DESIGN OF HIGHWAY EMBANKMENTS ON
UNSTABLE NATURAL SLOPES

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ABSTRACT

Sections of the new Alexandria-Ashland (AA) highway in northeastern Kentucky must be located on and through natural slopes of the Kope and Crab Orchard Formations that consist mainly of shales. Numerous highway embankment and cut slope failures have occurred in these two geologic formations in past years. The shales of these formations have very poor and undesirable engineering properties. Many stability problems encountered with the Kope and Crab Orchard shales are caused primarily by the tendency of those shales to breakdown when exposed to water and produce clays and clayey silts of relatively low shear strengths. Construction of highway embankments with, through, and on the shales of the Kope and Crab Orchard Formations has been necessary because of the vast aerial presence of these shales and the lack of more suitable and economical alternate construction materials. In the design of the AA highway, geotechnical engineers were faced with three difficult problems. First, in forming embankments, there was a question of how the shales should be compacted and what constituted proper compaction. Second, the most difficult problem, perhaps, was the selection of appropriate shear strength parameters of the overconsolidated clays and clayey shales of the natural slope foundations and the embankments constructed of the shales. Numerous landslide studies involving overconsolidated clays and clay shales show that use of peak strengths from triaxial tests may lead to underconservative designs while use of residual shear strengths may lead to uneconomical designs. A third factor complicating the design of shale embankments on weathered shale slopes is the seepage of water into the embankment. Exposure of the lower portions of the embankments composed of Kope and Crab Orchard shales to seepage and rapid-drawdown conditions created by the Ohio River make the shales susceptible to breakdown and swelling and may produce a progressive "softening" and decrease in shear strengths. The design of certain sections of the AA highway passing through the Kope and Crab Orchard shales was further complicated by the fact that some natural slopes and existing highway fills having slopes of 3 horizontal to 1 vertical are failing. The design problem became one of placing new highway embankments on existing failing natural slopes. This paper presents case histories to document existing failures and discusses the treatment of the three factors -- compaction, shear strength, and seepage. A discussion of the different aspects of the geotechnical design for new fill placement is presented. Emphasis is placed on the selection and justification of design parameters for foundation and embankment materials as well as the construction procedures and compaction specification that were finally adopted.

INTRODUCTION

The stability of natural slopes, cut slopes, and embankment slopes is a major concern to any highway designer. The Alexandria-Ashland (AA) Highway in northern Kentucky will cross two areas that historically have been difficult areas for both highway construction and maintenance activities due to slope stability and settlement

problems (1-16). Weak clayey shales that comprise the bedrock in these two areas are the major contributor to the problem. Major problems which roadways have experienced in the two areas are catastrophic landsliding and significant settlements of several inches to several feet within embankment sections.

The purpose of the AA Highway is to provide a direct link between the Covington-Cincinnati area and the Ashland-Greenup area in northeastern Kentucky as shown in a plan view in Figure 1. The AA Highway begins in Campbell County at the interchange of I

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* American Engineering Company is providing overall supervision of design and construction of the AA Highway for the Commonwealth of Kentucky. Geotechnical overview was provided by Mr. Bishop for American Engineering Company.
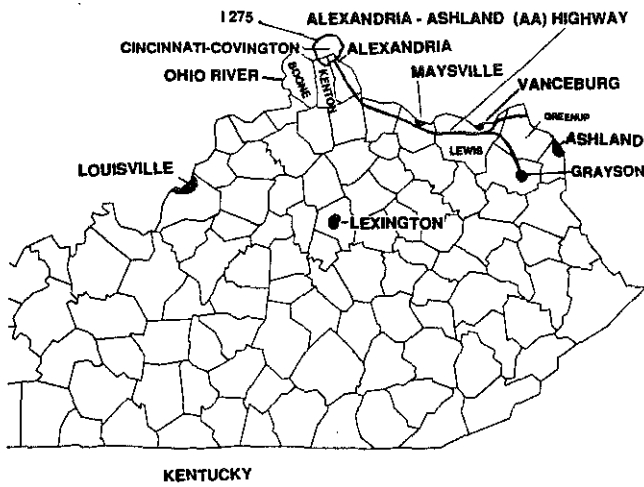
FIGURE 1. Location of the Alexandria-Ashland (AA) Highway in Northeastern Kentucky.

275 and KY 9 near Alexandria and extends eastward paralleling the Ohio River to Maysville and then to Vanceburg. Typically, the AA Highway is located about one to five miles south of the Ohio River. Near Vanceburg, the AA Highway splits, with one spur traversing due east to the Greenup Dam bridge, and the other spur traveling southeast to Grayson. Total project length is approximately 136 miles, of which 125 miles are new construction. The AA Highway will be a two-lane facility with truck passing lanes where required. In areas of heavy traffic, a four-lane roadway will be constructed.

Numerous large scale failures of shale embankments along major interstate routes in the early seventies prompted major research efforts by the Federal Highway Administration (FHWA), universities, and other governmental agencies. Kentucky experienced its share of failures on Interstates 64 and 75. The focus of the research was to develop guidelines for design and construction of new shale embankments as well as remedial measures for existing failing embankments. Incorporation of existing experience and research data into the design and construction of the AA Highway was a major project objective.

In the design of cut slopes and embankments the following factors must be considered:

* subsurface profile
* settlement properties
* shear strength properties
* surface and subsurface drainage
* construction specifications

The engineering performance of clayey shales in highway embankments and cut slopes has been demonstrated to be a function of the type of construction procedures used and a function of time. When first excavated, clayey shales appear and perform as rock. After exposure to weathering, the clayey shales break down, becoming a soil of low shear strength and high deformational properties. Overcoming the poor engineering properties of the clayey shales presented a design and construction challenge for

engineers working on the AA Highway project.

TOPOGRAPHIC AND GEOLOGIC SETTING

The alignment of the AA Highway crosses two areas of Kentucky where slope instability problems have severely affected highways and other structures. Northern Kentucky and southern Ohio counties surrounding Covington and Cincinnati have experienced slope stability problems since development began in that area. These problems are directly related to the rock types (the Kope and Fairview Formations) outcropping in the area and the topography. A second area to be crossed by the alignment where severely unstable natural slopes are present is in Lewis County. Between the communities of Tollesboro and Vanceburg, the Crab Orchard Formation outcrops and consists of clayey shale. Numerous natural slope failures and embankment and cut slope failures exist in this area.

The Kope and Fairview Formations outcrop in the Outer Bluegrass Physiographic Region of Kentucky. The area is a maturely dissected upland of rolling hills and small stream valleys. Elevations range from 500 feet above mean sea level adjacent to the Ohio River to 900 feet along ridge tops. Local relief ranges typically from 200 to 300 feet. Erosion has reduced the land surface to essentially an all-sloping landscape. Wide stream valleys are scarce.

The Kope and Fairview Formations are upper Ordovician in age and range in combined thickness from 260 feet to 370 feet. The Fairview overlays the Kope and comprises the ridge tops in many areas. The Kope Formation consists approximately of 80 percent shale and 20 percent limestone. The shale is gray to bluish or greenish gray in color, and laminations range from thick to thinly bedded. A few beds reach thicknesses of 6 feet. Slake durability indices (17-20, 8) are typically less than 50 percent and average in the 30's. Two types of limestone are present in the Kope Formation. About two-thirds of the limestone is medium gray in color, finely to coarsely crystalline, and bioclastic and occurs in indiscrete regular to irregular beds measuring as much as 12 inches in thickness. Generally, the beds are less than 8 inches in thickness. About one-third of the limestone is medium gray to dark gray in color, fine grained, argillaceous, and silty. This portion of the Kope occurs in beds measuring as much as 8 inches in thickness and commonly is laminated or cross-laminated. The limestones are interbedded with the shale throughout the unit, but interbeds occur somewhat more frequently near the top (21, 22).

The Fairview Formation consists of interbedded limestone and shale. Limestone comprises approximately 40 percent of the Formation near the base of the unit and about 65 percent near the top of the unit. The limestone is coarsely crystalline, sparry, and bioclastic and occurs in beds measuring as much as 3 feet in thickness. Generally, the beds of limestone are less than 8 inches in thickness. The shale is light bluish gray to medium gray in color and

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weathers olive gray to yellow. The formation is laminated to thinly bedded in beds less than 2 feet in thickness. Generally, the beds are less than 10 inches thick. Slake durability indices for the shale are typically less than 50 percent and average in the 30's. When exposed to weathering, shales of the Kope and Fairview Formations usually break down rapidly into soils that are plastic clays and silts with low shear strength (21, 22).

The Crab Orchard Formation outcrops at the juncture between the Outer Bluegrass and Knobs Physiographic Regions of Kentucky. The outcrop of Crab Orchard shales located in western Lewis County is approximately five miles wide. Topography of the area is a maturely dissected upland and consists of well-developed stream valleys. Elevations range from 600 to 850 feet above mean sea level near Tollesboro and 600 to 1100 feet at the eastern edge of the outcrop. Wide flat ridge tops are present at Tollesboro. Progressing eastward, the landscape becomes more mountainous and the local relief increases. Hillside slopes in the area are irregular and hummocky. Evidence of natural slope failures is numerous and indications of continual creep movements are prevalent. Two embankment landslide failures and numerous cut slope failures are present along (existing) KY. 10.

