SW-86-031. OSWER Policy Directive No. 9472.003.

CHAPTER 6.0
Run-On/Run-Off Controls

Surface water management is necessary at hazardous waste facilities to minimize erosion damage to earthen hazardous waste containment structures, and to prevent interference with the natural processes involved in certain hazardous waste treatment methods. Design of a surface water management system requires a knowledge of local precipitation patterns, surrounding topographic features, geologic conditions, and facility design. Surface water management systems do not have to be expensive or complex to be effective. The equipment and materials used for construction of the surface water management system are the same as those used for general earthwork and foundation construction. Construction may include excavation of a series of shallow channels to direct surface water flow, or in some cases, installation of basins to retain rainfall accumulation from sudden, intense storms. Surface water management systems are required for all hazardous waste treatment, storage and disposal areas. Because these systems are integral with hazardous waste facilities, the system components should be constructed during facility construction. This chapter introduces the regulatory requirements for surface water management systems, provides a general discussion of design criteria for those systems, and describes the types of system components, materials, and construction techniques that may be employed to control run-on/run-off at land disposal units.

6.1 The Regulations and Performance Standards

The requirements of 40 CFR Parts 264 and 270 concerning surface water management are comparable for most hazardous waste management units. For landfills (and waste piles), the objective of surface water management is to prevent increased generation of hazardous wastes through the mixing of run-on and precipitation with hazardous waste or leachate. For surface impoundments and secondary containment structures, control measures are intended to prevent overtopping of the structures by accumulated precipitation. (For land treatment operations, the prevention of run-on will prevent reduced efficiency of biodegradation processes and impaired tilling operations.)

The performance standards of 40 CFR Part 264 require the owner/operator of landfills to "design, construct, operate, and maintain run-off and run-on management systems capable of collecting and controlling at least the water volume resulting from a 24-hr, 25-yr storm". Surface impoundments must be designed to prevent overtopping resulting from run-on based on a 24-hour, 100-year storm. Collection and holding structures that are part of the system design must be expeditiously emptied or otherwise managed after every storm to maintain design capacities. Inspection is required during construction to ensure quality workmanship and materials. During operation, continued inspection is required to prevent erosion or other deterioration of the primarily earthen structures. Run-on/run-off control is required in both the closure and post-closure phases of operation.

To demonstrate that the standards will be met, the permit application requirements of 40 CFR Part 270 include for each unit, a provision for detailed plans and an engineering report that show the design and supporting calculations for run-on/run-off controls including collection and holding units. (The only exception to this requirement is for land treatment units where a description, rather than actual design plans, is specified.) Information commonly used in demonstrating that standards for run-on/run-off controls are met is provided in Exhibit 6-1.

6.2 Design and Operation

In accordance with the mixture rule [40 CFR 261.3(a)(2)(iii) and (b)(2)], precipitation run-off that "mixes" with hazardous waste is also considered a hazardous waste and must be treated, stored, or disposed of as such. The first step in the design process is to properly categorize the surface waters that drain across the facility as either uncontaminated, potentially contaminated, or contaminated. These classifications are described as follows:

- *Uncontaminated waters*: run-off from areas not used for treatment, storage, or disposal of hazardous wastes and from closed sites

Exhibit 6-1. Information Commonly Used in Demonstrating That Performance Standards for Run-On/Run-Off Controls are Met

Information	Typical Parameters
Description of Run-on Control System	Description of: • System used to prevent run-on • Design of run-on control system • Include plan view, drawing details, profiles, cross sections, and calculations used to size system
Calculation of Peak Run-on Flow	Calculate peak surface water flow expected from design storm. Include: • Data sources • Methods used to calculate peak flow
Description of Run-off Control System	Description of: • System used to collect and control run-off from active portions • Design of system • Include plan view, drawing details, cross sections, and calculations demonstrating that system has sufficient capacity to collect total run-off volume • Procedures for determination of whether run-off is hazardous waste
Calculation of Peak Run-off Flow	Calculate total run-off volume expected from the design storm. Include: • Data sources • Methods used to calculate peak run-off flow
Management of Collection and Holding Units	Description of: • Procedures for emptying collection and holding units to maintain capacity • Fate and management of discharged liquids
Description of Construction and Maintenance	Provide: • Detailed construction and material specifications • Construction quality control program • Description of required maintenance activities

- *Potentially contaminated waters*: run-off from all on-site roadways over which hazardous waste materials are transported. This classification could also include processing areas, operations areas, partially closed cells, and all equipment storage/parking areas. Potentially contaminated run-off must be collected in a hazardous waste surface impoundment or tank that meets the minimum technical requirements of the RCRA regulations. The collected run-off must be tested. If it is not contaminated, it can be discharged to the storm water drainage system.

