Embankment Construction Using Shale

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Research Report KTC-98-2

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in cooperation with
Transportation Cabinet
Commonwealth of Kentucky

and

Federal Highway Administration
U.S. Department of Transportation

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January 1998
### Abstract

Shales have been used extensively in the construction of highway embankments, and other earthen structures, because of the vast amounts of these materials located in many areas of the country and the lack of economical and alternate available materials. Because shales exhibit a wide range of engineering properties and behaviors, many problems have occurred. Numerous shale embankment failures have occurred generally some 1 to 10 years after construction. Settlements of 1 to 3 feet (0.3 - 0.9 m) have been observed in many old embankments and required numerous asphaltic overlays. Shale embankments that settle continuously have been observed to fail eventually. Each year millions of dollars are spent repairing embankments built with shales. This report presents a discussion of some of the research conducted by the University of Kentucky Transportation Center in the seventies and eighties and attempts to address some of the problems that arise in constructing shale embankments. A brief overview of the engineering properties of shales located in Kentucky is presented. Some important factors that need to be considered in designing and building shale embankments are briefly discussed. Finally, a description of the construction of three experimental shale embankments in 1986 is given. These embankments were constructed to evaluate a special shale compaction provision adopted by the Kentucky Transportation Cabinet to avoid large long-term settlements and instabilities. Observations of long-term settlements of the embankments are presented.
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Numerous problems have been encountered when constructing highway embankments and cuts through shale formations. Embankment failures occur due to slope instability and large settlements. Uneven pavements occur due to weathering of shales and differential settlement of shale embankments. Weathering of highway shale cuts causes premature filling of drainage ditches and rock falls have occurred commonly throughout Kentucky and other areas of the world. Large expenditures have been required for major remedial work, as well as for routine maintenance. For example, major repairs of shale embankments of portions of Interstates 71 and 75 located in the northern regions of Kentucky have exceeded 120 million dollars over the past two decades. These shale embankments were constructed of Kope and Fairview Shales of the Ordovician Geologic Age. The main objectives of this report are to discuss important aspects of past shale research pertaining to the construction of highway embankments using shale materials and to present observations and results of long-term monitoring of experimental shale embankments constructed about one decade ago.

In particular, an overview of shales located in Kentucky and their general engineering properties (index and physical properties) are described. Kentucky contains large amounts and a wide variety of shales that have very poor engineering properties. As shown by laboratory and field test results, shear strength, bearing strength, settlement behavior, and durability of compacted shales change dramatically when exposed to water. Settlements increase while shear strength, bearing strength and durability decrease as water is absorbed by shales. Some important factors to consider in the design and construction of shale embankments are presented. These factors include geology, water content, durability, strength, settlement behavior, compaction and lift thickness, laboratory compaction of shales, and the embankment foundation. Successful use of shale materials requires good planning, design, and construction. Two very important considerations include using thin lifts and heavy compactors (and disks) to break down the shales. When hard rocks and durable shales are used as rock fill, the amount of soil (minus No.4 material) should be limited to about 20 percent, or less.

Based on an analysis of field compaction techniques used at the three experimental test fills constructed of intermediate and soil-like shales and compacted with extra heavy compactors, the special compaction provision (which was formulated in the early eighties) shown in the Appendix of this report was generally successful. The heavy compactors included a tamping-foot roller and a vibratory roller. Slaking with water, using heavy compactors, and using a heavy-duty disk proved successful in breaking down or degrading the intermediate and soil-like shales. Results of field and laboratory gradation tests performed on shale samples from the test fills show that the percent passing the 3/4-inch (19-mm) and No. 4 (4.75-mm) sieves averaged about 88 and 60 percent, respectively. Compaction requirements were generally met. Relative compaction generally averaged approximately 95 percent. Field water contents, after adjusting for oversized material, generally met specifications.

Field dry densities and water contents from the nuclear gage averaged about 2 lbs/ft³ (32 kg/m³) and 2 percent higher than values from the sand cone. The tendency of the nuclear gage to register
Executive Summary

slightly higher values than those obtained from the sand cone tests was probably due to the hydrocarbons in the black and gray shales at the site. It is recommended that nuclear gages be calibrated against the sand cone on project materials and that adjustments be made to values obtained with the nuclear gage. The drive sampler yielded average values much lower than the sand cone or nuclear gage. However, average water contents obtained with the drive sampler were similar to those from the sand cone. For measuring dry densities of intermediate and soil-like shales, the drive sampler is not recommended because of the ends of the samples cannot be trimmed smooth.

Results of observations of the three experimental shale test embankments constructed about one decade ago using experimental compaction procedures described below and in the Appendix are discussed. The results of settlement observations performed over a period of some ten years show that settlements of shale embankments can be minimized using the experimental compaction procedures formulated in the early eighties. Long-term settlements of well-compacted shale embankments as high as 65 feet (19.8 m) in height were generally less than about 6-7 inches (15.2-17.8 cm) in a period of about 27 years. Previous observations show that similar embankments compacted as loose fill in lifts as large as 3 feet (0.91 m) usually settled from 1 to 3 feet (0.30 to 0.91 m) in periods ranging from 1 to 10 years. Basically, good results can be obtained by compacting shales in about 8-inch (20.3 cm) loose lifts using heavy compactors. The minimum recommended weight for the static tamping-foot roller is 60,000 pounds (27.2 metric tons). The minimum, recommended total compactive effort for the vibratory tamping-foot roller is 55,000 pounds (24.9 metric tons).

Findings and knowledge acquired from several past studies and the results of long-term monitoring of three experimental shale embankments were valuable in formulating and influencing design standards of the Alexandria-Ashland (AA, or KY 9) highway in Northern Kentucky. This highway involved some 125 miles (201 km) of new construction that passes through the Kope and Crab Orchard shale formations. As previous studies have shown, numerous embankment and settlement failures occurred on stretches of Interstates 75 and 71 in Northern Kentucky that pass through the Kope and Fairview shales and roadways that pass through the Crab Orchard Formations. For example, some 120 embankments on a stretch of I 75 were identified as failures requiring remedial work. As noted previously, some 120 million dollars have been spent repairing failures on I 75 and 71--repair costs have averaged some 1.5 to 2 million dollars per mile. In a series of meetings held at the Division of Materials (Geotechnical Branch) of the Kentucky Transportation Cabinet, four general recommendations (Hopkins 1985) were made in the mid-eighties regarding the design of the AA roadway. These were, as follows:

- All shale embankments should be compacted using the Special Shale Compaction Provision (see Appendix), especially fills over about 40 - 50 feet (12 - 15 m) in height.

- For embankments over about 30 - 40 feet (9 - 12 m) in height, it was recommended that minimum slope designs of about 2.5 horizontal to 1 vertical or 3 horizontal to 1 vertical be considered-- values of $\phi'$ and $c'$.
Executive Summary

Normally obtained from triaxial tests for the clayey shales of that area should be reduced in the slope stability analysis.

- Good drainage should be placed at the base of large shale fills.
- All pavement soil and shale subgrades should be stabilized.

Generally, those recommendations were adopted. Perhaps, the most important recommendation pertaining to the compaction of the shales was generally observed. After some 11 years, no serious problems have been reported concerning the shale embankments on this roadway. Based on this observation, and the lack of massive failures (of the type that have occurred on I 71 and 75 in similar materials) of the shale embankments on the AA-Highway (KY 9), which appears to be the result of using the special shale compaction provision, some 188 to 250 million dollars (1.5-2 million repair dollars/mile x 125 miles) will potentially be saved.

Use of the special shale compaction provision will save millions of dollars each year by preventing large shale embankment failures.
Introduction

Problem

Over the last four decades, the geometry of modern highways has been vastly changed and improved. These changes have required constructing large highway embankments, especially in mountainous areas. As the heights and widths of modern highway embankments have increased, more materials, even undesirable materials such as certain types of shales, have been required. Large quantities and numerous varieties of shales are located in Kentucky, as well as many other areas of the country. Consequently, construction of highway embankments using shale materials is necessary in many instances because of their availability and the lack of more suitable and economical alternate construction materials. These materials have been used extensively in the construction of highway embankments.

Shale may be defined as a fine-grained, indurated, detrital sedimentary rock formed by the consolidation, such as compression or cementation, of clay, silt, or mud. Examples of shale include claystone, siltstone, and mudstone. These materials are characterized by a finely stratified structure, or fissility, that is parallel to the bedding. The composition of shale generally consists of about one-third of clay minerals, one-third of quartz, and one-third of miscellaneous materials. Generally, when first excavated most shales are very hard and have the appearance of sound, durable rock.

