STATE OF NEW MEXICO
BEFORE THE WATER QUALITY CONTROL COMMISSION

In the Matter of: **PROPOSED AMENDMENT TO 20.6.6 NMAC (Dairy Rule)**	)))))))	**No. WQCC 13-08 (R)**

DIRECT TESTIMONY OF I. KEITH GORDON

1.0 Experience and Qualifications

1.1 What is your name?

I. Keith Gordon, P.E.

1.2 Who is you employer?

Gordon Environmental, Inc. (GEI)
President and Principal Engineer

1.3 What is your role with the firm?

I am responsible for managing a staff of 15 engineers, specialized scientists and support professionals in the execution of complex environmental projects.

1.4 Please describe your education and degrees.

BS, Civil Engineering; Geotechnical Specialty
Northwestern University, 1977
(See ***Gordon - 1****, CV)*

1.5 What professional licenses do you hold?

Professional Engineer, NM & 25 other states
Certified Geosynthetics Liner Expert

1.6 What are the standards to achieve Professional Engineering (P.E.) Registration?

- *Four-year Bachelor of Science in Engineering degree, at an accredited University*
- *Five years of applicable experience under the direct supervision of a qualified Professional Engineer*
- *References by five Professional Engineers testifying to the quality of your work and your professional ethics*
- *30 hours (biannual) of Professional Development Hours (PDH) certified to maintain currency of technical competency*

- *Reciprocity in 25 states for P.E. Registration requiring compliance with the PDH standards, in addition to state-specific testing (i.e., seismicity, CA; ethics, MT; etc.)*

1.7 Please describe your experience related to your testimony.

Since 1977, I have been responsible for the design, permitting, and construction assurance (CQA) for soil and geosynthetic liner systems for waste containment facilities.

1.8 Approximately how many liner systems have you designed as the engineer of record?

I have designed, permitted, and supervised the installation of over 2,500 acres of liner systems in 20 states; comprising 200 projects ranging from 1 to 50 ± acres.

1.9 For what types of facilities?

These facilities include:

- *MSW landfills (i.e., Subtitle D composite liners)*
- *MSW landfills with compacted soil liners*
- *Hazardous waste facilities with double liners and leak detection system (i.e., RCRA Subtitle C)*
- *Water impoundments for utilities deploying both compacted soil liners and geomembrane liners*
- *Composite liners for combined nuclear/hazardous wastes*
- *The Las Uvas Valley Dairy (LUVD) impoundments in Hatch, NM*
- *OCD regulated facilities for oil/gas waste management (fluids and solids)*

1.10 What is the extent of your experience as an engineer or consultant on projects requiring ground water discharge permits under the WQCC's Regulations?

GEI has extensive recent experience with discharge permits managed by the Ground Water Protection Bureau (i.e., sludge management, nutrient contamination, voluntary closures, surface water/groundwater interaction, oil field waste management, as well as the Dairy Rule).

1.11 Have you designed any liner systems required by the Dairy Rule, particularly 20.6.6.17(D) NMAC?

GEI prepared a comprehensive demonstration of "liner equivalency" for constructed compacted clay liners for LUVD (DP347) submitted on 03/2012.

1.12 What is the extent of your experience regarding inspections, evaluations, and repairs of impoundment liner systems?

I have extensive experience in performing CQA, inspections, evaluations and repairs for both soil and geosynthetic liner systems. CQA is the single most important element in ensuring design performance of engineered liner systems. I have certified over 500 acres of soil and geosynthetic liner in NM since 1989.

1.13 What is your experience in investigation, corrective action and abatement of contamination resulting from leaking liner systems?

I have been involved in the investigation and corrective action for waste containment liner systems since 1977. These facilities include a multitude of different liner types; and over a dozen CERCLA (Superfund) sites, as well as landfills and impoundments. We are working on several groundwater issues for unlined NM MSW Landfills, and well as routine compliance monitoring for 25 waste management facilities.

2.0 Current Dairy Rule Requirements

2.1 Where in the Rule are the minimum requirements for liner systems required by the Dairy Rule for new surface impoundments used to contain dairy wastewater?

The primary standard for liners for new surface waste impoundments designed to contain dairy wastewater are spelled out in the current 20.6.6.17 NMAC; "Engineering and Surveying Requirements for All Dairy Facilities":

- *A. P.E. Seal*
- *B. Licensed Surveyor*
- *C. Engineering Plans and Specifications*
  - *(1) Design and Construction Plans/Specifications*
  - *(2) Construction Quality Assurance (CQA)*
  - *(3) Wastewater Management*
  - *(4) Grading and Drainage*
- *D. Engineering Design Requirements*
  - ***(5) Impoundment Design and Construction - liner***

2.2 How do you expect that a liner system constructed in accordance with the minimum requirements in the Dairy Rule as specified in 20.6.6.will perform with regard to containment of dairy wastewater?

The prescriptive single geosynthetic liner layer will provide a reduction in vertical flow of fluids. The current prescriptive geomembrane design can be very effective at containing impoundment fluids; but a quantitative flow rate cannot be established without an understanding of potential defects. These systems are potentially susceptible to manufacturing flaws, installation impacts and operating conditions that potentially reduce their functionality and reliability. ***Figure 1 of Gordon-2*** *illustrates the mechanisms for potential flow through permeation, and more importantly defects in a single 60-mil geomembrane when deployed as an independent unit; as opposed to one component of a liner system.*

2.3 What are the causes of leakage or seepage from a synthetic liner system meeting the minimum requirements specified by the Dairy Rule?

The performance of geomembrane liners, i.e., 60-mil HDPE is highly reliant upon the subgrade upon which they are placed, and direct contact with the prepared subgrade. Construction Quality Assurance in the field, and pre-qualification of the subgrade soils, *are essential in the performance of the liner system. HDPE liners may also be subject to long-term degradation via ultraviolet (UV) light (i.e., sunlight).*

2.4 What is the range of seepage rates expected for a synthetic liner system meeting the minimum requirements specified by the Dairy Rule?

*As shown on **Figure 1 of Gordon–2**, there is nominal seepage through the prescriptive liner without any flaws. The fluid head (i.e., water depth) is directly proportional to the rate of seepage. In my 37 years of experience with geosynthetic line design and CQA; when leak testing was conducted; in no case were no deficiencies identified.*

*As shown on **Figure 1 of Gordon–2**, the potential leakage rates for a single 60-mil HDPE geomembrane liner can range from 10 to 40,000 gallons/acre/day. It is extremely important that qualified CQA be performed during liner installation; that the materials delivered meet MQA standards; and that the geomembrane be installed in **direct contact** with a well-prepared subgrade.*

2.5 What information do you rely on for your response to the question above?

***Gordon–3** provides 2 relevant pages from "Waste Containment Systems, Waste Stabilization, and Landfill Design and evaluation" that tabulate leakage rates for various liner systems. These finding are consistent with other published finding (**Gordon-5**, Bibliography) and my professional experience.*

3.0 DIGCE's Proposed Amendment

3.1 What liner provisions of the existing Dairy Rule would be amended if the Commission adopts DIGCE's proposed amendments?

The proposed amendments would pertain to 20.6.6.17; Engineering and Surveying Requirements for all Dairy Facilities § D. Engineering design requirements:

(5) Impoundment design and construction – liner
(6) Impoundment liner – wastewater or wastewater/stormwater combination
(7) Impoundment liner – stormwater

The amended language reads as follows:

(5) Impoundment design and construction – ~~synthetic~~ liner. An applicant or permittee proposing or required to construct a new or to improve an existing impoundment liner, shall, at a minimum, use a 2' thick compacted soil liner with a maximum demonstrated permeability of 1 x 10^{-7} cm/sec or other materials having equivalent performance characteristics with regard to permeability, resistance to degradation by ultraviolet light compatibility with the liquids anticipated to be collected in the impoundment, tensile strength, and tear and puncture resistance.

Synthetic impoundment liners shall include a liner component that is at least 60-mil HDPE and meet the following additional design and construction requirements.

(a) The liner shall be installed with sufficient slack in the liner material to accommodate shrinkage due to temperature changes. Folds in the liner material shall not be present in the completed liner.

(b) The sub-grade shall be free of sharp rocks, vegetation and stubble to a depth of at least six inches below the liner. The surface in contact with the liner shall be smooth to allow for good contact between liner and sub-grade. The surface shall be dry during liner installation. The liner installer shall provide the owner with a sub-grade acceptance certificate prior to installing the liner indicating acceptance of the earthwork.

(c) The liner shall be anchored in an anchor trench. The trench shall be a minimum of 12 inches wide, 12 inches deep and shall be set back at least 24 inches from the top inside edge of the impoundment.

(d) The liner panels shall be oriented such that all sidewall seams are vertical.

(e) If practicable, decomposing organic materials shall be removed from areas over which a liner will be installed. If such materials remain, a liner vent system shall be installed.

(f) Any opening in the liner through which a pipe or other fixture protrudes shall be sealed in accordance with the liner manufacturer's requirements. Liner penetrations shall be detailed in the construction plans and record drawings.

(g) The liner shall be installed by, or the installation supervised by, an individual that has the necessary training and experience as required by the liner manufacturer.

(h) Manufacturer's installation and field seaming guidelines shall be followed.

(i) Liner seams shall be field tested by the installer and verification of the adequacy of the seams shall be submitted to department along with the record drawings.

(j) Concrete slabs installed on top of a liner for operational purposes shall be completed in accordance with manufacturer and installer recommendations to ensure liner integrity.

(6) Impoundment liner – wastewater or wastewater/stormwater combination. An applicant or permittee proposing or required to construct a new or to improve an existing wastewater or combination wastewater/stormwater impoundment. Shall, at a minimum use a ~~single liner that is at least 60 mil HDPE~~ liner that meets the requirements of paragraph (5) of this subsection. ~~or other materials having equivalent characteristics with regard to permeability, resistance to degradation by ultraviolet light, compatibility with the liquids anticipated to be collected in the impoundment, tensile strength, and tear and puncture resistance.~~

(7) **Impoundment liner – stormwater.** Any applicant or permittee required to improve an existing stormwater impoundment pursuant to Subsection A~~B~~ of 20.6.6.27 NMAC shall, at a minimum, use a liner that ~~is at least 60 mil HDPE or other material having equivalent characteristics with regard to permeability, resistance to degradation by ultraviolet light, compatibility with the liquids anticipated to be collected in the impoundments, tensile strength, and tear and puncture resistance.~~ meets the requirements in paragraph (5) of this subsection.

The rationale for amending paragraphs (6) and (7) is straightforward. It is reasonable to assume that a wastewater impoundment would have as high or higher levels of constituents to be contained than stormwater. It also follows that a wastewater/stormwater mixture would be more dilute than the wastewater impoundment alone. Therefore, the engineered containment design prescribed in Section (5) should be sufficient for all three permutations. All engineered liner designs are predicated on an understanding of the materials to be contained and the environmental setting.

The proposed amendments to paragraph (5) are focused on providing the design engineer with flexibility to prescribe the most appropriate liner that meets the applicable "performance standards". The liner design must necessarily address the range of waste types, material compatibility, constructability, sustainability, costs, etc.

The current Regulations impose a prescriptive "Design Standard" (i.e., 60 mil HDPE geomembrane) vs. a more appropriate and proven "Performance Standard". The design standard prescribed does not establish an acceptable flow rate, and prohibits the design engineer from deploying more suitable technologies that would improve on that flow rate. As shown on ***Figure 1 of Gordon–2****, the unpredictable number of defects in HDPE makes it impossible to determine the numerical "performance standard"; which could be equated to gal/acre/day.*

On the other hand, the flow rate for a compacted soil liner, as shown on ***Figure 2 of Gordon–2*** *is strictly a function of the permeability and thickness. For the design proposed in amended paragraph (5), the performance standard is 400 gal/arce/day. The CSL meets all of the structural standards of the current Regulations (i.e., UV resistance, Compatibility, Tear/Puncture Resistance); and offers additional advantages of the geomembrane option (****Figure 2 of Gordon–2****).*

For the reasons stated above, the proposed CSL would be a better choice for the prescriptive liner that the FML. It would provide the design engineer the flexibility to meet a numerical "performance standard" using the most appropriate materials and construction methods based on a demonstration of equivalency. In summary, the CSL provides a demonstrated and predictable level of performance against which to compare other liner design options for "equivalency".

3.2 What type or types of liner systems would be allowed under DIGCE's proposed amendment to 20.6.6.17(D)(5) NMAC?

The proposed amendments add compacted soil liners (CSL's) as an approved alternative to the prescriptive HDPE geomembrane liner for both wastewater and stormwater management systems.

3.3 As an engineer, within the last ten years have you recommended or designed a compacted soil liner system of the nature described in DIGCE's proposed amendments? If so, please describe the application for which the liner system was intended and the liner system design.

I have prescribed a compacted soil layer system for Las Uvas Valley Dairies (LUVD) wastewater management impoundments. The liner system design includes a two-foot thick compacted soil subject to laboratory pre-qualification and rigid field CQA. Other comparable existing soil-lined impoundments at the facility are functioning in accordance with applicable regulatory and performance standards. I have also recommended CSL as a secondary liner for several municipal solid waste (MSW) landfills to meet NMED Solid Waste Bureau (SWB) Rules.

3.4 What materials could be used to construct a compacted soil liner as described in DIGCE's proposed amendments?

The liner materials would be pre-qualified fine-grained soils meeting the standards enumerated on ***Figure 2 of Gordon–2****, and further described in* ***Gordon–4****.*

3.5 How do those materials compare to HDPE liner materials with regard to permeability, resistance to degradation by ultraviolet light, compatibility with the liquids anticipated to be collected in the impoundment, tensile strength, and tear and puncture resistance?

Proposed Section 20.6.6.17.D.(5) NMAC provides a set of technical parameters to establish "equivalency" to a single 60-mil HDPE summarized as follows:

- ***Permeability:*** *the compacted soil liner (CSL) can have a permeability (i.e., leakage rate) at least as favorable as the prescriptive FML.*
- ***Resistance to degradation by ultraviolet light:*** *the HDPE is susceptible to UV light degradation; the CSL is not. Potential desiccation of the CSL will not occur below the water line; and can be minimized/repaired in the freeboard zone.*
- ***Compatibility with the liquids anticipated to be collected in the impoundment:*** *due to the relatively inert nature of the liquid manure waste, neither the FML nor the CSL has compatibility issues with the fluids contained. Incompatibility concerns for both materials are typically associated with organic solvents. In fact, the quality of the CSL is enhanced as the manure solids plug the surface pores* ***(Gordon-3)****.*
- ***Tensile strength, and tear and puncture resistance:*** *compacted soil is obviously superior to HDPE in tensile strength, as well as tear and puncture resistance, as it is a plastic medium that is self-healing.*

In summary, the CSL is at least equivalent with regard to permeability, UV light impacts, liquids compatibility, and structural integrity.

3.6 What does permeability mean?

Permeability *is an intrinsic property of materials (e.g., soils) to promote or restrict the rate of fluid flow.*

Hydraulic conductivity*, symbolically represented as K, is a property that describes the ease with which a fluid (usually water) can move through pore spaces or fractures. Saturated hydraulic conductivity, K_{sat}, describes* ***water*** *movement through saturated media (i.e., soil liners). This is the common reference to "permeability", which can be translated to flow velocity; and flow rate (e.g., 400 gal/acre/day).*

3.7 Why does DIGCE's proposed amendment specify a maximum demonstrated permeability of 1 X 10^{-7} cm/sec?

Permeability in this context equates to hydraulic conductivity (K_{sat}) governing the rate of fluid flow through a saturated soil. A value of 1×10^{-7} cm/sec is an intrinsic property of fine-grained soils (e.g., clays) that are highly impermeable. $1 \times 10^{-7} = 0.0000001$ cm/sec, which is equivalent to .002 miles/hour.

3.8 What does such a demonstrated permeability mean in terms of the maximum seepage that could exit the bottom of a compacted soil liner system meeting that permeability requirement?

*The rate of seepage is typically measured by engineers in gallons per acre per day (gpd). For water, using 1 x 10^{-7} cm/sec; the resultant seepage rate is 400 gpd for a CSL; and considerably less for fluids that contain solids (e.g., manure wastes). A reduction of one order of magnitude (i.e., to 1 x 10^{-8}) is reasonable based on focused research (**Gordon-5**); essentially slowing flow from 400 gpd to 40 gpd (i.e., 10% of the proposed performance standard).*

3.9 How would a permit applicant demonstrate that a proposed compacted soil liner system would meet the maximum permeability of 1 X 10^{-7} cm/sec?

The most common and accepted method to measure permeability (i.e., K_{sat}) is to subject representative samples of the pre-qualified soils to laboratory testing. Essentially, soils from the borrow source are collected for testing in the laboratory, where they are compacted to meet conditions expected in the field. Water is pressurized to flow through cylinders of the compacted soils in order to predict the flow velocity (i.e., cm/sec or gallons/acre/day).

3.10 Are there standard methods that an engineer would use to make such a demonstration? If so, what are those methods and where can they be found?

The Engineer would typically prescribe the following series of standard ASTM laboratory tests on a pre-established number of samples enumerated in the Construction Quality Assurance (CQA) Plan:

Table 1
DIGCE Proposed Dairy Rule Amendments
Compacted Soil Liner - Testing Specifications

Test Type	*Testing Frequency Density*	*ASTM No.*
1. SOILS PREQUALIFICATION		
1.1 Grain Size Distribution	*1/1000 cy*	*D422*
1.2 Plasticity - Atterberg Limits	*1/5000 cy*	*D4318*
1.3 Compaction – Standard Proctor	*1/5000 cy*	*D698*
1.4 Moisture Content	*1/5000 cy*	*D2216; or D4643*
1.5 Hydraulic Conductivity (Permeability)	*1/5000 cy*	*D5084*
2. CONSTRUCTION QUALITY ASSURANCE (CQA)		
2.1 Field Moisture/Density - Nuclear Test Method	*4/acre/lift*	*D6938*

Notes:

- *cy = cubic yard*
- *min. = 1 test per project*
- *Most recent ASTM updates should be applied*
- *Accepted industry standards alternatives may be approved upon technical demonstrations to NMED*

3.11 Why is a two-foot thick compacted soil liner required by DIGCE's proposed amendment?

A two-foot thickness of compacted soil at 1×10^{-7} cm/sec is the industry standard for waste containment applications. USEPA Subtitle D establishes this design as the prescriptive secondary liner for MSW landfills, and there is a wealth of data supporting its performance. The performance of the compacted soil liner has been studied and documented for decades ***(Gordon-3 and Gordon-4)****, and the results are highly predictable. CSL deployment is specified in countless regulatory settings, including NMED's Solid and Hazardous Waste Bureau, OCD Regulations, USEPA Hazardous Waste standards, etc. The 2' thickness, in conjunction with the permeability, establishes the performance standard (i.e., 400 gal/acre/day).*

3.12 What information supports the two-foot thick specification?

The 24" (60 cm) CSL design is placed and compacted in 3 to 4 bonded layers. There are numerous references ***(Gordon-5)*** *that document CSL performance standards and USDA which states:*

> *"for lagoons that are sealed with clay or materials liners and that comply with maximum leakage requirements, groundwater impacts are minimized"*

Ref. No. 17 is a document specifically prepared by USDA to assist Dairy operators in the deployment of CSL technology nationwide (US Department of Agriculture. Agricultural Waste Management Field Handbook. Appendix 10D Design and Construction Guidelines for Impoundments Lined with Clay or Amendment-Treated Soil. Washington: 2008). ***Figure 3 of Gordon–2*** *illustrates how the 2-ft thick CSL would perform in a typical dairy application; with the solids reducing the K_{sat} by at least one order of magnitude (i.e., to 1×10^{-8} cm/sec).*

3.13 Why would a dairy operator proposed to use a compacted soil liner rather than a synthetic liner?

The use of a CSL system is attractive to Dairy operators from a cost standpoint, particularly if there are suitable materials on-site or nearby. In addition, the CSL:

- *can be constructed using locally available material (i.e., sustainability)*
- *can be readily repaired by regrading or adding material*
- *performance is enhanced over time by the settling of solids*
- *not subject to ultraviolet degradation*
- *does not require third-party liner contractors for installation and repair*
- *proper installation can be confirmed by standard laboratory and field testing techniques*
- *potential susceptibility to desiccation occurs only in the freeboard zone, and can be addressed by engineering specifications (i.e., compaction dry of optimum) and routine maintenance*

*These and other advantages are summarized on **Figure 4 of Gordon-2**.*

3.14 What are the costs of a compacted soil liner meeting the specifications in DIGCE's proposed amendments?

Assuming that suitable soils are available on-site or nearby, the liner costs for a typical 10-acre installation are approximately $12,000 - $20,000 per acre.

3.15 How do those costs compare to a synthetic liner system meeting the specifications of 20.6.6.17(D) NMAC?

The costs for a typical single 60-mil HDPE installation (10-acre impoundment) are approximately $25,000 - $50,000 per acre. Applying the assumptions identified herein, there is no resultant increase in performance for the additional costs.

3.16 What types of construction plans and specifications and supporting design calculations and other work products requiring the practice of engineering typically would be required for a proposed compacted soil liner under 20.6.6.17(A) and (C)(1) NMAC?

There is an extensive set of engineering documents required for the proper construction and documentation of a CSL, whose protocol are well established:

- *Construction Plans (i.e., engineering drawings)*
- *Technical Specifications*
- *Construction Quality Assurance (CQA) Plan*
- *Engineering Certification Report*

*Section 3.10 provides the proven test methods and frequencies for CSL's, and **Gordon -4** provides a comprehensive summary of proven CQA/CQC methods for documenting CSL liners. CSLs have been deployed for decades for waste containment applications, with a wealth of field and laboratory data demonstrating their efficacy under a range of conditions.*

3.17 Would those have to bear the seal and signature of a licensed New Mexico engineer?

All of these documents, most importantly the "CQA Plan" and "Engineering Certification Report" of completed construction, are required to be sealed by a NM Professional Engineer qualified in geotechnical engineering and waste containment facility design/CQA.

3.18 What does an engineer's seal and signature on plans, specifications and supporting design calculations signify?

The certifying Engineer, by sealing the documents, affirms that he/she:

- *Is registered in good standing with the NM State Board of Licensure for Professional Engineers and Professional Surveyors*
- *Directly prepared or supervised the Plans and Specifications*
- *Reviewed the laboratory and field testing results to confirm compliance with the Plans, Specifications, Regulations, prevailing industry standards, etc. (see **Figure***

5 of Gordon-2) in order to prepare and submit the "Engineering Certification Report".

3.19 Would the impoundment CQA/CQC requirements in 20.6.6.17(C)(2) apply to a proposed compacted soil liner system under DIGCE's amendments?

The applicable standards of 20.6.6.17.(C)(2) for geomembrane impoundments CQA/CQC are not proposed for amendment and would apply comparably to CSL's as appropriate as well as FML's (e.g., subgrade preparation).

3.20 Would a CQA/CQC plan for a compacted soil liner differ from such a plan for a synthetic liner and, if so, how?

The CQA/CQC Plans for both CSL and FML systems are comparable in many ways, and should be prepared by qualified Professional Engineers. The materials specifications are subject to different ASTM standards, as well as field and laboratory installation techniques.

3.21 Does the U.S. Environmental Protection Agency specify or recommend any particular liner systems for dairy wastewater impoundments?

USEPA establishes water quality protection requirements and guidelines for dairies under its "Confined Animal Feedlot Operations" (CAFO) standards (i.e., 40 CFR, Parts 122 & 412). No prescriptive liner system designs are specified in EPA's CAFO Rule. EPA Region 6 has issued a General Permit No. NMG010000 for CAFO's in New Mexico. That permit also does not prescribe a liner system design, but Condition III.D.1.b of the permit refers to NRCS guidance and states that if a site-specific assessment has not been performed by a NRCS or Professional Engineer, "the liner shall be constructed to have hydraulic conductivities no greater than 1X10 (-7) cm/sec, with a thickness of 1.5 feet or greater or its equivalency in other materials. The liner requirements proposed in DIGCE's amendments have the same permeability (hydraulic conductivity) criteria and is more robust in requiring two feet, rather than 1.5 feet of liner thickness.

3.22 How do the liner systems specified in DIGCE's proposed amendments compare to liner systems specified for other facilities in New Mexico? *The NMED SWB Rules provide for the use of both HDPE and CSL's as liner layers in composite liner systems, as do the NMED Hazardous Waste Bureau and NM Oil Conservation Division (OCD). Single FML liners (or stand-alone CSL's) are not identified as prescriptive designs; although there are equivalency demonstration opportunities.*

3.23 What facts and circumstances regarding dairy impoundments warrant different approaches to liner systems compared to those required for other types of facilities?

The liner designs prescribed for the range of regulatory environments all have the goal of environmental protection; and more specifically stewardship of groundwater and surface water quality. The factors that govern the type of liner prescribed by the Engineer-of-Record are based on site-specific conditions (i.e., depth-to-groundwater, groundwater quality, intervening soil layers, etc.); nature of the waste (i.e., toxicity, solid content, *mobility, etc.) and use of groundwater. The types of contaminants associated with dairy operations (i.e., nitrates) do not rise to the level of concern related to heavy metals (e.g., hexavalent chromium) and organic solvents that may be disposed of at Subtitle C (Hazardous), Subtitle D (MSW) facilities, and OCD regulated oil field waste management facilities.*

3.24 Explain DIGCE's proposed amendments to 20.6.6.17(D)(5) NMAC.

The proposed amendments by DIGCE to §(D)(5) allow for the Applicant to propose for a properly engineered CSL to be used in lieu of the prescriptive 60-mil HDPE geomembrane. DIGCE's proposed amendments to §(D)(7) provide a baseline for technical standards to be applied for an approved compacted soil liner (CSL), including PE certification. The amendments do not propose to nullify the current 20.6.6.17(C)(2) requirements for liner CQA/CQC specific to HDPE's; but instead would prescribe materials-specific CQA standards for CSL's (Section 3.10).