- *Contaminated waters*: run-off collection from active landfill cells; sumps receiving or containing materials resulting from the generation of hazardous wastes; and leachate collected from the secure cell leachate collection system. Contaminated run-off must be collected in a hazardous waste surface impoundment or tank that meets the minimum technology requirements of the RCRA regulations. Within an active landfill cell, the contaminated run-off should be channeled into the leachate collection system. Run-off within the cell must be treated as a hazardous waste until an intermediate cover is placed over the cell.

The following sections provide an introduction to the basic concepts involved in the design of run-on and run-off controls, with guidance to sources providing more detailed information.

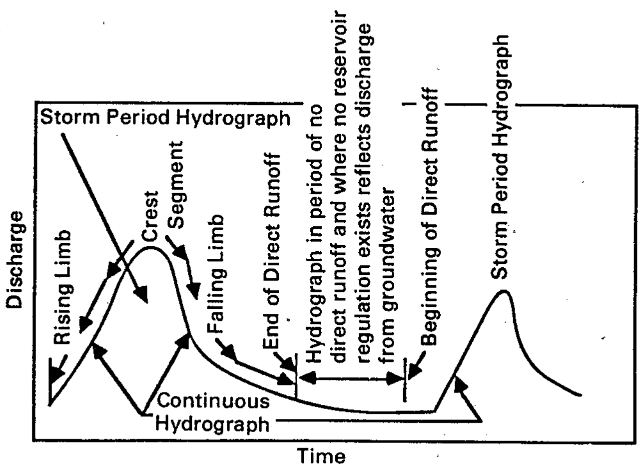
6.2.1 Design Overview

The two methods commonly recommended by EPA for use in designing surface water management structures are the Soil Conservation Service (SCS) method and the Rational method. Both will be introduced here. Before discussing these methods, a brief review of basic hydrologic concepts is helpful.

In surface water management, the study of rainfall effects is limited to the natural drainage basin within which the facility lies. This basin is referred to as the local watershed. The designer must understand the watershed's responses to precipitation, such as how much of the total volume of rainfall will infiltrate the ground surface, how much will evaporate, and how much rainfall will flow across the earth's surface as run-off. Site characteristics that influence rainfall infiltration rates include soil type, soil moisture, antecedent rainfall, cover type, impervious surfaces, and surface retention. Travel time for run-off from the most distant boundary of the watershed to the point of outfall must be determined. Calculations for travel time are based on slope, length of flow path, depth of flow, and roughness of flow surfaces. The relationship of these parameters, along with the total drainage area of the watershed, determines peak run-off discharges and the total drainage area of the watershed. Other factors which impact final run-off discharge calculations include the effect of any flood control works or other natural or manmade storage, and the time distribution of rainfall during a storm. (Reference 1, p.1-1). Chapters 1 and 2 of Reference 1 provide a good overview of surface hydrology and the factors that impact run-off calculations.

Hydrologists graph the run-off accumulation, with respect to time, in a hydrograph. Exhibit 6-2 provides an example of a hydrograph. For a complete discussion of the development and use of hydrographs, including elementary precipitation - streamflow relationships, see Reference 2, Chapter 4.

Exhibit 6-2. Hydrograph



Source: Viessman et al, 1977 p. 102

6.2.2 Design Approach

The standard design approach for a surface water control system is to:

- Identify the intensity of the design storm;
- Determine the peak discharge rate;
- Calculate the run-off volume during peak discharge;
- Determine the control system design criteria and the required capacity for the control systems; and
- Design the control system.