As shown in Figure 1, the geology of Kentucky consists of Ordovician, Silurian, Devonian, Mississippian, Pennsylvanian, Cretaceous, Tertiary, and the Quaternary system deposits. With the

![Large quantities of clay shales](image-url)
exception of Quaternary Deposits, shales are associated with all periods. Shales of the Ordovician, Silurian, Devonian, and Mississippian have been involved in many road-building problems. Certain formations have been particularly troublesome. The Kope and Fairview Formations (interbedded shales and limestone) of Northern Kentucky have caused embankment problems on Interstate 75 near Covington and Interstate 71 and have caused extensive failures of KY 8 (along the Ohio River, east of Covington). In the Knobs region to the east of the Cincinnati Arch, the Crab Orchard Formation (Silurian) has been involved in many embankment failures. Many Mississippian shales behave poorly. For instance, the lower Borden Formation (Nancy and New Providence Members) tends to cause many pavement problems. The Henley shale bed caused construction problems on Interstate 64 east of Morehead. Also, shales of the Eastern (Pennsylvanian) and Western (Mississippian) Kentucky Coal Fields, such as the Breathitt Formation (near Jackson KY) and Tradewater (Western Kentucky) have been troublesome.

Numerous problems have been encountered when constructing highway embankments and cuts through shale formations. Embankment failures occur due to slope instability and large settlements. Uneven pavements occur due weathering of shales and differential settlement of shale embankments. Weathering of highway shale cuts causes premature filling of drainage ditches and rock falls have occurred commonly throughout Kentucky and other areas of the world. Large expenditures have been required for major remedial work, as well as for routine maintenance. For example, major repairs of shale embankments of portions of Interstates 71 and 75 located in the northern regions of Kentucky have exceeded 120 million dollars over the past two decades. These shale embankments were constructed of Kope and Fairview Shales of the Ordovician Geologic Age. Approximate repair costs have averaged some 1.5 to 2 million dollars per mile for the portions of those interstates passing through the clayey shale formations. Several additional major repairs of shale embankments on I 71 will be required in the future.

Shales, when used as a construction material, create major problems because in many cases they tend to degrade from a hard, or indurated mass, to a fine-grained mass of soil. This degradation of normally, hard intact shale frequently produces weak, or low strength clays and silts. Degradation of shale particles in embankments frequently occurs over a long period of time and many problems do not occur until several years after construction. A typical view of the large settlements of many shale embankments of I 71 and I 75 is shown in Figure 2. Settlements of 1 to 3 feet (0.3-0.9 m) in shale embankments have frequently been observed and required overlays. Mudjacking of concrete bridge approaches, or the overlaying of asphaltic bridge approaches have been required. In numerous cases, the settlements of the embankments continue and eventually slope failures occur.

Objectives

To address the problems of the failures of shale embankments, the Kentucky Transportation Center at the University of Kentucky, in cooperation with the Kentucky Transportation Cabinet, conducted several research studies during the seventies and eighties. The main objective of this report is to discuss briefly aspects of the past shale research. In particular, the general engineering properties of shales located in Kentucky are described. Some important factors to consider in the design and construction of shale embankments and the long-term observations of three experimental
Embankment Construction Using Shales—Hopkins and Beckham

Figure 2. Typical view of large shale embankment settlement observed on such roadways as Interstates 71 and 75.

Shale test embankments constructed about one decade ago using compaction procedures described below and in the Appendix are discussed.

Scope

Although shales have played major roles in causing the instability of cut slopes and unstable highway subgrades, this report mainly focuses on the construction of embankments using shales. Basically, this report deals with research performed in the seventies and eighties at the University of Kentucky Transportation Center and attempts to provide some evaluation of provisional shale compaction specifications developed in that period of time.

Overview of the Engineering Properties of Kentucky Shales

Sample Collection

To obtain an overview of the engineering properties of shales located in Kentucky, some forty types of shale materials, which exhibit a wide range of field performances, were selected from various physiographic regions and geologic periods for testing. Samples were obtained from the Jackson Purchase, Western Coal Field, Knobs, Mississippian Plateaus, Bluegrass, and Eastern Coal Field Physiographic Regions of Kentucky. An effort was made to collect both hard and soft shales and shales that were considered to be durable and nondurable. The samples were obtained using hand tools and a drill rig. Shale pieces for bag samples were loosened using a rock hammer and mattock. The unweathered samples were generally collected from highway cuts. During sampling, freshly dug and unweathered shale specimens were placed in sealed containers to obtain natural in-situ moisture contents. The testing program consisted of two categories:

- Index tests
- Physical properties.
Physical properties of shales located in Kentucky are briefly described below. Although this discussion focuses only on shales in Kentucky, the varieties of shales and engineering properties are representative of many shales found in adjacent states and other East Central States of the country.

Testing Program

Index Properties

The index tests consisted of Atterberg limits, in-situ water contents (unweathered), specific gravity, and particle-size analysis. ASTM standards, where applicable, were followed in the testing program. The index tests were performed on unweathered shales. The shales were crushed using a rock crusher. Liquid and plastic limits were performed on material passing the minus No. 40 (0.425-mm) sieve while the grain-size analysis was performed on material passing the minus No. 10 (2.0-mm) sieve. Results of the index tests and slake-durability tests of unweathered tests are summarized in Table 1. Results of tests performed on selected weathered shales are summarized in Table 2. The weathered materials were collected from talus piles of highway cuts.

Liquid and plastic limits of the unweathered shales ranged from 19 to 38 percent and non-plastic (NP) to 18 percent, respectively. Liquid and plastic limits of the selected weathered shales ranged from NP to 40 and NP to 15, respectively. Natural in-situ water contents of the materials (unweathered) ranged from 1.7 to 23.2 percent. Particle sizes finer than 0.002-mm size of unweathered and weathered shales ranged from 3 to 38 percent and 8 to 39 percent, respectively. The index properties of the weathered and unweathered shales were similar. Classifications of the weathered shales were ML and CL according to the Unified Classification System. Specific gravities of the weathered shales ranged from 2.52 to 2.85. The percentages of particles smaller than the 0.002 mm of weathered and unweathered shales are compared in Figure 3. Generally, the percent finer of the weathered shales averaged about 3 percent more than the percent finer of the processed unweathered shales. However, the values were similar.

Physical Properties

Slake-Durability Properties

Slake-durability tests were performed on unweathered shales.

Figure 3. Comparison of the percent finer than 0.002-mm size particles of unweathered and weathered shales.
Table 1. Engineering properties of Kentucky (unweathered) Shales.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Geologic Name</th>
<th>Geologic Period</th>
<th>In-Situ M. C. (%)</th>
<th>LL (%)</th>
<th>PI (%)</th>
<th>Finer than 0.002 mm (%)</th>
<th>SDI&lt;sub&gt;10&lt;/sub&gt; (%)</th>
<th>SDI&lt;sub&gt;60&lt;/sub&gt; (%)</th>
<th>Jar Slake Durability Index</th>
</tr>
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<tbody>
<tr>
<td>1-2</td>
<td>New Albany</td>
<td>Devonian</td>
<td>1.7</td>
<td>22</td>
<td>NP</td>
<td>3.3</td>
<td>99</td>
<td>99</td>
<td>6</td>
</tr>
<tr>
<td>17-2</td>
<td>Sunbury</td>
<td>Mississippian</td>
<td>2.8</td>
<td>22</td>
<td>1</td>
<td>7.4</td>
<td>99</td>
<td>98</td>
<td>6</td>
</tr>
<tr>
<td>13-1</td>
<td>Hance</td>
<td>Pennsylvanian</td>
<td>2.9</td>
<td>21</td>
<td>5</td>
<td>12.4</td>
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<td>91</td>
<td>5</td>
</tr>
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<td>8-1A</td>
<td>Lower Clays Ferry</td>
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<td>7</td>
<td>18.2</td>
<td>83</td>
<td>75</td>
<td>5</td>
</tr>
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<td>Upper Drakes</td>
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<td>20</td>
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<td>Crab Orchard (No.3)</td>
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<td>11-3</td>
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<td>Pennsylvanian</td>
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Table 2. Engineering properties of Kentucky (weathered) Shales

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<th>PI (%)</th>
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<th>Percent Finer than 0.002 mm</th>
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Following procedures described elsewhere (Hopkins and Gilpin, 1981). Some ten different testing procedures were initially examined (Hopkins and Gilpin, 1981). Results of the procedure proposed by Gamble (1971)-a modification of the original procedure proposed by Franklin and Chandra (1972) and the procedure essentially used by most agencies-- are shown in Table 1. Also, the results of a testing procedure proposed by Hopkins (1981;1986) are shown in Table 1. In the former standard procedure, two ten-minute cycles are used. The test is performed on ten pieces of oven-dried shale each having a dry mass of 40 to 60 grams. In the latter case, one 60-minute cycle and air-dried material are used. The latter test requires less total testing time and uses only one cycle of testing. A wide variety of slake-durability indices (SDI) and jar slake numbers (I) were obtained for Kentucky shales.

Generally, the method that uses one 60-minute cycle and air-dried material appeared to yield a better distribution of slake-durability indices for the wide range of shales tested than the procedure that uses two ten-minute cycles and oven-dried material, as illustrated in Figure 4. Apparently, oven-drying the shales tends to...
make the shales more resistant to slaking. However, since the latter procedure became embedded into the testing community, there was no way to change the SDI specifications. The slake-durability tests were performed in an apparatus—first prototype in Kentucky—designed and built by the University of Kentucky Transportation Center. To promote the use of this testing method of shales, working drawings of this apparatus were supplied to numerous consultants, other universities, and governmental agencies (Hopkins and Gilpin, 1981).