The Crab Orchard Formation is Middle Silurian in age and is differentiated into an upper and lower unit. The upper unit consists entirely of clayey shale and is the source of most slope instability problems. Thickness of the upper Crab Orchard ranges from 130 feet to 160 feet as mapped on the Tollesboro's and Charters' Geologic Quadrangle Maps (23, 24). This portion of the Crab Orchard consists totally of clayey shale. The upper unit is mainly greenish gray in color, but thin red and brown zones are present. The unit is laminated to thinly bedded, fissile to chunky, and when exposed to water expands and becomes very plastic. Slake durability indices are typically less than 40 percent and average in the low twenties and upper teens. A prominent characteristic of the lithologies associated with the Crab Orchard Formation is the overlying Bisher Limestone. It is a dolomitic limestone, light grey in color, fine to coarse grained in texture with irregular bedding. Thickness of this unit ranges from 0 to 70 feet. Many natural landslides occur on slopes just below the outcrop of the Bisher Limestone.

Soils along the alignment where the Kope and Fairview outcrop are primarily residual in origin. They are clays and silty clays that classify as ML, MH, CH, and CL by the United Soil Classification System. Thicknesses of these soils range from 1 foot or less on steep slopes to 10 or more feet on relatively flat terrain. Alluvial soils are present along major stream valleys. Thickness of the alluvium in the stream valleys was directly related to the distance from the Ohio River. The thicker alluvium is located immediately adjacent to the river.

Random and erratic areas of glacial soils exist in Campbell County on the alignment of the AA Highway. Thickness of these deposits range from a few feet to as much as 60 feet. The glacial deposits are the result of both Illinoian and Wisconsin glaciation. Glacial outwash, drift, lacustrine, and alluvial deposits are present.

Soils along the alignment in the area where the Crab Orchard outcrops are residual silts and clays that classify as ML, MH, CL, and CH by the United Soil Classification System. Thicknesses of these soils range from a few feet to more than 20 feet in some areas. In some areas, it is difficult to establish a soil-rock interface due to weathering of the shale. Only in major stream valleys is alluvium present.

ENGINEERING PROPERTIES
INDEX PROPERTIES

Typical engineering characteristics of the residual overburden soils and weathered clayey shales along the roadway foundation of the AA Highway are generally poor. Based on an analysis of 290 tests, average values of liquid limit and plasticity index of the residual soils and weathered clayey shales of the Kope and Fairview Formations were about 46 (±10) percent and 22 (±6) percent, respectively. Natural water contents of soils and the clayey shales in the rock disintegration zone generally averaged about 27.3 (±2.0) percent based on an analysis of 46 tests. However, natural water contents of the unweathered Kope shale typically are about 8 to 9 percent (8). Liquidity indices of the weathered materials are typically about 0.15 and range from about 0.5 to 0.0. Liquidity indices of the intact unweathered clayey shales are typically -0.7. Based on the Unified Soil Classification System and the AASHTO Classification System, the weathered soils and clayey shales predominantly classify as CL and CH and A-7-5 and A-7-6, respectively. Hence, the weathered soils and clayey shales of the Kope and Fairview Formations are generally moist to wet, firm, highly overconsolidated, plastic clays.

Based on an analysis of 63 tests performed on weathered overburden soils and clayey shales of the Crab Orchard Formation and geological formations similar to the Crab Orchard, the liquid limit and plasticity index of the weathered materials along the roadway foundation typically averaged 51 (±14) percent and 26 (±10) percent, respectively. Natural water contents of the weathered materials averaged 24 percent (based on 194 tests). A typical average value of liquidity index was 0.0. Natural water content of the unweathered intact clayey shales of the Crab Orchard Formation is typically 7.4 percent (8). Liquidity index of the unweathered shale is about -0.65. Average values of activity were 0.7 (±0.2). Percentages of clay particles finer than the 0.002-mm size averaged 38.4 (±11). Based on the Unified Soil Classification System and AASHTO Classification System, the weathered materials of the Crab Orchard and associated geologic formations generally classified as CL and CH and A-7-5 and A-7-6. Hence, the weathered materials were typically moist to wet, firm, highly overconsolidated, plastic clays.

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BEARING RATIO

Bearing ratios (CBR) of soaked specimens of the weathered soils and clayey shales of the Crab Orchard and Kope Formations, and similar formations, were generally very low. Soaked bearing ratios of weathered materials of the Kope Formation averaged about 3.4 (±1.0) percent. Soaked CBR values of the weathered materials of the Crab Orchard and formations similar to the Crab Orchard averaged about 6.6 (±4.5) percent. Frequently, CBR values were less than 3.0. CBR values of compacted shales from the Crab Orchard and Kope Formations, and reported elsewhere (25), were 1.9 and 2.0 percent, respectively. CBR values before soaking were 23 and 32 percent, respectively. Moreover, high swelling strains occurred during those tests. Maximum vertical strains due to swelling during soaking were 12.4 percent and 10.6 percent, respectively, for the Crab Orchard and Kope shales. The large reduction in bearing strengths and large increase in volume of these shales when exposed to water has caused many failures of pavements and embankments and illustrates the poor engineering properties of these shales.

SLAKE DURABILITY

Slake-durability indices (SDI) (17-20, 8) of unweathered shales of the Kope and Crab Orchard encountered along roadway are typically in the middle of the intermediate range while indices of the weathered shales in the rock disintegration zone are typically less than 50 percent and classify as "soil-like." Typical jar slake numbers of these shales are 1 or 2, that is, the shales slake rapidly and breakdown immediately into soil when exposed to water.

MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT OF NATURAL OVERBURDEN SOILS

Average values of maximum dry density and optimum moisture content as determined from AASHTO T-99 (26) of the weathered soils from the Kope and Fairview Formations were 104.3 (±3.2) pounds per cubic foot and 19.3 (±2) percent (based on an analysis of 42 tests). Average values of maximum dry density of the weathered soils of the Crab Orchard Formation (based on an analysis of 21 tests) were 102.5 (±5) pounds per cubic foot and 21 (±3.6) percent. In both cases, the weathered overburden soils of the Kope, Fairview, and Crab Orchard Formations frequently had natural water contents larger than optimum moisture contents. Hence, aeration of the overburden soils was required before compaction in many cases. Since the overburden soils were generally very thin and shallow (varying generally from 2 to 15 feet in thickness), major portions of the embankments will be constructed of unweathered shales of the Kope, Fairview, Crab Orchard, and similar Formations.

Results of standard compaction tests -- AASHTO T99 -- (reported elsewhere (27)) performed on shales from the Kope and Crab Orchard Formations yielded values of maximum dry density of 116.3 and 118.6 pounds per cubic foot and optimum moisture contents of 13 and 11.4 percent, respectively. Since natural water contents of unweathered shales of the Kope and Crab Orchard shales are typically about 7 to 8 percent, then additional water will be required to obtain the optimum moisture content. The addition of water to the shales before compaction, in areas of Kentucky where water is not plentiful, was one major economical factor that had to be considered and strongly influenced the formulation of embankment compaction specifications. The addition of water to the unweathered shales was desirable since the water would cause the shales to slake, breakdown, and facilitate compaction.

SHEAR STRENGTH OF NATURAL OVERBURDEN SOILS

Based on 62 triaxial failure envelopes from tests (isotropically consolidated-undrained triaxial compression tests with pore-pressure measurements) on weathered, overburden soils and shales of the Kope Formation, average peak effective stress parameters, ϕ'_p and c'_p, were 28.7 (±3) degrees and 232 (±125) pounds per square foot (psf), respectively. Based on the percentage ($P_{0.002mm}$) of clay particles finer than 0.002-mm size, and a correlation ($\phi'_r = 68.2 - 30.2 \log(P_{0.002mm})$) (9), the estimated residual shear strength parameter, ϕ'_r, for weathered soils derived from the Kope is 22 degrees. Based on one standard deviation, the estimated ϕ'_r-values may range from about 19 to 27 degrees. Average peak effective stress parameters for the weathered overburden soils derived from the shales of the Crab Orchard Formation (based on four triaxial failure envelopes) were about 28 (±5) degrees and 43 (±51) psf, respectively. Based on the percentage of particles finer than 0.002 mm, and the correlation presented elsewhere (9), the estimated residual strength parameter, ϕ'_r, is about 20 degrees. The ϕ'_r-value, however, is estimated to range from 17 to 25 degrees.