Each step will be discussed briefly in the following sections.

6.2.2.1 Identify Design Storm

The regulations require the owner/operator to use data from a design storm of 24-hour, 25-year frequency for land disposal units, waste piles, and land treatment facilities. Surface impoundment freeboard must be able to contain the rainfall accumulation from a 24-hour, 100-yr storm.* Designers should obtain the design storm information from local planning agencies, civil works departments, or zoning boards for use in designing housing developments and roadway drainage systems. Actual precipitation data for at least the past 25 years, if available, should be reviewed to verify the intensity curves developed.

Permit reviewers do not need to independently gather local precipitation data for their initial review. The Precipitation-Frequency Atlas of the Western United States, the NOAA Atlas 2, is generally regarded as a good reference to verify the acceptable range of values for local data. If the local data used by the designer does not fall within acceptable ranges, then additional research by the permit reviewer into local data is necessary. For a complete list of the most current 24-hour rainfall data published by the National Weather Service (NWS), refer to Reference 3, p. B-3.

6.2.2.2 Determining Peak Discharge Rate/Calculating Run-off: SCS Method

A method that is most often appropriate for estimating run-on/run-off and peak discharge rate from a storm's rainfall is the Soil Conservation Service (SCS) Method. The SCS Method was originally designed to determine run-off volumes for small agricultural watersheds where insufficient long-term stream flow and precipitation data had been collected, but where soil types, topography, vegetative cover, and agricultural practices had been documented.

The model assumes that the rate and amount of rainfall is uniform throughout the watershed over a specified period of time. The calculated mass rainfall is converted to mass run-off by using a run-off Curve Number (CN). CNs were developed to account for the effects of soils, plant cover, amount of impervious areas, interception, and surface storage. Run-off is plotted on a hydrograph and routing procedures that depend on run-off travel time through segments of the watershed. For a further description of the development and use of the SCS Method, refer to Reference 1.

6.2.2.3 Rational Method

The Rational Method can be applied when determining peak discharge rates for significantly urbanized areas with largely impervious surface covers. The Rational Method is based on the premise that maximum run-off resulting from steady, uniformly intense precipitation will occur when the entire watershed, upstream of the site location, contributes to the discharge. The point in time at which this condition occurs after precipitation begins is called the Time of Concentration (TC). TC is normally estimated from consideration of the hydraulic characteristics of the watershed (Reference 4, p. 4-17).

6.2.3 Control System Structures

To achieve the standards set forth in 40 CFR Part 264, the designer will incorporate several structures, both temporary and permanent, into the system

*The design storm requirement for surface impoundments is not specifically stated in the regulations, but is specified in the preamble to the surface impoundment regulations, which was published in the Federal Register of July 26, 1982, 47 FR 32357.

design. Exhibit 6-3 provides a list of the most frequently used structures.

Exhibit 6-3. Surface Water Diversion and Collection Structures

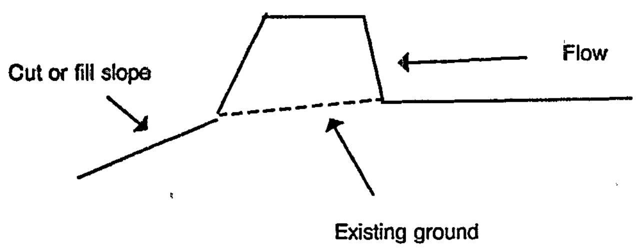
Technology	Duration of Normal Use
Dikes and berms	Temporary
Channels (earthen and CMP)	Temporary
Waterways	Permanent
Terraces and benches	Temporary and permanent
Chutes	Permanent
Downpipes	Temporary
Seepage ditches and basins	Temporary
Sedimentation basins	Temporary

Source: USEPA, Remedial Measures Handbook, 1985, p.3-80.