**Moisture-Density Relations**

Standard compaction tests were performed on the weathered shales. Three different compactive energies were used. Tests were performed using ASTM D 1557-78, Method A. This procedure is referred to as modified compaction. Standard compaction was performed following ASTM D 698-78, Method A. A low energy compactive effort was used and consisted of using a 1.84 pound (0.83 kg) rammer, a 12-inch (30.5-cm) drop, three layers, and 15 blows per layer. A 4-inch (101.6-mm) diameter mold was used. The relationship between maximum dry density and optimum moisture content of the shales at different compactive energies is shown in Figure 5. Values of maximum dry density and optimum moisture content ranged from 91 to 138 lbs/ft.³ (1,457 - 2,210 kg/m³) and 6.6 to 24.6 percent, respectively. These tests showed that moisture-density tests could be performed on shales having a wide range of slake-durability indices. Essentially all of the shales exhibited soil-like characteristics.  

![](image)  

**Figure 5.** Moisture-density relationship of shales.

![Graph showing moisture-density relationship of shales.](image)  

**Figure 6.** Comparison of soaked and unsoaked values of KYCBR.
Bearing Ratio

California Bearing Ratio (CBR) tests were performed on the weathered shale samples following procedures outlined elsewhere and according to Kentucky Method KM-64-501-76. Only material passing the No. 4 (4.75-mm) sieve was used. Soaked and unsoaked values of KYCBR are compared in Figure 6. Unsoaked values of KYCBR ranged from 15 to 46 while the soaked values of KYCBR ranged from about 1.5 to 33. Values of KYCBR of the clay shales decreased dramatically after soaking. To illustrate this decrease, a KYCBR specimen of the Kope shale from Northern Kentucky was soaked for 2.5 years. The KYCBR decreased from a value of 33 to 0.5 after soaking, as shown in Figure 7. A useful relationship between KYCBR and natural water content of the shales for use in preliminary design is shown in Figure 8.

Triaxial Shear Strengths

To examine the range of shear strengths of compacted Kentucky shales, consolidated-undrained tests with pore pressure measurements were performed on nine selected (weathered) shales. The shales that were selected exhibit a wide range of slake-durability indices. The triaxial specimens were compacted to the maximum dry densities and optimum moisture contents determined from three different compactive energies. The specimens were remolded to standard compaction (ASTM D 698-78), modified compaction (ASTM D 1552-78), and a low-energy compaction. This low-energy procedure involves
the use of a 1.84 pound (0.83 kg) hammer, a 12-inch (30.5-cm) drop, three layers of material, and 15 blows per layer. Results of these tests, as well as slake-durability values are shown in Table 3. The effective stress parameter, $\phi'$, of the weathered shales ranged from about 22 degrees to 40 degrees for SDI10 values ranging from 1 to 99 percent. As shown in Figure 9, the value of $\phi'$ for a given type of shale varied slightly for different compactive energies. However, as shown in Figure 10, the values of $c'$ generally increased significantly when the compactive energy increased. Hence, using a large compactive energy greater than standard compaction indicated that the strengths of shales could be increased significantly in the field. General relationships between slake-durability indices and $\phi'$ are shown in Figure 11.

**Design and Construction Considerations**

Some important considerations involved in the design of shale embankments against instability and large settlements are as follows:

- Geology
- Water
- Durability
- Strength
- Settlement
- Compaction and Lift
- Thickness
- Foundation.
Table 3. Effective stress parameters obtained from consolidated-undrained triaxial compression tests with pore pressure measurements

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<td>( c' )</td>
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From Stress-Difference Criterion

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From Stress-Ratio Criterion

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</table>

Geology

During highway corridor subsurface studies, rock core samples are normally obtained to identify geologic formations and to determine the types of materials that will be used in the highway embankments. Identification of different geologic formations and slake-durability indices is useful in designing the embankments because large quantities of the geologic formations will be used to construct the embankments. Such factors as the orientation and thicknesses of the layers will determine how the shales and other layers of rocks will be excavated and placed. For example,
where layers are oriented at large angles, the layers may have to be blasted and compacted as soil-like materials. Generally, a minimum of two rock core borings should be performed in each cut or borrow area to define the weathered shale depths and the thickness and inclination of different strata. However, where layers are fairly horizontal, one hole may be sufficient.

Water

To detect water-bearing layers, it may be necessary to extend the depth of the borings in the cut sections. Groundwater levels in the core borings should be monitored to define potential subsurface seepage that could enter the shale embankments. Moreover, during the design phase, consideration should be given to constructing surface structures that will convey surface runoff from the shale embankments. The intent here is to prevent runoff from spilling over onto the fill which could seep into the shale fill. All areas of groundwater seepage should be noted.

It is essential that efforts are made to prevent the infiltration of surface and subsurface water into the shale embankment. Use of drainage blankets and filter fabrics at the base of shale embankments can prevent the infiltration of subsurface water from deep residual soil overburdens and seepage from rock formations. When embankments are placed on side hill situations, the embankment acts as a dam and groundwater levels build up in the fill. Hence, pore pressures increase and reduce stability. Generally, for fill heights greater than the 40 or 50 feet (12 - 15 m), drainage measures at benches should be considered. The intent of benching is to provide a level surface for compacting the first lift of shale and to improve stability.

Use of surface drainage structures can control the amount of surface water entering the embankment. Oftentimes, it has been observed that runoff from the pavement surface spills over onto the shale slope and in cases involving soft shales, the materials soften and become unstable near the surface of the slope. Numerous shallow failures have occurred because of this situation. In addition to using surface drainage structures, the designer should consider using layers of geosynthetic materials spaced at appropriate heights throughout to reinforce the outer slope of the shale fill. Usually, the outer slope does not receive adequate compaction because compaction equipment cannot operate near the edge of the shale fill slope. Hence, the slope contains a zone of weak material.

Durability

The long-term durability of shales is one of the primary considerations in the design of shale embankments. The shales obtained from the core borings should be classified according to their long-term durability. For purposes of classification, slake-durability and jar slake tests should be performed. Shales classified as mechanically hard (SDI_{10} > 95%; I_{j} = 6) (Changed to SDI_{10} > 90% recently) and durable could be used as rock fill. These materials could be placed in lifts of not more than 24 inches (61 cm). Shales classified as soft and nondurable (SDI < 50%; I_{j} <2) should be compacted as a soil in thin lifts approximately (8-12 inches, 20 - 30.5 cm). Intermediate shales (50 % < SDI < 95 %; I_{j} = 3-5) classified as hard and nondurable are difficult to compact and require heavy equipment to compact.
Table 4. Suggested shear strength parameters based on slake-durability indices

<table>
<thead>
<tr>
<th>Slake-Durability Index (SDI) (%)</th>
<th>Jar Slake Test (I) (%)</th>
<th>Description of Shale</th>
<th>Shear Strength Parameters</th>
<th>Undrained Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>( \phi' ) (degrees)</td>
<td>( c' ) (lbs/ft²)</td>
</tr>
<tr>
<td>SDI &lt; 50</td>
<td>( \leq 2 )</td>
<td>Soil-like Shale</td>
<td>20-25</td>
<td>200</td>
</tr>
<tr>
<td>50 &lt; SDI &lt; 95</td>
<td>3-5</td>
<td>Intermediate Shale</td>
<td>26-30</td>
<td>200</td>
</tr>
<tr>
<td>SDI &gt; 95</td>
<td>6</td>
<td>Hard Shale</td>
<td>35-45</td>
<td>0</td>
</tr>
</tbody>
</table>

**Strength**

In the design of slopes of highway shale embankments, a knowledge of the effective stress (peak) strength parameters, \( \phi' \) and \( c' \), the angle of internal friction and cohesion, respectively, of the compacted shales as they will exist in the embankment is essential to determine stability of the slopes. These parameters may be estimated by performing triaxial tests on compacted specimens of the shales that are to be used to construct the embankment. However, this can be a time-consuming task. This approach should probably be reserved for important embankments over 40 to 50 feet (12-5 m) in height, bridge approach embankments, or in other special cases. Oftentimes, the parameters are estimated, as shown in Table 4.

Additionally, the relationship between the natural, in-situ water content (unweathered) and \( \phi' \) may be used, as shown in Figure 12. The cohesive component, \( c' \), for hard-like shales could be assumed to be zero while for soil-like and intermediate shales a value of about 100-200 lbs/ft² (1,602 - 3,204 kg/m²) could be assumed. The peak value of \( \phi_p' \) can also be estimated from a relationship between \( \phi' \) and plasticity index, PI, or (Hopkins 1982):

\[
\phi_p' = 44.7 - 12\log PI
\] (1)
The residual shear strength parameter, $\phi_r$, may be approximated by the following relationship:

$$\phi_r' = 68.2 - 30.2 \log CF$$

(2)

where $CF$ is the clay fraction, or the percent finer than 0.002-mm size particles (Hopkins, 1986).