SHEAR STRENGTH OF COMPACTED SHALES

Shear strength parameters, ϕ'_p and c'_p, of compacted or remolded specimens of shales from the Kope and Crab Orchard Formations are shown in Figures 2 and 3 (27). In these series of tests, the shales were compacted at three different densities and compactive energies using modified compaction (ASTM D 1558-78), standard compaction (ASTM D 698-78), and a low-energy compactive effort. The specimens were compacted to maximum dry density and optimum moisture at each level of compactive energy. Isotropically consolidated-undrained triaxial tests with pore-pressure measurements were performed. Effective stress parameters, ϕ'_p, obtained for the compacted shales of the Kope Formation, corresponding to the three different compactive energies, were 25.4, 26.6, and 26.2 degrees, respectively. Values of c'_p were 958, 272, and 320 psf, respectively. Values of ϕ'_p obtained for the shales of the Crab Orchard Formation were 23.0, 23.9, and 24.5 degrees, respectively. Corresponding values of c'_p were 1210, 768, and 550 psf. Variations of the parameters ϕ'_p and c'_p with dry density are illustrated in Figures 2 and 3. While the ϕ'_p-value changes only
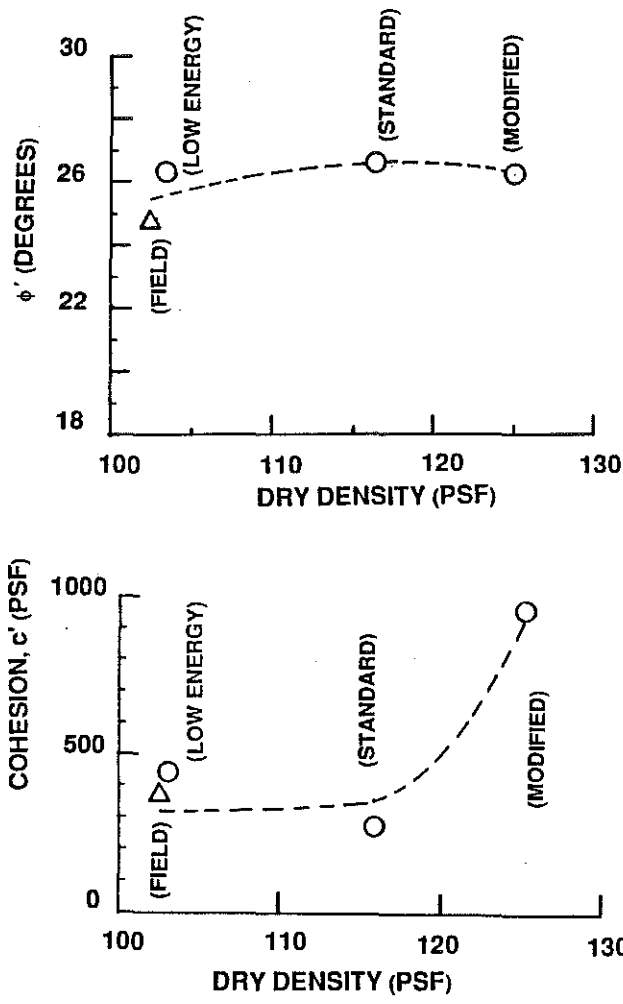
FIGURE 2. Variation of Effective Stress Parameters, ϕ'_p and c'_p, for Compacted Kope Shales as a Function of Dry Density (27).

slightly with an increase in dry density and compactive effort, the c'_p value increases significantly as the compactive energy increases.

In 1981, extensive studies of numerous highway embankment failures on 75 in Boone, Grant, and Kenton counties were conducted (10). Embankments in these well-documented studies were constructed of materials from the Kope and Fairview Formations in the late 1950's and 1960's and have been in place some 20 to 25 years. Numerous settlement and stability problems have occurred. During the studies, some 336 consolidated-undrained triaxial compression tests with pore-pressure measurements were performed on undisturbed samples from numerous embankments experiencing settlement and stability problems. The average peak effective stress parameters, ϕ'_p and c'_p, obtained from the triaxial tests were 24.7 (±5.3) degrees and 368 (±252) psf. These parameters are similar to parameters obtained from triaxial tests (27) performed on remolded samples of the Kope shale compacted at standard compaction (ASTM D 698). Those tests yielded values of 26.6 degrees and 272 psf. Moreover, the average dry density
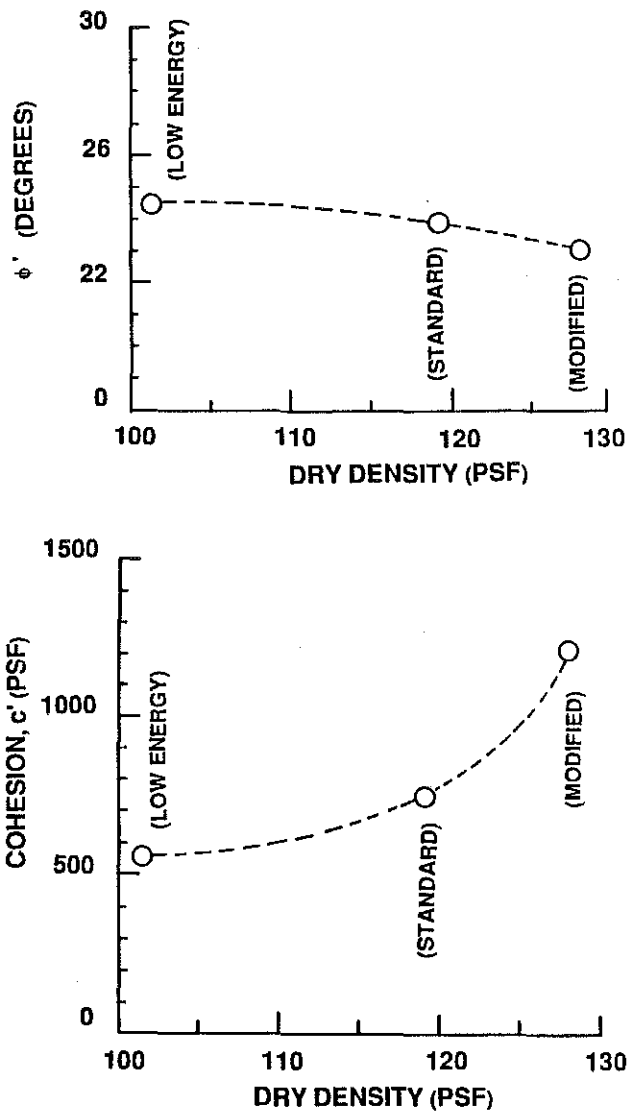
FIGURE 3. Variation of Effective Stress Parameters, ϕ'_p and c'_p, for Compacted Crab Ochard Shales as a Function of Dry Density (27).

percent. That value is similar to average values of maximum dry density and optimum moisture contents (ASTM D 698) obtained from tests performed on natural overburden soils in the Kope areas -- 102.5 (±5) psf and 19.3 (±2) percent. Hence, the long-term (>20 years) average dry density and optimum moisture content of embankments constructed of shales from the Kope and Fairview are about the same as the maximum dry density and optimum moisture content (AASHTO T 99) of the natural overburden soils. However, the in situ average dry density and moisture content are lower than the maximum dry density and optimum moisture content obtained from standard compaction tests on Kope shales. Based on those

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data, the relative compaction of the I-75 embankments was about 90 percent.

The low value of relative compaction is probably partly due to the fact that lift thicknesses up to 3 feet were permitted 20 years ago and partly due to swelling that occurred when the shales were subjected to seepage and surface infiltration of water. Also, the compactive effort was insufficient to breakdown mixtures of limestone and shales. Insufficient compaction left voids in the fill. As illustrated in Figures 2 and 3, increasing the compactive effort above standard compaction increases the c'-value and therefore increases the factor of safety and stability.

Although greater compactive effort may be used in the field, the question that arises is whether the cohesion, c', will decrease over a long period of time as relaxation and creep of the embankment occurs and seepage and surface water infiltrate into the embankment, which tend to negate high negative pore pressures imparted by compaction.

Past studies (28, 9) have shown that, when clays of highway embankments and foundations (natural overburden soils) have liquidity indices less than about 0.4, stability analyses based on either a total stress analysis or an effective stress analysis and based on peak shear strength parameters, S_u or ϕ'_p and c'_p, respectively, usually yields unreliable results. Triaxial tests overestimate the in situ shear strength of the overconsolidated materials. This observation is based on a review of 75 national and international, well-documented geotechnical failures. For example, stability analysis (10) of several embankment failures on I 75 using the peak strength parameters, ϕ'_p and c'_p, obtained from an analysis of 336 triaxial tests yielded factors of safety ranging from 1.7 to 2.0. Liquidity indices of those embankment clays were less than 0.4. Consequently, because of uncertainties that arise in performing stability analyses of slopes and foundations (LI < 0.4) composed of overconsolidated plastic clays and clayey shales and based on past experience, the concept of reducing the peak shear strengths from triaxial tests was adopted in designing embankments on the AA Highway.

Guidelines based on slake-durability classifications for selecting design shear strength parameters, ϕ'_d and c'_d, contained in the Kentucky Geotechnical Manual (20) and NAVFACS (29) suggest the following strength parameters for shale embankments:

Soil-like Shale (SDI < 50) S_u = 1,000 to 1,500 psf
                                             ϕ' = $20°$ to $25°$
                                           c' = 200 psf

Intermediate Shale (50<SDI<95) S_u = 1,000 to 1,500 psf
                                           ϕ' = $26°$ to $30°$
                                           c' = 200 psf

Based on the past, poor performance of Kope and Fairview shales, and considering that these shales generally classify as soil-like (SDI < 50), design parameters of ϕ'_d equal 20 degrees and c'_d equal to 200 psf were selected. These values are smaller than values for ϕ'_p and c'_p obtained from compacted specimens of Kope shale (26.6 degrees and 272 psf) and average parameters (ϕ'_p equal to 24.7 degrees and c'_p equal to 368 psf) obtained from an analysis of 336 triaxial tests performed on undisturbed specimens from several embankments constructed of Kope and Fairview shales, or

$$\tan \phi'_d = 3 \tan \phi'_p / 4 \qquad (1)$$

and

$$c'_d = 3 c'_p / 5. \qquad (2)$$

To check the selection of the design strength parameters, two embankment failures on I 75 (slides identified in the report by Munson, et al. (10) as 90 and 95) were reanalyzed. Those embankments were constructed of shales from the Kope Formation. The embankment failures were reanalyzed using the ICES LEASE computer program (Bishop model (30)) and HOPK-I (a new stability model and computer program (31)), the shear surfaces as indicated from slope inclinometers and observed scarps, monitored pore pressures, and the reduced shear strength parameters. Factors of safety of 1.07 and 1.08 were obtained for Slide 95 from the ICES LEASE and HOPK-I programs, respectively. For Slide 90 factors of safety of 0.89 and 0.90 were obtained. Hence, the reduced shear strength parameters for the Kope shale embankments appeared to be a reasonable choice since the factors of safety were near 1.0.