3.25 Explain DIGCE's proposed amendments to 20.6.6.17(D)(6) and (7) NMAC.

DIGCE recommends that the same standards prescribed for wastewater impoundments be applied for liner design/installation for wastewater/stormwater combinations.

I. Keith Gordon P.E.

Professional Resumé (Summary)
I. Keith Gordon, P.E.

Gordon Environmental, Inc.
Consulting Engineers

Responsibilities:
- President, Principal Engineer
  Gordon Environmental, Inc.
- Manage staff of 15 (Bernalillo)
  Environmental Engineers & Scientists

Education
- BS Civil Engineering
  Northwestern University, 1977
  Geotechnical Engineering

Certifications/Memberships/Officer
- Professional Engineer: NM & 25 additional States
- NICET, NCEES, SWANA, ASCE, NMSPE, NSPE, NW&RA

Areas of Expertise
- Environmental and Geotechnical Engineering
- Waste Facility Siting, Design, & Permitting
- Liner Design & QA/QC, Final Covers
- Regulatory Compliance & Rules Assistance, Hearing Testimony

Advisor/Trainer/Publications
- New Mexico Environment Department
- United State Environmental Protection Agency
- National Waste & Recycling Association
- Solid Waste Association of North America
- Various State Regulatory Agencies (IL, IA, CA, MI, MN)

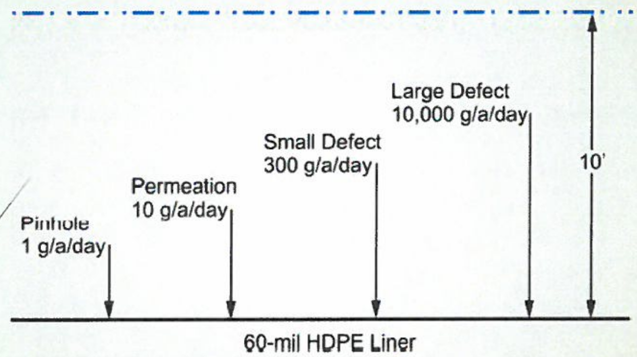
Applicable Experience
- 37 years experience in waste facility design, permitting, & construction
- Extensive liner expertise in soils & geosynthetics design, procurement, & CQA
- Over 2,500 acres of constructed soil & geosynthetics liners in 20 states
- Over 50 waste containment cells in NM encompassing over 500 acres
- Expertise with single soil/geosynthetic liners; composite (i.e., RCRA Subtitle D) systems; & double (i.e., RCRA Subtitle C) configurations, HDPE, PVC, etc.
- NM experience with the new "Part 36" OCD landfill liner standards, WIPP containment designs for salt management, Dairy Rule comments

GEI Clients
- Las Uvas Valley Dairies
- 26 of 33 Counties in NM, plus most of the Municipalities & Solid Waste Agencies
- All of the Private Sector "Subtitle D" Landfills in NM
- 50 other Waste Containment Projects in NM, TX, & 20 other states
- Several CERCLA (i.e., Superfund) projects in 10 states involving the historical use of containment liner designs

SWANA Excellence Awards (Waste Management Facilities)
- 2012: Sandoval County Landfill, Rio Rancho, NM
- 2000: Cerro Colorado Landfill, Albuquerque, NM
- 1998: South Central Transfer/MRF, Las Cruces, NM
- 1997: Camino Real Landfill, Sunland Park, NM
- 1986: Miller Road Landfill, Saginaw, MI

Figure 1

HDPE Liner Leakage Rates



Range: 10 – 40,000 gal/acre/day
Typical: 300 gal/acre/day (with "GOOD" QA/QC)

Controlling Factors:

- ✓ **Quality Assurance**
- ✓ **Site-Specific Conditions**

Figure 2

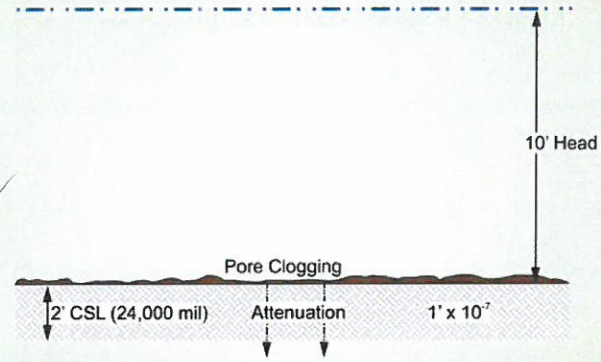
Soil Liners
Materials Classification – Laboratory Testing

- Grain Size (≥ 50 passing 200 sieve)
- Plasticity (PI ≥ 15, LL ≥ 30)
- Proctor Density/optimum moisture
- <u>Permeability</u> (k ≤ 1.0 x 10^{-7} cm/sec)
- One test per source
- ASTM methods or alternatives

NMED Dairy Technical Working Group
June 24, 2014

Figure 3

Compacted Soil Liner (CSL) Leakage Rates



Range: 40 – 400 gal/acre/day
Typical: 100 gal/acre/day

Controlling Factors:
- ✓ **Quality Assurance**
- ✓ **Site-Specific Conditions**

Figure 4

Compacted Soil Liners Advantages

- **Permeability**
- **UV Resistance**
- **Compatibility**
- **Tear/Puncture Resistance**
- Natural Attenuation
- Self-Healing
- Pore-Clogging
- Sustainable (Local Resources)
- Easily Repaired
- Breakthrough

Bold *= Regulatory Criteria*

Controlling Factors:
- ✓ **Quality Assurance**
- ✓ **Site-Specific Conditions**

Figure 5

Soil Liners Field Testing/Installation/ CQA

- Installation conditions (i.e., weather, subgrade)
- Subgrade preparation, scarification
- Lift thickness (i.e., 6" compacted)
- Field density/moisture (4/acre/lift)
- Testing frequency
- Engineering Certification (i.e., 1 x 10^{-7} cm/sec)

NMED Dairy Technical Working Group
June 24, 2014

TABLE 8.1 Calculated Unitized Leakage Rates Through a 40-mil HDPE Geomembrane[a]

	Water Depth Above Geomembrane, h_w				
	0.01 ft	0.1 ft	1 ft	10 ft	100 ft
Permeation (gal/acre/day)	0.00001	0.001	0.1	10	30
Pinhole (gal/acre/day)	0.001	0.01	0.1	1	10
Small hole (gal/acre/day)	10	30	100	300	1,000
Large hole (gal/acre/day)	300	1,000	3,000	10,000	30,000

Source: Giroud and Bonaparte (1989a). Reproduced by permission of Elsevier.

[a]Assumes infinitely permeable material above and below the geomembrane; one pinhole or one hole per acre; small hole has 0.005 in^2 surface area; large hole has 0.16 in^2 surface area.

The number of pinholes occurring within a geomembrane is dependent on quality assurance and quality control (QA/QC) during manufacturing, whereas the number of observable holes is influenced by field QA/QC. Pinholes are generally rare and the flow rates through pinholes in a good-quality geomembrane is minimal compared with the flow rates through holes. Flow through holes generally occurs at the seams or due to defects caused by punctures or tears. Giroud and Bonaparte (1989a) recommend the following assumptions regarding geomembrane defects:

- A frequency of one hole per acre with good QA/QC and a frequency of 10 holes per acre or greater with poor QA/QC
- A large hole, 0.16 in^2 (1 cm^2), for sizing lining system and LCRS components
- A small hole, 0.005 in^2 (3.1 mm^2), for performance calculations such as estimating the flow in the leakage collection layer under typical operating conditions

Table 8.1 summarizes leakage rates for a 40-mil HDPE geomembrane due to permeation, pinholes, and holes. It should be noted that the geomembrane thickness has an influence only on flow due to permeation and pinholes. It is also interesting to note that flow due to permeation and pinholes is several orders of magnitude less than that through large holes and that at heads greater than 1 foot, flow due to permeation is greater than flow through pinholes.

Reference: Sharma, Hari D. and Sangeeta P. Lewis. *Waste Containment Systems, Waste Stabilization, and Landfills Design and Evaluation.* New York, New York: 1994

TABLE 8.3 Comparison of Approximate Leakage Rates Through Various Types of Liners[a] (gallons/acre/day)

Type of Liner	Leakage Mechanism	Liquid Depth Above Liner, h_w				
		0.01 ft	0.1 ft	1 ft	10 ft	100 ft
Clay liner	Flow through porous media	93	96	125	400	3,200
Geomembrane liner	Permeation	0.00001	0.001	0.1	10	30
	Small hole	10	30	100	300	1,000
	Large hole	300	1,000	3,000	10,000	30,000
Composite liner						
Good field conditions	Permeation	0.00001	0.001	0.1	10	30
	Small hole	0.002	0.015	0.01	0.9	8
	Large hole	0.002	0.02	0.15	1	9
Poor field conditions	Permeation	0.00001	0.01	0.1	10	30
	Small hole	0.01	0.08	0.6	5	40
	Large hole	0.01	0.1	0.7	6	50

Source: Giroud and Bonaparte (1989b). Reproduced by permission of Elsevier.

[a] Assumes a 3-ft-thick clay liner with a 1×10^{-7} cm/s coefficient of permeability and a 40-mil HDPE geomembrane. A small hole has an area of 0.005 in^2 and a large hole has an area of 0.16 in^2. One hole is assumed per acre.

Reference: Sharma, Hari D. and Sangeeta P. Lewis. *Waste Containment Systems, Waste Stabilization, and Landfills Design and Evaluation*. New York, New York: 1994

EPA/600/R-93/182
September 1993

Technical Guidance Document:

QUALITY ASSURANCE AND QUALITY CONTROL FOR WASTE CONTAINMENT FACILITIES

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Chapter 2

Compacted Soil Liners

2.1 Introduction and Background

2.1.1 Types of Compacted Soil Liners

Compacted soil liners have been used for many years as engineered hydraulic barriers for waste containment facilities. Some liner and cover systems contain a single compacted soil liner, but others may contain two or more compacted soil liners. Compacted soil liners are frequently used in conjunction with geomembranes to form a *composite liner,* which usually consists of a geomembrane placed directly on the surface of a compacted soil liner. Examples of soil liners used in liner and cover systems are shown in Fig. 2.1.

Compacted soil liners are composed of clayey materials that are placed and compacted in layers called *lifts.* The materials used to construct soil liners include natural mineral materials (natural soils), bentonite-soil blends, and other material

2.1.1.1 Natural Mineral Materials

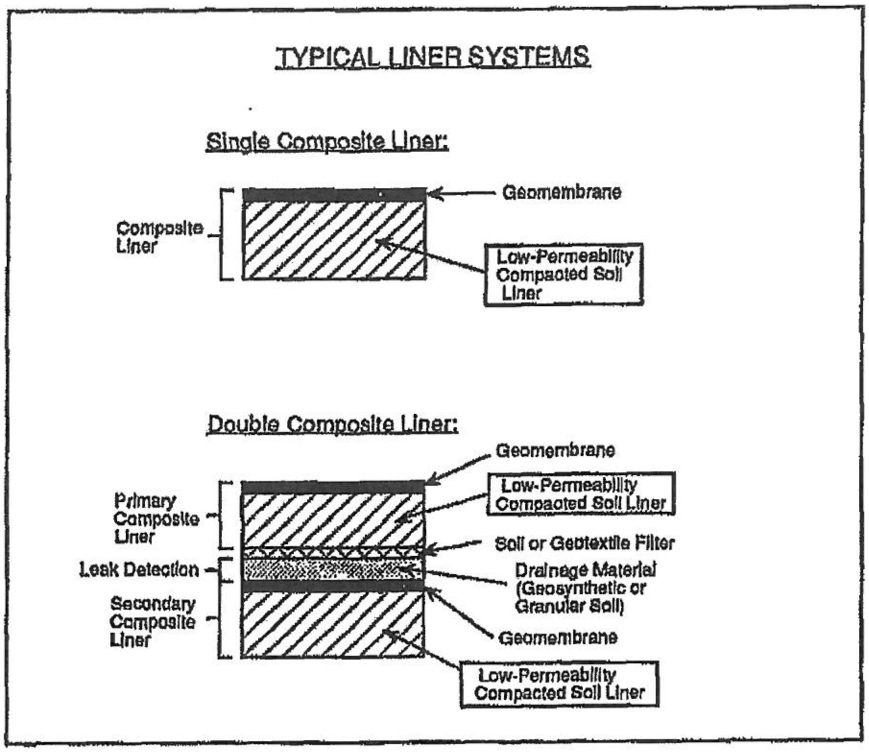
The most common type of compacted soil liner is one that is constructed from naturally occurring soils that contain a significant quantity of clay. Soils are usually classified as CL, CH, or SC soils in the Unified Soil Classification System (USCS) and ASTM D-2487. Soil liner materials are excavated from locations called *borrow pits.* These borrow areas are located either on the site or offsite. The soil in the borrow pit may be used directly without processing or may be processed to alter the water content, break down large pieces of material, or remove oversized particles. Sources of natural soil liner materials include lacustrine deposits, glacial tills, aeolian materials, deltaic deposits, residual soils, and other types of soil deposits. Weakly cemented or highly weathered rocks, e.g., mudstones and shales, can also be used for soil liner materials, provided they are processed properly.

2.1.1.2 Bentonite-Soil Blends

If the soils found in the vicinity of a waste disposal facility are not sufficiently clayey to be suitable for direct use as a soil liner material, a common practice is to blend natural soils available on or near a site with bentonite. The term *bentonite* is used in different ways by different people. For purposes of this discussion, bentonite is any commercially processed material that is composed primarily of the mineral smectite. Bentonite may be supplied in granular or pulverized form. The dominant adsorbed cation of commercial bentonite is usually sodium or calcium, although the sodium form is much more commonly used for soil sealing applications. Bentonite is mixed with native soils either in thin layers or in a pugmill.

2.1.1.3 Other

Other materials have occasionally been used for compacted soil liners. For example, bentonite may be blended with flyash to form a liner under certain circumstances. Modified soil minerals and commercial additives, e.g., polymers, have sometimes been used.



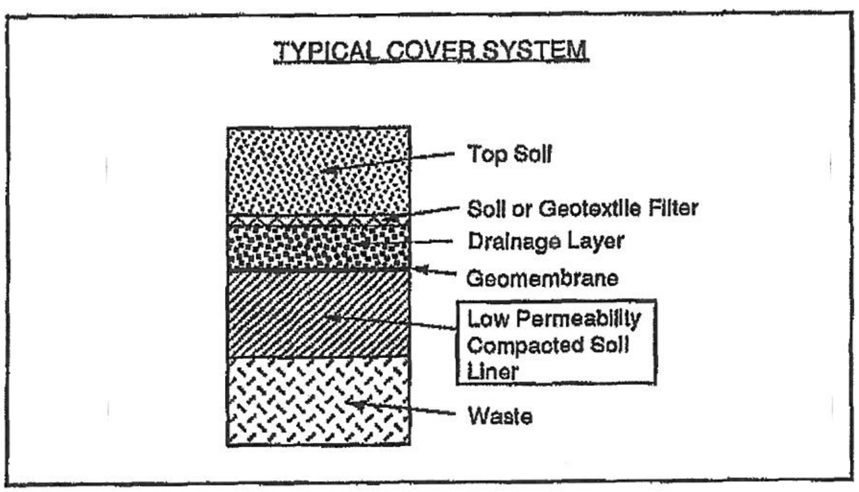


Figure 2.1 - Examples of Compacted Soil Liners in Liner and Cover Systems

2.1.2 Critical CQC and CQA Issues

The CQC and CQA processes for soil liners are intended to accomplish three objectives:

1. Ensure that soil liner materials are suitable.

2. Ensure that soil liner materials are properly placed a. ⌄ compacted.

3. Ensure that the completed liner is properly protected.

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

As discussed in Chapter 1, the objective of CQA is to ensure that the final product meets specifications. A detailed program of tests and observations is necessary to accomplish this objective. The objective of CQC is to control the manufacturing or construction process to meet project specifications. With geosynthetics, the distinction between CQC and CQA is obvious: the geosynthetics installer performs CQC while an independent organization conducts CQA. However, CQC and CQA activities for soils are more closely linked than in geosynthetics installation. For example, on many earthwork projects the CQA inspector will typically determine the water content of the soil and report the value to the contractor; in effect, the CQA inspector is also providing CQC input to the contractor. On some projects, the contractor is required to perform extensive tests as part of the CQC process, and the CQA inspector performs tests to check or confirm the results of CQC tests.

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

2.1.3 Liner Requirements

The construction of soil liners is a challenging task that requires many careful steps. A blunder concerning any one detail of construction can have disastrous impacts upon the hydraulic conductivity of a soil liner. For example, if a liner is allowed to desiccate, cracks might develop that could increase the hydraulic conductivity of the liner to above the specified requirement.

As stated in Section 2.1.2, the CQC and CQA processes for soil liners essentially consist of using suitable materials, placing and compacting the materials properly, and protecting the completed liner. The steps required to fulfill these requirements may be summarized as follows:

1. The subgrade on which the soil liner will be placed should be properly prepared.

2. The materials employed in constructing the soil liner should be suitable and should conform to the plans and specifications for the project.

3. The soil liner material should be preprocessed, if necessary, to adjust the water content, to remove oversized particles, to break down clods of soil, or to add amendments such as bentonite.

4. The soil should be placed in lifts of appropriate thickness and then be properly remolded and compacted.

5. The completed soil liner should be protected from damage caused by desiccation or freezing temperatures.

6. The final surface of the soil liner should be properly prepared to support the next layer that will be placed on top of the soil liner.

The six steps mentioned above are described in more detail in the succeeding subsections to provide the reader with a general introduction to the nature of CQC and CQA for soil liners. Detailed requirements are discussed later.

2.1.3.1 <u>Subgrade Preparation</u>

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

Sometimes it is necessary to "tie in" a new section of soil liner to an old one, e.g., when a landfill is being expanded laterally. It is recommended that a lateral excavation be made about 3 to 6 m (10 to 20 ft) into the existing soil liner, and that the existing liner be stair-stepped as shown in Fig. 2.2 to tie the new liner into the old one. The surface of each of the steps in the old liner should be scarified to maximize bonding between the new and old sections.

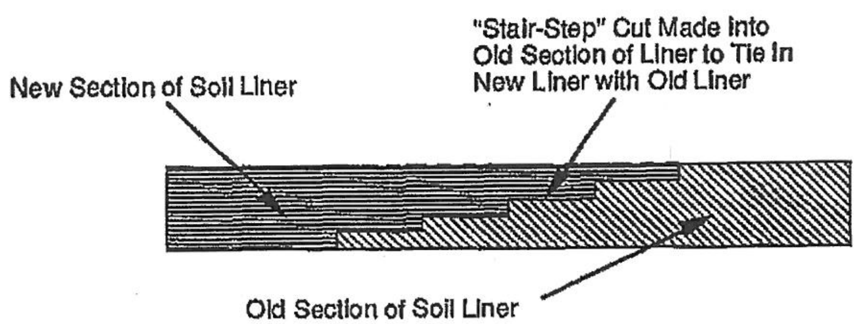


Figure 2.2 - Tie-In of New Soil Liner to Existing Soil Liner

2.1.3.2 Material Selection

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

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

1. A potential borrow source is located and explored to determine the vertical and lateral extent of the source and to obtain representative samples, which are tested for properties such as liquid limit, plastic limit, percent fines, etc.

2. Once construction begins, additional CQC and CQA observations and tests may be performed in the borrow pit to confirm the suitability of materials being removed.

3. After a lift of soil has been placed, additional CQA tests should be performed for final verification of the suitability of the soil liner materials.

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

A variety of tests is performed at various stages of the construction process to ensure that the soil liner material conforms with specifications. However, tests alone will not necessarily ensure an adequate material -- observations by qualified CQA inspectors are essential to confirm that deleterious materials (such as stones or large pieces of organic or other deleterious matter) are not present in the soil liner material.

2.1.3.3 Preprocessing

Some soil liner materials must be processed prior to use. The principal preprocessing steps that may be required include the following:

1. Drying of soil that is too wet.

2. Wetting of soil that is too dry.

3. Removal of oversized particles.

4. Pulverization of clods of soil.

5. Homogenization of nonuniform soil.

6. Addition of bentonite.

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

2.1.3.4 Placement, Remolding, and Compaction

Soil liners are placed and compacted in lifts. The soil liner material must first be placed in a loose lift of appropriate thickness. If a loose lift is too thick, adequate compactive energy may not be delivered to the bottom of a lift.

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

2.1.3.5 Protection

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

2.1.3.6 Final Surface Preparation

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

2.1.4 Compaction Requirements

One of the most important aspects of constructing soil liners that have low hydraulic conductivity is the proper remolding and compaction of the soil. Background information on soil compaction is presented in this subsection.

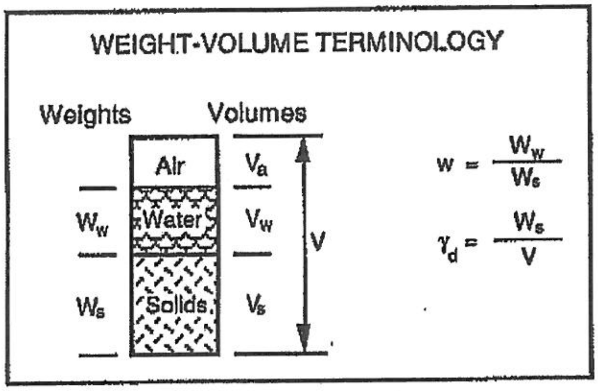
2.1.4.1 Compaction Curve

A compaction curve is developed by preparing several samples of soil at different water contents and then sequentially compacting each of the samples into a mold of known volume with a specified compaction procedure. The total unit weight (γ), which is also called the wet density, of each specimen is determined by weighing the compacted specimen and dividing the total weight by the total volume. The water content (w) of each compacted specimen is determined by oven drying the specimen. The dry unit weight (γ_d), which is sometimes called the dry density, is calculated as follows:

$$\gamma_d = \gamma/(1 + w) \tag{2.1}$$

The (w, γ_d) points are plotted and a smooth curve is drawn between the points to define the compaction curve (Fig. 2.3). Judgment rather than an analytic algorithm is usually employed to draw the compaction curve through the measured points.

The *maximum dry unit weight* ($\gamma_{d,max}$) occurs at a water content that is called the *optimum water content*, w_{opt} (Fig. 2.3). The main reason for developing a compaction curve is to determine the optimum water content and maximum dry unit weight for a given soil and compaction procedure.



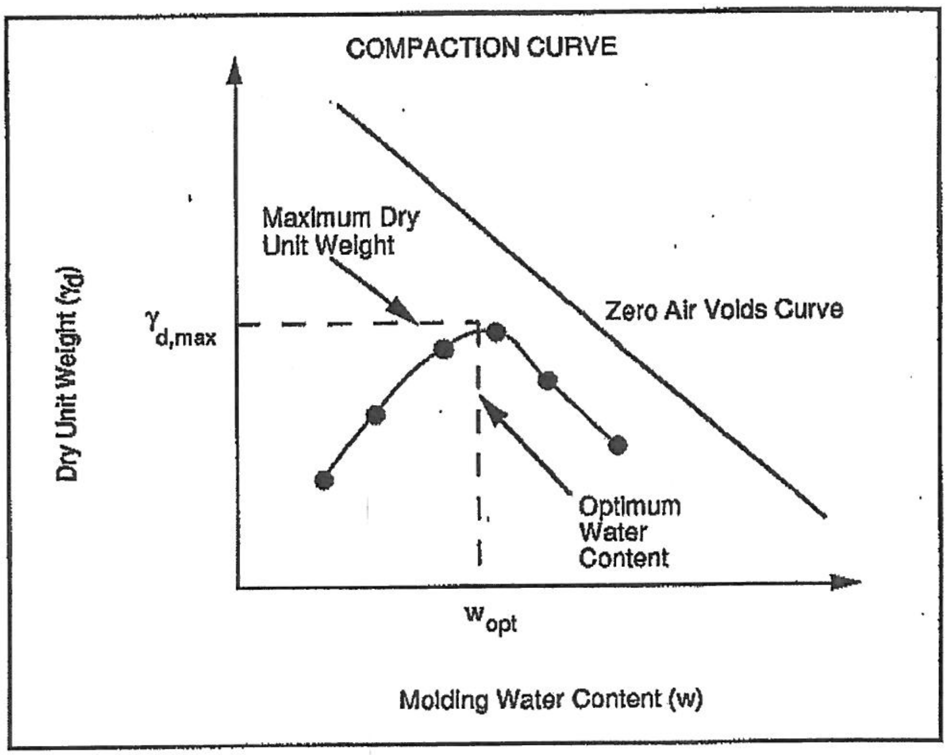


Figure 2.3 - Compaction Curve

The *zero air voids curve* (Fig. 2.3), also known as the *100% saturation curve,* is a curve that relates dry unit weight to water content for a saturated soil that contains no air. The equation for the zero air voids curve is:

$$\gamma_d = \gamma_w/[w + (1/G_s)] \tag{2.2}$$

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

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

2.1.4.2 Compaction Tests

Several methods of laboratory compaction are commonly employed. The two procedures that are most commonly used are standard and modified compaction. Both techniques usually involve compacting the soil into a mold having a volume of 0.00094 m^3 (1/30 ft^3). The number of lifts, weight of hammer, and height of fall are listed in Table 2.1. The compaction tests are sometimes called *Proctor* tests after Proctor, who developed the tests and wrote about the procedures in several 1933 issues of Engineering News Record. Thus, the compaction curves are sometimes called Proctor curves, and the maximum dry unit weight may be termed the *Proctor density*.