**Settlement**

Prior to about 1966, slopes of 2 horizontal to 1 vertical (2H:1V; 26.6 degrees) were commonly used in the design of all embankments regardless of the height of the embankment. However, numerous instability problems and a pronounced number of failures occurred in areas of Kentucky containing nondurable, clayey shales, such as the Kope and Crab Orchard shales. Generally, slopes of 2 horizontal to one vertical will be adequate for embankments less than about 40 or 50 feet (12 - 15 m), as shown in Figure 13 (Hopkins 1985). Field measurements shown in Figure 14 indicate that the long-term factor of safety should be equal to or greater than approximately 1.5 to minimize settlement. Hence, special design and construction are suggested in the following cases:

- Shale embankments greater about 40-50 feet (12 - 15 m)
- Shale embankments subject to rapid draw down
- Shale embankments subject to subsurface infiltration
- Bridge approach shale embankments.

In those cases, slopes milder than 2H:1V may be required. For example, in cases where settlements must be minimized, flatter slopes, extra compaction, and base drainage may have to be considered. However, as the embankment height increases above 40 to 50 feet (12 - 15 m), settlements increase rapidly as shown in Figure 13. Hence, special consideration must be given to the slope design. In cases where nondurable shales are used, slope designs milder than 2H:1V should be considered, especially in situations where materials
Embankment Construction Using Shales—Hopkins and Beckham

Figure 14. Long-term settlement of several case studies as a function of the long-term factor of safety (Hopkins 1985).

Consolidation rings are too small to accommodate the large particles. In a relatively dry state, compacted shales exhibit small compression. However, as shown by Strohm et al. (1981), compacted shales (using large 6-inch -- 152.4-mm -- diameter molds) when soaked may exhibit large and excessive settlements. An approximate correlation of slake-durability indices and compression of soaked shale specimens has been given by Strohm et al.

Secondary compression and settlement due to shear strain may occur even for well compacted fills. These settlements may amount to approximately 0.3 to 0.6 percent of the fill height over a period of 15 to 20 years (see NAVFAC, 1982). As shown in Figure 13, settlements of embankments greater than about 50 feet (15 m) become significant, especially when they are not well compacted. Embankments constructed of clayey shales exhibit the greatest settlements. Large embankments settlements observed on I 75 also confirm these observations. Many of the fills in excess of 50 feet (15 m) settled some 1 to 3 feet (0.3 - 0.9 m) over a period of some 1 to 10 years.

An approximate method of estimating the settlement of embankments is given by the following empirical equation (Hopkins, 1985):

$$H_{50} = 12H_0 10^{(1.5 \log F_r - 4.68)(\log (t_s / t_o))(\text{inches})}$$

(3)
where

\[ H_s = \text{estimated embankment settlement in inches}, \]

\[ H_e = \text{height of embankment in feet}, \]

\[ F_s = \text{ratio of } H_e \text{ to the long-term factor of safety}, \]

\[ t_c = \text{time of placement of pavement or completion time (the time between the start of construction and placement of the pavement), and} \]

\[ t_{ss} = \text{time at the end of significant secondary compression and shear strain of the embankment (assumed value)}. \]

In developing this empirical equation, observed long-term settlements (projected to 27.4 years) of several embankments were plotted as a function of the long-term factor of safety. Observed pore pressures and slope-inclinometer data obtained over a period of many years were used in stability analysis of those sites. Also, shear strength parameters, \( \phi' \) and \( c' \), were obtained from triaxial tests performed on undisturbed samples from the embankments several years after construction. Settlements of approach foundations and embankments were monitored for several years. Embankment settlement was obtained by subtracting the settlement of the foundation from the total settlement measured at the top of the embankment. The settlements were plotted as a function of the logarithm of time. The relationship between settlement and the log of time was linear. Therefore, the relationship could be projected to 27.4 years. Settlements occurring in the next log cycle (27.4 to 274 years) could be considered insignificant. Curves given by Equation 3 are shown graphically in Figure 15 for different factors of safety. The estimated settlement varies widely as the factor of safety varies. For example, for an embankment 60 feet (18 m) in height, the estimated settlements would be 1.5, 4, 5, and 8 inches (38.1, 101.6, 127.0, and 203.2 mm) for factors of safety of 3, 2, 1.5, and 1, respectively. According to this concept, as the factor of safety increases, the settlement decreases. This approach can be used to design and minimize the long-term settlement of bridge approach embankments.

Figure 15. Estimated settlement as a function of embankment height (Hopkins 1985).
Compaction and Lift Thickness

The importance of meeting unit weight and lift thickness criteria is illustrated by data published by Lutton (1977) and shown in Figure 16. Indices obtained from slake-durability tests performed on shales from various highway embankments (fifteen states) were plotted as a function of lift thickness. The criterion lines, or envelopes, shown are attempts to separate problem embankments from non-problem embankments. Those data indicate that as the slake-durability indices increase and lift thicknesses decrease the susceptibility of a shale embankment to develop settlement and instability decreases. These data demonstrate that materials excavated from shale cut formations requires good compaction to prevent instability and large settlements.

When durable rock and durable shales are placed as clean rock, the amount of soil (minus No.4 -- 4.75 mm) should be limited to 20 percent or less for lifts as thick as 2 feet (0.6 m). This requirement insures adequate contact of the durable rocks. One of the most difficult situations occurs when both nondurable shale and hard durable rock are present in the cut, or borrow area. Obtaining adequate compaction is very difficult when the hard durable rock is too large because the hard rock cannot be broken down by the compaction equipment. In this case, the percentage of plus 6-inch (15.2 cm) size should be limited to about 20 percent or less. The maximum size of rock in the plus 6-inch (15.2-cm) material should not be greater than 12 inches (0.3 m). Controlled blasting to beak down the materials into small fragments may be necessary.

Laboratory Compaction of Shales

To control field compaction of shales, laboratory moisture-density tests are necessary. Since shales exist in an indurated and massive form in nature, a question arises concerning how a given shale should be broken down or what gradation should be used in the laboratory tests that would simulate
the gradation in the field. Consequently, in approaching this problem, laboratory moisture-density
tests were performed using different methods of preparing the shale samples. In one series of
compaction tests, the shales were crushed using a rock crus her and tests were performed on material
passing the 3/4-inch (19
-mm) sieve. In another
series, only material
passing the No. 4 sieve
(4.75 mm) was used.

In ASTM (American Standards for Testing Materials) Method D
698-78, four procedures
are cited for determining
the moisture-density
relations of soils and
soil-aggregate mixtures.
 These are listed in
Figure 17. If the
material retained on the
No. 4 sieve (4.75 mm) is
less than seven percent,
then Methods A and B
may be used. In Method
A, a 4-inch (101.6-mm)
mold is used while in
Method B, a 6-inch
(152.4-mm) mold is used. In both methods, only material passing the No. 4-sieve is used. The plus
No. 4 (4.75 mm) material is discarded. When the material retained on the No. 4 (4.75 mm) sieve
is greater than 7 percent, then the ASTM standard recommends using Method C. In this procedure,
a 6-inch (152.4-mm) mold is used. This method is used when the material retained on the 3/4-inch
(19-mm) sieve is less than 10 percent. The plus 3/4-inch (19-mm) material is discarded. However,
if the material retained on the 3/4-inch sieve is greater than 10 percent, but less than 30 percent, then
the ASTM standard recommends using Method D. In method D, a 6-inch (152.4-mm) mold is used.
When material passing the 3/4-inch sieve is greater than or equal 10 percent, but less 30 percent,
then method D is used. Material passing the 3-inch (76-mm) sieve and retained on the 3/4-inch (19-
mm) sieve is replaced by an equal amount of material passing the 3/4-inch sieve and retained on the
No. 4 (4.75-mm) sieve. The material retained on the No. 4 (4.75-mm) sieve is discarded. Later,
ASTM standards use the 3/8-inch (9.5-mm) sieve in place of the No. 4 (4.75-mm) sieve as a grain
size control.

Figure 17. ASTM 1991 laboratory compaction methods.

If more than 30 percent of the material is retained on the 3/4-inch (19-mm) sieve, then none of the
ASTM Methods (A,B,C, and D–1991) may be used to determine the maximum dry density and
optimum moisture content. Another approach may be used. As given in NAVFAC (1982) the laboratory maximum dry density and optimum moisture content obtained from standard compaction (ASTM D 698-78) may be used as reference values to which results of field density tests of materials having oversized particles may be compared. To adjust the laboratory maximum dry density and optimum moisture content obtained from standard compaction tests for comparison with field density data of materials having oversized particles, the following equations may be used:

\[
\gamma_m = \frac{1-(0.05)F}{F} \frac{F}{162} \frac{1-F}{\gamma_1} 
\]

and

\[
w_j = Fw_g + (1-F)w_0
\]

where:

- \(\gamma_m\) = adjusted maximum dry density (lbs/ft\(^3\))
- \(F\) = fraction of oversized particles by weight determined by sieve analyzes in the field during field density testing (Oversized particles are larger than the maximum size allowed using a given size of mold-- No. 4 (4.75-mm) sieve for a 4-inch (101.6-mm) mold, No. 3/4-inch (19-mm) sieve for a 6-inch (152.4-mm) mold, and No. 2-inch (51-mm) sieve for a 12-inch (305-mm) mold).
- \(w_j\) = adjusted optimum moisture content,
- \(w_g\) = moisture content of oversize particles (obtained from field data), and
- \(w_0\) = laboratory optimum moisture content without oversize particles per cubic feet (bulk specific gravity of 2.60 times 62.4 lbs/ft\(^3\) (1,000 kg/m\(^3\)).