Various other analyses were performed using average parameters obtained from 336 triaxial tests and parameters obtained from specimens compacted at different densities. In all cases, both stability programs gave nearly identical results. Differences were less than 0.5 percent. The various factors of safety based on the different shear strength parameters are shown in Figure 4 as a function of the effective stress parameter c'. Use of effective stress parameters obtained from modified compaction yields factors of safety that range from 2.3 to 3.6 and illustrates the potential benefits of using high compactive efforts.

Considering the past poor engineering performance of Crab Orchard shales and since these shales generally classify as soil-like (SDI < 50), effective stress parameters, ϕ'_d and c'_d, selected for design purposes were 18 degrees and 100 psf. Also, these parameters were selected on the basis of back-calculations of two highway failures and specimens remolded at different relative compactions. The first massive sidehill highway embankment failure occurred in 1972 on 64 (Milepost 118). Details of that failure are given elsewhere (4). Pore pressures (observation wells) and lateral movements (five slope inclinometers) were monitored over a period of 4 years prior to failure. The 100-foot sidehill embankment was constructed of Crab Orchard shales and rested on a 20-foot thick foundation of residual soils derived from Crab Orchard shales.
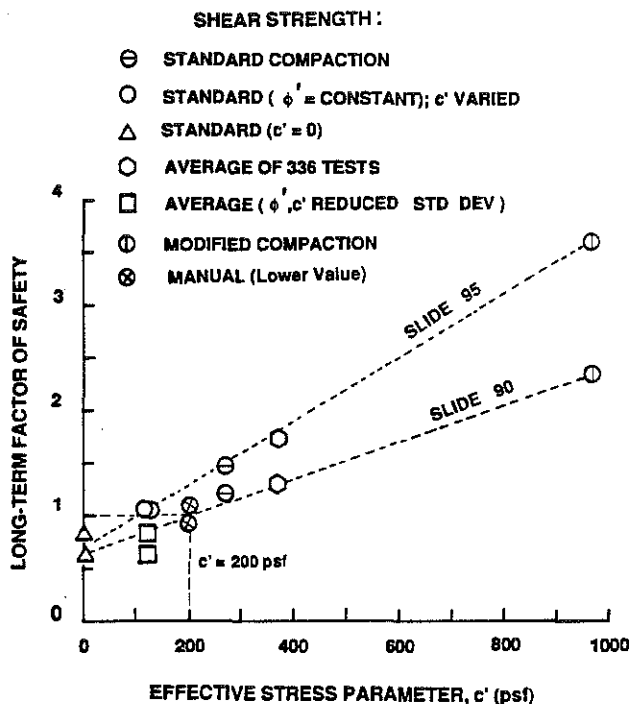
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SHEAR STRENGTH :

⊖   STANDARD COMPACTION
○   STANDARD (ϕ' = CONSTANT); c' VARIED
△   STANDARD (c' = 0)
○   AVERAGE OF 336 TESTS
☐   AVERAGE (ϕ',c' REDUCED STD DEV)
⊕   MODIFIED COMPACTION
⊗   MANUAL (Lower Value)

FIGURE 4.   Factors of Safety as a Function of c'_p -- Reanalyses of Slides 90 and 95 (Kope Shale Embankments) on I 75.

Based on inclinometer data, the unstable mass was wedge-shaped. A major portion of the shear surface passed through the foundation soils. To test the selected design parameters, a stability analysis was performed using the HOPK-I stability computer program. A factor of safety of 0.93 was obtained. Analyses also were performed using the ϕ'-value (equal to 23.9 degrees) obtained from compaction tests and varying c' until a factor of safety of 1.0 was obtained. A cohesive value of zero yielded a factor of safety of 1.12. Details of the stability analysis of the second slope failure, which occurred on KY Route 10 along the AA Highway alignment, are given in the Section entitled "CASE HISTORIES."

Although minimizing settlements of shale embankments was a major consideration, major slides in the Kope and Crab Orchard areas have generally resulted from overstressing of weak clays and shales at or below the bases of embankments on sidehill locations (1-9). Numerous embankment failures have occurred at sidehill locations due to a bearing capacity failure of the foundation. In most cases, back analyses show that laboratory triaxial shear strengths overestimate field shear strengths. Consequently, special considerations were given to formulating design criteria for sidehill locations.

SETTLEMENT

At many locations on the AA Highway, embankment foundations consist of relatively thin (less than 10 feet), overconsolidated soils. Overconsolidated soils usually settle (32) a small fraction of settlements predicted from Terzaghi's (cf 32)

consolidation theory and initial and primary consolidation occur rapidly. Hence, these types of settlements will occur before the placement of the pavements. Although secondary compression may be appreciable over a long period when thick foundation deposits are present (5, 33), this type of settlement was not considered significant because of the thinness of many of the embankment foundations. Except in thick deposits or in alluvial areas, foundation settlement due to primary and secondary compression was not considered significant.

Settlement of embankments constructed of shale is a common problem and is difficult to predict and control. Use of conventional consolidation testing to define deformational properties generally are not applicable because of large particle sizes of the shales. Consolidation rings are usually too small to accommodate the large particles. In a relatively dry state, compacted shales exhibit small compression. However, as shown by Shamburger, et al. (15) and Drnevich, et al. (34), compacted shales (using large molds) when soaked may exhibit large and excessive settlements (creep or secondary compression and shear strain) with increasing time. Based on an approximate correlation (15) of slake-durability indices (20) and compression of soaked shale specimens, gross estimates of settlement of an 80-foot embankment (soaked) constructed of either Kope or Crab Orchard shale is about 0.4 to 0.6 percent of fill height, or 4 to 8 inches for SDI's ranging from about 20 to 40 percent.

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 (according to NAVFAC (29)). As shown in Figure 5 (33), estimated settlements due to secondary compression and shear strain for embankments constructed of fine-grained plastic soils (CL, CH, OL, MN) become significant for embankment heights greater than about 50 feet. Long-term settlements from field measurements and reported elsewhere (33) are compared to the criteria from NAVFACS in Figure 5. Embankments constructed of clays and clayey shales and greater than about 50 feet in height exhibited the most settlement. Large embankment settlements (10) observed on I 75 also confirm these observations. Many of these fills in excess of about 50 feet had settled several inches in a 20- to 25-year period after construction.

As shown by A. W. Bishop (cf 35), an embankment having a factor of safety below about 1.8 will exist in a state of plastic equilibrium. As the factor of safety decreases below this value, the shear strain potential increases. This aspect of embankment settlement is, perhaps, illustrated in Figure 6. Observed long-term settlements (projected to 27.4 years) of several embankments were plotted as a function of the long-term factors of safety (33). Observed pore pressures and slope-inclinometer data obtained over a period of several years were used in stability analyses of those sites. Also, shear strength parameters were obtained from triaxial
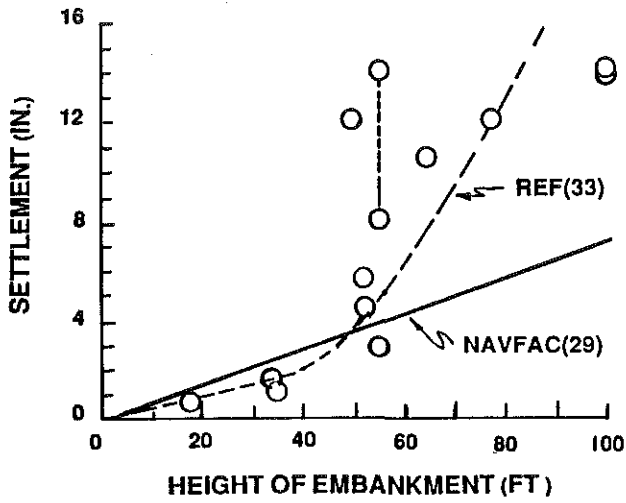
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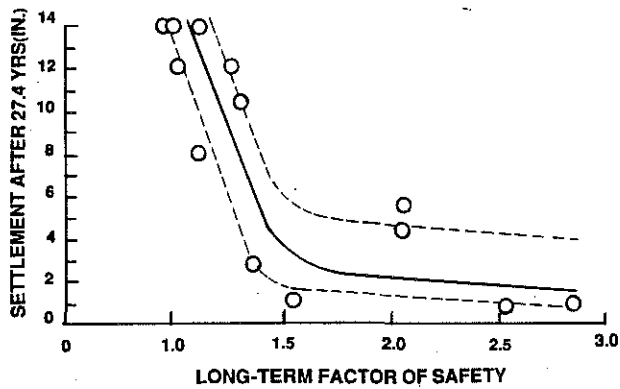FIGURE 5. Long-Term Settlement as a Function of Embankment Height (after Hopkins (33)).