6.2.3.1 Dikes/Berms

Dikes and berms are well-compacted earthen ridges or ledges constructed immediately upslope from, or along the perimeter, of the intended area of protection. A typical dike design is shown in Exhibit 6-4. Dikes are intended as short term protection of critical areas by intercepting storm run-off and diverting the flow to natural or manmade drainage channels, manmade outlets, or sediment basins. Typically, dikes and berms should be expected to maintain their integrity for about one year, after which they should be rebuilt. Dikes are generally classified into two groups: interceptor dikes, designed to reduce slope length, and diversion dikes, designed to divert surface flow and to reduce slope length. Dikes can also prevent mixing of incompatible wastes and can reduce the amount of leachate produced in a landfill cell by diverting the water available to infiltrate the soil cover. Due to their temporary nature, dikes and berms are designed for run-off from no larger than a five-acre watershed (Reference 5, Chapter 3).

Exhibit 6-4 Typical temporary diversion dike.



Source: USEPA, 1976
Adapted: USEPA, Remedial Measures Handbook, 1985 p. 3-39

A detailed design is not usually required for construction of interceptor and diversion dikes. Common design criteria are found in Reference 5, p. 3-38.

6.2.3.2 Swales, Channels and Waterways

Channels are excavated ditches that are generally wide and shallow with trapezoidal, triangular, or parabolic cross-sections. A typical channel design is shown in Exhibit 6-5. Diversion channels are used primarily to intercept run-off or reduce slope length. Channels stabilized with vegetation or stone rip-rap are used to collect and transfer diverted water off site or to on-site storage or treatment. Applications and limitations of channels and waterways differ depending upon their specific design. Reference 5, Chapter 3 includes a good summary discussion of their applications (Reference 5, pp. 3-40).

Swales are placed along the perimeter of a site to keep off-site run-off from entering the site and to carry surface run-off from a land disposal unit. They are distinguished from earthen channels by side slopes which are less steep and have vegetative cover for erosion control (Reference 5, p. 3-42).

The specific design for channels, swales, and waterways must consider local drainage patterns, soil permeability, annual precipitation, area land use, and other pertinent characteristics of the contributing watershed. To comply with the permit regulations, channels and waterways should accommodate the maximum rainfall expected in a 25-year period. Manning's formula for steady uniform flow in open channels is used to design channels and waterways. Refer to Reference 5, Chapter 3 for a detailed discussion of the application of Manning's formula in channel and waterway design.

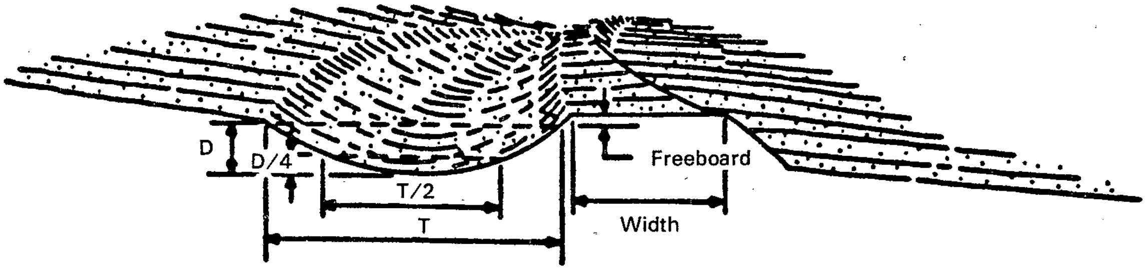
6.2.3.3 Terraces

Terraces are embankments constructed along the contour of very long or very steep slopes to intercept and divert flow of surface water and to control erosion of slopes by reducing slope length. A typical terrace design is shown in Exhibit 6-6. Terraces may function to hydrologically isolate sites, control erosion of cover materials on sites which have been capped, or collect contaminated sediments eroded from disposal areas. For disposal sites undergoing final grading, construction terraces may be included as part of the site closure plan (Reference 5, pp. 3-52). Refer to Reference 5 for a complete discussion of design considerations.