The density of oversize particles is assumed to be 162 lbs/ft\(^3\) (2,595 kg/m\(^3\))—bulk specific gravity of 2.60 times 62.4 lbs/ft\(^3\) (1,000 kg/m\(^3\)). Equations 4 and 5 are suitable for well-graded materials and when the oversize particles are less than 60 percent by weight. The density of oversize particles is assumed to be 162 lbs/ft\(^3\) (2,595 kg/m\(^3\)). Suggested procedures to follow in checking field density and moisture content are shown in Figure 18. Recent ASTM specifications use the 3/8-inch (9.5-mm) sieve in place of the No. 4 (4.75-mm) sieve.
Foundation

Although adequate compaction of shales is essential to prevent instability of the embankment and control settlement, a large number of major slides have occurred at side hill locations when the clays or weathered shales were over stressed. This situation is particularly difficult when the overburden contains over consolidated clays. In numerous studies involving over consolidated clays, the back calculated strengths were generally much lower than strengths obtained from triaxial tests, that is, in both cases, total stress and effective stress analysis yielded much higher factors of safety than the actual factor of safety of one—the case when an embankment fails. Based on an analysis of numerous failures, it appears that when the liquidity index is greater than about 0.4 (Hopkins, 1975, 1986) the effective stress and total stress analysis (based on peak triaxial strength parameters)
yields reasonably reliable results, as shown in Figure 19. However, when the liquidity index of the foundation materials is below 0.4, the stability analysis overestimates the factor of safety. Indeed, numerous studies have shown this to be the case. Use of residual strength oftentimes yields factors of safety that are less than one. Hence, the designer faces a dilemma. Defining the “correct” shear strength requires “engineering experience.” Consideration must be given to using a lower shear strength than that obtained from triaxial tests. Much more research needs to be performed on this aspect.

Experimental Shale Test Embankments

Embarkment Location and Configurations

To test the new provisional shale compaction specifications (See Appendix), three experimental shale test embankments were constructed on KY 11 in Lee and Wolfe Counties in the summer of 1987. Configurations of the embankments and limits of the shale placements are shown in Figures 20, 21, and 22. The three test embankments are located at Stations 288+50, 498+50, and 556+50. Data for a fourth embankment are included, although that embankment was not closely monitored during construction.
This reconstruction project is located approximately 6 miles (9.6 km) south of Natural Bridge State Park. Project alignment begins at Station 260+00 about 1,300 feet (396 m) south of the intersection of existing KY 11 and KY 498. The end of the project is located at Station 576+00 and the total length of the project is about 6 miles (9.6 km). The project is situated in the Eastern Coal Field Physiographical Region. Local relief is about 200 ft (61 m) and reaches from elevation 1,040 (317 m) to 1,240 (378 m). Bedrock along the alignment consists of sandstone, shale, and coal of the Breathitt Formation and sandstone of the Lee Geologic Formation (Pennsylvanian).

**Slake-Durability Indices of Shales Used in Test Embankments**

In each case, the lower portions of the three experimental embankments were constructed of shales that classified either as intermediate (50 < SD<sub>10</sub> < 95 percent) or soil-like (SD<sub>10</sub> < 50 percent). As shown in Figures 23 and 24, slake-durability indices, which are based on 125 SDI tests performed on rock
cores during corridor studies, of approximately 73 percent of the shales used to construct the test embankments had values that ranged from 61 to 97 percent (essentially intermediate shales). SDI<sub>10</sub>-values of only 10 percent of the shales ranged from about 51 to 60 percent (lower intermediate). SDI<sub>10</sub>-values of some 17 percent of the shales were less than 50 percent (soil-like shales). Values of SDI<sub>10</sub> ranged from 10 to 90 percent. Indicated, maximum lift thickness for 73 percent of the shales (61 < SDI<sub>10</sub> < 97 percent) ranges from 12 to 22 inches. For values of SDI<sub>10</sub> below 61 percent, the maximum lift thickness must be smaller to avoid major problems. Although those values are maximum, to insure that no major problems occur, a much smaller lift thickness was selected for the three test fills. A lift thickness of about 8 inches (20.3 cm) was used, although this thickness may have been as much as 12 inches (30.5 cm) on some occasions. Maximum lift thickness which could have been used and target lift thicknesses are summarized in Table 5 and illustrated in Figure 25.

**Special Shale Compaction Provision**

The shale embankments were compacted following the special shale compaction provision shown in the Appendix. The intermediate and soil-like shales were generally compacted in about 8-inch lifts, although this value may have been slightly larger in some instances.
Table 5. Indicated and target lift thicknesses.

<table>
<thead>
<tr>
<th>Station No.</th>
<th>Fill Height (feet)</th>
<th>Reported Factor of Safety</th>
<th>SDI of Fill Shales (%)</th>
<th>Indicated Lift Thickness (Inches)</th>
<th>Target Lift Thickness (Inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>288+50</td>
<td>55</td>
<td>1.4</td>
<td>69</td>
<td>8-14</td>
<td>8-12</td>
</tr>
<tr>
<td>498+50</td>
<td>65</td>
<td>1.5</td>
<td>48</td>
<td>8</td>
<td>8-12</td>
</tr>
<tr>
<td>556+50</td>
<td>65</td>
<td>1.6</td>
<td>89</td>
<td>20</td>
<td>8-12</td>
</tr>
</tbody>
</table>

After a lift of loose material had been spread, water was added, as shown in Figure 26, in an effort to slake the shales. After watering, the shales were broken down using a 24-inch (0.61-m) gang disk, as shown in Figure 27. The special provision shown in the Appendix requires two heavy duty compactors.

The first compactor must be a static tamping-foot roller having a minimum weight of 60,000 pounds (27.2 metric tons). The second compactor is a vibrating tamping-foot roller having a minimum compactive effort of 55,000 pounds (24.9 metric tons). Total compactive effort is defined as that portion of the static weight acting upon the unsprung compaction drum added to the centrifugal force provided by the drum. Compaction equipment used to compact the fills is listed in Table 6. The three test fills were compacted using a CAT® 825 tamping-foot roller weighing 71,429 pounds (32.3 metric tons).

Figure 25. Relationship between slake-durability index and lift thickness (After Lutton 1977).

Figure 26. View of water truck used to slake the shales.
Table 6. Listing of compaction equipment.

<table>
<thead>
<tr>
<th>Equipment Type</th>
<th>Operating Weight (lbs)</th>
<th>Centrifugal Weight (lbs)</th>
<th>Total Operating Weight (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CAT®825 Static Tamping-Foot Roller</td>
<td>-</td>
<td>-</td>
<td>71,429</td>
</tr>
<tr>
<td>Raygo® 4200 (Vibratory Roller)</td>
<td>26,900</td>
<td>50,000</td>
<td>76,700</td>
</tr>
<tr>
<td>Dynapac® CA 25 (Vibratory Roller)</td>
<td>24,400</td>
<td>44,000</td>
<td>68,500</td>
</tr>
<tr>
<td>Ingersoll Rand® SPF 56 (Vibratory Roller)</td>
<td>22,500</td>
<td>42,200</td>
<td>64,500</td>
</tr>
</tbody>
</table>

A minimum of two passes, as specified in the special provision, was used. Actually, more than two passes were used on many occasions. Three different compactors were used at different times to complete the final compaction. These included a Raygo® 4200, Dynapac® CA25, and an Ingersoll Rand® SPF 56. The operating weights of the three different compactors were 26,900, 24,400, and 22,500 pounds (12.2, 12.1, and 10.2 metric tons), respectively. Centrifugal force of the three compactors were 50,000,

44,000, and 42,200 pounds (22.2, 19.9, and 19.1 metric tons), respectively. Total compactive weights were 76,700, 68,400, and 64,500 pounds (34.7, 31.0, and 29.9 metric tons), respectively. The weights of all compactors exceeded the weights required by the special provision. Views of the static tamping-foot roller and a vibratory roller are shown in Figure 28.

Figure 27. View of disk used to breakdown the shales.

Figure 28. View of static tamping-foot and vibratory rollers.
Field Compaction Control

Gradation of Shale Fills

Gradation tests were performed on field samples collected during the field density tests. Samples were obtained from each of the three embankments. The tests were performed to determine the percent of particles passing the No. 4 (4.75-mm) and the 3/4-inch (19-mm) sieves. These data are summarized in Figures 29 and 30 and Table 7. The percent passing the 3/4-inch (19-mm) sieve ranged from 70 to 98 and averaged 88 percent, as shown in Figure 29. As shown in Figure 30, the percent of shale particles passing the No. 4 (4.75-mm) sieve ranged from 42 to 78 and averaged about 60 percent. Hence, the watering, discing, and compaction procedure was effective in breaking down the shale particles since the percent of particle sizes greater than the 3/4-inch (19-mm) sieve averaged about 12 percent.