FIGURE 6. Long-Term Settlements as a Function of Long-Term Factors of Safety (after Hopkins (33)).

shear strain and secondary compression, C_{ss} (an empirical coefficient obtained from field measurements). Settlement due to secondary compression and shear strain may be estimated from the following empirical equation:

$$H_{ss} = C_{ss} H_e \log_{10}(t_{ss}/t_c) \qquad (3)$$

and;

$$H_{ss} = (10^{(1.53 \log_{10} F_r - 4.676)}) H_e \log_{10}(t_{ss}/t_c) \qquad (4)$$

in which H_{ss} = settlement of the embankment due to secondary compression and shear strain,

H_e = height of embankment,

t_c = time of placement of pavement (the time between the start of construction and placement of the pavement),

t_{ss} = time at the end of significant secondary compression and shear strain of the embankment (assumed), and

F_r = Ratio of H_e and long-term factor of safety.

Equation 4 relates shear strain and secondary settlement to the height of embankment, the factor of safety, and time. The factor of safety, of course, is a function of shear strength (ϕ' and c'), geometry, dry density (and compactive effort), and pore pressures. As shown in Figures 2 and 3, the shear strength is a function of dry density. Consequently, as the dry density increases, the factor of safety increases since the cohesion, c', increases with dry density. In Figure 7, the settlement, H_{ss}, obtained from equation 4 is plotted as a function of the height of embankment, H_e, for various assumed values of factors of safety. The value of log (t_{ss}/t_c) was taken to be 1.1 for these curves.

sites. Also, shear strength parameters were obtained from triaxial tests performed on "undisturbed" samples obtained several years after construction. Settlements of approach pavements and foundation settlements were monitored for several years. Embankment settlement was obtained by subtracting the foundation settlement from the bridge approach settlement. Settlements obtained in this manner were plotted as a function of the logarithm of time. The relationship between embankment settlement and the logarithm of time was linear. Hence, the linear relationship could be projected with time to obtain settlements at some future date. Each case was projected to 27.4 years (10,000 days). Generally, settlements that occur after 27.4 years were insignificant. Although a considerable scatter of data was present, the data in Figure 6 show that a factor of safety of 1.5 or greater tends to decrease long-term settlements.

As one approach to estimating (gross) long-term settlements of embankments constructed of Kope and Crab Orchard shales, the method proposed by Hopkins (33) was used. The slope of the linear settlement-log time relationship is referred to as the coefficient of
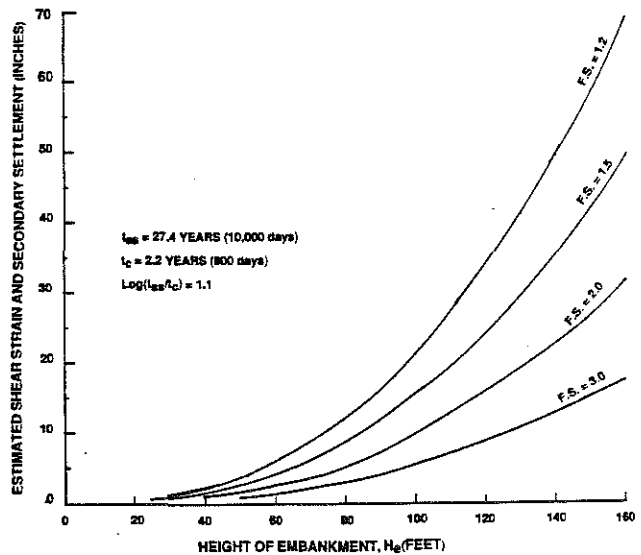

FIGURE 7. Estimated Embankment Settlement as a Function of Embankment Height and the Long-Term Factor of Safety (after Hopkins (33)).

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Based on Figure 7, the effect of the factor of safety on settlement is readily apparent. Based on these curves, and assuming a design factor of safety of 1.5, the long-term settlement of a 50-foot embankment (or pavement) is estimated to be 2.7 inches at the end of 27.4 years. Based on criteria given by NAVFAC, the settlement is estimated to be about 3.6 inches. Based on the approximate method by Shamburger, et al. (15), settlements are estimated to be about 4 inches. Consequently, for embankments 50 feet or less in height on the AA highway, special (high compactive efforts) compaction was not specified since settlements of about 2 to 4 inches were considered tolerable and acceptable.

COMPACTION OF SHALES

Numerous large scale failures of shale embankments along major interstate routes in the early seventies prompted a major research effort into the engineering uses and properties of shale. The research focused on development of guidelines for design and construction of new shale embankments as well as compiling a list of remedial measures that may be used to correct existing embankments in distressed or failure states. A cursory review of existing literature regarding this research and a review of shale compaction procedures of states surrounding Kentucky are discussed below. Finally, a brief discussion of the Special Shale Embankment Construction Specification adopted for design and construction of the AA Highway is presented. A copy of this specification is presented in Appendix A.

LITERATURE REVIEW – COMPACTION

Numerous papers have been published regarding the engineering properties and uses of shales in highway construction. Topics range from geologic classification of shales to case histories of embankment failures. The number of reports is too extensive for a thorough review in this paper. Two major works that influenced current shale compaction procedures for Kentucky and adjacent states containing large areas of shales will be discussed. The most comprehensive study is contained in five reports (12-16) prepared by the U.S. Army Waterways Experiment Station for the Federal Highway Administration (FHWA).

Research Studies Sponsored by FHWA

In 1974, after numerous failures of existing shale embankments, the Federal Highway Administration initiated and sponsored a three-phase study to develop design and construction guidelines for remedial actions on existing failures and for design and construction of new shale embankments. The research was conducted by the U.S. Army Engineers Waterways Experimental Station, Vicksburg, Mississippi. As part of the preliminary work conducted for Phase 1 (12), the researchers made a very important observation that provided the impetus for their, as well as other, research:

"... The underlying cause of excessive settlement and slope failures in highway shale embankments appears to be deterioration or softening of certain shales with time after construction. Inadequate compaction and saturation are two other primary causes of shale embankment problems."

Phase 1 of the research focused on available information regarding classification and material properties, physical and chemical tests, design guidelines and construction control procedures, and sampling and testing procedures for in situ and compacted shales. As part of Phase 1, the researchers summarized current construction procedures used at that time by some of the state highway departments and found that acceptance or rejection criteria varied considerably among the states. They also found that most of the cited causes of failures could be linked to the lack of tests for predicting shale performance with time. The researchers also determined that "... the major factor is the degree of durability exhibited by the shale material and how this durability can be expected to change with time."

The second phase (13) of the research dealt mainly with the evaluation and remedial treatment of shale embankments that were exhibiting distress. Results of this part of the research provided a process by which a highway geotechnical engineer could assess the current overall stability of an existing embankment and also provided recommendations for correcting shale embankment problems. Perhaps the most important part of their results was the statement that surface and subsurface drainage is a critical part of most remedial measures. Therefore, considerations of surface and subsurface drainage are extremely important in design of new embankments.

Phase 3 (14) was designed to fill gaps identified in the first two phases of the study and to provide a comprehensive manual for design and construction of shale embankments. As part of this phase, information and interviews supplied by 15 state highway departments regarding shale embankment performance was correlated with the slake durability index to provide a guideline for placing shale in embankments. As shown in Figure 8, slake-durability indices were generally correlated with lift thickness and embankment performance.

Also as part of Phase 3, the researchers provided conclusions and recommendations regarding the use of shale in highway embankments. Based on those findings, the primary causes of large settlements and slope stability problems are inadequate compaction, saturation, and shale deterioration. Another conclusion was that classification of shales according to long-term durability was absolutely essential in development of design measures. Recommendations included classifying shales as either soil-like or rock-like and then placing the material in lift thicknesses according to classification.
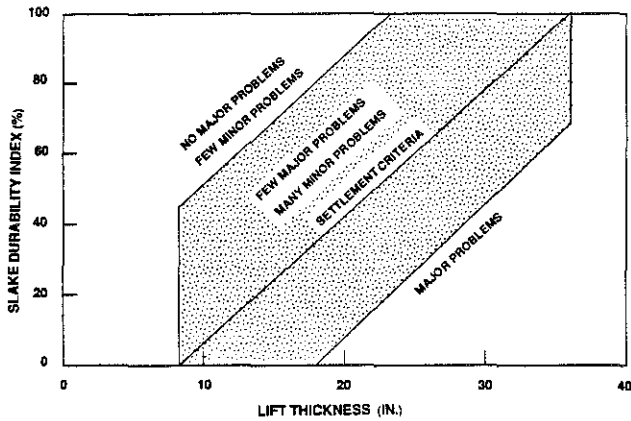
FIGURE 8.   Slake-Durability Index as a Function of Lift Thickness.