6.2.3.4 Chutes and Downpipes

Chutes and downpipes are usually temporary structures which can play an important role in preventing erosion while landfill and surface impoundment covers are "stabilizing" with vegetation. A typical chute design is shown in Exhibit 6-7. Chutes are excavated earthen channels lined with non-erodible materials such as bituminous concrete or grouted rip-rap. Downpipes are constructed of rigid piping or flexible tubing and installed with

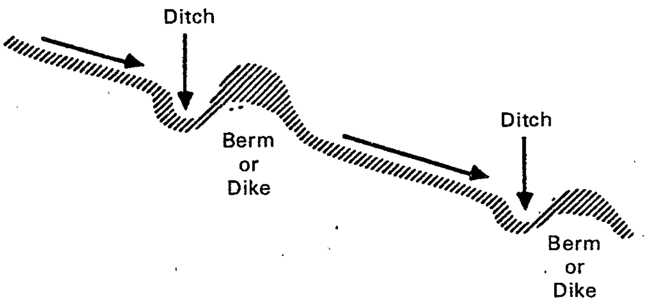
Exhibit 6-5. **Typical Channel Design**



Parabolic cross-section

Source: USEPA, 1976
Reprinted: USEPA, Remedial Measures Handbook, 1985, p. 3-43

Exhibit 6-6. **Typical Terrace Design**



Source: USEPA, 1976
Reprinted: USEPA, Remedial Measures Handbook, 1985, p 3-43

prefabricated inlet sections. As a general rule, chutes should not be used when hydraulic heads are expected to be more than 18 feet. Downpipes should not be used when the drainage basin is estimated to be larger than five acres (Reference 5, Chapter 3).

6.2.3.5 Seepage Basins and Ditches

Seepage basins and ditches are used to discharge water collected from surface water diversions, ground-water pumping or leachate treatment. They may also be used as part of an in-situ treatment process to force treatment reagents into the subsurface. A typical seepage basin design is shown in Exhibit 6-8. They are most effective in highly permeable soils where recharge can occur. They are not applicable at sites where collected run-off or ground water is contaminated. Typically, they are used in areas with shallow ground-water tables. Seepage ditches distribute water over a larger area than achievable with basins. They can be used for all soil where permeability exceeds about 0.9 inches per day (Reference 5, Chapter 3).

A seepage basin typically consists of the actual basin, a sediment trap, a by-pass for excess flow, and an emergency overflow. A considerable amount of recharge occurs through the sidewalls of the basin, and therefore it is preferable that these be constructed of pervious material such as packed gravel. For general design parameters refer to Reference 5, Chapter 3.

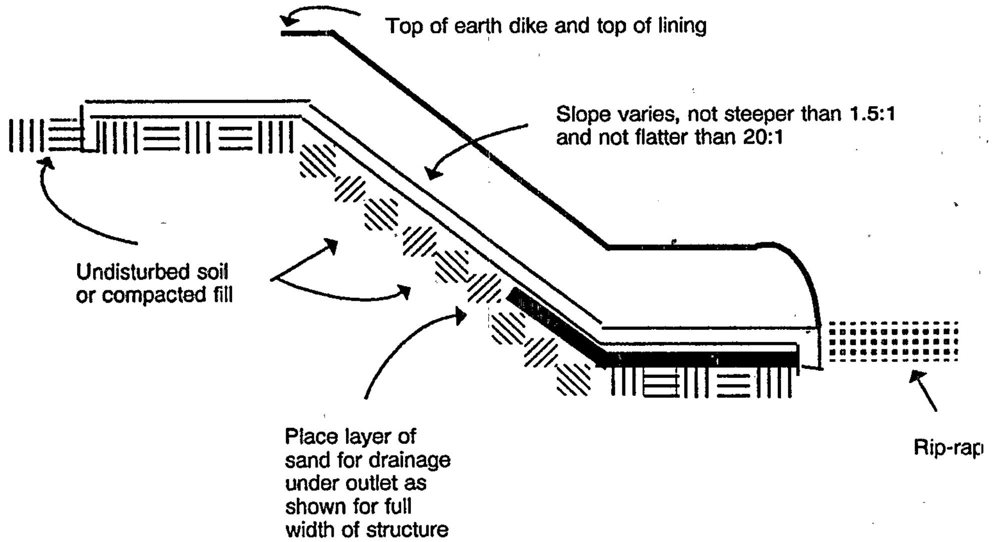
6.2.3.6 Sedimentation Basins

Sedimentation basins are used to retard surface water flow such that suspended particulates can settle. Sedimentation basins serve as the final step in control of diverted, uncontaminated surface run-off, prior to discharge. A typical basin design is shown in Exhibit 6-9. Basins are especially useful in areas where surface run-off has a high silt or sand content. The major components include a principal and emergency spillway, an anti-vortex device and the basin. The principal spillway consists of a vertical pipe or riser joined to a horizontal pipe that extends through the dike and has an outlet beyond the impoundment. The riser is topped by the anti-vortex device and trash rack which improves the flow of water into the spillway and prevents floating debris from being carried out of the basin. For additional design information, refer to Reference 5, Chapter 3.