Laboratory-Moisture Density

Standard laboratory compaction tests were performed following procedures of ASTM D 698-72. The tests were performed on minus 3/4-inch (19-mm) and minus No. 4 (4.75-mm) materials. The two different sizes of shales were obtained by processing the shales through a rock crusher. Test results are summarized in Table 8 for the three different stations.
Table 7. Particles sizes finer than the 3/4-inch sieve and the No. 4 sieve.

<table>
<thead>
<tr>
<th>Lift No.</th>
<th>El. (Ft)</th>
<th>Minus 3/4-inch Sieve Material</th>
<th>Minus No.-4 Sieve Material</th>
<th>Minus No.-4 Sieve Material from Field Density test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Station Number 288+50</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>1077</td>
<td>80</td>
<td>59</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>1078</td>
<td>82</td>
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<td>84</td>
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<td>18</td>
<td>1088</td>
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<td>Station Number 498+50</td>
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<tr>
<td>1</td>
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Field Density Testing

Field density tests were performed using a nuclear density gage, a sand cone apparatus (6-inch -- 152.4-mm -- diameter), and a drive cylinder (4-inch -- 101.6-mm diameter). The sand cone tests were performed in accordance with ASTM D 1556-82 and the drive cylinder tests were performed in accordance with ASTM D 2937-83. The nuclear density tests were performed in accordance with procedures described by the equipment manufacturer, the Troxler® Electronic Laboratories, Inc.
Table 8. Laboratory moisture-density relations.

<table>
<thead>
<tr>
<th>Station Number</th>
<th>Minus 3/4-inch</th>
<th>Minus No.-4 inch</th>
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<tbody>
<tr>
<td></td>
<td>Maximum</td>
<td>Optimum</td>
</tr>
<tr>
<td></td>
<td>Dry Density</td>
<td>Moisture Content</td>
</tr>
<tr>
<td></td>
<td>(lb/ft³)</td>
<td>(%)</td>
</tr>
<tr>
<td>288+50¹</td>
<td>125.2</td>
<td>11.6</td>
</tr>
<tr>
<td>498+50²</td>
<td>129.0</td>
<td>9.5</td>
</tr>
<tr>
<td>498+50²</td>
<td>128.7</td>
<td>11.1</td>
</tr>
<tr>
<td>556+50³</td>
<td>132.6</td>
<td>10.0</td>
</tr>
<tr>
<td>556+50³</td>
<td>121.8</td>
<td>11.5</td>
</tr>
</tbody>
</table>

1. Shale obtained from cut at Station 285+00.
2. Shale obtained from cut at Station 479+00.
3. Shale obtained from cut at Station 544+50

Test Fill, Station 288+50

Standard compaction tests performed on minus 3/4-inch (19-mm) and minus No. 4 sieve (4.75-mm) yielded optimum moisture contents and maximum dry densities of 11.6 and 12.8 percent and 125.2 and 120.1 lbs/ft³ (2,005 and 1,924 kg/m³), respectively, as shown in Table 8. The average water content obtained from sand-cone tests was 8.3 percent and ranged from 6.9 to 11.6 percent, as shown in Table 9. The average water contents obtained from the drive sampler and the nuclear gage were 8.1 and 9.8 percent, respectively. Hence, the moisture contents obtained from the nuclear gage were about 1.5 percent higher than those obtained from sand-cone or drive cylinder tests. Readings of moisture contents from the nuclear gage may have been higher due to hydrocarbons present in the black intermediate shales used in the test fills. Base on the optimum moisture contents of 11.6 percent [minus 3/4-inch (19-mm) material] or 12.8 percent [minus No. 4 (4.75-mm) material], the field moisture content was some 3 to 5 percent lower than the unadjusted optimum moisture content.

The special compaction note allows a field value of 2 percent lower than optimum water content, and the field water content was some 1 to 3 percent below a value required by the specifications. Although Kentucky specifications, KM 64-512, adjust densities for oversized aggregates (greater than No. 4 -- 4.75-mm sieve), no adjustments are made for water content. The average material retained on the No. 4 (4.75-mm) sieve was about 41 percent. Using the moisture content of the oversized material (a measured value of about 6 percent), the adjusted, average optimum moisture content is given by Equation 5, or

\[ W_j = (0.41)(6.0) + (1-0.41)12.8 = 10\% \]
As shown in Figure 31, the field moisture contents are slightly below the minimum specified optimum moisture content (based on the fractions retained on the No.4 --4.75- mm screen). The average percent of shale particles retained on the 3/4- inch (19-mm) sieve was 15.1 percent. Using 11.6 percent as the optimum moisture content, and 15.1% retained on the 3/4-inch (19-mm) sieve, and Equation 5, the adjusted water content (figure 32) is about 10.7 percent and the lower limit is about 8.7 percent. The average field moisture content was about 0.5 percent lower than the lower specification limit of 8.7 percent. Hence, the field water contents are slightly below the field specification requirements. Based on results obtained from the nuclear gage, the field water contents met specifications. However, for practical purposes, these results were generally acceptable. In some specifications, if two-thirds of the values are within a plus or minus value of 3 percent of optimum moisture, then the field values are acceptable (NAVFAC, 1982).

The average dry densities (uncorrected for oversized particles) obtained from the sand-cone tests,

<table>
<thead>
<tr>
<th>Lift No.</th>
<th>El. (Ft)</th>
<th>Sand Cone Water Content (%)</th>
<th>Sand Cone Dry Density (Pcf)</th>
<th>Drive Sampler Water Content (%)</th>
<th>Drive Sampler Dry Density (Pcf)</th>
<th>Nuclear Gage Water Content (%)</th>
<th>Nuclear Gage Dry Density (Pcf)</th>
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</thead>
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<td>1</td>
<td>1071</td>
<td>5.8</td>
<td>117.9</td>
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<tr>
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<td>1082</td>
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<td>123.2</td>
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<td>123.2</td>
</tr>
</tbody>
</table>
Using Equation 4 and the average percent retained on the No. 4 (4.75-mm) sieve, the average adjusted dry density is 131.6 lbs/ft\(^3\) (2,108 kg/m\(^3\)). This value is essentially the same as the value obtained from the nomograph in Kentucky Method 64-512. Using the average percent retained on the 3/4-inch (19-mm) sieve and Equation 4, the average, adjusted dry density is 128.7 lbs/ft\(^3\) (2,061 kg/m\(^3\)). Hence, adjusting the dry density on the basis of minus No. 4 (4.75-mm) or minus 3/4-inch (19-mm) material gave about the same adjusted dry densities.
Using the nomograph in KM 64-512, the adjusted dry density is about 132.0 lbs/ft$^3$ (2,114 kg/m$^3$). A maximum dry density of 120.1 lbs/ft$^3$ (1,924 kg/m$^3$) was obtained from standard compaction tests on minus No. 4 (4.75-mm) materials. Forty one percent retained on the No. 4 (4.75-mm) sieve, and a bulk specific gravity of 2.65 were used to obtain the adjusted dry density. Obtaining a bulk density of intermediate shale particles from a laboratory test is difficult. The particles would slake or degrade during testing. The value of 2.65 used is, perhaps too high for shales at this site. A more realistic value for dry density of the compacted shale particles is 155 lbs/ft$^3$ (2,483 kg/m$^3$), or a bulk specific gravity of 2.48. Using this value and the nomograph, the adjusted dry density is about 128.5 lbs/ft$^3$ (2,058 kg/m$^3$).

Based only on the sand cone tests and using the maximum dry density of material passing the 3/4-inch sieve, the unadjusted values of relative compaction ranged from 92 to 106 percent. Values adjusted according to Equation 4 ranged from 100.5 to 103.6 percent (see Figure 33). Based on the maximum dry density of minus No. 4 (4.75-mm) material, the relative compaction ranged from about 100.5 to 110 percent (Figure 34). Based on those results, the compaction of the embankment at Station 288+50 met density specifications.
Test Fill, Station 498 + 50

Standard compaction tests were performed on material passing a 3/4-inch (19-mm) and No. 4 (4.75-mm) sieve. Two tests were performed for each material. For material passing the No. 4 (4.75-mm) sieve, optimum moisture contents were 10.3 and 11.2 percent, respectively. Maximum dry densities were 124.6 and 125.2 lbs/ft$^3$ (1,996 and 2,005 kg/m$^3$), respectively. From the nomograph in KM 64-512, assuming a bulk specific gravity of 2.65, and using a value of 48.3 percent retained on the No. 4 (4.75-mm) sieve, the adjusted average maximum dry density is about 136.0 lbs/ft$^3$ (2,178 kg/m$^3$). Based on an assumed bulk specific gravity of 2.48, the adjusted maximum dry density is 132.0 (2,114 kg/m$^3$). Based on the one test value from the sand-cone test (Table 10), the uncorrected relative compaction was 96.6, as shown in Figure 35. After correction, the relative compaction was 104.2 percent.