Franklin Study

As an extension of the FHWA study and in an effort to provide additional information as to the long-term durability of shales in highway embankments, Franklin in 1981 devised a shale classification system (36). In Franklin's classification system, a shale rating "R" was determined based on slake durability index and either plasticity or point load strength. Plasticity is used in conjunction with the slake durability index when the slake durability index is less than 80 percent. If the slake durability index is greater than 80 percent, the point-load strength is used. Based on these correlations, a shale rating on a scale of 0.0 to 9.0 is determined for the sample. Based on the FHWA studies (12-16), Franklin devised a tentative correlation, as shown in Figure 9, between the R value, lift thickness, and expected performance of shale embankments.
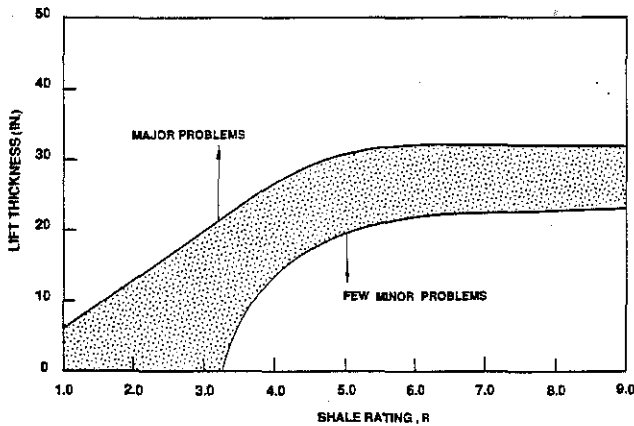

FIGURE 9.   Lift Thickness as Function of Shale Rating ("R") and Expected Performance.

SHALE COMPACTION SPECIFICATIONS
OF ADJACENT STATES

As part of the research effort described above, the Kentucky Transportation Research Program, University of Kentucky, conducted a survey (27) in 1985 of surrounding states with regard to shale compaction specifications. The states of Indiana,

Pennsylvania, Tennessee, Virginia, Ohio, and West Virginia were contacted. Specifications of states responding to this survey are discussed briefly below.

Indiana

The most detailed set of specifications were those obtained from Indiana (37). Those specifications require classifying the shale according to the Franklin system. Any material with a R-value below 5 is considered to be a shale or "soft rock." For embankments constructed of these materials, the Indiana specifications require that the embankment materials be placed in maximum 8-inch loose lifts. After placing, the shales are brought to within -2 percent to +1 percent of optimum moisture content by adding water and "uniformly incorporating" the water with a minimum 24-inch gang disk. The shales are then compacted using three passes of a 60,000-pound static tamping-foot roller, and then using two passes of a 55,000-pound dynamic tamping-foot roller. The newly placed material is bladed between the static and dynamic rollers. Roller speed is restricted to 3 mph.

Indiana also has special requirements when the geologic formation consists of a combination of limestone and shale. The compaction requirements include items mentioned above, but also limits the size of limestone rocks. Limestone fragments of 6 inches in thickness and 1.5 feet in any other direction are prohibited. Moreover, the specifications require encasing the slopes with 10 feet of non-shale, non-erodible material. The number of passes of the static roller is also increased to six from three.

Pennsylvania

The Pennsylvania specifications are not nearly as detailed as the Indiana's, and yet they do require a maximum lift thickness of 8 inches. Shales are placed approximately at optimum moisture content and at 97 percent of the maximum dry density. However, the specifications do not give procedural specifications on how this density is to be obtained. The Pennsylvania specifications also require placing coarse material on the out slopes while requiring that finer material must be placed on the inside of the embankment. Additionally, the specifications require "large" pieces of rock to be broken down until "most" of the voids are filled.

Tennessee/Virginia

Neither Tennessee nor Virginia had special provisions for shale embankments at the time the research survey was conducted. Tennessee handles shale as unclassified material and Virginia treats shale as rock. The Virginia Department of Highways did, however, state that their method of handling shales has created some performance problems.

Kentucky Standard Specifications

Current Kentucky "standard" specifications, adopted in 1975,

make distinctions between durable and nondurable shales and places some requirements on the manner in which shales are to be compacted. Shales having SDI's less than or equal to 95 percent are considered nondurable and are placed in 12-inch loose lifts. According to "standard" procedures, shales are then compacted to 95 percent of maximum dry density. For durable shales (SDI greater than 95), placement is permitted in maximum loose layers up to 3 feet with compaction achieved by blading or dozing so that voids, pockets, and bridging will be minimized. Dimensions of boulders are limited to 3 feet vertically and approximately 4.5 feet horizontally. Recognizing the limitations of the standard compaction specification when applied to the compaction of shales, Kentucky adopted a special compaction provision in 1984. Statewide application of this provision was approved by the Federal Highway Administration in 1985. The provision was modeled after the Indiana shale compaction specifications.

SPECIAL SHALE EMBANKMENT CONSTRUCTION SPECIFICATION

Comparison of Kentucky "standard" specifications to FHWA and Franklin correlations for shale embankment performance indicates a major portion of these embankments will exhibit some form of distress. Based on these correlations and past experience on I 75 and other major highways, a "Special Shale Embankment Construction Specification" was formulated for use in the design and construction of the AA highway. A complete copy of the Special Shale Embankment Construction Specification is presented in Appendix A.

The "Special Shale Embankment Construction Specification" for the AA Highway was patterned after specifications currently used by Indiana. Shales that exhibit SDI's less than 95 are subject to the special compaction criteria as determined by the designer. These materials are to be placed in maximum 8-inch loose layers. After placing and blading to a uniform thickness, water is added to obtain a moisture content of -4 to +2 percent of optimum moisture. Compaction is achieved using a 60,000-pound static tamping foot roller and 50,000-pound vibrating tamping foot roller. Each lift must receive three passes of the static roller and two passes of the vibratory roller. Also, the AA special compaction criteria requires large limestone slabs to be either broken down or removed from the fill.

DESIGN APPROACHES

Three major concerns along the AA highway were slope stability, embankment settlement, and surface and subsurface drainage. Stability and settlement problems were addressed by the selective use of the special shale embankment construction specification. Surface and subsurface drainage considerations are addressed on a site-specific basis using currently accepted and documented practices (38).

During the final design stage of the AA highway, engineers realized that indiscriminate use of the special shale embankment construction specification would be very costly. As a result, general criteria and geometric configurations were developed to aid in determining the applicability of using the special method. Using these general guidelines, consulting engineers were able to design economical embankment configurations for locations where the special criteria were deemed necessary. The following paragraphs describe in more detail how slope stability and settlement were treated.

SLOPE STABILITY

Embankments

Slope stability analyses were conducted using a systematic trial-and-error procedure. Considering preliminary analyses and past performance of shale embankments, initial configurations were chosen based on proposed embankment height. Generally, special compaction techniques were not employed at this stage of the analysis. Table 1 presents a summary of the initial embankment geometry, shear strength parameters, and required factors of safety.

TABLE 1. PARAMETERS USED IN INITIAL SLOPE
                STABILITY ANALYSIS

| EMBANKMENT HEIGHT (ft)	SIDE SLOPES (H:V)	SHEAR STRENGTH PARAMETERS	REQUIRED FACTOR OF SAFETY
<25	2:1	Lower bound*	1.5
25 - 50	2:1 or 2.5:1	Lower bound*	1.5
>50	2.5:1 or 3:1	Lower bound*	1.4

* From Geotechnical Manual, Division of Materials, Kentucky Department of Highways(20)

Based on results of initial stability analyses, embankments that did not meet the required factor of safety were reanalyzed using the special shale compaction specification. In this case, ϕ' and c' were set equal to about 23 to 25 degrees and 200 pounds per square foot, respectively. If the special shale compaction criteria were used as a slope stabilization technique, the entire lower portion of the embankment up to a specified elevation was designated to receive special compaction. Generally, designers selected the top elevation based on trial analyses and tried to keep the specially compacted areas in the passive portion of the embankment. Typical configurations for embankments stabilized using the shale embankment construction specification is shown in Figure 10.

At sidehill locations where major highway failures have occurred due to overstressing of weak soils and shales in Kope and Crab Orchard areas and where the soil underlying the embankment and overconsolidated, plastic soils classified as soil-like (SDI < 95 percent), benching techniques were specified. Typical embankment foundation benching used at sidehill locations are shown in Figure

TYPICAL CROSS SECTIONS

USED FOR FOR SLOPE STABILITY ANALYSIS

SHALE EMBANKMENT COMPACTION SPECIFICATION (S.E.C.S.)
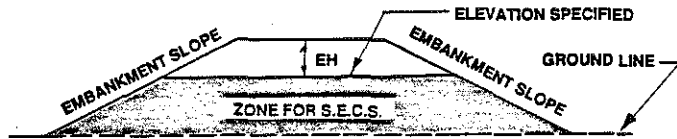
EH = HEIGHT OF EMBANKMENT ABOVE ZONE OF MATERIALS COMPACTED
ACCORDING TO SPECIAL SHALE COMPACTION SPECIFICATION
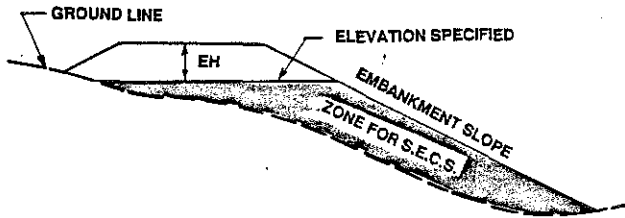WILL TYPICALLY RANGE 20 TO 40 FEET

FIGURE 10.   Typical Configurations for Embankments Stabilized Using the Special Shale Embankment Construction Specification.