6.3 Materials

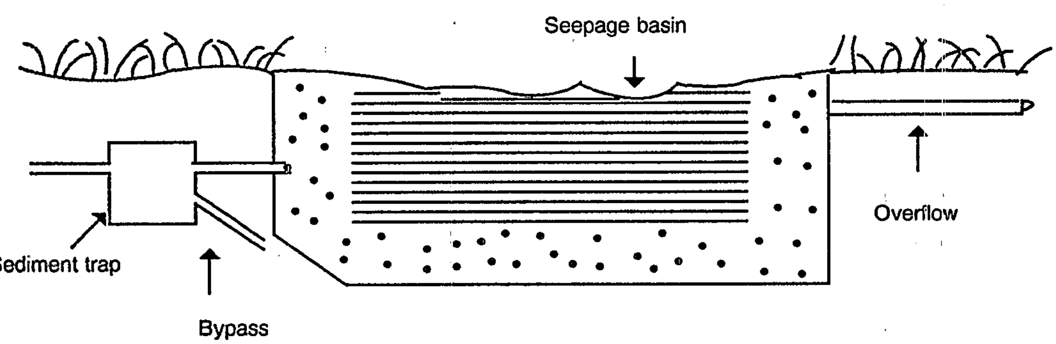
The materials used for construction of surface water management structures are generally local site materials such as sand, gravel and soils. Common techniques used to stabilize the structures include seeding, mulching and the application of soil additives or rip-rap. Other synthetic erosion control materials routinely used include woven jute multifilament fiber matting, polyester fiber matting, and three dimensional polyethylene net. Channels and downpipes may require the use of corrugated metal, concrete pipe or flexible tubing made of heavy fabric. Terraces and chutes are typically lined with

Exhibit 6-7 Typical paved chute design.



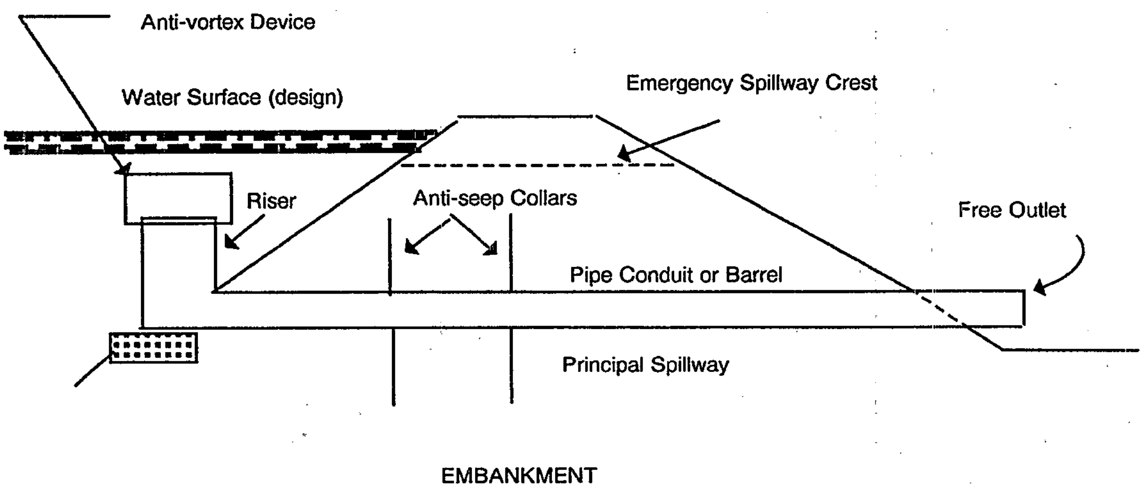
Source: USEPA, 1976
Adapted: USEPA, Remedial Measures Handbook, 1985 p. 3-58

Exhibit 6-8 Typical seepage basin design.