Table 2.1 - Compaction Test Details

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

Proctor's original test, now frequently called the *standard Proctor compaction test*, was developed to control compaction of soil bases for highways and airfields. The maximum dry unit weights attained from the standard Proctor compaction test were approximately equal to unit weights observed in the field on well-built fills using compaction equipment available in the 1920s and 1930s. During World War II, much heavier compaction equipment was developed and the unit weights attained from field compaction sometimes exceeded the laboratory values. Proctor's original procedure was modified by increasing compactive energy. By today's standards:

- Standard Compaction (ASTM D-698) produces maximum dry unit weights approximately equal to field dry unit weights for soils that are well compacted using modest-sized compaction equipment.
- Modified Compaction (ASTM D-1557) produces maximum dry unit weights approximately equal to field dry unit weights for soils that are well compacted using the heaviest compaction equipment available.

2.1.4.3 Percent Compaction

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

$$P = \gamma_d / \gamma_{d,max} \times 100\% \qquad (2.3)$$

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

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

2.1.4.4 Estimating Optimum Water Content and Maximum Dry Unit Weight

Many CQA plans require that the water content and dry unit weight of the field-compacted soil be compared to values determined from laboratory compaction tests. Compaction tests are a routine part of nearly all CQA programs. However, from a practical standpoint, performing compaction tests introduces two problems:

1. A compaction test often takes 2 to 4 days to complete -- field personnel cannot wait for the completion of a laboratory compaction test to make "pass-fail" decisions.

2. The soil will inevitably be somewhat variable -- the optimum water content and maximum dry unit weight will vary. The values of w_{opt} and $\gamma_{d,max}$ appropriate for one location may not be appropriate for another location. This has been termed a "mismatch" problem (Noorany, 1990).

Because dozens (sometimes hundreds) of field water content and density tests are performed, it is impractical to perform a laboratory compaction test each and every time a field measurement of water content and density is obtained. Alternatively, simpler techniques for estimating the maximum dry unit weight are almost always employed for rapid field CQA assessments. These techniques are subjective assessment, one-point compaction test, and three-point compaction test.

2.1.4.4.1 Subjective Assessment

Relatively homogeneous fill materials produce similar results when repeated compaction tests are performed on the soil. A common approach is to estimate optimum water content and maximum dry unit weight based on the results of previous compaction tests. The results of at least 2 to 3 laboratory compaction tests should be available from tests on borrow soils prior to actual compaction of any soil liner material for a project. With subjective assessment, CQA personnel estimate the optimum water content and maximum dry unit weight based upon the results of the previously-completed compaction tests and their evaluation of the soil at a particular location in the field. Slight variations in the composition of fill materials will cause only slight variations in w_{opt} and $\gamma_{d,max}$. As an approximate guide, a relatively homogeneous borrow soil would be considered a material in which w_{opt} does not vary by more than $\pm$ 3 percentage points and $\gamma_{d,max}$ does not vary by more than $\pm$ 0.8 kN/ft^3 (5 pcf). The optimum water content and maximum dry unit weight should not be estimated in this manner if the soil is heterogeneous -- too much guess work and opportunity for error would exist.

2.1.4.4.2 One-Point Compaction Test

The results of several complete compaction tests should always be available for a particular borrow source prior to construction, and the data base should expand as a project progresses and additional compaction tests are performed. The idea behind a one-point compaction test is shown in Fig. 2.4. A sample of soil is taken from the field and dried to a water content that appears to be just dry of optimum. An experienced field technician can usually tell without much difficulty when the water content is just dry of optimum. The sample of soil is compacted into a mold of known volume according to the compaction procedure relevant to a particular project, e.g., ASTM D-698 or D-1557. The weight of the compacted specimen is measured and the total unit weight is computed. The sample is dried using one of the rapid methods of measurement discussed later to determine water content. Dry unit weight is computed from Eq. 2.2. The water content-dry unit weight point from the one-point compaction test is plotted as shown in Fig. 2.4 and used in conjunction with available compaction curves to estimate w_{opt} and $\gamma_{d,max}$. One assumes that the shape of the compaction is similar to the previously-developed compaction curves and passes through the one point that has been determined.

The dashed curve in Fig. 2.4 is the estimated compaction curve. The one-point compaction test is commonly used for variable soils. In extreme cases, a one-point compaction test may be required for nearly all field water content and density measurements for purposes of computing percent compaction. However, if the material is so variable to require a one-point compaction test for nearly all field density measurements, the material is probably too variable to be suitable for use in a soil liner. The best use of the one-point compaction test is to assist with estimation of the optimum water content and maximum dry unit weight for questionable materials and to fill in data gaps when results of complete compaction tests are not available quickly enough.

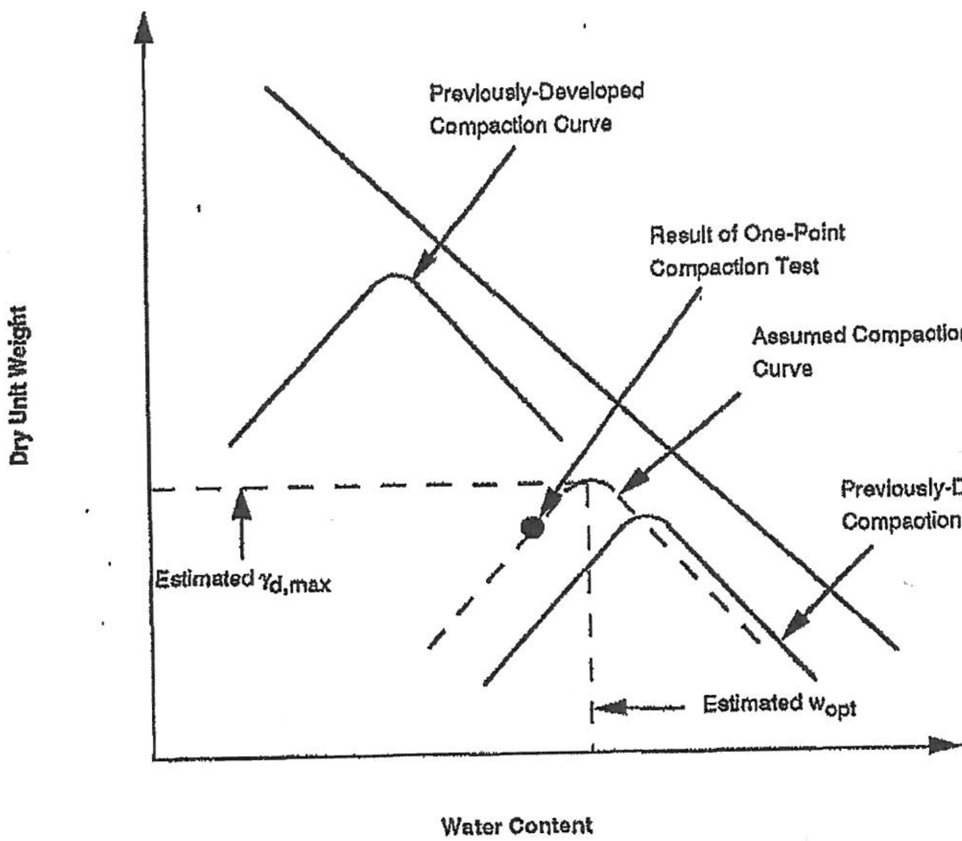


Figure 2.4 - One-Point Compaction Test

2.1.4.4.3 Three-Point Compaction Test (ASTM D-5080)

A more reliable technique than the one-point compaction test for estimating the optimum water content and maximum dry unit weight is to use a minimum of three compaction points to define a curve rather than relying on a single compaction point. A representative sample of soil is obtained from the field at the same location where the in-place water content and dry unit weight have been measured. The first sample of soil is compacted at the field water content. A second sample is prepared at a water content two percentage points wetter than the first sample and is compacted. However, for extremely wet soils that are more than 2% wet of optimum (which is often the case for soil liner materials), the second sample should be dried 2% below natural water content. Depending on the outcome of this compaction test, a third sample is prepared at a water content either two percentage points dry of the first sample or two percentage points wet of the second sample (or, for wet soil liners, 2 percentage points dry of the second sample). A parabola is fitted to the three compaction data points and the optimum water content and maximum dry unit weight are determined from the equation of the best-fit parabola. This technique is significantly more time consuming than the one-point compaction test but offers 1) a standard ASTM procedure and 2) greater reliability and repeatability in estimated w_{opt} and $\gamma_{d,max}$.

2.1.4.5 Recommended Procedure for Developing Water Content-Density Specification

One of the most important aspects of CQC and CQA for soil liners is documentation of the water content and dry unit weight of the soil immediately after compaction. Historically, the method used to specify water content and dry unit weight has been based upon experience with structural fill. Design engineers often require that soil liners be compacted within a specified range of water content and to a minimum dry unit weight. The "Acceptable Zone" shown in Fig. 2.5 represents the zone of acceptable water content/dry unit weight combinations that is often prescribed. The shape of the Acceptable Zone shown in Fig. 2.5 evolved empirically from construction practices applied to roadway bases, structural fills, embankments, and earthen dams. The specification is based primarily upon the need to achieve a minimum dry unit weight for adequate strength and limited compressibility. As discussed by Mundell and Bailey (1985), Boutwell and Hedges (1989), and Daniel and Benson (1990), this method of specifying water content and dry unit weight is not necessarily the best method for compacted soil liners.

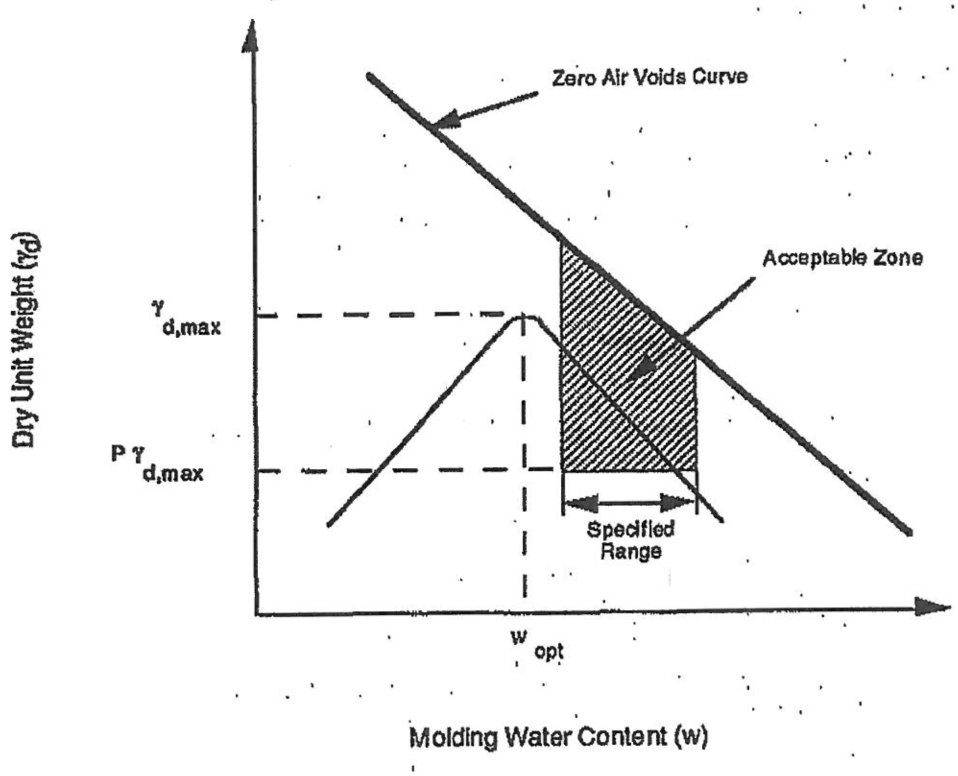


Figure 2.5 - Form of Water Content-Dry Unit Weight Specification Often Used in the Past

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

For most earthwork projects, modified Proctor effort represents a reasonable upper limit on the compactive effort likely to be delivered to the soil in the field. Standard compaction effort (ASTM D-698) likely represents a medium compactive effort. It is conceivable that soil in some locations will be compacted with an effort less than that of standard Proctor compaction. A reasonable lower limit of compactive energy is the "reduced compaction" procedure in which standard compaction procedures (ASTM D-698) are followed except that only 15 drops of the hammer per lift are used instead of the usual 25 drops. The reduced compaction procedure is the same as the 15 blow compaction test described by the U.S. Army Corps of Engineers (1970). The reduced compactive effort is expected to correspond to a reasonable minimum level of compactive energy for a typical soil liner or cover. Other compaction methods, e.g., kneading compaction, could be used. The key is to span the range of compactive effort expected in the field with laboratory compaction procedures.

One satisfactory approach is as follows:

1. Prepare and compact soil in the laboratory with modified, standard, and reduced compaction procedures to develop compaction curves as shown in Fig. 2.6a. Make sure that the soil preparation procedures are appropriate; factors such as clod size reduction may influence the results (Benson and Daniel, 1990). Other compaction procedures can be used if they better simulate field compaction and span the range of compactive effort expected in the field. Also, as few as two compaction procedures can be used if field construction procedures make either the lowest or highest compactive energy irrelevant.

2. The compacted specimens should be permeated, e.g., per ASTM D-5084. Care should be taken to ensure that permeation procedures are correct, with important details such as degree of saturation and effective confining stress carefully selected. The measured hydraulic conductivity should be plotted as a function of molding water content as shown in Fig. 2.6b.

3. As shown in Fig. 2.6c, the dry unit weight/water content points should be replotted with different symbols used to represent compacted specimens that had hydraulic conductivities greater than the maximum acceptable value and specimens with hydraulic conductivities less than or equal to the maximum acceptable value. An "Acceptable Zone" should be drawn to encompass the data points representing test results meeting or exceeding the design criteria. Some judgment is usually necessary in constructing the Acceptable Zone from the data points. Statistical criteria (e.g., Boutwell and Hedges, 1989) may be introduced at this stage.

4. The Acceptable Zone should be modified (Fig. 2.6d) based on other considerations such as shear strength. Additional tests are usually necessary in order to define the acceptable range of water content and dry unit weight that satisfies both hydraulic conductivity and shear strength criteria. Figure 2.7 illustrates how one might overlap Acceptable Zones defined from hydraulic conductivity and shear strength considerations to define a single Acceptable Zone. The same procedure can be applied to take into consideration other factors such as shrink/swell potential relevant to any particular project.

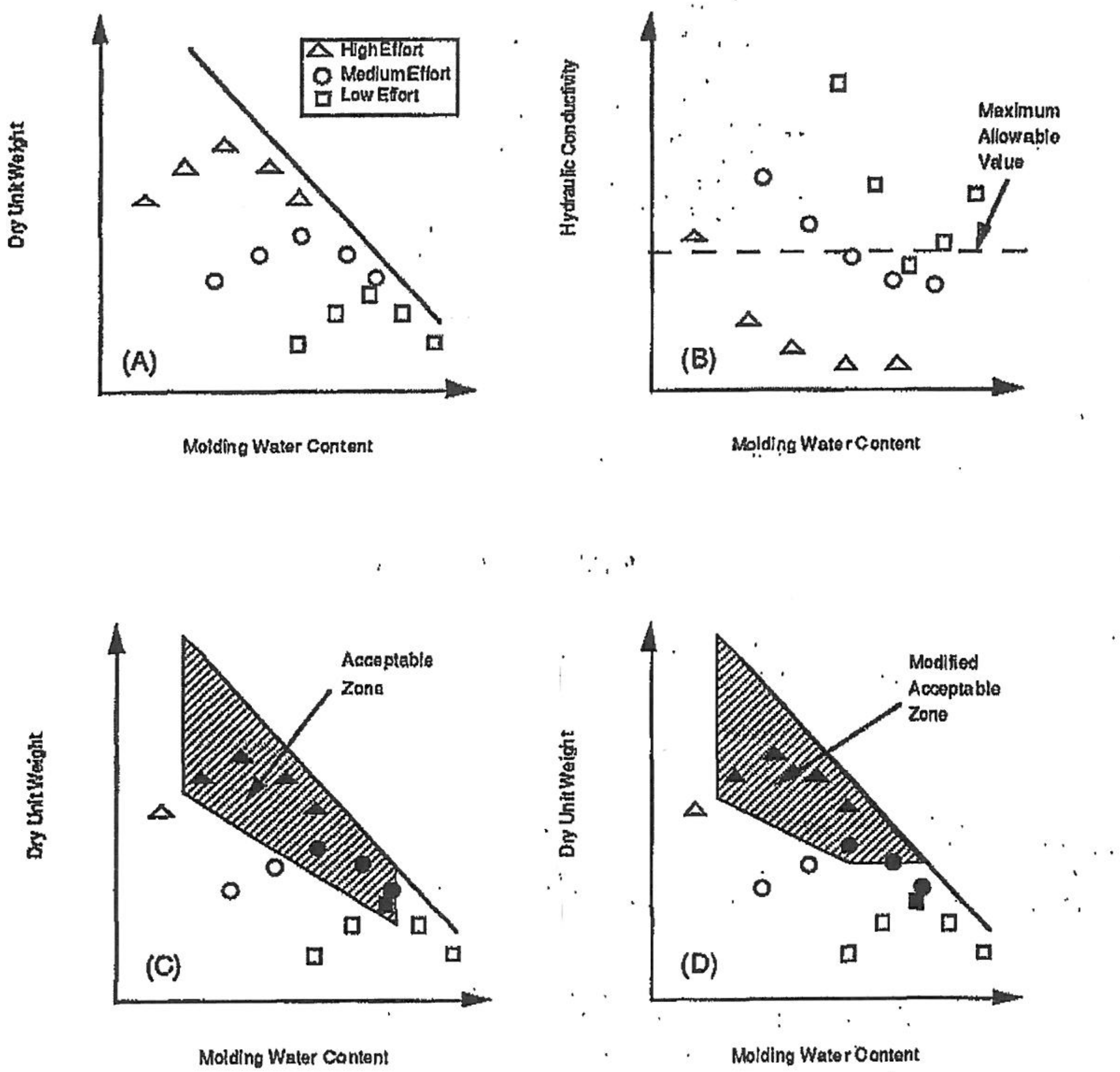


Figure 2.6 - Recommended Procedure to Determine Acceptable Zone of Water Content/Dry Unit Weight Values Based Upon Hydraulic Conductivity Considerations (after Daniel and Benson, 1990).

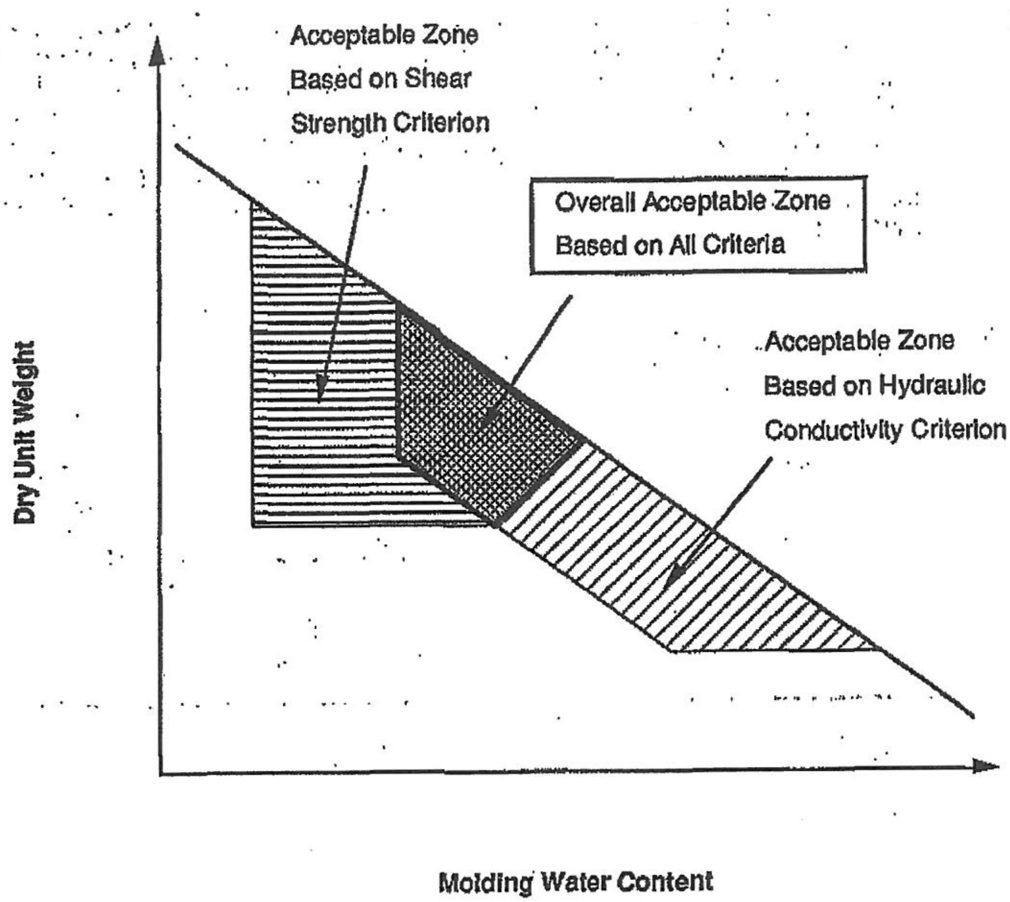


Figure 2.7 - Acceptable Zone of Water Content/Dry Unit Weights Determined by Superposing Hydraulic Conductivity and Shear Strength Data (after Daniel and Benson, 1990).

The same general procedure just outlined may also be used for soil-bentonite mixtures. However, to keep the scope of testing reasonable, the required amount of bentonite should be determined before the main part of the testing program is initiated. The recommended procedure for soil-bentonite mixes may be summarized as follows:

1. The type, grade, and gradation of bentonite that will be used should be determined. This process usually involves estimating costs from several potential suppliers. A sufficient quantity of the bentonite likely to be used for the project should be obtained and tested to characterize the bentonite (characterization tests are discussed later).

2. A representative sample of the soil to which the bentonite will be added should be obtained.

3. Batches of soil-bentonite mixtures should be prepared by blending in bentonite at several percentages, e.g., 2%, 4%, 6%, 8%, and 10% bentonite. Bentonite content is defined as the weight or mass of bentonite divided by the weight or mass of soil mixed with bentonite. For instance, if 5 kg of bentonite are mixed with 100 kg of soil, the bentonite content is 5%. Some people use the gross weight of bentonite rather than oven dry weight. Since air-dry bentonite usually contains 10% to 15% hygroscopic water by weight, the use of oven-dry, air-dry, or damp weight can make a difference in the percentage. Similarly, the weight of soil may be defined as either moist or dry (air- or oven-dry) weight. The contractor would rather work with total (moist) weights since the materials used in forming a soil-bentonite blend do contain some water. However, the engineering characteristics are controlled by the relative amounts of dry materials. A dry-weight basis is generally recommended for definition of bentonite content, but CQC and CQA personnel must recognize that the project specifications may or may not be on a dry-weight basis.

4. Develop compaction curves for each soil-bentonite mixture prepared from Step 3 using the method of compaction appropriate to the project, e.g., ASTM D-698 or ASTM D-1557.

5. Compact samples at 2% wet of optimum for each percentage of bentonite using the same compaction procedure employed in Step 4.

6. Permeate the soils prepared from Step 5 using ASTM D-5084 or some other appropriate test method. Graph hydraulic conductivity versus percentage of bentonite.

7. Decide how much bentonite to use based on the minimum required amount determined from Step 6. The minimum amount of bentonite used in the field should always be greater than the minimum amount suggested by laboratory tests because mixing in the field is usually not as thorough as in the laboratory. Typically, the amount of bentonite used in the field is one to four percentage points greater than the minimum percent bentonite indicated by laboratory tests.

8. A master batch of material should be prepared by mixing bentonite with a representative sample of soil at the average bentonite content expected in the field. The procedures described earlier for determining the Acceptable Zone of water content and dry unit weight are then applied to the master batch.

2.1.5 Test Pads

Test pads are sometimes constructed and tested prior to construction of the full-scale compacted soil liner. The test pad simulates conditions at the time of construction of the soil liner. If conditions change, e.g., as a result of emplacement of waste materials over the liner, the properties of the liner will change in ways that are not normally simulated in a test pad. The objectives of a test pad should be as follows:

1. To verify that the materials and methods of construction will produce a compacted soil liner that meets the hydraulic conductivity objectives defined for a project, hydraulic conductivity should be measured with techniques that will characterize the large-scale hydraulic conductivity and identify any construction defects that cannot be observed with small-scale laboratory hydraulic conductivity tests.

2. To verify that the proposed CQC and CQA procedures will result in a high-quality soil liner that will meet performance objectives.

3. To provide a basis of comparison for full-scale CQA: if the test pad meets the performance objectives for the liner (as verified by appropriate hydraulic conductivity tests) and the full-scale liner is constructed to standards that equal or exceed those used in building the test pad, then assurance is provided that the full-scale liner will also meet performance objectives.

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

In terms of CQA, the test pad can provide an extremely powerful tool to ensure that performance objectives are met. The authors recommend a test pad for any project in which failure of the soil liner to meet performance objectives would have a potentially important, negative environmental impact.