11. In applying this technique, thin overburden soils are removed and benches are excavated into the rock disintegration zone (RDZ). Although it was certainly desirable to locate the bottom of the benches below the lower level of the RDZ material, excavation costs were uneconomical because the RDZ in many cases extended several tens of feet below the bottom of the overburden soils.

Cut Slopes

Cut slopes along the AA Highway alignment are predominantly in rock. Only in areas where glacial materials were present and where the Crab Orchard Formation outcrops were special design considerations given to cut slopes. Rock cut slope designs followed standard practice for the lithologies encountered
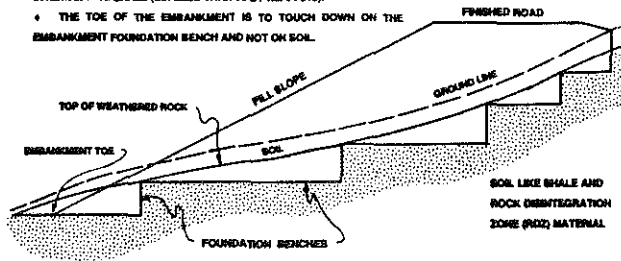

FIGURE 11.   Typical Configuration Showing Benching Scheme for Sidehill Embankment Locations.

(20). Typically, slopes of one horizontal to one vertical with 18-foot wide intermediate benches spaced at 30-foot lift intervals were used in the Kope Formation. Where overburden soils were thin, typically less than 10 feet, cut slopes through the RDZ materials and soils were typically constructed on slopes at two horizontal to one vertical.

Cut slopes through glacial soils presented problems in some areas due to high ground-water tables. At several sites along the alignment, the existing watertable was above the roadway grade. In those areas, slopes of three horizontal to one vertical and subsurface drainage provisions were specified. Also, surface water diversions were placed around the heads of the slopes.

Cut slopes in the Crab Orchard Formation presented unusual problems. The boundary between the overburden soils and the shale bedrock was difficult to establish due to the softness of the shale. Also, soils on existing hill slopes are unstable and are slowly creeping down slope. Cut slopes of two horizontal to one vertical were typically used and extra wide benches were specified.

Slope stability analyses were performed on all cut slopes involving overburden soils of the Crab Orchard Formation greater than 10 feet in thickness. Shear strength parameters used in the analysis were obtained from consolidated-undrained triaxial tests with pore-pressure measurements. However, c'_p was set equal to zero in the analysis to account for "softening" that may occur when a slope is excavated. Generally, ground-water levels obtained during the corridor studies were used in the stability analyses.

Settlement

Settlement of shale embankments due to compression within the fills is a documented problem. Along the AA Highway alignment, numerous embankments greater than 50 feet in height will exist. The tallest embankment on the project is 160 feet in height measured from the toe of the embankment to the top of the embankment. To minimize settlements expected in embankments of this height, increasing the density was the only practical and economical approach. The special shale embankment construction specification was used in these cases to achieve densities higher than those normally obtained from standard compaction (AASHTO T 99).

Settlement was considered to be a problem for all embankments over 50 feet in height and at all bridge approaches. Ideally, it was desirable to construct the entire embankment following the special specification. However, because of increased construction costs, it was not economical to compact the entire embankment using the special compaction specification. The next best alternative was to compact just the core zone of the embankment under the roadway using the special compaction techniques. Figure 12 presents typical cross sections showing the zoned embankment configuration where special compaction techniques were applied. The configuration is a flat-topped pyramidoidal zone within the center of the embankment. The top

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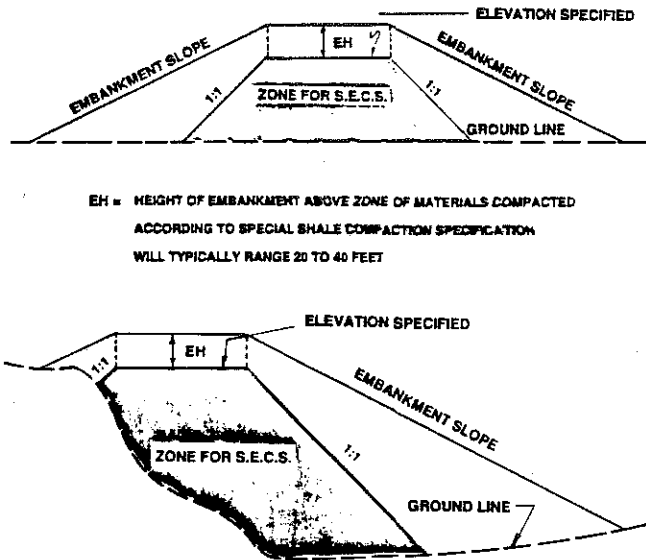
FIGURE 12. Typical Cross Sections Illustrating Zoned Embankment Configurations.

FIGURE 13. Cross Section of Embankment Located at Station 1667+00.

elevation of the zone is typically 25 to 50 feet below roadway grade. The width of the zone at the specified elevation is the width of the roadway template, as measured from shoulder to shoulder. Below the specified elevation, the zone widens on both sides on one horizontal to one vertical slopes. The pyramidoidal zone in the center of the embankment was used because it will provide high densities in the core of the embankment where the highest stresses exist. All bridge approach embankments were compacted according to the special compaction techniques.

CASE HISTORIES

STATION 1667+00

The highest embankment on the AA Highway project is located in Bracken County between Stations 1664+00 and 1672+50. Total height of the embankment at Station 1667+00, measured from the toe of embankment to the shoulder or crest of the embankment, is 166 feet (see Figure 13). The maximum depth of the fill measured along the centerline, which occurs at Station 1668+25, is 133 feet. The embankment is a cross-valley fill and the centerline of the roadway is perpendicular to the existing valley and stream.

Both the Kope and Fairview Formations outcrop in the area of the embankment. Foundation soils average 5 feet in thickness. The Kope Formation outcrops over the entire area. Fill material for this embankment will consist of both Kope and Fairview shales. Foundation soils classified as CL and A-7-6. Triaxial and unconfined compression tests were performed on the foundation soils.
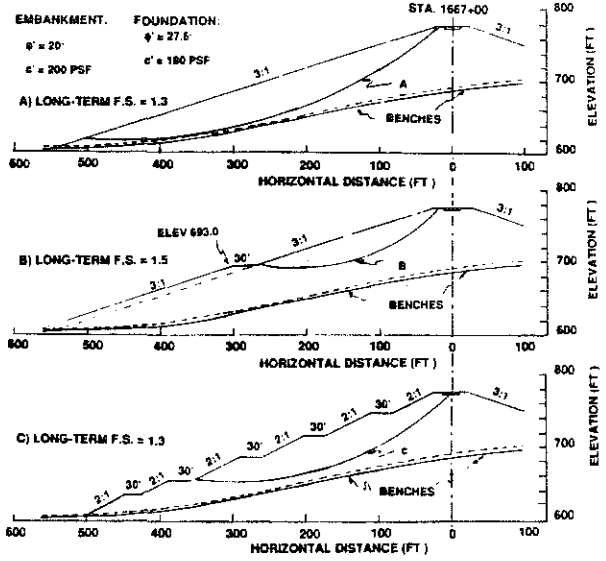
Undrained shear strengths were 4,060, 3,940, 6,920 and 3,800 psf. Results obtained from triaxial tests were ϕ' equal to 27.6 degrees and c' equal to 190 psf. Slope stability analyses were performed using effective stress parameters of the embankment and foundation materials.

Stabilities of three slope configurations were evaluated. Due to the large volumes of material required to construct the embankment, the application of the special compaction specification to the entire embankment was not economical. Considering the great height of this fill, the primary problem was to determine an embankment configuration having stable slopes while attempting to minimize embankment settlements. Fortunately, there were no right-of-way restrictions to limit the location of the embankment toe and large volumes of excavated materials were available to construct flatten slopes. For these reasons, slope stability calculations, based on a ϕ' equal to 20 degrees and c' equal to 200 psf (embankment), were performed to find a stable embankment configuration having an adequate factor of safety. Results of the analyses are given in Figure 13. The selected configuration consisted of a three horizontal to one vertical slope with a berm (middle section in Figure 13).

The special shale embankment construction specification was applied to the core zone to control settlement. Gross estimates of settlement of this embankment based on the approximate correlation given by Shamburger, et al. (15) were 12 to 16 inches. Based on criteria given in NAVFAC (29), estimated settlements of 12 to 15 inches may occur over a period of 15 to 20 years after construction. Using the method by Hopkins (33), a factor of safety of 1.5, and Equation 4, gross estimates of settlements of 35 inches may occur over a period of 25 to 30 years. However, if dry densities in the field approximate dry densities obtained from modified compaction, which may result from the special compaction specification, then factors of

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safety may be nearer to values of 2 to 3. Constructing drains at the base of this fill also will minimize ground-water seepage into the fill. By minimizing seepage, the factor of safety will remain at a high level. In this case, long-term settlements of the fill are estimated (Equation 4 and factors of safety equal to 2 and 3, respectively) to range from 21 to 11 inches. Approximately half of the embankment height in the core zone located under roadway centerline will be constructed according to the special compaction specification.