The drive sampler yielded an average dry density that was some 7.1 lbs/ft$^3$ (113.7 kg/m$^3$) lower than the average values obtained from nuclear gage and sand-cone tests. Based on Equation 4 and the percent retained on the No. 4 (4.75-mm) sieve -- 48.3 -- the average dry density is about 136.2 lbs/ft$^3$ (2,182 kg/m$^3$). Based on the percent retained on the 3/4-inch (19-mm) sieve -- 18.2 -- and using Equation 4, the adjusted dry density is 132.6 lbs/ft$^3$ (2,124 kg/m$^3$).
Table 10. Results of field density tests at Station 498+50.

<table>
<thead>
<tr>
<th>Lift No.</th>
<th>El. (Ft)</th>
<th>Sand Cone</th>
<th>Drive Sampler</th>
<th>Nuclear Gage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Water</td>
<td>Dry Density</td>
<td>Water</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Content (%)</td>
<td>(lbs/ft³)</td>
<td>Content (%)</td>
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<tr>
<td>1</td>
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<td>9.2</td>
<td>114.4</td>
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<td>7.4</td>
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</tbody>
</table>

Average field moisture contents from the sand-cone test (only one test), drive sampler, and the nuclear gage were 7.3, 7.9, and 10.0 percent, respectively. The average nuclear gage value was some 2.1 percent higher than the drive sampler. As shown in Figures 36 and 37, the field moisture contents were slightly lower than the adjusted optimum moisture content.
### Table 11. Results of field density tests at Station 556+50.

<table>
<thead>
<tr>
<th>Lift No.</th>
<th>El. (Ft)</th>
<th>Sand Cone Water Content (%)</th>
<th>Sand Cone Dry Density (Pcf)</th>
<th>Drive Sampler Water Content (%)</th>
<th>Drive Sampler Dry Density (Pcf)</th>
<th>Nuclear Gage Water Content (%)</th>
<th>Nuclear Gage Dry Density (Pcf)</th>
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<td>7</td>
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</table>

**Test Fill, Station 556 + 50**

Two sets of laboratory tests were performed. Values of optimum moisture contents and maximum dry densities for the minus No. 4 (4.75-mm) and minus 3/4-inch (19-mm) sieves are shown in Table 8.

From the nomograph in KM 64-512, assuming a bulk specific gravity of 2.65, and using an average percent retained on the No. 4 (4.75-mm) sieve of 35.4, the adjusted dry densities are 128.5 and 129.5 lbs/ft³ (2,058 and 2,074 kg/m³), respectively (Equation 4 yielded adjusted dry densities of 128.6 and 128.7 lbs/ft³ (2,060 and 2,061 kg/m³). If a bulk specific gravity of 2.48 (γ

**Figure 39. Field relative compactions and relative compactions adjusted on the basis of the plus No. 4 material, Station 556+50.**
The uncorrected relative compaction from the drive sampler and nuclear gage ranged from 89 to 102 percent. Relative compaction, corrected for the plus No. 4 (4.75-mm) material ranged from 103 to 108 percent. See Figure 39. Relative compaction, corrected for the plus 3/4-inch material, as shown in Figure 40, ranged from 100 to 102 percent. Based on nuclear gage results, average relative densities ranged from 89 to 102 percent. The uncorrected, relative compaction of shales in this fill was slightly lower than 95 percent. The nuclear gage yielded dry densities that averaged about 6.4 lbs/ft³ (102.4 kg/m³) higher than dry densities obtained from sandcone tests. The drive sampler yielded an average dry density that was about 3.2 lbs/ft³ (51.2 kg/m³) higher than the sand-cone test results and about 3.1 lbs/ft³ (49.6 kg/m³) lower than average nuclear gage results.

The average water contents obtained from sand-cone tests, the drive sampler, and nuclear gage were 11.5, 8.9, and 11.9 percent, respectively.

The nuclear gage and sand-cone yielded similar results. Based on the uncorrected values, field moisture contents met specifications. The unadjusted moisture content values, using equation 5 and assuming a moisture content of about 6.0 percent (natural water content of the shales), were about 9.5 and 10.4 percent, respectively (based on the fraction retained on the No. 4 (4.75-mm) sieve). Generally field moisture contents, as shown in Figures 41 and 42, were close to the adjusted moisture contents. Based on those values, the field water contents were close to specifications. Generally, requirements of the special compaction note were met.
Figure 42. Field moisture contents and optimum moisture contents adjusted on the basis of the plus No. 4 material, Station 556+50.

Figure 43. Relative compactions adjusted on the basis of material retained on the 3/4-inch sieve and the No. 4 sieve.

As shown in Figure 43, and based on adjusted values, relative compaction of the three test embankments met specifications.

Settlements

Test Fill, Station 288+50

Slake-durability index (KM 64-513) of the shales used in the lower portion of this fill was near a value of 69 percent. Based on this value, the shale classified as intermediate. Using the coordinates of 69 percent for SDI and 8 inches (20.3 cm) for the lift thickness, the point plots in the zone labeled “No Major Problems, Few Minor Problems” in Figure 24. For the upper portion of the embankment, the parameters $\phi'$ and $c'$ used in the design analysis were 22 degrees and 200 psf (6.9 kPa). For the intermediate shales used in the lower portion, the reported design parameters were 27 degrees and 200 psf (6.9 kPa), respectively. A factor of safety of 1.4 was reported. Using a fill height of 55 feet (16.8 m), a factor of safety of 1.4, and Equation 3, the predicted long-term settlement of this fill is 3.8 inches (9.7 cm). Based on the approximate method by Strohm et al. (1981), the long-term settlement is 3 to 5 inches (7.6 to 12.7 cm). Based on criterion of NAVFAC (1982), the long-term settlement is 2 to 4 inches (5.1 to 10.2 cm). Measured settlements were discontinued when the surveying bench mark was destroyed.

Test Fill, Station 498 + 50

The slake durability index of these shales was near 89 percent. Hence, these shales were classified in the intermediate range. These shales were more durable than those placed in the fills at stations...
288 + 50 and 556 + 50. Liquid and plastic limits were 39 and 21 percent, respectively. Material finer than 0.002 mm was about 18 percent. Reported design parameters, $\phi'$ and $c'$, of the shales placed in the lower portion of the embankment were 27 degrees and 200 psf (9.6 kPa). The $\phi'$ and $c'$ parameters assigned to the upper portion of this embankment were 22 degrees and 200 psf (9.6 kPa). The long-term safety factor was reported to be about 1.5. Using Equation 3, the long-term settlement is estimated to be about 5.2 inches (13.2 cm) for this 65-foot (19.8 m) high embankment. Based on the approximate method by Strohm et al. (1981) and using a SDI of 89 percent, the estimated long-term settlement is estimated to be about 2 inches (5.1 cm). Based on criterion in NAVFAC (1982), the estimated long-term settlement ranges from 2.3 to 4.6 inches (5.8 to 11.7 cm). As shown in Figure 44, the long-term settlement of this fill, projected to 10,000 days, or 27.4 years, ranges from about 4.8 inches to 6.8 inches (12.2 - 17.3 cm).

**Test Fill, Station 556 + 50**

The slake durability indices of shales in the lower part of this embankment were near 48 percent. Since this value is less than 50, the shales classified as soil-like. Based on an 8-inch (20.3-cm) lift and SDI equal 48 percent, these shales plot in Figure 24 in the zone identified as: "No Major Problems." In the upper portion of the embankment, the parameters of 22 degrees and 200 psf (9.6 kPa), $\phi'$ and $c'$, respectively, were used for design. In the lower portion of the embankment, the parameters were 27 degrees and 200 psf (9.6 kPa). The long-term safety factor was estimated to be 1.6. This embankment is about 65 feet (19.8 m) in height. Based on the approximate method by Strohm et al. (1981) and using a SDI of 48 percent, the estimated long-term settlement is estimated to be 4 to 5 inches (10.2 to 12.7 cm). Based on NAVFAC (1981), the estimated long-term settlement ranges from 2.5 inches (6.4 to 12.7 cm). From Equation 3, the long-term settlement is estimated to be 4.7 inches (12 cm). Because the surveying bench mark was destroyed, settlement measurements at this station were discontinued.
**Test Fill, Station 317+50**

Although this fill had originally been scheduled to be one of the three test fills, plans for this fill were changed due to construction scheduling. However, the settlement of the fill was monitored for about 3.3 years until the surveying bench mark was destroyed. Height of this embankment is slightly less than 50 feet, as shown in Figure 45. The fill was built using a mixture of soil and shales. Settlement of the fill is shown in Figure 46. The fill settled about 1.9 to 2.3 inches (4.8 - 5.8 cm) in 3 years. Based on the last readings, the settlement appears to be linear with the logarithm of time. Assuming that this is the case and projecting the settlement to a time of 10,000 days, or 27.4 years, the long-term settlement of this fill is about 2.9 to 3.2 inches (7.4 - 9.1 cm). If safety factors of 1.5 and 1.75 are assumed, and based on equation 5, the estimated settlement ranges from about 3.0 to 2.1 inches (7.6 - 5.3 cm), respectively. Estimated and measured long-term settlements for all fills are summarized in Table 12.