A test pad need not be constructed if results are already available for a particular soil and construction methodology. By the same token, if the materials or methods of construction change, an additional test pad is recommended to test the new materials or construction procedures. Specific CQA tests and observations that are recommended for the test pad are described later in Section 2.10.

2.2 Critical Construction Variables that Affect Soil Liners

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

2.2.1 Properties of the Soil Material

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

2.2.1.1 Plasticity Characteristics

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

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

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

- Plastic Limit (PL): The water content corresponding to the arbitrary limit between the plastic and solid states of consistency of a soil.

- Plasticity Index (PI): The numerical difference between liquid and plastic limits, i.e., LL - PL.

The liquid limit and plastic limit are measured using ASTM D-4318.

Experience has shown that if the soil has extremely low plasticity, the soil will possess insufficient clay to develop low hydraulic conductivity when the soil is compacted. Also, soils that have very low PI's tend to grade into non-plastic soils in some locations. The question of how low the PI can be before the soil is not sufficiently plastic is impossible to answer universally. Daniel (1990) recommends that the soil have a PI $\geq$ 10% but notes that some soils with PI's as low as 7% have been used successfully to build soil liners with extremely low in situ hydraulic conductivity (Albrecht and Cartwright, 1989). Benson et al. (1992) compiled a data base from CQA documents and related the hydraulic conductivity measured in the laboratory on small, "undisturbed" samples of field-compacted soil to various soil characteristics. The observed relationship between hydraulic conductivity and plasticity index is shown in Fig. 2.8. The data base reflects a broad range of construction conditions, soil materials, and CQA procedures. It is clear from the data base that many soils with PI's as low as approximately 10% can be compacted to achieve a hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s.

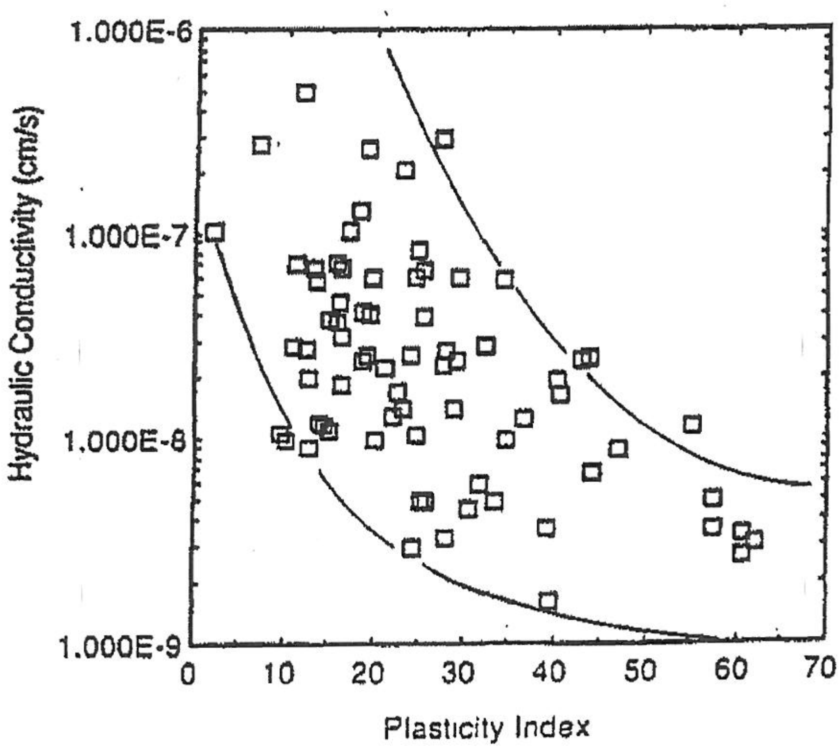


Figure 2.8 - Relationship between Hydraulic Conductivity and Plasticity Index (Benson et al., 1992)

Soils with high plasticity index (>30% to 40%) tend to form hard clods when dried and sticky clods when wet. Highly plastic soils also tend to shrink and swell when wetted or dried. With highly plastic soils, CQC and CQA personnel should be particularly watchful for proper processing of clods, effective remolding of clods during compaction, and protection from desiccation.

2.2.1.2 Percentage Fines

Some earthwork specifications place a minimum requirement on the percentage of fines in the soil liner material. *Fines* are defined as the fraction of soil that passes through the openings of the No. 200 sieve (opening size = 0.075 mm). Soils with inadequate fines typically have too little silt- and clay-sized material to produce suitably low hydraulic conductivity. Daniel (1990) recommends that the soil liner materials contain at least 30% fines. Data from Benson et al. (1992), shown in Fig. 2.9, suggest that a minimum of 50% fines might be an appropriate requirement for many soils. Field inspectors should check the soil to make sure the percentage of fines meets or exceeds the minimum stated in the construction specifications and should be particularly watchful for soils with less than 50% fines.

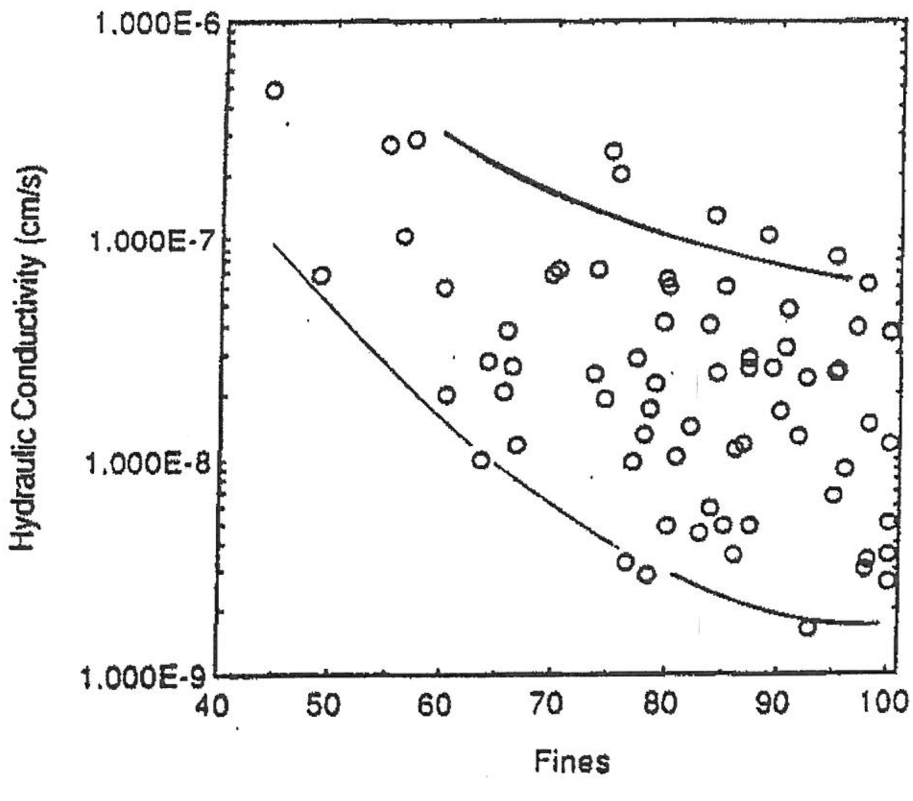


Figure 2.9 - Relationship between Hydraulic Conductivity and Percent Fines (Benson et al., 1992)

2.2.1.3 Percentage Gravel

Gravel is herein defined as particles that will not pass through the openings of a No. 4 sieve (opening size = 4.76 mm). Gravel itself has a high hydraulic conductivity. However, a relatively large percentage (up to about 50%) of gravel can be uniformly mixed with a soil liner material without significantly increasing the hydraulic conductivity of the material (Fig. 2.10). The hydraulic conductivity of mixtures of gravel and clayey soil is low because the clayey soil fills the voids between the gravel particles. The critical observation for CQA inspectors to make is for possible segregation of gravel into pockets that do not contain sufficient soil to plug the voids between the gravel particles. The uniformity with which the gravel is mixed with the soil is more important than the gravel content itself for soils with no more than 50% gravel by weight. Gravel also may possess the capability of puncturing geosynthetic materials -- the maximum size and the angularity of the gravel are very important for the layer of soil that will serve as a foundation layer for a geomembrane.

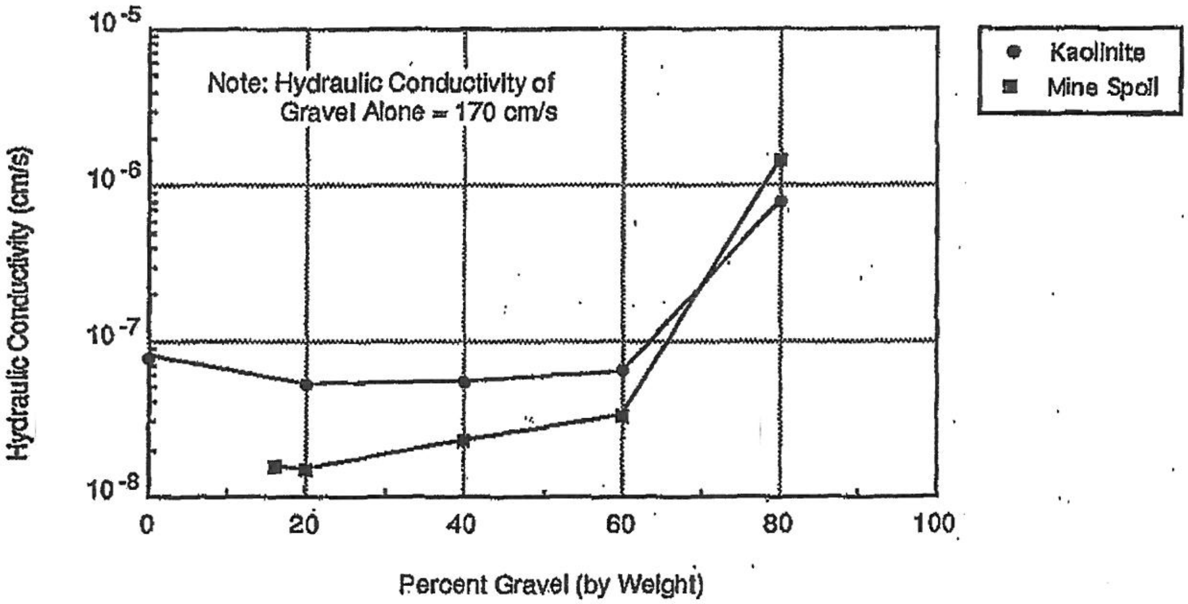


Figure 2.10 - Relationship between Hydraulic Conductivity and Percentage Gravel Added to Two Clayey Soils (after Shelley and Daniel, 1993).

2.2.1.4 Maximum Particle Size

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

2.2.1.5 Clay Content and Activity

The clay content of the soil may be defined in several ways but it is usually considered to be the percentage of soil that has an equivalent particle diameter smaller than 0.005 or 0.002 mm, with 0.002 mm being the much more common definition. The clay content is measured by sedimentation analysis (ASTM D-422). Some construction specifications specify a minimum clay content but many do not.

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

Benson et al. (1992) related hydraulic conductivity to clay content (defined as particles < 0.002 mm) and reported the correlation shown in Fig. 2.11. The data suggest that soils must have at least 10% to 20% clay in order to be capable of being compacted to a hydraulic conductivity $\leq 1 \times 10^{-7}$ cm/s. However, Benson et al. (1992) also found that clay content correlated closely with plasticity index (Fig. 2.12). Soils with PI >10% will generally contain at least 10% to 20% clay.

It is recommended that construction specification writers and regulation drafters indirectly account for clay content by requiring the soil to have an adequate percentage of fines and a suitably large plasticity index -- by necessity the soil will have an adequate amount of clay.

2.2.1.6 Clod Size

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

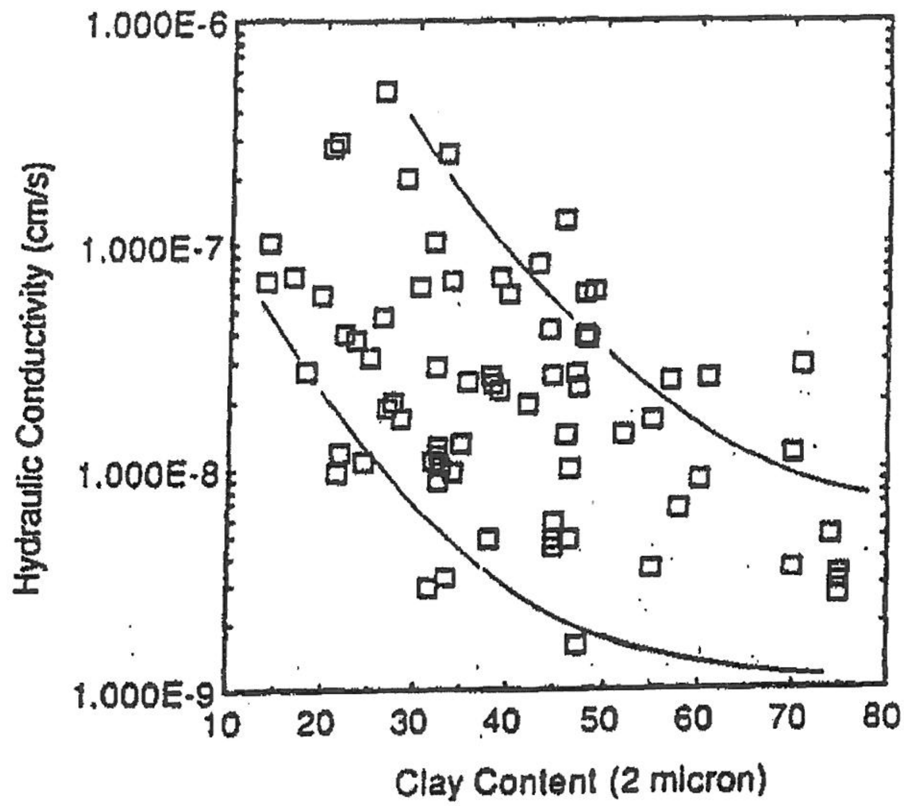


Figure 2.11 - Relationship between Hydraulic Conductivity and Clay Content (Benson et al., 1992)

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

2.2.1.7 Bentonite

Bentonite may be added to clay-deficient soils in order to fill the voids between the soil particles with bentonite and to produce a material that, when compacted, has a very low hydraulic conductivity. The effect of the addition of bentonite upon hydraulic conductivity is shown in Fig. 2.13 for one silty sand. For this particular soil, addition of 4% sodium bentonite was sufficient to lower the hydraulic conductivity to less than 1×10^{-7} cm/s.

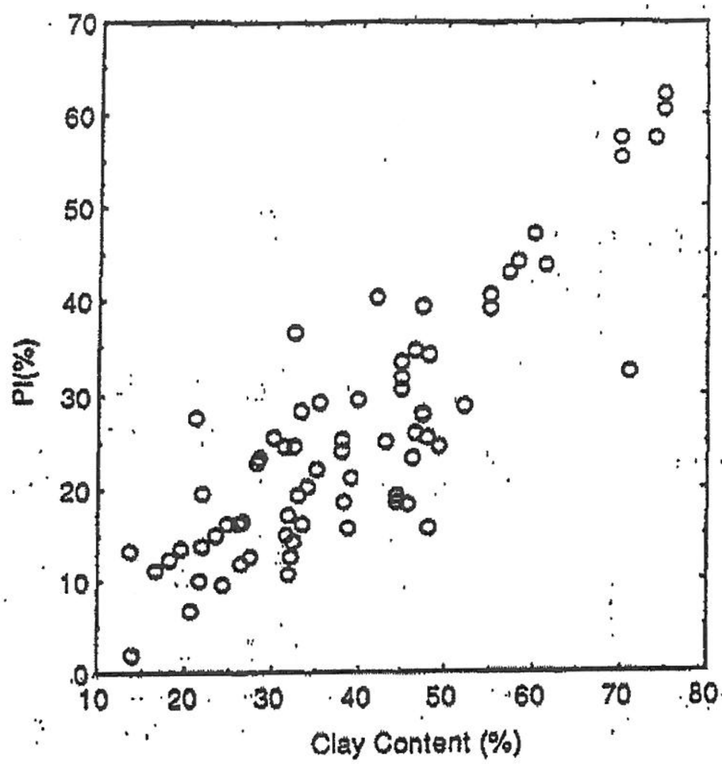


Figure 2.12 - Relationship between Clay Content and Plasticity Index (Benson et al., 1992)

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

Any given type of bentonite may be available in several grades. The grade is a function of impurities in the bentonite, processing procedures, or additives. Some calcium bentonites are processed with sodium solutions to modify the bentonite to a sodium form. Some companies add polymers or other compounds to the bentonite to make the bentonite more absorbent of water or more resistant to alteration by certain chemicals.

Another variable is the gradation of the bentonite. A facet often overlooked by CQC and CQA inspectors is the grain size distribution of the processed bentonite. Bentonite can be ground to different degrees. A fine, powdered bentonite will behave differently from a coarse, granular bentonite -- if the bentonite was supposed to be finely ground but too coarse a grade was delivered, the bentonite may be unsuitable in the mixture amounts specified. Because bentonite is available in variable degrees of pulverization, a sieve analysis (ASTM D422) of the processed dry bentonite is recommended to determine the grain size distribution of the material.

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

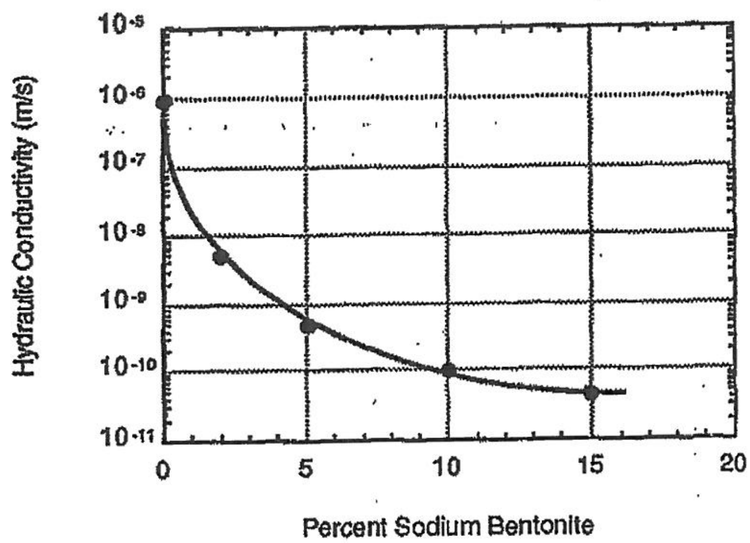


Figure 2.13 - Effect of Addition of Bentonite to Hydraulic Conductivity of Compacted Silty Sand

2.2.2 Molding Water Content

For natural soils, the degree of saturation of the soil liner material at the time of compaction is perhaps the single most important variable that controls the engineering properties of the compacted material. The typical relationship between hydraulic conductivity and molding water content is shown in Fig. 2.14. Soils compacted at water contents less than optimum (*dry of optimum*) tend to have a relatively high hydraulic conductivity; soils compacted at water contents greater than optimum (*wet of optimum*) tend to have a low hydraulic conductivity and low strength. For some soils, the water content relative to the plastic limit (which is the water content of the soil when the soil is at the boundary between being a solid and plastic material) may indicate the degree to which the soil can be compacted to yield low hydraulic conductivity. In general, if the water content is greater than the plastic limit, the soil is in a plastic state and should be capable of being remolded into a low-hydraulic-conductivity material. Soils with water contents dry of the plastic limit will exhibit very little "plasticity" and may be difficult to compact into a low-hydraulic-conductivity mass without delivering enormous compactive energy to the soil. With soil-bentonite mixes, molding water content is usually not as critical as it is for natural soils.

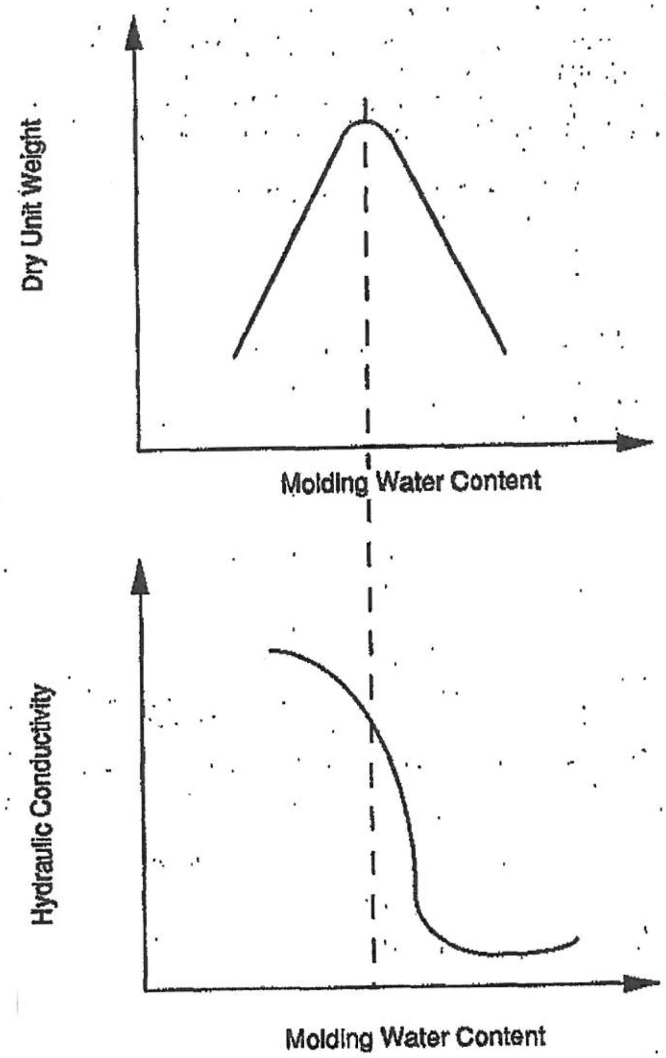


Figure 2.14 - Effect of Molding Water Content on Hydraulic Conductivity

The water content of highly plastic soils is particularly critical. A photograph of a highly plastic soil (PI = 41%) compacted 1% dry of the optimum water content of 17% is shown in Fig. 2.15. Large inter-clod voids are visible; the clods of clay were too dry and hard to be effectively remolded with the compactive effort used. A photograph of a compacted specimen of the same soil moistened to 3% wet of optimum and then compacted is shown in Fig. 2.16. At this water content, the soft soil could be remolded into a homogenous, low-hydraulic-conductivity mass.

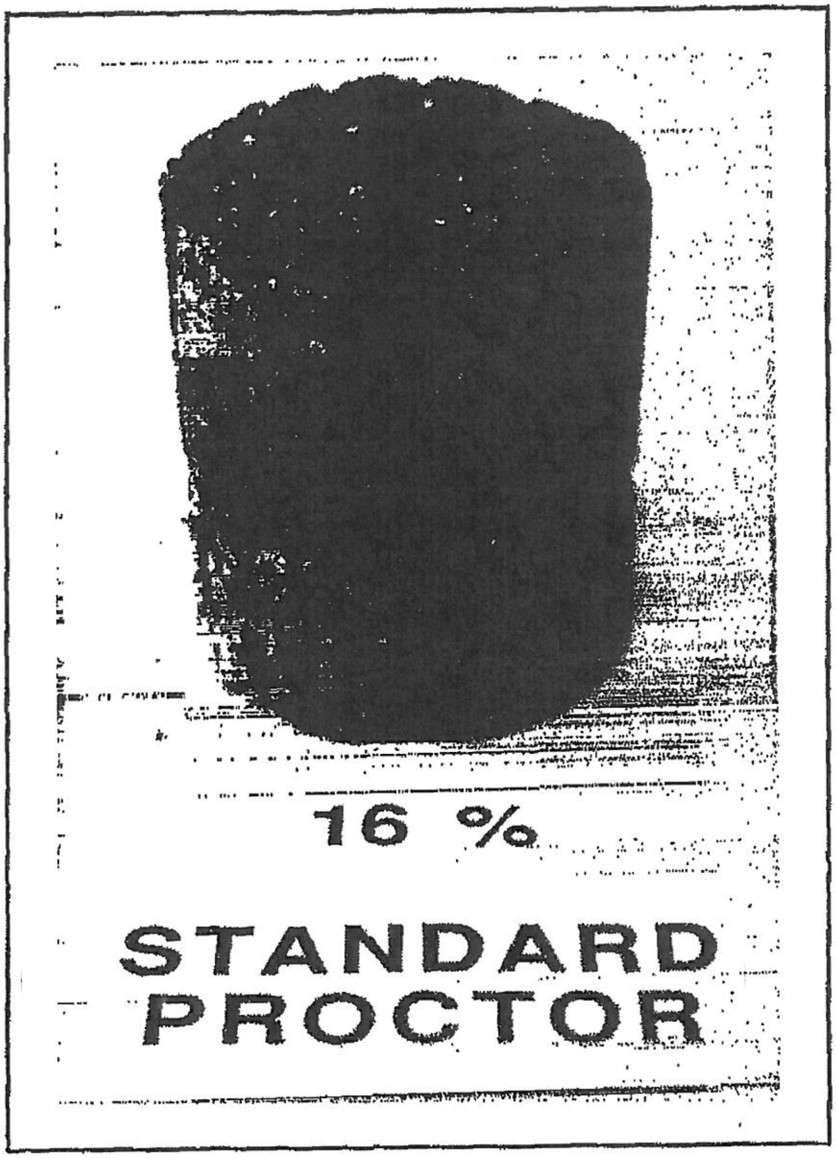


Figure 2.15 - Photograph of Highly Plastic Clay Compacted with Standard Proctor Effort at a Water Content of 16% (1% Dry of Optimum).

