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 materials.

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 and 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 as 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 Subgrade Preparation

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.

!["Stair-Step" Cut Made into Old Section of Liner to Tie In New Liner with Old Liner](image)

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.
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 ($\gamma$), 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 ($\gamma_d$), which is sometimes called the dry density, is calculated as follows:

$$\gamma_d = \frac{\gamma}{1 + w}$$  \hspace{1cm} (2.1)

The ($w$, $\gamma_d$) 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 ($\gamma_{d,\text{max}}$) occurs at a water content that is called the optimum water content, $w_{\text{opt}}$ (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.
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:

\[ w = \frac{W_w}{W_s} \]

\[ \gamma_d = \frac{W_s}{V} \]
\[ \gamma_d = \gamma_w / [w + (1/G_s)] \]  

(2.2)

where \( G_s \) is the specific gravity of solids (typically 2.6 to 2.8) and \( \gamma_w \) 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>
<thead>
<tr>
<th>Compaction Procedure</th>
<th>Number of Lifts</th>
<th>Weight of Hammer</th>
<th>Height of Fall</th>
<th>Compactive Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard</td>
<td>3</td>
<td>24.5N (5.5 lbs)</td>
<td>305 mm (12 in.)</td>
<td>594 kN-m/m(^3) (12,375 ft-lb/ft(^3))</td>
</tr>
<tr>
<td>Modified</td>
<td>5</td>
<td>44.5N (10 lbs)</td>
<td>457 mm (18 in.)</td>
<td>2,693 kN-m/m(^3) (56,250 ft-lb/ft(^3))</td>
</tr>
</tbody>
</table>

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:
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:

\[
P = \frac{\gamma_d}{\gamma_{d,\text{max}}} \times 100\%
\]

where \(\gamma_d\) 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 \((\gamma_{d,\text{max}})\) 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 \( w_{\text{opt}} \) and \( \gamma_{d,\text{max}} \) 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 \( w_{\text{opt}} \) and \( \gamma_{d,\text{max}} \). As an approximate guide, a relatively homogeneous borrow soil would be considered a material in which \( w_{\text{opt}} \) does not vary by more than \( \pm 3 \) percentage points and \( \gamma_{d,\text{max}} \) 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 \( w_{\text{opt}} \) and \( \gamma_{d,\text{max}} \). 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 \( w_{\text{opt}} \) and \( \gamma_{d,\text{max}} \).

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](image)

Molding Water Content \( (w) \)

Dry Unit Weight \( (\gamma_d) \)

\( \gamma_{d,\text{max}} \)

\( P \gamma_{d,\text{max}} \)

\( w_{\text{opt}} \)

Acceptable Zone

Zero Air Voids Curve

Specified Range

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).
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)](image)
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 ≤ A ≤ 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 ≤ 1 \times 10^{-7} \text{ 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.
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 $1 \times 10^{-7}$ cm/s.
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.

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 of 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](image-url)
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)
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).
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 (\(\gamma_{d,A} = \gamma_{d,B}\)) 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 \(\gamma_{d,A} = \gamma_{d,B}\). The percent compaction for the two compacted specimens is computed as follows:

![Graph showing compaction curves for Energy A and Energy B](image)

**Figure 2.21 - Illustration of Why Dry Unit Weight Is a Poor Indicator of Hydraulic Conductivity
for Soil Compacted Wet of Optimum**
\[ P_A = \gamma_{d,A}/[\gamma_{d,\text{max}}]_A \times 100\% \]

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

Since \( \gamma_{d,A} = \gamma_{d,B} \) but \([\gamma_{d,\text{max}}]_B > [\gamma_{d,\text{max}}]_A\), 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 \((w_{\text{opt}}, \gamma_{d,\text{max}})\) points for compaction curves developed on the same soil with different compactive energies (Fig. 2.22). The greater the percentage of actual \((w, \gamma_d)\) 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 \((w, \gamma_d)\) 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.

![Good Bonding of Lifts vs Poor Bonding of Lifts](image)

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

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.
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.

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/m³).
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.
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.
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.