Source: Tourbier and Westmacott, 1974
Adapted: USEPA, Remedial Measures Handbook, 1985 p. 3-65

Exhibit 6-9 Typical sedimentation basin design.



Adapted: USEPA, Remedial Measures Handbook, 1985 p. 3-68 bituminous concrete, concrete, grouted rip-rap or similar non-erodible material. For a complete discussion of available materials see Reference 5, Chapter 3.

6.4 Construction/Equipment

The equipment used for construction of surface water management structures is standard construction excavation and grading equipment including dozers, scrapers, and graders. Well established construction techniques are used. Briefly, prior to construction, the site should be cleared, grubbed, and stripped of topsoil to eliminate trees, vegetation, roots, and other debris. All material should be compacted to prevent unequal settlement. Earthen dikes should be compacted to reduce erosion. For a detailed discussion of equipment and construction techniques for each structure, see Reference 5, Chapter 3.

6.5 Quality Assurance (QA)/Quality Control (QC)

Prior to inspecting any structure during construction, the Quality Control inspector should thoroughly review all design drawings and specifications to ensure the constructability of the design. Factors to consider in the review include: actual site conditions, completeness of design, and consistency between design drawings and specifications. For complete guidance on QA inspections during construction, see Reference 3.

6.5.1 Materials

All materials should be inspected prior to being installed in the surface water management system. The material submittals (catalog cuts) that were submitted for review during the initial construction phases should be checked to confirm that the materials proposed are the same as those used in the actual construction. Specific material details that should be field-verified include pipe sizes, soil gradation and strength, concrete slump, and specified pipe perforations.

6.5.2 Erosion Control Systems

Inspection activities for erosion control systems include visual observations and surveys to ensure that the dimensions of the completed structures are as specified. Slopes are the most important items to check; if slopes are too steep, they may be unstable and eventually could fail. Other items to be checked include berm width, crest width, overall height, thickness and actual dimensions. Vegetative cover should be inspected at regular intervals to ensure that vegetation is properly established (Reference 3, pp. 18 and 19).

6.5.3 Operations/Maintenance QA

During operations, inspections are required periodically and after each storm to ensure that erosion has not damaged the integrity of the berms, dikes, or channels. Inspections are also required to ensure that collection basins and sumps are kept free of standing water to continue to satisfy the capacity requirements cited in the permit.

6.6 References

1. U.S. Department of Agriculture, Soil Conservation Service. 6/1986. Urban Hydrology for Small Watersheds. PB87-101580
2. Viessman, Knapp, et al. *Introduction to Hydrology*, Harper and Row, Publishers, 1977 (Second Edition)
3. U.S. Environmental Protection Agency. 10/1986. Technical Guidance Document: Construction Quality Assurance for Hazardous Waste Land Disposal Facilities, OSWER Policy Directive No. 9472.003. EPA/530-SW-86-031
4. U.S. Environmental Protection Agency. 9/1985. Covers for Uncontrolled Hazardous Waste Sites. EPA/540/2-85/002. Hazardous Waste Engineering Research Laboratory, Cincinnati, Ohio
5. U.S. Environmental Protection Agency. 10/1985. Handbook; Remedial Action at Waste Disposal Sites (revised). EPA/625/6-85/006. Hazardous Waste Engineering Research Laboratory, Cincinnati, Ohio

☆U.S. GOVERNMENT PRINTING OFFICE: 1993 - 750-002/60122

United States
Environmental Protection Agency
Center for Environmental Research Information
Cincinnati, OH 45268

Official Business
Penalty for Private Use
$300

EPA/625/6-88/018

Please make all necessary changes on the below label, detach or copy, and return to the address in the upper left-hand corner.

If you do not wish to receive these reports CHECK HERE ☐; detach, or copy this cover, and return to the address in the upper left-hand corner.

BULK RATE
POSTAGE & FEES PAID
EPA
PERMIT No. G-35