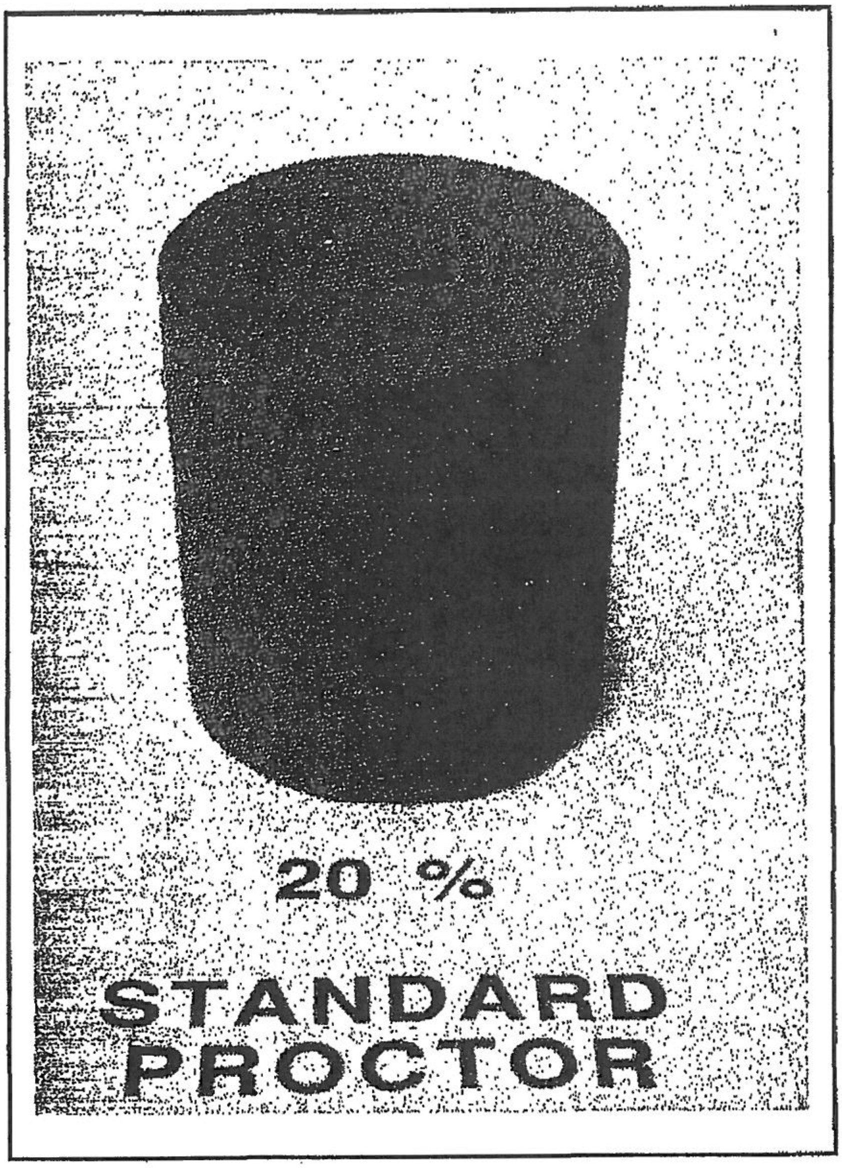


Figure 2.16 - Photograph of Highly Plastic Clay Compacted with Standard Proctor Effort at a Water Content of 20% (3% Wet of Optimum).

It is usually preferable to compact the soil wet of optimum to minimize hydraulic conductivity. However, the soil must not be placed at too high a water content. Otherwise, the shear strength may be too low, there may be great risk of desiccation cracks forming if the soil dries, and ruts may form when construction vehicles pass over the liner. It is critically important that CQC and CQA inspectors verify that the water content of the soil is within the range specified in the construction documents.

2.2.3 Type of Compaction

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

1. Impact Compaction: A ram is repeatedly raised and dropped to compact a lift soil into a mold (Fig. 2.17a), e.g., standard and modified Proctor.

2. Static Compaction: A piston compacts a lift of soil with a constant stress (Fig. 2.17b).

3. Kneading Compaction: A "foot" kneads the soil (Fig. 2.17c).

4. Vibratory Compaction: The soil is vibrated to densify the material (Fig. 2.17d).

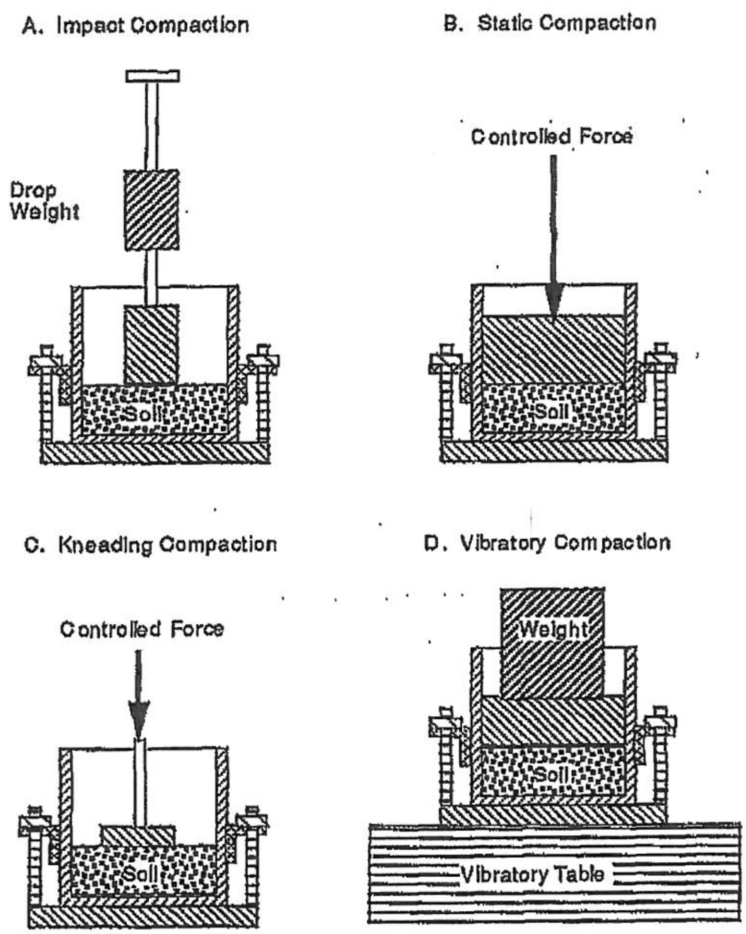


Figure 2.17 - Four Types of Laboratory Compaction Tests

Experience from the laboratory has shown that the type of compaction can affect hydraulic conductivity, e.g., as shown in Fig. 2.18. Kneading the soil helps to break down clods and remold the soil into a homogenous mass that is free of voids or large pores. Kneading of the soil is particularly beneficial for highly plastic soils. For certain bentonite-soil blends that do not form clods, kneading is not necessary. Most soil liners are constructed with "footed" rollers. The "feet" on the roller penetrate into a loose lift of soil and knead the soil with repeated passages of the roller. The dimensions of the feet on rollers vary considerably. Footed rollers with short feet (≈ 75 mm or 3 in.) are called "pad foot" rollers; the feet are said to be "partly penetrating" because the foot is too short to penetrate fully a typical loose lift of soil. Footed rollers with long feet (≈ 200 mm or 8 in.) are often called "sheepsfoot" rollers; the feet fully penetrate a typical loose lift. Figure 2.19 contrasts rollers with partly and fully penetrating feet.

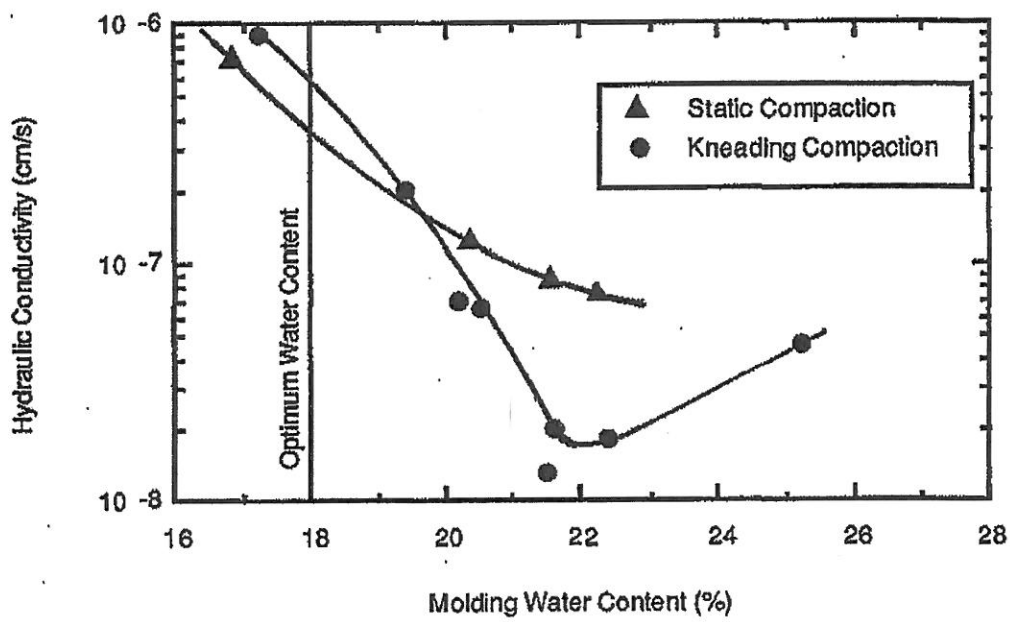


Figure 2.18 - Effect of Type of Compaction on Hydraulic Conductivity (from Mitchell et al., 1965)

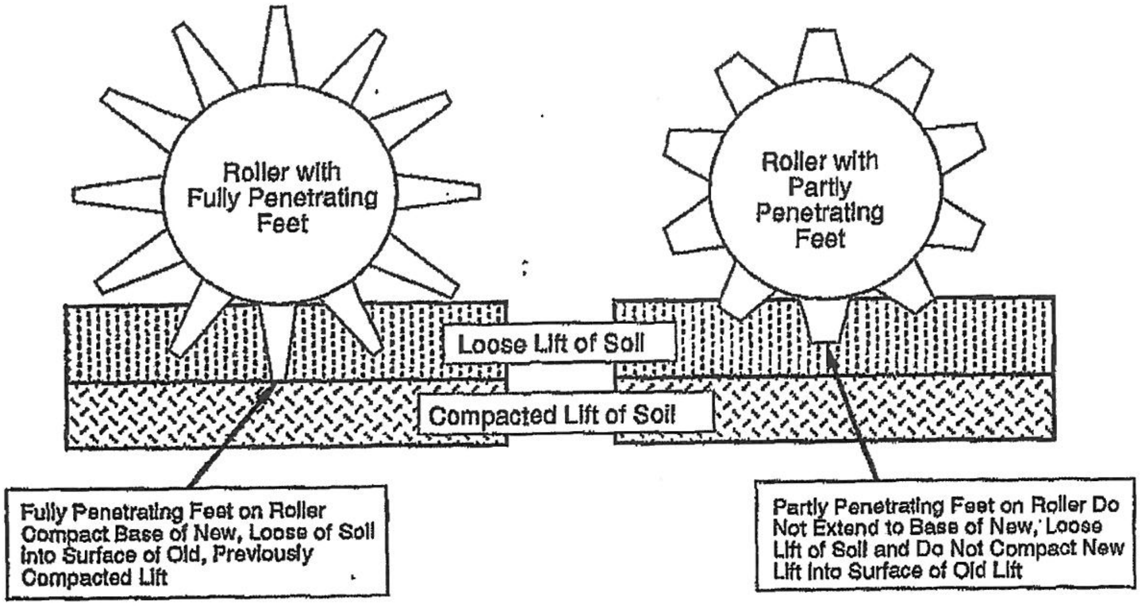


Figure 2.19 - Footed Rollers with Partly and Fully Penetrating Feet

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

2.2.4 Energy of Compaction

The energy used to compact soil can have an important influence on hydraulic conductivity. The data shown in Fig. 2.20 show that increasing the compactive effort produces soil that has a greater dry unit weight and lower hydraulic conductivity. It is important that the soil be compacted with adequate energy if low hydraulic conductivity is to be achieved.

In the field, compactive energy is controlled by:

1. The weight of the roller and the way the weight is distributed (greater weight produces more compactive energy).

2. The thickness of a loose lift (thicker lifts produce less compactive energy per unit volume of soil).

3. The number of passes of the compactor (more passes produces more compactive energy).

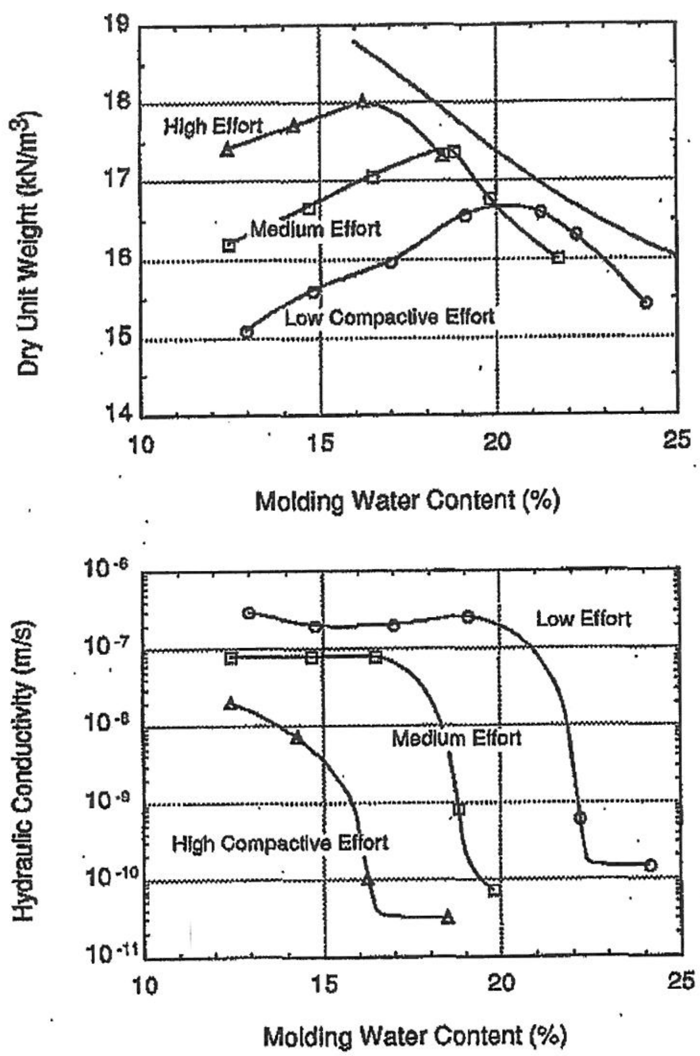


Figure 2.20 - Effect of Compactive Energy on Hydraulic Conductivity (after Mitchell et al., 1965)

Many engineers and technicians assume that percent compaction is a good measure of compactive energy. Indeed, for soils near optimum water content or dry of optimum, percent compaction is a good indicator of compactive energy: if the percent compaction is low, then the compactive energy was almost certainly low. However, for soil compacted wet of optimum, percent compaction is not a particularly good indicator of compactive energy. This is illustrated by the curves in Fig. 2.21. The same soil is compacted with Compactive Energy A and Energy B (Energy B > Energy A) to develop the compaction curves shown in Fig. 2.21. Next, two specimens are compacted to the same water content ($w_A = w_B$). The dry unit weights are practically identical ($\gamma_{d,A} \approx \gamma_{d,B}$) despite the fact that the energies of compaction were different. Further, the hydraulic conductivity (k) of the specimen compacted with the larger energy (Energy B) has a lower hydraulic conductivity than the specimen compacted with Energy A despite the fact that $\gamma_{d,A} \approx \gamma_{d,B}$. The percent compaction for the two compacted specimens is computed as follows:

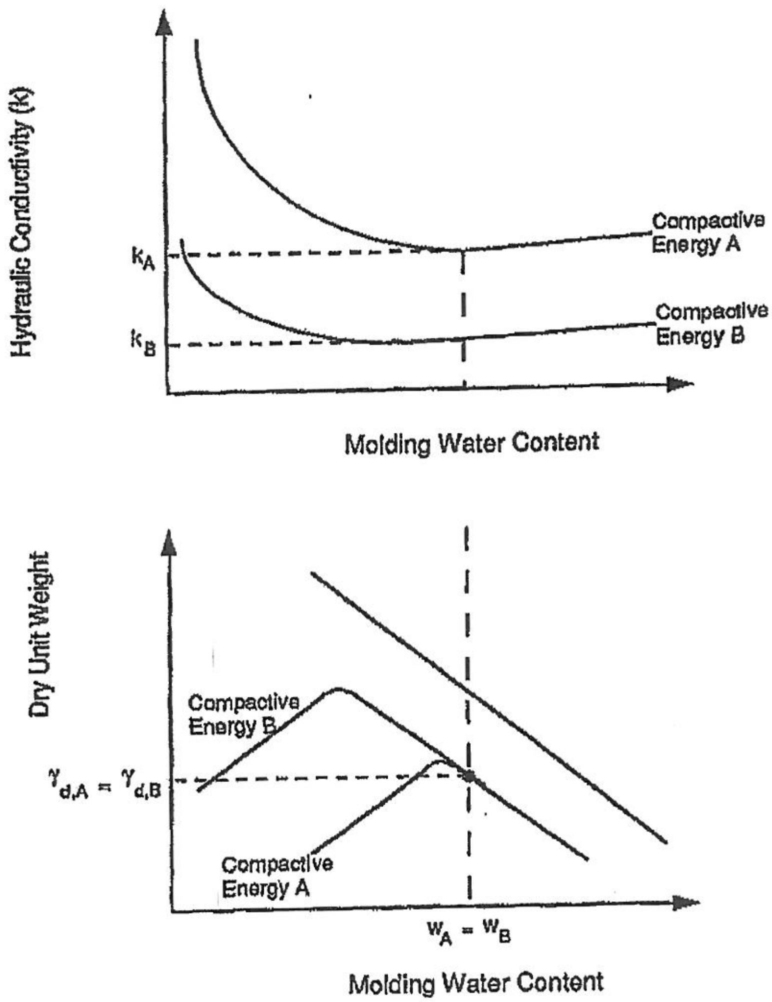


Figure 2.21 - Illustration of Why Dry Unit Weight Is a Poor Indicator of Hydraulic Conductivity for Soil Compacted Wet of Optimum

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

$$P_B = \gamma_{d,B}/[\gamma_{d,max}]_B \times 100\%$$

Since $\gamma_{d,A} = \gamma_{d,B}$ but $[\gamma_{d,max}]_B > [\gamma_{d,max}]_A$, then $P_A > P_B$. Thus, based on percent compaction, since $P_A > P_B$, one might assume Soil A was compacted with greater compactive energy than Soil B. In fact, just the opposite is true. CQC and CQA personnel are strongly encouraged to monitor equipment weight, lift thickness, and number of passes (in addition to dry unit weight) to ensure that appropriate compactive energy is delivered to the soil. Some CQC and CQA inspectors have failed to realize that footed rollers towed by a dozer must be filled with liquid to have the intended large weight.

Experience has shown that effective CQC and CQA for soil liners can be accomplished using the line of optimums as a reference. The "line of optimums" is the locus of (w_{opt}, $\gamma_{d,max}$) points for compaction curves developed on the same soil with different compactive energies (Fig. 2.22). The greater the percentage of actual (w,γ_d) points that lie above the line of optimums the better the overall quality of construction (Benson and Boutwell, 1992). Inspectors are encouraged to monitor the percentage of field-measured (w,γ_d) points that lie on or above the line of optimums. If the percentage is less than 80% to 90%, inspectors should carefully consider whether adequate compactive energy is being delivered to the soil (Benson and Boutwell, 1992).

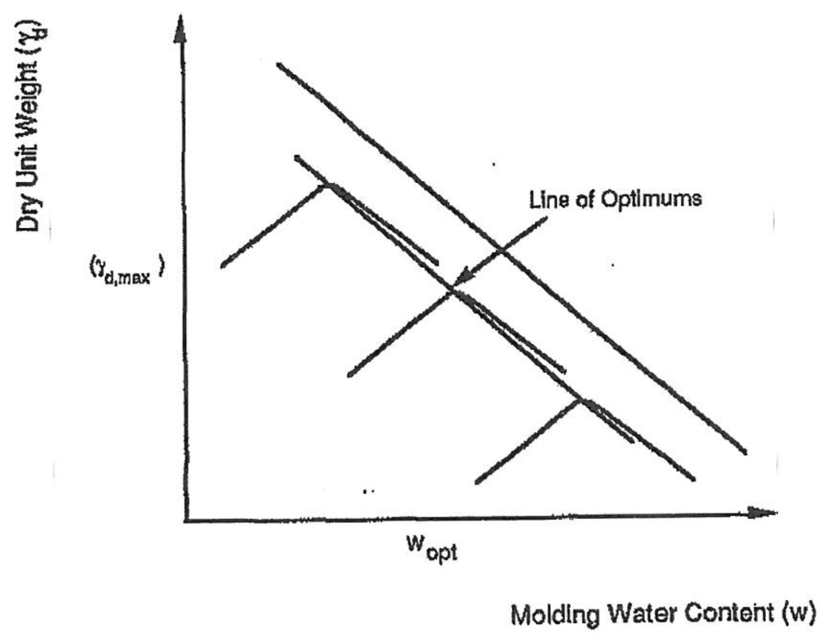


Figure 2.22 - Line of Optimums

2.2.5 Bonding of Lifts

If lifts of soil are poorly bonded, a zone of high hydraulic conductivity will develop at interfaces between lifts. Poorly bonded lift interfaces provide hydraulic connection between more permeable zones in adjacent lifts (Fig. 2.23). It is important to bond lifts together to the greatest extent possible, and to maximize hydraulic tortuosity along lift interfaces, in order to minimize the overall hydraulic conductivity.

Bonding of lifts is enhanced by:

1. Making sure the surface of a previously-compacted lift is rough before placing the new lift of soil (the previously-compacted lift is often scarified with a disc prior to placement of a new lift), which promotes bonding and increased hydraulic tortuosity along the lift interface..

2. Using a fully-penetrating footed roller (the feet pack the base of the new lift into the surface of the previously-compacted lift).

Inspectors should pay particular attention to requirements for scarification and the length of feet on rollers.

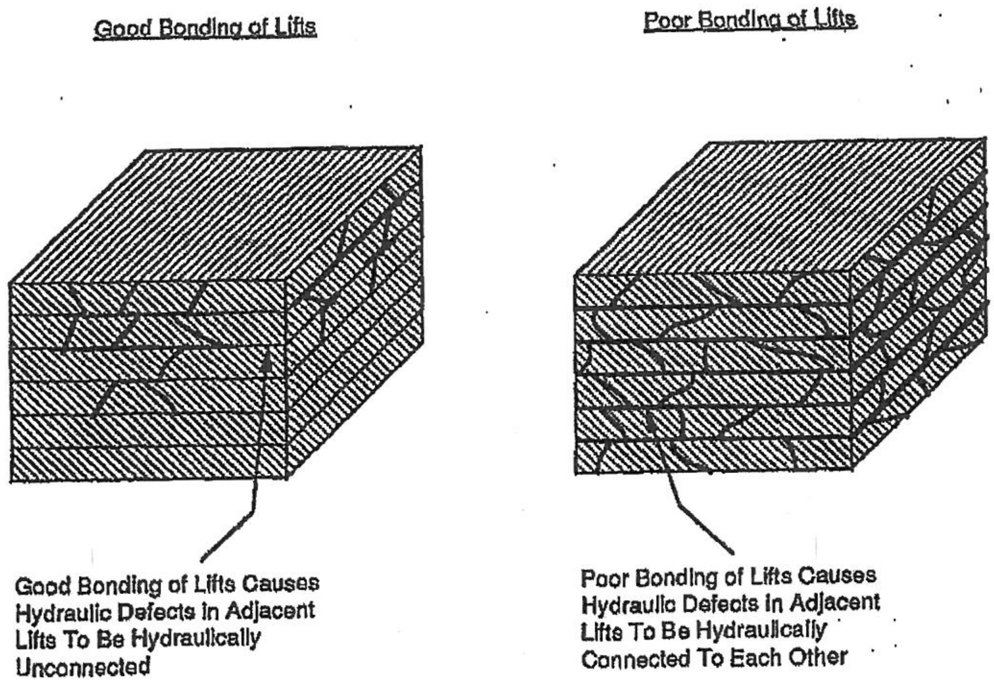


Figure 2.23 - Flow Pathways Created by Poorly Bonded Lifts

2.2.6 Protection Against Desiccation and Freezing

Clay soils shrink when they are dried and, depending on the amount of shrinkage, may crack. Cracks that extend deeper than one lift can be disastrous. Inspectors must be very careful to make sure that no significant desiccation occurs during or after construction. Water content should be measured if there are doubts.

Freezing of a soil liner will cause the hydraulic conductivity to increase. Damage caused by superficial freezing to a shallow depth is easily repaired by rerolling the surface. Deeper freezing is not so easily repaired and requires detailed investigation discussed in Section 2.9.2.3. CQC & CQA personnel should be watchful during periods when freezing temperatures are possible.