**Benefits of Special Shale Compaction Provision**

Table 12. Estimated and measured settlements.

<table>
<thead>
<tr>
<th>Station Number</th>
<th>Fill Height (feet)</th>
<th>Reported Factor of Safety</th>
<th>SDI of Fill Shales (%)</th>
<th>Long-Term Settlement (inches)</th>
<th>Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>288+50</td>
<td>55</td>
<td>1.4</td>
<td>69</td>
<td>2-4</td>
<td>3-5</td>
</tr>
<tr>
<td>498+50</td>
<td>65</td>
<td>1.5</td>
<td>48</td>
<td>2.3-4.6</td>
<td>2</td>
</tr>
<tr>
<td>556+50</td>
<td>65</td>
<td>1.6</td>
<td>89</td>
<td>2.5-6</td>
<td>4-5</td>
</tr>
<tr>
<td>317+50</td>
<td>50</td>
<td>(1.5-1.75 (Intm.)</td>
<td>1.8-3.6</td>
<td>2.1 - 3.0</td>
<td></td>
</tr>
</tbody>
</table>

As previous studies (Munson and Mathis, 1981-1983) have shown, numerous embankment and settlement failures occurred on stretches of Interstates 75 and 71 in Northern Kentucky that pass through the Kope and Fairview shales and roadways that pass through the Crab Orchard Formations. For example, some 120 embankments on a stretch of I 75 were identified as failures requiring remedial work. As noted previously, some 120 million dollars have been spent repairing failures on I 75 and 71—repair costs have averaged some 1.5 to 2 millions per mile for roadways passing through the Kope and Fairview shales.

In a series of meetings held at the Division of Materials (Geotechnical Branch, Kentucky Transportation Cabinet), in the mid-eighties regarding design standards of the Alexander-Ashland Highway, four general recommendations (Hopkins 1985) were made to avoid embankment and subgrade failures experienced in previous years with the Kope, Fairview, and the Crab Orchard shales on Interstates 75 and 71. These were, as follows:

- All shale embankments should be compacted using the Special Shale Compaction Provision (see Appendix), especially fills over about 40 - 50 feet (12 - 15 m) in height.
- For embankments over about 30 to 40 feet (9 - 12 m) in height, it was recommended that minimum slope designs of about 2.5 horizontal to 1 vertical or 3 horizontal to 1 vertical be considered—values of $\phi'$ and $c'$ normally obtained from triaxial tests for the clayey shales of that area should be reduced in the slope stability analysis.
- Good drainage should be placed at the base of large shale fills.
- All pavement soil and shale subgrades should be stabilized.

Generally, those recommendations were adopted. Perhaps, the most important recommendation pertaining to the compaction of the shales was generally observed. After some 11 years, no serious problems have been reported concerning the shale embankments on the AA-Highway. Based on this observation, and the lack of massive failures (of the type that have occurred on I 71 and I 75 in similar materials) of the shale embankments on the AA-highway(KY9), which appears to be the
result of using the special shale compaction provision, some 188 to 250 dollars (1.5-2 million repair dollars/mile x 125 miles) will potentially be saved.

**Summary and Conclusions**

Numerous failures of embankments constructed with shales have occurred in the past and continue to occur. Millions of dollars are spent each year repairing shale embankments. Because of the vastness of these materials in many regions of the country and the lack of economical alternate construction materials, the shale materials must be used in constructing highway embankments. Shales exhibit a wide range of engineering properties, as illustrated by the results of tests performed on Kentucky shales.

Successful use of these materials requires good planning, design, and construction. Some important factors that should be considered in constructing shale embankments include geological features, infiltration of water, durability, placement and compaction of shales, and foundation conditions. Two very important considerations include using thin lifts and heavy compactors (and disks) to break down the shales. When hard rocks and durable shales are used as rock fill, the amount of soil (minus No.4 material) should be limited to about 20 percent, or less.

Based on an analysis of field compaction techniques used at the three tests fills constructed of intermediate and soil-like shales and compacted with extra heavy compactors, the special compaction provision shown in the Appendix was generally successful. The heavy compactors included a tamping-foot roller and a vibratory roller. Slaking with water, using heavy compactors, and using a heavy-duty disk proved successful in breaking down or degrading the intermediate and soil-like shales. The percent passing the 3/4-inch (19-mm) and No. 4 (4.75-mm)sieves averaged about 88 and 60 percent, respectively. Compaction requirements were generally met. Relative compaction generally averaged approximately 95 percent. Field water contents, after adjusting for oversized material, generally met specifications.

Field dry densities and water contents from the nuclear gage averaged about 2 lbs/ft³ (32 kg/m³) and 2 percent higher than values from the sand cone. The tendency of the nuclear gage to register slightly higher values than those obtained from the sand cone tests was probably due to the hydrocarbons in the black and gray shales at the site. It is recommended that nuclear gages be calibrated against the sand cone on project materials and that adjustments be made to values obtained with the nuclear gage. The drive sampler yielded average values much lower than the sand cone or nuclear gage. However, average water contents obtained with the drive sampler were similar to those from the sand cone. For measuring dry densities of intermediate and soil-like shales, the drive sampler is not recommended because of the ends of the samples cannot be trimmed smooth.

Use of the special shale compaction provision will save millions of dollars each year by preventing large shale embankment failures.
Appendix

SPECIAL NOTE FOR COMPACTION OF SHALE EMBANKMENTS
Special Note Number 2T

I. DESCRIPTION

This work shall consist of constructing and compacting embankments composed predominately of nondurable shale (SDI less than 90 by KM 64-513) when designated on the plans, utilizing the construction techniques specified herein. These requirements are in addition to Sections 207 and 208 of the current Standard Specifications for Road and Bridge Construction.

II. CONSTRUCTION REQUIREMENTS

Nondurable shale and/or these materials interbedded with thin seams less than 101.6 mm (4 inches) thick or harder rock shall be compacted utilizing an approved static tamping-foot roller in conjunction with a vibratory tamping-foot roller. The minimum weight for the static tamping-foot roller shall be 27.2 metric tons (60,000 pounds). The minimum total compactive effort for the vibratory tamping-foot roller shall be 24.9 metric tons (55,000 pounds). Total compactive effort is defined as the portion of the static weight acting upon the unsprung drum added to the centrifugal force provided by that drum. If the manufacturer’s charts do not list the static weight acting upon the compaction drum, the Contractor will be required to have the roller weighed to the satisfaction of the Engineer, and that weight shall be added to the centrifugal force, rated in accordance with the Construction Industry Manufacturer’s Association (CIMA). Each tamping-foot on the vibratory tamping-foot roller shall project from the drum a minimum of 10.16 mm (4 inches). The surface area of the end of each foot on each roller shall be no less than 3,548.4 mm² (5 ½ square inches).

Shale shall be placed in 203.2 mm (8 inches) maximum loose lifts to the full width of the cross section. Excavation and blasting procedures shall accommodate the selective placement of the material. Each lift shall be bladed as required prior to insure uniform layer thickness. Large rock fragments or limestone slabs having thickness greater than 101.6 mm (4 inches) and/or any dimension greater than 0.46 m (1 ½ feet) shall be removed from the layer to be compacted, or broken down and incorporated into the lift.

If the shale is dry, the Contractor shall apply water to accelerate the slaking action (breakdown) and to facilitate compaction. The water shall be distributed by an approved method which provides uniform application of the required quantity of water. The water shall uniformly incorporated throughout the entire lift by a multiple gang disk with a minimum disk wheel diameter of 609.6 mm (24 inches). The amount of water shall be that required to achieve a moisture content of optimum ± 2.0 percent as determined by KM 64-511. This moisture content requirement shall have equal weight with the density requirements specified herein when determining the acceptability of a layer. Moisture content tests will be conducted at such a frequency as deemed necessary to
assure that the entire layer conforms to the specified moisture content.

Unless otherwise approved in writing by the Engineer, each embankment lift shall receive a minimum of 3 passes with the static roller followed by blading and a minimum of 2 passes with the vibratory roller. The rollers shall not exceed 4.8 kph (3 mph) during these passes. Each embankment layer shall be compacted to a minimum of 95 percent of maximum dry density as determined by KM 64-511. The number of passes will, at the direction of the Engineer, be adjusted upward if necessary to obtain 95 percent of maximum dry density.

The in-place density will be determined by using nuclear gages. Tests will be conducted at such frequency as deemed necessary to assure that the entire layer is compacted to the specified density.

III. METHOD OF MEASUREMENT

No separate measurement or payment will be made for compaction, as specified herein. Payment for all labor, machinery, materials, and other costs associated with the compaction of shale embankments to the specified density, except water, is considered incidental to earthwork items in the contract.

Water used as directed for providing the specified moisture will be measured by weight or volume (tank capacity or meter) and converted to 1,000 liter (1,000 gallon) units.

IV. BASIS OF PAYMENT

The accepted quantity of water will be paid for at the contract price per 1,000 liter (1,000 gallon) unit, which shall be full compensation for all work necessary to furnish and properly apply and incorporate the water.

May 16, 1991

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