2.3 Field Measurement of Water Content and Dry Unit Weight

2.3.1 Water Content Measurement

2.3.1.1 Overnight Oven Drying (ASTM D-2216)

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

Were it not for the fact that one has to wait overnight to determine water content with this method, undoubtedly ASTM D-2216 would be the only method of water content measurement used in the CQC and CQA processes for soil liners. However, field personnel cannot wait overnight to make decisions about continuation with the construction process.

2.3.1.2 Microwave Oven Drying (ASTM D-4643)

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

To guard against overdrying the soil, ASTM method D-4643 requires that the soil be dried for three minutes and then weighed. The soil is then dried for an additional minute and reweighed. The process of drying for one minute and weighing the soil prevents overheating of the soil and forces the operator to cease the drying process once the weight of the soil has stabilized.

Under ideal conditions, microwave oven drying can yield water contents that are almost indistinguishable from values measured with conventional overnight oven drying. Problems that are sometimes encountered with microwave oven drying include problems in operating the oven if the soil contains significant metal and occasional problems with samples exploding from expansion of gas in the interior of the sample during microwave oven drying. Because errors can occasionally arise with microwave oven drying, the water content determined with microwave oven drying should be periodically checked with the value determined by conventional over-night oven drying (ASTM D-2216).

2.3.1.3 Direct Heating (ASTM D-4959)

Direct heating of the soil was common practice up until about two decades ago. To dry a soil with direct heating, one typically places a mass of soil into a metallic container (such as a cooking utensil) and then heats the soil over a flame, e.g., a portable cooking stove, until the soil first appears dry. The mass of the soil plus container is then measured. Next, the soil is heated some more and then re-weighed. This process is repeated until the mass ceases to decrease significantly (i.e., to change by < 0.1% or less).

The main problem with direct heating is that if the soil is overheated during drying, the water content that is measured will be too large. Although ASTM D-4959 does not eliminate this problem, the ASTM method does warn the user not to overheat the soil. Because errors can do arise with direct heating, the water content determined with direct heating should be regularly checked with the value determined by conventional over-night oven drying (ASTM D-2216).

2.3.1.4 Calcium Carbide Gas Pressure Tester (ASTM D-4944)

A known mass of moist soil is placed in a testing device and calcium carbide is introduced. Mixing is accomplished by shaking and agitating the soil with the aid of steel balls and a shaking apparatus. A measurement is made of the gas pressure produced. Water content is determined from a calibration curve. Because errors can occasionally arise with gas pressure testing, the water content determined with gas pressure testing should be periodically checked with the value determined by conventional over-night oven drying (ASTM D-2216).

2.3.1.5 Nuclear Method (ASTM D-3017)

The most widely used method of measuring the water content of compacted soil is the nuclear method. Measurement of water content with a nuclear device involves the moderation or thermalization of neutrons provided by a source of fast neutrons. Fast neutrons are neutrons with an energy of approximately 5 MeV. The radioactive source of fast neutrons is embedded in the interior part of a nuclear water content/density device (Fig. 2.24). As the fast neutrons move into the soil, they undergo a reduction in energy every time a hydrogen atom is encountered. A series of energy reductions takes place when a neutron sequentially encounters hydrogen atoms. Finally, after an average of nineteen collisions with hydrogen atoms, a neutron ceases to lose further energy and is said to be a "thermal" neutron with an energy of approximately 0.025 MeV. A detector in the nuclear device senses the number of thermal neutrons that are encountered. The number of thermal neutrons that are encountered over a given period of time is a function of the number of fast neutrons that are emitted from the source and the density of hydrogen atoms in the soil located immediately below the nuclear device. Through appropriate calibration, and with the assumption that the only source of hydrogen in the soil is water, the nuclear device provides a measure of the water content of the soil over an average depth of about 200 mm (8 in.).

There are a number of potential sources of error with the nuclear water content measuring device. The most important potential source of error is extraneous hydrogen atoms not associated with water. Possible sources of hydrogen other than water include hydrocarbons, methane gas, hydrous minerals (e.g., gypsum), hydrogen-bearing minerals (e.g., kaolinite, illite, and montmorillonite), and organic matter in the soil. Under extremely unfavorable conditions the nuclear device can yield water content measurements that are as much as ten percentage points in error (almost always on the high side). Under favorable conditions, measurement error is less than one percent. The nuclear device should be calibrated for site specific soils and changing conditions within a given site.

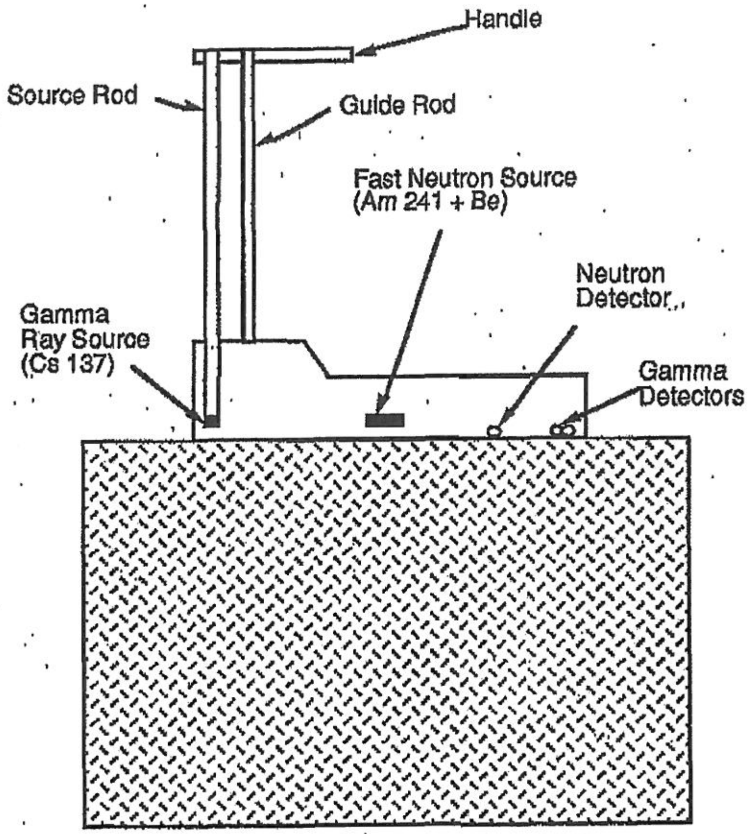


Figure 2.24 - Schematic Diagram of Nuclear Water Content - Density Device

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

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

2.3.2 Unit Weight

2.3.2.1 Sand Cone (ASTM D-1556)

The sand cone is a device for determining the volume of a hole that has been excavated into soil. The idea is to determine the weight of sand required to fill a hole of unknown volume. Through calibration, the volume of sand that fills the hole can be determined from the weight of sand needed to fill the hole. A schematic diagram of the sand cone is shown in Fig. 2.25.

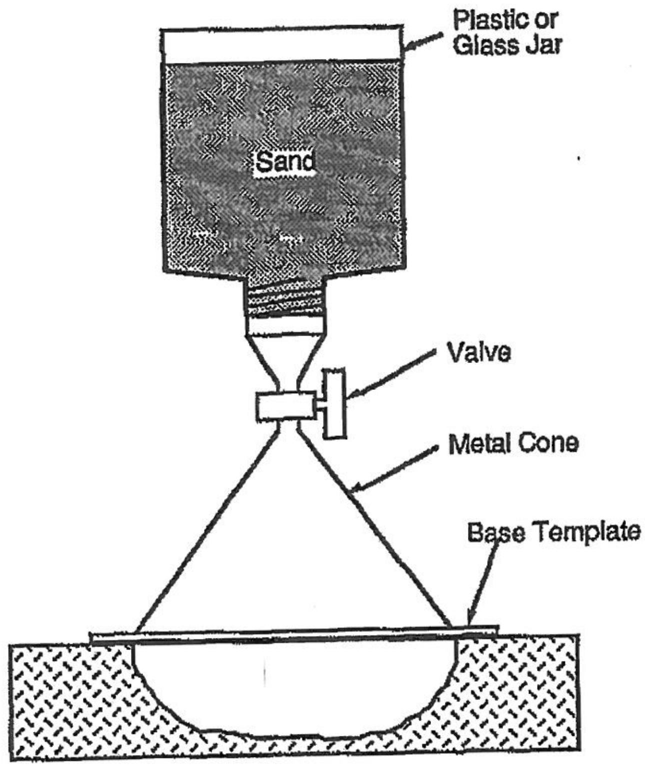


Figure 2.25 - Sand Cone Device

The sand cone is used as follows. First, a template is placed on the ground surface. A circle is scribed along the inside of the hole in the template. The template is removed and soil is excavated from within the area marked by the scribed circle. The soil that is excavated is weighed to determine the total weight (W) of the soil excavated. The excavated soil is oven dried (e.g., with a microwave oven) to determine the water content of the soil. The bottle in a sand cone device is filled with sand and the full bottle is weighed. The template is placed over the hole and the sand cone device is placed on top of the template. A valve on the sand cone device is opened, which allows sand to rain down through the inverted funnel of the device and inside the excavated hole.

When the hole and funnel are filled with sand, the valve is closed and the bottle containing sand is weighed. The difference in weight before and after the hole is dug is calculated. Through calibration, the weight of sand needed to fill the funnel is subtracted, and the volume of the hole is computed from the weight of sand that filled the hole. The total unit weight is calculated by dividing the weight of soil excavated by the computed volume of the excavated hole. The dry unit weight is then calculated from Eq. 2.1.

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

2.3.2.2 Rubber Balloon (ASTM D-2167)

The rubber balloon is similar to the sand cone except that water is used to fill the excavated hole rather than sand. A rubber balloon device is sketched in Fig. 2.26. As with the sand cone test, the test is performed with the device located on the template over the leveled soil. Then a hole is excavated into the soil and the density measuring device is again placed on top of a template at the ground surface. Water inside the rubber balloon device is pressurized with air to force the water into the excavated hole. A thin membrane (balloon) prevents the water from entering the soil. The pressure in the water forces the balloon to conform to the shape of the excavated hole. A graduated scale on the rubber balloon device enables one to determine the volume of water required to fill the hole. The total unit weight is calculated by dividing the known weight of soil excavated from the hole by the volume of water required to fill the hole with the rubber balloon device. The dry unit weight is computed from Eq. 2.1.

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

2.3.2.3 Drive Cylinder (ASTM D-2937)

A drive cylinder is sketched in Fig. 2.27. A drop weight is used to drive a thin-walled tube sampler into the soil. The sampler is removed from the soil and the soil sample is trimmed flush to the bottom and top of the sampling tube. The soil-filled tube is weighed and the known weight of the sampling tube itself is subtracted to determine the gross weight of the soil sample. The dimensions of the sample are measured to enable calculation of volume. The unit weight is calculated by dividing the known weight by the known volume of the sample. The sample is oven dried (e.g., in a microwave oven) to determine water content. The dry unit weight is computed from Eq. 2.1.

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

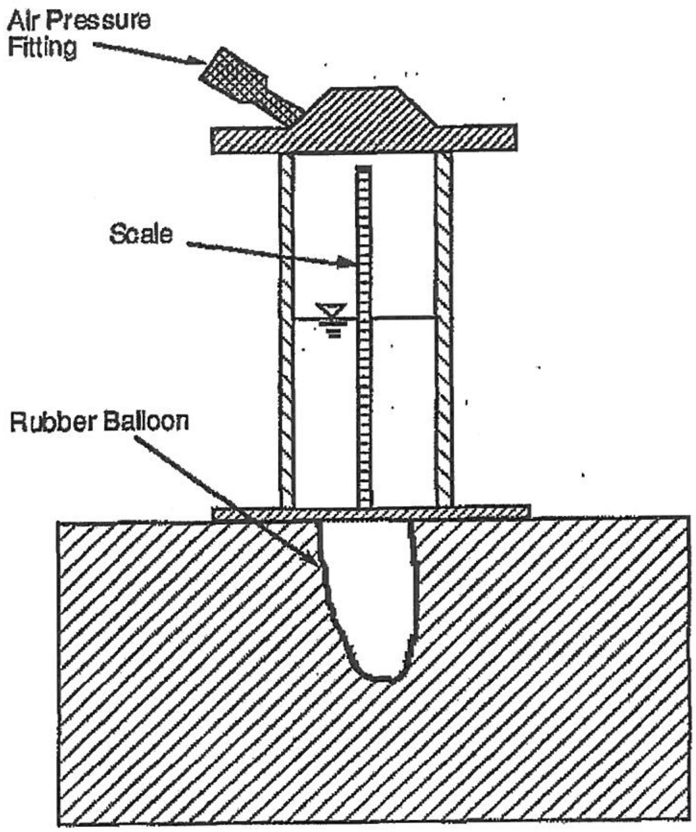


Figure 2.26 - Schematic Diagram of Rubber Balloon Device

2.3.2.4 Nuclear Method (ASTM D-2922)

Unit weight can be measured with a nuclear device operated in two ways as shown in Fig. 2.28. The most common usage is called *direct transmission* in which a source of gamma radiation is lowered down a hole made into the soil to be tested (Fig. 2.28a). Detectors located in the nuclear density device sense the intensity of gamma radiation at the ground surface. The intensity of gamma radiation detected at the surface is a function of the intensity of gamma radiation at the source and the total unit weight of the soil material. The second mode of operation of the nuclear density device is called *backscattering*. With this technique the source of gamma radiation is located at the ground surface (Fig. 2.28b). The intensity of gamma radiation detected at the surface is a function of the density of the soil as well as the radioactivity of the source. With the backscattering technique, the measurement is heavily dependent upon the density of the soil within the upper 25 to 50 mm of soil. The direct transmission method is the recommended technique for soil liners because direct transmission provides a measurement averaged over a greater depth than backscattering.

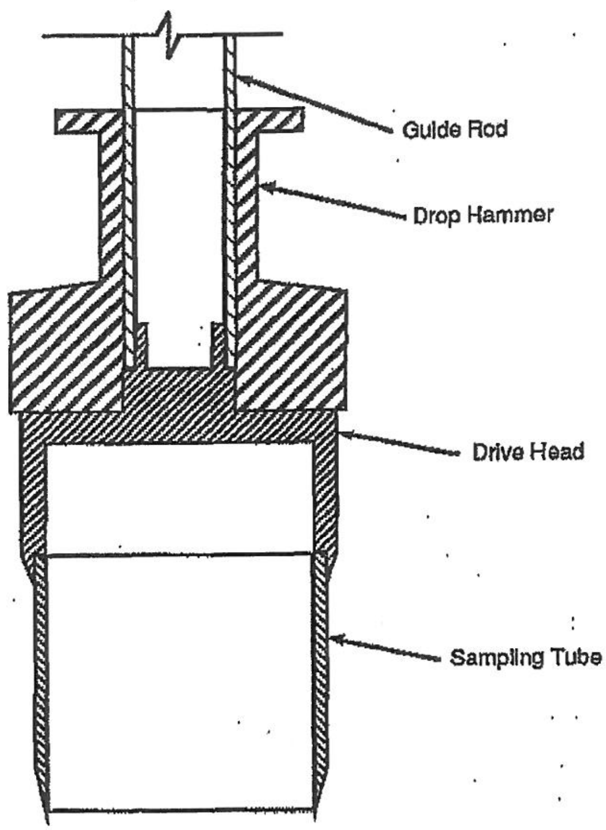


Figure 2.27 - Schematic Diagram of Drive Ring

The operation of a nuclear density device in the direct transmission mode is as follows. First, the area to be tested is smoothed, and a hole is made into the soil liner material by driving a rod (called the *drive rod*) into the soil. The diameter of the hole is approximately 25 mm (1 in.) and the depth of the hole is typically 50 mm (2 in.) greater than the depth to which the gamma radiation source will be lowered below the surface. The nuclear device is then positioned with the source rod directly over the hole in the soil liner material. The source rod is then lowered to a depth of approximately 50 mm (2 in.) above the base of the hole. The source is then pressed against the surface of the hole closest to the detector by pulling on the nuclear device and forcing the source to bear against the side of the hole closest to the detector. The intent is to have good contact between the source and soil along a direct line from source to detector. The intensity of radiation at the detector is measured for a fixed period of time, e.g., 30 or 60 s. The operator can select the period of counting. The longer the counting period, the more accurate the measurement. However, the counting period cannot be extended too much because productivity will suffer.

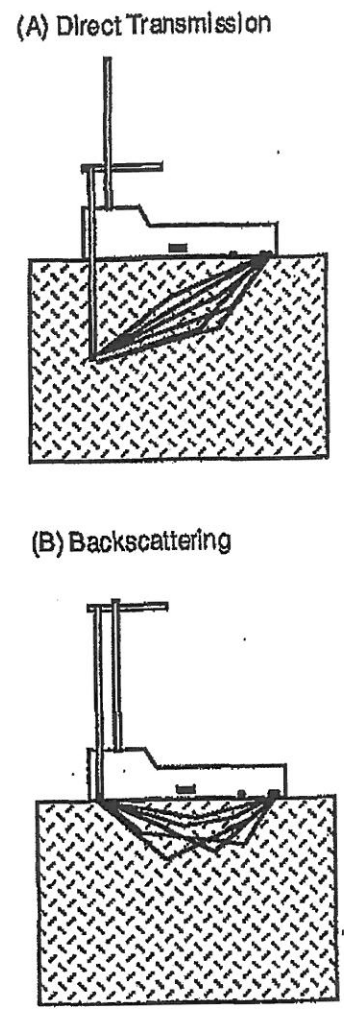


Figure 2.28 - Measurement of Density with Nuclear Device by (a) Direct Transmission and (B) Backscattering

After total unit weight has been determined, the measured water content is used to compute dry unit weight (Eq. 2.1). The potential sources of error with the nuclear device are fewer and less significant in the density-measuring mode compared to the water content measuring mode. The most serious potential source of error is improper use of the nuclear density device by the operator. One gross error that is sometimes made is to drive the source rod into the soil rather than inserting the source rod into a hole that had been made earlier with the drive rod. Improper separation of the source from the base of the hole, an inadequate period of counting, inadequate warm-up, spurious sources of gamma radiation, and inadequate calibration are other potential sources of error.

2.4 Inspection of Borrow Sources Prior to Excavation

2.4.1 Sampling for Material Tests

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

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

2.4.2. Material Tests

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

The material tests that are normally performed on borrow soil are water content, Atterberg limits, particle size distribution, compaction curve, and hydraulic conductivity (Table 2.2). Each of these tests is discussed below.

Table 2.2 - Materials Tests

Parameter	ASTM Test Method	Title of ASTM Test
Water Content	D-2216	Laboratory Determination of Water (Moisture) Content of Soil and Rock
	D-4643	Determination of Water (Moisture) Content of Soil by the Microwave Oven Method
	D-4944	Field determination of Water (Moisture) Content of Soil by the Calcium Carbide Gas Pressure Tester Method
	D-4959	Determination of Water (Moisture) Content by Direct Heating Method
Liquid Limit, Plastic Limit, & Plasticity Index	D-4318	Liquid Limit, Plastic Limit, and Plasticity Index of Soils
Particle Size Distribution	D-422	Particle Size Analysis of Soil
Compaction Curve	D-698	Moisture-Density Relations for Soils and Soil-Aggregate Mixtures Using 5.5-lb. (2.48-kg) Rammer and 12-in. (305-mm) Drop
	D-1557	Moisture-Density Relations for Soils and Soil-Aggregate Mixtures Using 10-lb. (4.54-kg) Rammer and 18-in. (457-mm) Drop
Hydraulic Conductivity	D-5084	Measurement of Hydraulic Conductivity of Saturated Porous Materials Using A Flexible Wall Permeameter

2.4.2.1 Water Content

It is important to know the water content of the borrow soils so that the need for wetting or drying the soil prior to compaction can be identified. The water content of the borrow soil is normally measured following the procedures outlined in ASTM D-2216 if one can wait overnight for results. If not, other test methods described in Section 2.3.1 and listed in Table 2.2 can be used to produce results faster.

2.4.2.2 Atterberg Limits

Construction specifications for compacted soil liners often require a minimum value for the liquid limit and/or plasticity index of the soil. These parameters are measured in the laboratory with the procedures outlined in ASTM D-4318.

2.4.2.3 Particle Size Distribution

Construction specifications for soil liners often place limits on the minimum percentage of fines, the maximum percentage of gravel, and in some cases the minimum percentage of clay. Particle size analysis is performed following the procedures in ASTM D-422. Normally the requirements for the soil material are explicitly stated in the construction specifications. An experienced inspector can often judge the percentage of fine material and the percentage of sand or gravel in the soil. However, compliance with specifications is best documented by laboratory testing.

2.4.2.4 Compaction Curve

Compaction curves are developed utilizing the method of laboratory compaction testing required in the construction specifications. Standard compaction (ASTM D-698) and modified compaction (ASTM D-1557) are two common methods of laboratory compaction specified for soil liners. However, other compaction methods (particularly those unique to state highway or transportation departments) are sometimes specified.

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

2.4.2.5 Hydraulic Conductivity

The hydraulic conductivity of compacted samples of borrow material may be measured periodically to verify that the soil liner material can be compacted to achieve the required low hydraulic conductivity. Several methods of laboratory permeation are available, and others are under development. ASTM D-5084 is the only ASTM procedure currently available. Care should be taken not to apply excessive effective confining stress to test specimens. If no value is specified in the CQA plan, a maximum effective stress of 35 kPa (5 psi) is recommended for both liner and cover systems.

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

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

1. Obtain a large, bulk sample of representative material with a mass of approximately 20 kg.

2. Develop a laboratory compaction curve using the procedure specified in the construction specifications for compaction control, e.g., ASTM D-698 or D-1557.

3. Determine the target water content (w_{target}) and dry unit weight ($\gamma_{d,target}$) for the hydraulic conductivity test specimen. The value of w_{target} is normally the lowest acceptable water content and $\gamma_{d,target}$ is normally the minimum acceptable dry unit weight (Fig. 2.29).

4. Enough soil to make several test specimens is mixed to w_{target}. The compaction procedure used in Step 2 is used to prepare a compacted specimen, except that the energy of compaction is reduced, e.g., by reducing the number of drops of the ram per lift. The dry unit weight (γ_d) is determined. If $\gamma_d \approx \gamma_{d,\,target}$, the compacted specimen may be used for hydraulic conductivity testing. If $\gamma_d \neq \gamma_{d,target}$, then another test specimen is prepared with a larger or smaller (as appropriate) compactive energy. Trial and error preparation of test specimens is repeated until $\gamma_d \approx \gamma_{d,\,target}$. The procedure is illustrated in Fig. 2.29. The actual compactive effort should be documented along with hydraulic conductivity.

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

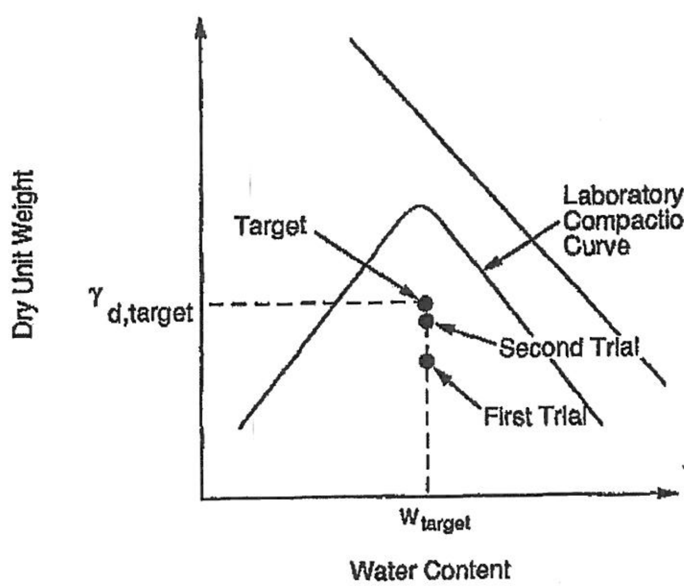


Figure 2.29 - Recommended Procedure for Preparation of a Test Specimen Using Variable (But Documented) Compactive Energy for Each Trial

2.4.2.6 Testing Frequency

The CQA plan should stipulate the frequency of testing. Recommended minimum values are shown in Table 2.3. The tests listed in Table 2.3 are normally performed prior to construction as part of the characterization of the borrow source. However, if time or circumstances do not permit characterization of the borrow source prior to construction, the samples for testing are obtained during excavation or delivery of the soil materials.

Table 2.3 - Recommended Minimum Testing Frequencies for Investigation of Borrow Source

Parameter	Frequency
Water Content	1 Test per 2000 m^3 or Each Change in Material Type
Atterberg Limits	1 Test per 5000 m^3 or Each Change in Material Type
Percentage Fines	1 Test per 5000 m^3 or Each Change in Material Type
Percent Gravel	1 Test per 5000 m^3 or Each Change in Material Type
Compaction Curve	1 Test per 5000 m^3 or Each Change in Material Type
Hydraulic Conductivity	1 Test per 10,000 m^3 or Each Change in Material Type

Note: 1 $yd^3 = 0.76\ m^3$

2.5 Inspection during Excavation of Borrow Soil

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

A key factor for inspectors to observe is the plasticity of the soil. Experienced technicians can often determine whether or not a soil has adequate plasticity by carefully examining the soil in the field. A useful practice for field identification of soils is ASTM D-2488, "Description and Identification of Soils (Visual-Manual Procedure)." The following procedure is used for identifying clayey soils.

- Dry strength: The technician selects enough soil to mold into a ball about 25 mm (1 in.) in diameter. Water is added if necessary to form three balls that each have a diameter of about 12 mm (1/2 in.). The balls are allowed to dry in the sun. The strength of the dry balls is evaluated by crushing them between the fingers. The dry strength is described with the criteria shown in Table 2.4. If the dry strength is none or low, inspectors should be alerted to the possibility that the soil lacks adequate plasticity.

- Plasticity: The soil is moistened or dried so that a test specimen can be shaped into an elongated pat and rolled by hand on a smooth surface or between the palms into a thread about 3 mm (1/8 in.) in diameter. If the sample is too wet to roll easily it should be spread into a thin layer and allowed to lose some water by evaporation. The sample threads are re-rolled repeatedly until the thread crumbles at a diameter of about 3 mm (1/8 in.). The thread will crumble at a diameter of 3 mm when the soil is near the plastic limit. The plasticity is described from the criteria shown in Table 2.5, based upon observations made during the toughness test. Non-plastic soils are usually unsuitable for use as soil liner materials without use of amendments such as bentonite.

Table 2.4 - Criteria for Describing Dry Strength (ASTM D-2488)

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

Table 2.5 - Criteria for Describing Plasticity (ASTM D-2488)

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

2.6 Preprocessing of Materials

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

2.6.1 Water Content Adjustment

Soils that are too wet must first be dried. If the water content needs to be reduced by no more than about three percentage points, the soil can be dried after it has been spread in a loose lift just prior to compaction. If the water content must be reduced by more than about 3 percentage points, it is recommended that drying take place in a separate processing area. The reason for drying in a separate processing area is to allow adequate time for the soil to dry uniformly and to facilitate mixing of the material during drying. The soil to be dried is spread in a lift about 225 to 300 mm (9 to 12 in.) thick and allowed to dry. Water content is periodically measured using one or more of the methods listed in Table 2.2. The contractor's CQC personnel should check the soil periodically to determine when the soil has reached the proper water content.

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

If the soil must be moistened prior to compaction, the same principles discussed above for drying apply; water content adjustment in a separate preprocessing area is recommended if the water content must be increased by more than about 3 percentage points. Inspectors should be careful to verify that water is distributed uniformly to the soil (a spreader bar on the back of a water truck is the recommended device for moistening soil uniformly), that the soil is periodically mixed with a disc or rototiller, and that adequate time has been allowed for uniform hydration of the soil. If the water content is increased by more than three percentage points, at least 24 to 48 hours would normally be required for uniform absorption of water and hydration of soil particles. The construction specifications may limit the type of water that can be used; in some cases, contaminated water, brackish water, or sea water is not allowed.

2.6.2 Removal of Oversize Particles

Oversized stones and rocks should be removed from the soil liner material. Stones and rocks interfere with compaction of the soil and may create undesirable pathways for fluid to flow through the soil liner. The construction specifications should stipulate the maximum allowable size of particles in the soil liner material.

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

2.6.3 Pulverization of Clods

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

2.6.4 Homogenizing Soils

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

2.6.5 Bentonite

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

The parameters that are specified for the bentonite quality vary considerably from project to project. The construction specifications should stipulate the criteria to be met by the bentonite and the relevant test methods. The quality of bentonite is usually measured with some type of measurement of water adsorption ability of the clay. Direct measurement of water adsorption can be accomplished using the plate water adsorption test (ASTM E-946). This test is used primarily in the taconite iron ore industry to determine the effectiveness of bentonite, which is used as a binder during the pelletizing process to soak up excess water in the ore. Brown (1992) reports that thousands of plate water adsorption tests have been performed on bentonite, but experience has been that the test is time consuming, cumbersome, and extremely sensitive to variations in the test equipment and test conditions. The plate water adsorption test is not recommended for CQC/CQA of soil liners.

Simple, alternative tests that provide an indirect indication of water adsorption are available. One indirect test for water adsorption is measurement of Atterberg (liquid and plastic) limits via ASTM D-4318. The higher the quality of the bentonite, the higher the liquid limit and plasticity index. Although liquid and plastic limits tests are very common for natural soils, they have not been frequently used as indicators of bentonite quality in the bentonite industry. A commonly-used test in the bentonite industry is the free swell test. The free swell test is used to determine the amount of swelling of bentonite when bentonite is exposed to water in a glass beaker. Unfortunately, there is currently no ASTM test for determining free swell of bentonite, although one is under development. Until such time as an ASTM standard is developed, the bentonite supplier may be consulted for a suggested testing procedure.

The liquid limit test and free swell test are recommended as the principal quality control tests for the quality of bentonite being used on a project. There are no widely accepted cutoff values for the liquid limit and free swell. However, the following is offered for the information of CQC and CQA inspectors. The liquid limit of calcium bentonite is frequently in the range of 100 to 150%. Sodium bentonite of medium quality is expected to have a liquid limit of approximately 300 to 500%. High-quality sodium bentonite typically has a liquid limit in the range of about 500 to 700%. According to Brown (1992), calcium bentonites usually have a free swell of less than 6 cc. Low-grade sodium bentonites typically have a free swell of 8 - 15 cc. High-grade bentonites often have free swell values in the range of 18 to 28 cc. If high-grade sodium bentonite is to be used on a project, inspectors should expect that the liquid limit will be ≥ 500% and the free swell will be ≥ 18 cc.

The bentonite must usually also meet gradational requirements. The gradation of the dry bentonite may be determined by carefully sieving the bentonite following procedures outlined in ASTM D-422. The CQA inspector should be particularly careful to ensure that the bentonite has been pulverized to the extent required in the construction specifications. The degree of pulverization is frequently overlooked. Finely-ground, powdered bentonite will behave differently when blended into soil than more coarsely ground, granular bentonite. CQC/CQA personnel should be particularly careful to make sure that the bentonite is sufficiently finely ground and is not delivered in too coarse a form (per project specifications); sieve tests on the raw bentonite received at a job site are recommended to verify gradation of the bentonite.

The bentonite supplier is expected to certify that the bentonite meets the specification requirements. However, CQA inspectors should perform their own tests to ensure compliance with the specifications. The recommended CQA tests and testing frequencies for bentonite quality and gradation are summarized in Table 2.6.

Table 2.6 - Recommended Tests on Bentonite to Determine Bentonite Quality and Gradation

Parameter	Frequency	Test Method
Liquid Limit	1 per Truckload or 2 per Rail Car	ASTM D-4318, "Liquid Limit, Plastic Limit, and Plasticity Index of Soils"
Free Swell	1 per Truckload or 2 per Rail Car	No Standard Procedure Is Available
Grain Size of Dry Bentonite	1 per Truckload or 2 per Rail Car	ASTM D-422, "Particle Size Analysis of Soil"

2.6.5.1 Pugmill Mixing

A pugmill is a device for mixing dry materials. A schematic diagram of a typical pugmill is shown in Fig. 2.30. A conveyor belt feeds soil into a mixing unit, and bentonite drops downward into the mixing unit. The materials are mixed in a large box that contains rotating rods with mixing paddles. Water may be added to the mixture in the pugmill, as well.

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

2.6.5.2 In-Place Mixing

An alternative mixing technique is to spread the soil in a loose lift, distribute bentonite on the surface, and mix the bentonite and soil using a rototiller or other mixing equipment. There are several potential problems with in-place mixing. The mixing equipment may not extend to an adequate depth and may not fully mix the loose lift of soil with bentonite. Alternatively, the mixing device may dig too deeply into the ground and actually mix the loose lift in with underlying materials. Bentonite (particularly powdered bentonite) may be blown away by wind when it is placed on the surface of a loose lift, thus reducing the amount of bentonite that is actually incorporated into the soil. The mixing equipment may fail to pass over all areas of the loose lift and may inadequately mix certain portions of the loose lift. Because of these problems many engineers believe that pugmill mixing provides a more reliable means for mixing bentonite with soil. CQA personnel should carefully examine the mixing process to ensure that the problems outlined above, or other problems, do not compromise the quality of the mixing process. Visual examination of the mixture to verify plasticity (see Section 2.5 and Table 2.5) is recommended.

2.6.5.3 Measuring Bentonite Content

The best way to control the amount of bentonite mixed with soil is to measure the relative weights of soil and bentonite blended together at the time of mixing. After bentonite has been mixed with soil there are several techniques available to estimate the amount of bentonite in the soil. None of the techniques are particularly easy to use in all situations.

The recommended technique for measuring the amount of bentonite in soil is the methylene blue test (Alther, 1983). The methylene blue test is a type of titration test. Methylene blue is slowly titrated into a material and the amount of methylene blue required to saturate the material is determined. The more bentonite in the soil the greater the amount of methylene blue that must be added to achieve saturation. A calibration curve is developed between the amount of methylene blue needed to saturate the material and the bentonite content of the soil. The methylene blue test works very well when bentonite is added into a non-clayey soil. However, the amount of methylene blue that must be added to the soil is a function of the amount of clay present in the soil. If clay minerals other than bentonite are present, the clay minerals interfere with the determination of the bentonite content. There is no standard methylene blue test; the procedure outlined in Alther (1983) is suggested until such time as a standard test method is developed.

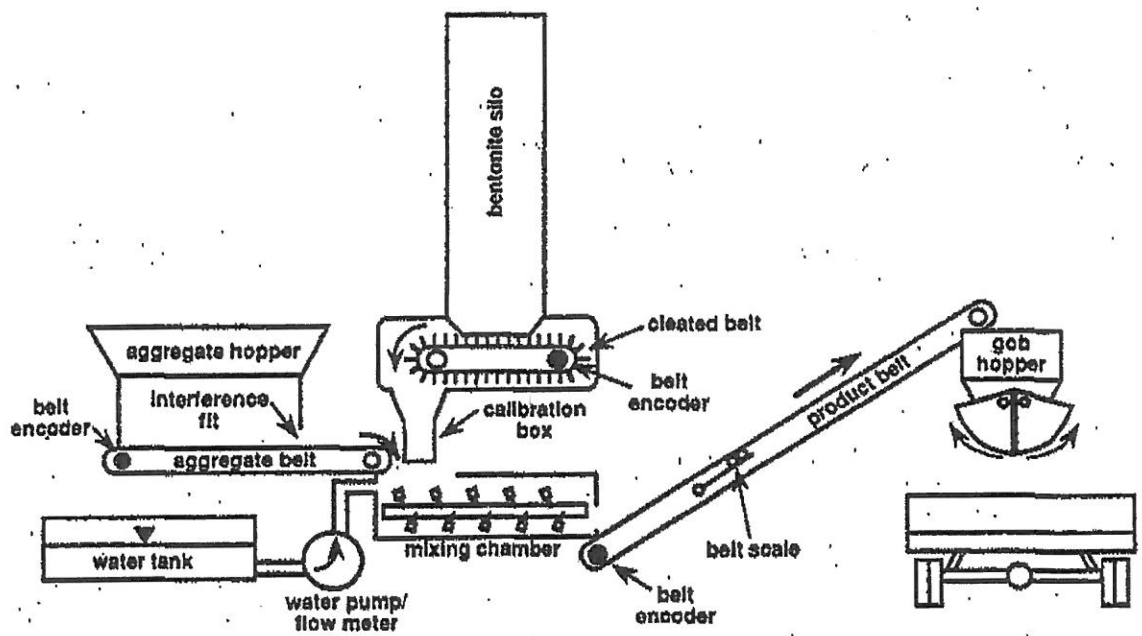


Figure 2.30 - Schematic Diagram of Pugmill

Another type of test that has been used to estimate bentonite content is the filter press test. This test is essentially a water absorbency test: the greater the amount of clay in a soil, the greater the water holding capacity. Like the methylene blue test, the filter press test works well if bentonite is the only source of clay in the soil. No specific test procedure was available at the time of this writing.

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

The recommended tests and testing frequencies to verify proper addition of bentonite are summarized in Table 2.7. However, the CQA personnel must realize that the amount of testing depends on the degree of control in the mixing process: the more control during mixing, the less is the need for testing to verify the proper bentonite content.

Table 2.7 - Recommended Tests to Verify Bentonite Content

Parameter	Frequency	Test Method
Methylene Blue Test	1 per 1,000 m^3	Alther (1983)
Compaction Curve for Soil-Bentonite Mixture (Needed To Prepare Hydraulic Conductivity Test Specimen)	1 per 5,000 m^3	Per Project Specifications, e.g., ASTM D-698 or D-1557
Hydraulic Conductivity of Soil-Bentonite Mixture Compacted to Appropriate Water Content and Dry Unit Weight	3/ha/Lift (1/Acre/Lift)	ASTM D-5084, "Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter"

Note: 1 $yd^3 = 0.76\ m^3$

2.6.6 Stockpiling Soils

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

2.7 Placement of Loose Lift of Soil

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

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

2.7.1 Surface Scarification

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

2.7.2 Material Tests and Visual Inspection

2.7.2.1 Material Tests

After a loose lift of soil has been placed, samples are periodically taken to confirm the properties of the soil liner material. These samples are in addition to samples taken from the borrow area (Table 2.3). The types of tests and frequency of testing are normally specified in the CQA documents. Table 2.8 summarizes recommended minimum tests and testing frequencies. Samples of soils can be taken either on a grid pattern or on a random sampling pattern (see Section 2.8.3.2). Statistical tests and criteria can be applied but are not usually applied to soil liners in part because enough data have to be gathered to apply statistics, and yet decisions have to be made immediately, before very much data are collected.

2.7.2.2 Visual Observations

Inspectors should position themselves near the working face of soil liner material as it is being placed. Inspectors should look for deleterious materials such as stones, debris, and organic matter. Continuous inspection of the placement of soil liner material is recommended to ensure that the soil liner material is of the proper consistency.

2.7.2.3 Allowable Variations

Tests on soil liner materials may occasionally fail to conform with required specifications. It is unrealistic to think that 100% of a soil liner material will be in complete conformance with specifications. For example, if the construction documents require a minimum plasticity index it may be anticipated that a small fraction of the soil (such as pockets of sandy material) will fail to conform with specifications. It is neither unusual nor unexpected that occasional failing material will be encountered in soil liners. Occasional imperfections in soil liner materials are expected. Indeed, one of the reasons why multiple lifts are used in soil liners is to account for the inevitable variations in the materials of construction employed in building soil liners. Occasional deviations from construction specifications are not harmful. Recommended maximum allowable variations (failing tests) are listed in Table 2.9.

Table 2.8 - Recommended Materials Tests for Soil Liner Materials Sampled after Placement in a Loose Lift (Just Before Compaction)

Parameter	Test Method	Minimum Testing Frequency
Percent Fines (Note 1)	ASTM D-1140	1 per 800 m^3 (Notes 2 & 5)
Percent Gravel (Note 3)	ASTM D-422	1 per 800 m^3 (Notes 2 & 5)
Liquid & Plastic Limits	ASTM D-4318	1 per 800 m^3 (Notes 2 & 5)
Percent Bentonite (Note 4)	Alther (1983)	1 per 800 m^3 (Notes 2 & 5)
Compaction Curve	As Specified	1 per 4,000 m^3 (Note 5)
Construction Oversight	Observation	Continuous

Notes:

1. Percent fines is defined as percent passing the No. 200 sieve.
2. In addition, at least one test should be performed each day that soil is placed, and additional tests should be performed on any suspect material observed by CQA personnel.
3. Percent gravel is defined as percent retained on the No. 4 sieve.
4. This test is only applicable to soil-bentonite liners.
5. 1 $yd^3 = 0.76\ m^3$.

Table 2.9 - Recommended Maximum Percentage of Failing Material Tests

Parameter	Maximum Allowable Percentage of Outliers
Atterberg Limits	5% and Outliers Not Concentrated in One Lift or One Area
Percent Fines	5% and Outliers Not Concentrated in One Lift or One Area
Percent Gravel	10% and Outliers Not Concentrated in One Lift or One Area
Clod Size	10% and Outliers Not Concentrated in One Lift or One Area
Percent Bentonite	5% and Outliers Not Concentrated in One Lift or One Area
Hydraulic Conductivity of Laboratory Compacted Soil	5% and Outliers Not Concentrated in One Lift or One Area

2.7.2.4 Corrective Action

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

The usual corrective action is to wet or dry the loose lift of soil in place if the water content is incorrect. The water must be added uniformly, which requires mixing the soil with a disc or rototiller (see Section 2.6.1). If the soil contains oversized material, oversized particles are removed from the material (see Section 2.6.2). If clods are too large, clods can be pulverized in the loose lift (see Section 2.6.3). If the soil lacks adequate plasticity, contains too few fines, contains too much gravel, or lacks adequate bentonite, the material is normally excavated and replaced.

2.7.3 Placement and Control of Loose Lift Thickness

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

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

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

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

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

2.8 Remolding and Compaction of Soil

2.8.1 Compaction Equipment

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

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

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

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

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

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

2.8.2 Number of Passes

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

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

$$C = [A_f/A_d] \times N \times 100\% \tag{2.4}$$

where N is the number of passes of the roller, A_f is the sum of the area of the feet on the drums of the roller, and A_d is the area the drum itself. Construction specifications sometimes require 150% -

200% coverage of the roller. For a given roller and minimum percent coverage, the minimum number of passes (N) may be computed.

The number of passes of a compactor over the soil can have an important influence on the overall hydraulic conductivity of the soil liner. It is recommended that periodic observations be made of the number of passes of the roller over a given point. Approximately 3 observations per hectare per lift (one observation per acre per lift) is the recommended frequency of measurement. The minimum number of passes that is reasonable depends upon many factors and cannot be stated in general terms. However, experience has been that at least 5 to 15 passes of a compactor over a given point is usually necessary to remold and compact clay liner materials thoroughly.

2.8.3 Water Content and Dry Unit Weight

2.8.3.1 Water Content and Unit Weight Tests

One of the most important CQA tests is measurement of water content and dry unit weight. Methods of measurement were discussed in Section 2.3. Recommended testing frequencies are listed in Table 2.10. It is stressed that the recommended testing frequencies are the minimum values. Some judgment should be applied to these numbers, and the testing frequencies should be increased or kept at the minimum depending on the specific project and other QA/QC tests and observations. For example, if hydraulic conductivity tests are not performed on undisturbed samples (see Section 2.8.4.2), more water content/density tests may be required than the usual minimum.

2.8.3.2 Sampling Patterns

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

A common method of selecting sample locations is to establish a grid pattern. The grid pattern is simple and ensures a high probability of locating defective areas so long as the defective areas are of a size greater than or equal to the spacing between the sampling points. It is important to stagger the grid patterns in successive lifts so that sampling points are not at the same location in each lift. One would not want to sample at the same location in successive lifts because repaired sample penetrations would be stacked on top of one another. The grid pattern sampling procedure is the simplest one to use that avoids the potential for bias described in the previous paragraph.

A third alternative for selecting sampling points is to locate sampling points randomly. Tables and examples are given in Richardson (1992). It is recommended that no sampling point be located within 2 meters of another sampling point. If a major portion of the area to be sampled has been omitted as a result of the random sampling process, CQA inspectors may add additional points to make sure the area receives some testing. Random sampling is sometimes preferred on large projects where statistical procedures will be used to evaluate data. However, it can be demonstrated that for a given number of sampling points, a grid pattern will be more likely to detect a problem area provided that the dimensions of the problem area are greater than or equal to the spacing between sampling points. If the problem area is smaller than the spacing between sampling points, the probability of locating the problem area is approximately the same with both a grid pattern and a random pattern of sampling.

Table 2.10 - Recommended Tests and Observations on Compacted Soil

Parameter	Test Method	Minimum Testing Frequency
Water Content (Rapid) (Note 1)	ASTM D-3017 ASTM D-4643 ASTM D-4944 ASTM D-4959	13/ha/lift (5/acre/lift) (Notes 2 & 7)
Water Content (Note 3)	ASTM D-2216	One in every 10 rapid water content tests (Notes 3 & 7)
Total Density (Rapid) (Note 4)	ASTM D-2922 ASTM D-2937	13/ha/lift (5/acre/lift) (Notes 2, 4 & 7)
Total Density (Note 5)	ASTM D-1556 ASTM D-1587 ASTM D-2167	One in every 20 rapid density tests (Notes 5, 6, & 7)
Number of Passes	Observation	3/ha/lift (1/acre/lift) (Notes 2 & 7)
Construction Oversight	Observation	Continuous

Notes:

1. ASTM D-3017 is a nuclear method, ASTM D-4643 is microwave oven drying, ASTM D-4944 is a calcium carbide gas pressure tester method, and ASTM D-4959 is a direct heating method. Direct water content determination (ASTM D-2216) is the standard against which nuclear, microwave, or other methods of measurements are calibrated for on-site soils.

2. In addition, at least one test should be performed each day soil is compacted and additional tests should be performed in areas for which CQA personnel have reason to suspect inadequate compaction.

3. Every tenth sample tested with ASTM D-3017, D-4643, D-4944, or D-4959 should be also tested by direct oven drying (ASTM D-2216) to aid in identifying any significant, systematic calibration errors.

4. ASTM D-2922 is a nuclear method and ASTM D-2937 is the drive cylinder method. These methods, if used, should be calibrated against the sand cone (ASTM D-1556) or rubber balloon (ASTM D-2167) for on-site soils. Alternatively, the sand cone or rubber balloon method can be used directly.

5. Every twentieth sample tested with D-2922 should also be tested (as close as possible to the same test location) with the sand cone (ASTM D-1556) or rubber balloon (ASTM D-2167) to aid in identifying any systematic calibration errors with D-2922.

6. ASTM D-1587 is the method for obtaining an undisturbed sample. The section of undisturbed sample can be cut or trimmed from the sampling tube to determine bulk density. This method should not be used for soils containing any particles > 1/6-th the diameter of the sample.

7. 1 acre = 0.4 ha.

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

2.8.3.3 Tests with Different Devices to Minimize Systematic Errors

Some methods of measurement may introduce a systematic error. For example, the nuclear device for measuring water content may consistently produce a water content measurement that is too high if there is an extraneous source of hydrogen atoms besides water in the soil. It is important that devices that may introduce a significant systematic error be periodically correlated with measurements that do not have such error. Water content measurement tests have the greatest potential for systematic error. Both the nuclear method as well as microwave oven drying can produce significant systematic error under certain conditions. Therefore, it is recommended that if the nuclear method or any of the rapid methods of water content measurement (Table 2.2) are used to measure water content, periodic correlation tests should be made with conventional overnight oven drying (ASTM D-2216).

It is suggested that at the beginning of a project, at least 10 measurements of water content be determined on representative samples of the site-specific soil using any rapid measurement method to be employed on the project as well as ASTM D-2216. After this initial correlation, it is suggested (see Tables 2.10) that one in ten rapid water content tests be crossed check with conventional overnight oven drying. At the completion of a project a graph should be presented that correlates the measured water content with a rapid technique against the water content from conventional overnight oven drying.

Some methods of unit weight measurement may also introduce bias. For example, the nuclear device may not be properly calibrated and could lead to measurement of a unit weight that is either too high or too low. It is recommended that unit weight be measured independently on occasion to provide a check against systematic errors. For example, if the nuclear device is the primary method of density measurement being employed on a project, periodic measurements of density with the sand cone or rubber balloon device can be used to check the nuclear device. Again, a good practice is to perform about 10 comparative tests on representative soil prior to construction. During construction, one in every 20 density tests (see Table 2.10) should be checked with the sand cone or rubber balloon. A graph should be made of the unit weight measured with the nuclear device versus the unit weight measured with the sand cone or rubber balloon device to show the correlation. One could either plot dry unit weight or total unit weight for the correlation. Total unit weight in some ways is more sensible because the methods of measurement are actually total unit weight measurements; dry unit weight is calculated from the total unit weight and water content (Eq. 2.1.).

2.8.3.4 Allowable Variations and Outliers

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

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

Periodic human errors are to be expected and should be addressed in the CQA plan.

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

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

Measurement techniques for compacted soils are imperfect and produce variable results. Turnbull et al. (1966) discuss statistical quality control for compacted soils. Noorany (1990) describes 3 sites in the San Diego area for which 9 testing laboratories measured water content and percent compaction on the same fill materials. The ranges in percent compaction were very large: 81-97% for Site 1, 77-99% for Site 2, and 89-103% for Site 3.

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

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

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

Another approach is to allow a small percentage of outliers but to require repair of any area where the water content is far too low or high or the dry unit weight is far too low. This approach is probably the simplest to implement -- recommendations are summarized in Table 2.11.

Table 2.11 - Recommended Maximum Percentage of Failing Compaction Tests

Parameter	Maximum Allowable Percentage of Outliers
Water Content	3% and Outliers Not Concentrated in One Lift or One Area, and No Water Content Less than 2% or More than 3% of the Allowable Value
Dry Density	3% and Outliers Not Concentrated in One Lift or One Area, and No Dry Density Less than 0.8 kN/m^3 (5 pcf) Below the Required Value
Number of Passes	5% and Outliers Not Concentrated in One Lift or One Area

2.8.3.5 Corrective Action

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

The usual problem requiring corrective action at this stage is inadequate compaction of the soil. The contractor is usually able to rectify the problem with additional passes of the compactor over the problem area.

2.8.4 Hydraulic Conductivity Tests on Undisturbed Samples

Hydraulic conductivity tests are often performed on "undisturbed" samples of soil obtained from a single lift of compacted soil liner. Test specimens are trimmed from the samples and are permeated in the laboratory. Compliance with the stated hydraulic conductivity criterion is checked.

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

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

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

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

Research is currently underway to determine if larger-sized samples from field-compacted soils can give more reliable results than the usual 75-mm (3 in.) diameter samples. Until further data are developed, the following recommendations are made concerning the approach to utilizing laboratory hydraulic conductivity tests for QA on field-compacted soils:

1. For gravely soils or other soils that cannot be consistently sampled without causing significant disturbance, laboratory hydraulic conductivity tests should not be a part of the QA program because representative samples cannot realistically be obtained. A test pad (Section 2.10) is recommended to verify hydraulic conductivity.

2. If a test pad is constructed and it is demonstrated that the field-scale hydraulic conductivity is satisfactory on the test pad, the QA program for the actual soil liner should focus on establishing that the actual liner is built of similar materials and to equal or better standards compared to the test pad -- laboratory hydraulic conductivity testing is not necessary to establish this.

3. If no test pad is constructed and it is believed that representative samples can be obtained for hydraulic conductivity testing, then laboratory hydraulic conductivity tests on undisturbed samples from the field are recommended.

2.8.4.1 Sampling for Hydraulic Conductivity Testing

A thin-walled tube is pushed into the soil to obtain a sample. Samples of soil should be taken in the manner that minimizes disturbance such as described in ASTM D-1587. Samples should be sealed and carefully stored to prevent drying and transported to the laboratory in a manner that minimizes soil disturbance as described in ASTM D-4220.

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

Sampling of gravely soils for hydraulic conductivity testing is often a futile exercise. The gravel particles that are encountered by the sampling tube tend to tumble and shear during the push, which caused major disturbance of the soil sample. Experience has been that QA/QC personnel may take several samples of gravely soil before a sample that is sufficiently free of gravel to enable proper sampling is finally obtained; in these cases, the badly disturbed, gravely samples are discarded. Clearly, the process of discarding samples because they contain too much gravel to enable proper sampling introduces a bias into the process. Gravely soils are not amenable to undisturbed sampling.

2.8.4.2 Hydraulic Conductivity Testing

Hydraulic conductivity tests are performed utilizing a flexible wall permeameter and the procedures described in ASTM D-5084. Inspectors should be careful to make sure that the effective confining stress utilized in the hydraulic conductivity test is not excessive. Application of excessive confining stress can produce an artificially low hydraulic conductivity. The CQA plan should prescribe the maximum effective confining stress that will be used; if none is specified a value of 35 kPa (5 psi) is recommended for both liner and cover systems.

2.8.4.3 Frequency of Testing

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

2.8.4.4 Outliers

The results of the above-described hydraulic conductivity tests are often given far too much weight. A passing rate of 100% does not necessarily prove that the liner was well built, yet some inexperienced individuals falsely believe this to be the case. Hydraulic conductivity tests are performed on small samples; even though small samples may have low hydraulic conductivity, inadequate construction or CQA can leave remnant macro-scale defects such as fissures and pockets of poorly compacted soil. The fundamental problem is that laboratory hydraulic conductivity tests are usually performed on 75-mm (3 in.) diameter samples, and these samples are too small to contain a representative distribution of macro-scale defects (if any such defects are present). By the same token, an occasional failing test does not necessarily prove that a problem exists. An occasional failing test only shows that either: (1) there are occasional zones that fail to meet performance criteria, or (2) sampling disturbance (e.g., from the sampling tube shearing stones in the soil) makes confirmation of low hydraulic conductivity difficult or impossible. Soil liners built of multiple lifts are expected to have occasional, isolated imperfections -- this is why the liners are constructed from multiple lifts. Thus, occasional failing hydraulic conductivity tests by themselves do not mean very much. Even on the best built liners, occasional failing test results should be anticipated.

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

2.8.5 Repair of Holes from Sampling and Testing

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

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

2.8.6 Final Lift Thickness

Construction documents may place restrictions on the maximum allowable final (after-compaction) lift thickness. Typically, the maximum thickness is 150 mm (6 in.). Final elevation surveys should be used to establish thicknesses of completed earthwork segments. The specified maximum lift thickness is a nominal value. The actual value may be determined by surveys on the surface of each completed lift, but an acceptable practice (provided there is good CQA on loose lift thickness) is to survey the liner after construction and calculate the average thickness of each lift by dividing the total thickness by the number of lifts.

Tolerances should be specified on final lift thickness. Occasional outliers from these tolerances are not detrimental to the performance of a multi-lift liner. It is recommended by analogy to Table 2.9 that no more than 5% of the final lift thickness determinations be out of specification and that no out-of-specification thickness be more than 25 mm (1 in.) more than the maximum allowable lift thickness.

2.8.7 Pass/Fail Decision

After all CQA tests have been performed, a pass/fail decision must be made. Procedures for dealing with materials problems were discussed in Section 2.7.2.4. Procedures for correcting deficiencies in compaction of the soil were addressed in Section 2.8.3.5. A final pass/fail decision is made by the CQA engineer based upon all the data and test results. The hydraulic conductivity test results may not be available for several days after construction of a lift has been completed. Sometimes the contractor proceeds at risk with placement of additional lifts before all test results are available. On occasion, construction of a liner proceeds without final results from a test pad on the assumption that results will be acceptable. If a "fail" decision is made at this late stage, the defective soil plus any overlying materials that have been placed should be removed and replaced.

2.9 Protection of Compacted Soil

2.9.1 Desiccation

2.9.1.1 Preventive Measures

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

A far better preventive measure is to water the soil periodically. Care must be taken to deliver water uniformly to the soil and not to create zones of excessively wet soil. Adding water by hand is not recommended because water is not delivered uniformly to the soil.

An alternative preventive measure is to cover the soil temporarily with a geomembrane, moist geotextile, or moist soil. The geomembrane or geotextile should be weighted down with sand bags or other materials to prevent transfer of air between the geosynthetic cover and soil. If a geomembrane is used, care should be taken to ensure that the underlying soil does not become heated and desiccate; a light-colored geomembrane may be needed to prevent overheating. If moist soil is placed over the soil liner, the moist soil is removed using grading equipment.

2.9.1.2 Observations

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

2.9.1.3 Tests

If there are questions about degree of desiccation, tests should be performed to determine the water content of the soil. A decrease in water content of one to two percentage points is not considered particularly serious and is within the general accuracy of testing. However, larger reductions in water content provide clear evidence that desiccation has taken place.

2.9.1.4 Corrective Action

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

2.9.2 Freezing Temperatures

2.9.2.1 Compacting Frozen Soil

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

2.9.2.2 Protection After Freezing

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

2.9.2.3 Investigating Possible Frost Damage

Inspectors usually cannot determine from an examination of the surface the depth to which freezing took place in a completed or partially completed soil liner that has been exposed to freezing. In such cases it may be necessary to investigate the soil liner material for possible frost damage. The extent of damage is difficult to determine. Freezing temperatures cause the development of tiny microcracks in the soil. Soils that have been damaged due to frost action develop fine cracks that lead to the formation of chunks of soil when the soil is excavated. The pushing of a sampling tube into the soil will probably close these cracks and mask the damaging effects of frost upon hydraulic conductivity. The recommended procedure for evaluating possible frost damage to soil liners involves three steps:

1. Measure the water content of the soil within and beneath the zone of suspected frost damage. Density may also be measured, but freeze/thaw has little effect on density and may actually cause an increase in dry unit weight. Freeze/thaw is often accompanied by desiccation; water content measurements will help to determine whether drying has taken place.

2. Investigate the morphology of the soil by digging into the soil and examining its condition. Soil damaged by freezing usually contains hairline cracks, and the soil breaks apart in chunks along larger cracks caused by freeze/thaw. Soil that has not been frozen should not have tiny cracks nor should it break apart in small chunks. The morphology of the soil should be examined by excavating a small pit into the soil liner and peeling off sections from the wall of the pit. One should not attempt to cut pieces from the sidewall; smeared soil will mask cracks. A distinct depth may be obvious; above this depth the soil breaks into chunks along frost-induced cracks, and below this depth there is no evidence of cracks produced by freezing.

3. One or more samples of soil should be carefully hand trimmed for hydraulic conductivity testing. The soil is usually trimmed with the aid of a sharpened section of tube of the appropriate inside diameter. The tube is set on the soil surface with the sharpened end facing downward, soil is trimmed away near the sharpened edge of the trimming ring, the tube is pushed a few millimeters into the soil, and the trimming is repeated. Samples may be taken at several depths to delineate the depth to which freeze/thaw damage occurred. The minimum diameter of a cylindrical test specimen should be 300 mm (12 in.). Small test specimens, e.g., 75 mm (3 in.) diameter specimens, should not be used because freeze/thaw can create morphological structure in the soil on a scale too large to permit representative testing with small samples. Hydraulic conductivity tests should be performed as described in ASTM D-5084. The effective confining stress should not exceed the

smallest vertical effective stress to which the soil will be subjected in the field, which is usually the stress at the beginning of service for liners. If no compressive stress is specified, a value of 35 kPa (5 psi) is recommended for both liner and cover system.

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

2.9.2.4 Repair

If it is determined that soil has been damaged by freezing, the damaged material is usually repaired as follows. If damage is restricted to a single lift, the lift may be disked, processed to adjust water content or to reduce clod size if necessary, and recompacted. If the damage extends deeper, damaged materials should be excavated and replaced.

2.9.3 Excess Surface Water

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

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

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

2.10 Test Pads

2.10.1 Purpose of Test Pads

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

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

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

2.10.2 Dimensions

Test pads (Fig. 2.31) normally measure about 10 to 15 m in width by 15 to 30 m in length. The width of the test pad is typically at least four times the width of the compaction equipment, and the length must be adequate for the compactor to reach normal operating speed in the test area. The thickness of a test pad is usually no less than the thickness of the soil liner proposed for a facility but may be as little as 0.6 to 0.9 m (2 to 3 feet) if thicker liners are to be employed at full scale. A freely draining material such as sand is often placed beneath the test pad to provide a known boundary condition in case infiltrating water from a surface hydraulic conductivity test (e.g., sealed double ring infiltrometer) reaches the base of the liner. The drainage layer may be drained with a pipe or other means. However, infiltrating water will not reach the drainage layer if the hydraulic conductivity is very low; the drainage pipe would only convey water if the hydraulic conductivity turns out to be very large. The sand drainage material may not provide adequate foundation support for the first lift of soil liner unless the sand is compacted sufficiently. Also, the first lift of soil liner material on the drainage layer is often viewed as a sacrificial lift and is only compacted nominally to avoid mixing clayey soil in with the drainage material.

2.10.3 Materials

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

2.10.4 Construction

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

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

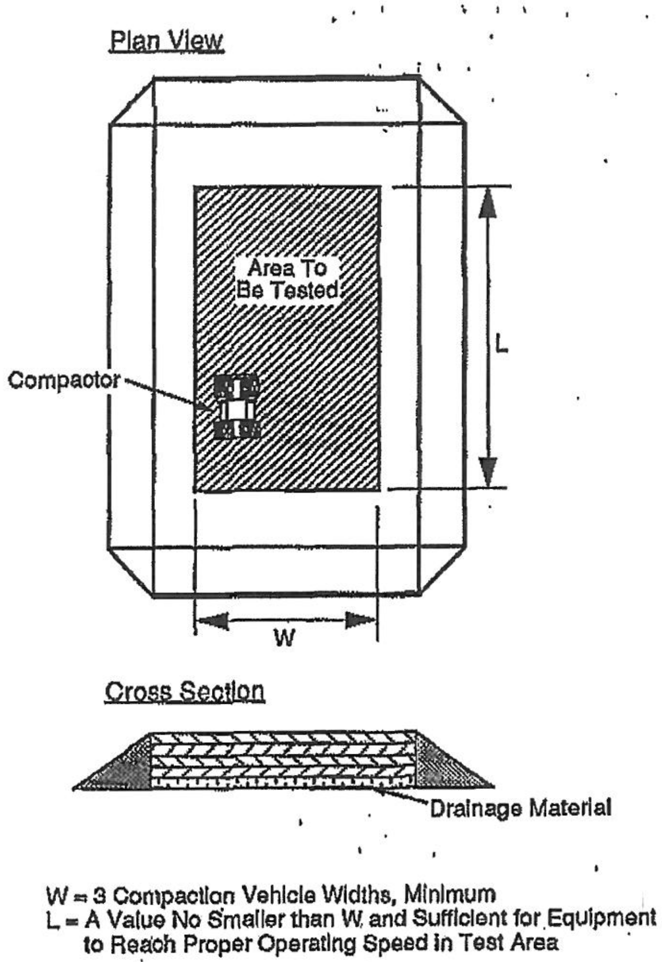


Figure 2.31 - Schematic Diagram of Soil Liner Test Pad

2.10.5 Protection

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

2.10.6 Tests and Observations

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

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

2.10.7 In Situ Hydraulic Conductivity

2.10.7.1 Sealed Double-Ring Infiltrometer

The most common method of measuring in situ hydraulic conductivity on test pads is the sealed double-ring infiltrometer (SDRI). A schematic diagram of the SDRI is shown Fig. 2.32. The test procedure is described in ASTM D-5093.

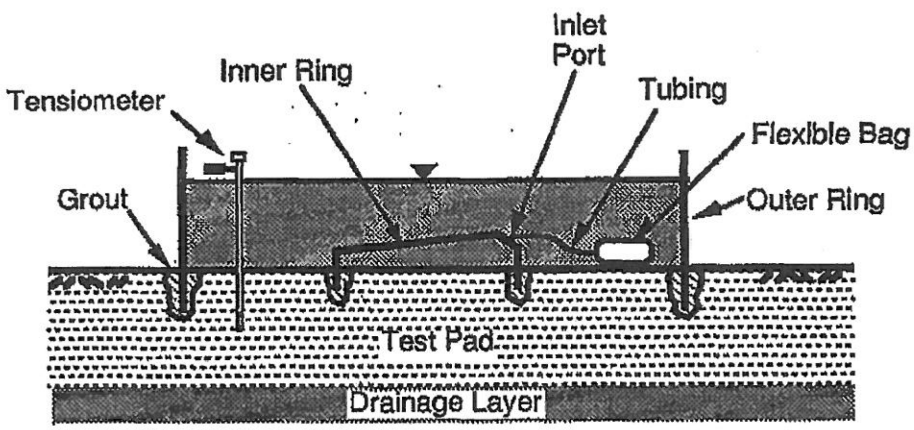


Figure 2.32 - Schematic Diagram of Sealed Double Ring Infiltrometer (SDRI)

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

$$I = Q/At \qquad (2.5)$$

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

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

$$K = I/i \tag{2.6}$$

Three procedures have been used to compute the hydraulic gradient. The procedures are called (1) apparent gradient method; (2) wetting front method; and (3) suction head method. The equation for computing hydraulic gradient from each method is shown in Fig. 2.33.

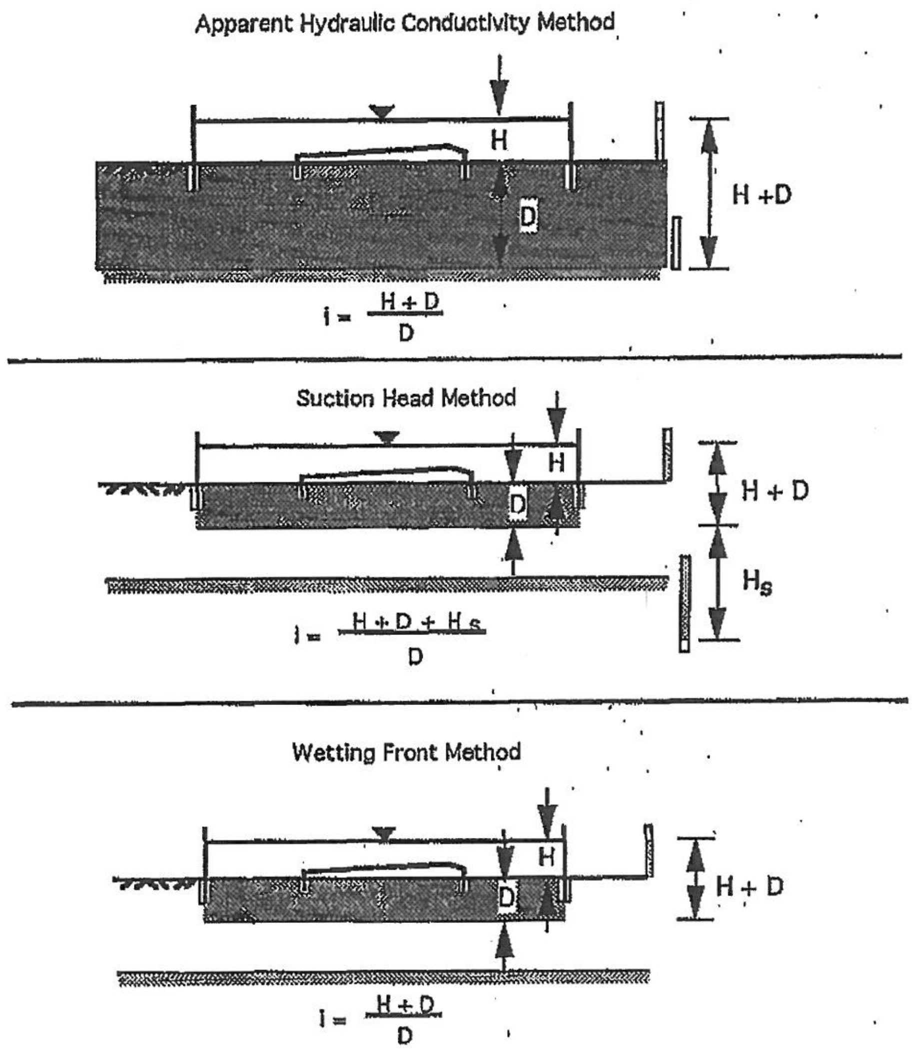


Figure 2.33 - Three Procedures for Computing Hydraulic Gradient from Infiltration Test

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

The second and most widely used method is the wetting front method. The wetting front is assumed to partly penetrate the test pad (Fig. 2.33) and the water pressure at the wetting front is conservatively assumed to equal atmospheric pressure. Tensiometers are used to monitor the depth of wetting of the soil over time, and the variation of water content with depth is determined at the end of the test. The wetting front method is conservative but in most cases not excessively so. The wetting front method is the method that is usually recommended.

The third method, called the suction head method, is the same as the wetting front method except that the water pressure at the wetting front is not assumed to be atmospheric pressure. The suction head (which is defined as the negative of the pressure head) at the wetting front is H_S and is added to the static head of water in the infiltration ring to calculate hydraulic gradient (Fig. 2.37). The suction head H_S is identical to the wetting front suction head employed in analyzing water infiltration with the Green-Ampt theory. The suction head H_S <u>is not</u> the ambient suction head in the unsaturated soil and is generally very difficult to determine (Brakensiek, 1977). Two techniques available for determining H_S are:

1. Integration of the hydraulic conductivity function (Neuman, 1976):

$$H_S = \int_{h_{sc}}^{0} K_r \, dh_s \tag{2.7}$$

   where h_{sc} is the suction head at the initial (presoaked) water content of the soil, K_r is the relative hydraulic conductivity (K at particular suction divided by the value of K at full saturation), and h_s is suction.

2. Direct measurement with air entry permeameter (Daniel, 1989, and references therein).

Reimbold (1988) found that H_S was close to zero for two compacted soil liner materials. Because proper determination of H_S is very difficult, the suction head method cannot be recommended, unless the testing personnel take the time and make the effort to determine H_S properly and reliably.

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

2.10.7.2 <u>Two-Stage Borehole Test</u>

The two-stage borehole hydraulic conductivity was developed by Boutwell (the test is sometimes called the Boutwell Test) and was under development as an ASTM standard at the time of this writing. The device is installed by drilling a hole (which is typically 100 to 150 mm in diameter), placing a casing in the hole, and sealing the annular space between the casing and borehole with grout as shown in Fig. 2.34. A series of falling head tests is performed and the hydraulic conductivity from this first stage (k_1) is computed. Stage one is complete when k_1 ceases to change significantly. The maximum vertical hydraulic conductivity may be computed by assuming that the vertical hydraulic conductivity is equal to k_1. However, the test may be continued for a second stage by removing the top of the casing and extending the hole below the casing as shown in Fig. 2.34. The casing is reassembled, the device is again filled with water, and falling head tests are performed to determine the hydraulic conductivity from stage two (k_2). Both horizontal and vertical hydraulic conductivity may be computed from the values of k_1 and k_2. Further details on methods of calculation are provided by Boutwell and Tsai (1992), although the reader is advised to refer to the ASTM standard when it becomes available.

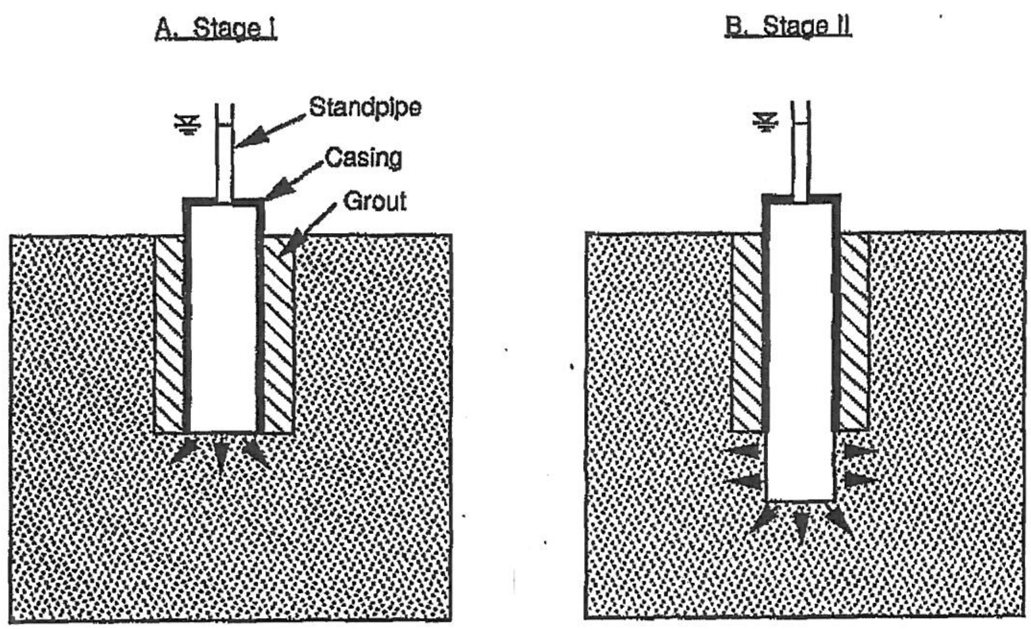


Figure 2.34 - Schematic Diagram of Two-Stage Borehole Test

The two-stage borehole test permeates a smaller volume of soil than the sealed double-ring infiltrometer. The required number of two-stage borehole tests for a test pad is a subject of current research. At the present time, it is recommended that at least 5 two-stage borehole tests be performed on a test pad if the two-stage test is used. If 5 two-stage borehole tests are performed, then one would expect that all five of the measured vertical hydraulic conductivities would be less than or equal to the required maximum hydraulic conductivity for the soil liner.

2.10.7.3 Other Field Tests

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

2.10.7.4 Laboratory Tests

Laboratory hydraulic conductivity tests may be performed for two reasons:

1. If a very large sample of soil is taken from the field and permeated in the laboratory, the result may be representative of field-scale hydraulic conductivity. The question of how large the laboratory test specimen needs to be is currently a matter of research, but preliminary results indicate that a specimen with a diameter of approximately 300 mm (12 in.) may be sufficiently large (Benson et al., 1993).

2. If laboratory hydraulic conductivity tests are a required component of QA/QC for the actual liner, the same sampling and testing procedures are used for the test pad. Normally, undisturbed soil samples are obtained following the procedures outlined in ASTM D-1587, and soil test specimens with diameters of approximately 75 mm (3 in.) are permeated in flexible-wall permeameters in accordance with ASTM D-5084.

2.10.8 Documentation

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

2.11 Final Approval

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

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ASTM D-2488, "Description and Identification of Soils (Visual-Manual Procedure)"

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ASTM D-2937, "Density and Unit Weight of Soil In Place by Drive-Cylinder Method"

ASTM D-3017, "Water Content of Soil and Rock In Place by Nuclear Methods (Shallow Depth)"

ASTM D-4220, "Preserving and Transporting Soil Samples"

ASTM D-4318, "Liquid Limit, Plastic Limit, and Plasticity Index of Soils"

ASTM D-4643, "Determination of Water (Moisture) Content of Soil by Microwave Oven Method"

ASTM D-4944, "Field Determination of Water (Moisture) Content of Soil by Calcium Carbide Gas Pressure Tester Method"

ASTM D-4959, "Determination of Water (Moisture) Content of Soil by Direct Heating Method"

ASTM D-5080, "Rapid Determination of Percent Compaction"

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LIST OF ACRONYMS

ASTM	-	American Society for Testing and Materials
CAFO	-	Confined Animal Feedlot Operations
CSL	-	Compacted Soil Liner
CQA	-	Construction Quality Assurance
DP	-	Discharge Permit
EMNRD	-	Energy, Minerals and Natural Resources Department
FML	-	Flexible Membrane Liner
GCL	-	Geosynthetic Clay Liner
GWQB	-	Ground Water Quality Bureau
HDPE	-	High Density Polyethylene
K_{sat}	-	Hydraulic Conductivity (i.e., permeability)
LUVD	-	Las Uvas Valley Dairies
MSW	-	Municipal Solid Waste
NMAC	-	New Mexico Administrative Code
NMED	-	New Mexico Environment Department
OCD	-	Oil Conservation Division
P.E.	-	Professional Engineer
PVC	-	Polyvinyl Chloride
SWB	-	Solid Waste Bureau
USDA	-	United States Department of Agriculture
USEPA	-	United States Environmental Protection Agency
UV	-	Ultraviolet Light (a component of sunlight)