The embankment at Station 1667+00 is an example of a situation where lower shear strength parameters were used in the slope stability analyses to obtain a stable embankment configuration. The lack of right-of-way restrictions and the availability of fill material allowed placement of the toe of the embankment some 540 feet left of roadway centerline. Constructing embankments of Kope shales to heights such as the embankment at Station 1667+00 presents a dilemma to highway designers since design stabilities and tolerable settlements obtained from analyses must be balanced against acceptable economical considerations. Obtaining acceptable stabilities and tolerable settlements for such large embankments are difficult because of the uncertainties involved in defining appropriate shear strength and settlement parameters of compacted shales.

HERRON HILL EMBANKMENT

Herron Hill is located east of Tollesboro in Lewis County near the community of Ribolt. The Crab Orchard Shale outcrops throughout the area. Kentucky 10 presently crosses the area. On the east side of Herron Hill, embankment slope failures are affecting the existing roadway. The alignment and grade for the AA Highway requires placement of an embankment on the existing slope below KY 10. Station 1826+00 is located in the center of the existing landslide area and was selected as the critical cross section for sampling, instrumentation, and analysis. Figure 14 presents the cross section at Station 1826+00.

The subsurface investigation consisted of obtaining four soil samples and installing slope inclinometers in two of the borings. Only one triaxial compression test with pore-pressure measurements was performed on an undisturbed sample because of difficulties in obtaining good quality soil samples. The triaxial test yielded effective shear strength parameters of $\phi' = 22.0$ degrees and c' equal to 0 psf.
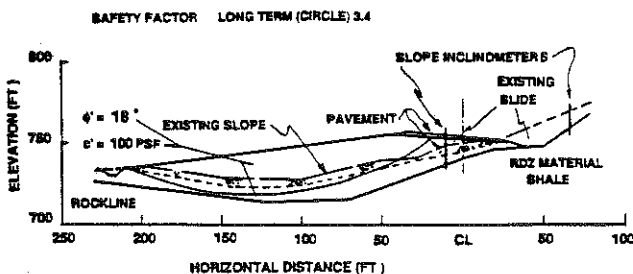
Slope inclinometer data were used to establish the configuration of the failure surface. A wedge method of analysis (31) was used to calculate effective stress strength parameters along the failure surface. In the back calculations, c' was assumed to be zero since the failure mass had moved along the failure surface (38) and ϕ' was varied until a factor of safety of unity was obtained. The resulting shear strength parameters were $\phi' = 18$ degrees and c' = 0 psf.

To evaluate the shear strength parameters of the compacted fill, samples of Crab Orchard Shale were remolded and tested. The samples were remolded at relative compactions of 90 and 95 percent and at different moisture contents. Results of the triaxial testing contained appreciable scatter. Compaction data and effective stress parameters are given in Table 2. Based on the results of these tests

TABLE 2. SHEAR STRENGTH PARAMETERS OF SPECIMENS OF CRAB ORCHARD SHALES COMPACTED AT DIFFERENT WATER CONTENTS AND VALUES OF RELATIVE COMPACTION

	MOLDING CONDITIONS				
TEST SET NUMBER	WATER CONTENT (%)	DRY DENSITY (pcf)	RELATIVE*** COMPACTION (%)	ϕ' (degrees)	c' (psf)
1*	11.0	118.5	100	23.9	768
2**	23.6	104.5	93.3	17.7	0.0
3**	19.8	95.7	90.5	15.9	0.0

* Research Study in Progress (27) -- Raw Shales
** Weathered Overburden Clayey Soils and Shales
*** Standard Compaction (AASHTO T99):
    Set 1: Opt. M.C.= 11.4%; Max. Dry Density = 118.6 psf
    Set 2: Opt. M.C.= 21.4%; Max. Dry Density = 112.0 psf
    Set 3: Opt. M.C.= 20.5%; Max. Dry Density = 105.8 psf

and the back calculations of the existing landslides, effective stress shear strength parameters of ϕ' equal to 18 degrees and c' equal to 100 psf were used for both the foundation soil and embankment materials. Based on these values, a factor of safety of 0.93 was obtained for the embankment at Milepost 118 on I 64 as described previously in the section entitled "Engineering Properties." Hence, these values appeared to be a reasonable choice for design purposes.

Slope stability analyses were performed using embankment slopes of five horizontal to one vertical and six horizontal to one vertical. Factors of safety of 2.1 and 3.4 were obtained for these slopes, respectively. The extremely flat embankment slopes were selected since right-of-way restrictions were not a problem and large volumes of waste materials existed on the project.

In the design, a subsurface drainage blanket consisting of a 5-foot layer of dolomite or a hard durable shale (SDI > 95 percent) will be constructed at the base of the embankment. The drainage layer will extend from the uphill shoulder to the toe of the embankment. Perforated pipes will be placed in all existing surface drainage channels prior to placement of the drainage blanket and embankment.

FIGURE 14. Cross Section of Embankment Located at Herron Hill, Station 1826+00.

The Crab Orchard shale is one of the weakest and most troublesome clayey shales in Kentucky (8, 27). By using waste materials to construct relatively flat slopes, extra compactive effort to densify the embankment shales, and subsurface drainage measures, the embankment at this site should remain stable.

SUMMARY

An overview of geotechnical criteria used in the design of the Alexandria-Ashland Highway was presented. Formulation of design criteria relied heavily on past experiences, local studies, and performances of cut slopes and embankments constructed through and with clayey shales of the Kope and Crab Orchard Formations. In past years, lift thicknesses up to 3 feet were permitted, and slopes of two horizontal to one vertical frequently were used. However, local experiences have shown that the past criteria have resulted in numerous settlement and instability problems and costly maintenance and remediation. Past experiences and case histories also have shown that peak shear strengths of the overconsolidated clays and clayey shales of the Kope and Crab Orchard Formations obtained from triaxial tests frequently overestimate the shear strengths available in the field. Criteria adopted for the design of the AA Highway were specifically formulated to minimize settlements and slope instability of embankments. Based on economical considerations, embankments measuring 50 feet or greater in height were zoned to minimize settlements. In the core zones, procedural specifications required a loose lift thickness of 8 inches and compactive efforts greater than compactive efforts normally used or required. To prevent slope instability, shear strengths smaller than peak shear strengths from triaxial tests were used in stability analyses. The selected design shear strength parameters were based on local experience and analyses of several case histories. One of the major objectives of this paper was to document the geotechnical criteria used in designing the AA Highway so that future evaluations of the criteria may be made by observing the performance of the cut slopes and embankments on the AA Highway.

The problem of predicting stabilities of earth structures was recognized by Terzaghi (39) when he stated the following:

"Comparing the results of shear tests performed in the laboratory with the shear values computed from shear failures on slopes in the field, I realized that the shear strength of cohesive soils in the field may depend on several factors which cannot be adequately reproduced in the laboratory. Hence in 1933 I suggested at the International Congress on Large Dams in Stockholm that the shear values obtained from laboratory tests should be divided by an empirical number which I called the "instability number." It may range from unity for materials which were

found to perform under field conditions in accordance with our forecast and five for those, like the Bearpaw shale, which start to move as soon as the shearing stresses exceed a small fraction of the laboratory strength ..... the need for purely empirical "instability numbers" continues to exist (1960) ..... However, our knowledge of the conditions which require the use of "instability numbers" or, more appropriately "coefficients of ignorance" is steadily increasing, and that is a great asset for the practicing engineer."

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APPENDIX A

SPECIAL SHALE EMBANKMENT CONSTRUCTION SPECIFICATION

Embankments or portions of embankments, as designated in these plans and cross-sections which are to be constructed of soil-like shales (SDI 50 or less by KM (Kentucky Methods) 64-513, slake durability index test) or intermediate shales (SDI greater than 50 but less than 95 by KM 64-513), shall be constructed according to the following construction techniques.

The embankments shall be constructed in successive uniform layers not to exceed 8 inches in loose depth thickness to the full width of the cross-section. Excavation and blasting procedures shall accommodate the selective placement of these materials. The material shall be bladed as required prior to compaction to insure uniform layer thickness.

Large rock fragments or limestone slabs having a thickness greater than six (6) inches and/or a dimension greater than two (2) feet shall be removed from the layer to be compacted, or broken down in place by hand or mechanical means and then incorporated into the layer. Large rock fragments removed from a layer are to be broken down and incorporated into that layer or subsequent layers. Where waste is present on a project the large rock fragments may be wasted at the contractors expense.

Water shall be added to each layer as required to obtain a moisture content near optimum (-4.0 to +2.0 percent of optimum moisture as determined by KM 64-511, moisture-density test) and to accelerate the slaking (breakdown) of the shales. Water shall be added using a spray bar on a truck or other methods of spraying that produce a uniform application as approved by the engineer. The water shall be uniformly incorporated throughout the entire layer by a multiple gang disk with a minimum disk wheel diameter of 24 inches.

Compaction shall be accomplished with a vibratory tamping-foot roller in conjunction with a static tamping-foot roller. The minimum weight for the static tamping-foot roller shall be 60,000 pounds. The minimum total compactive effort for the vibratory tamping-foot roller shall be 50,000 pounds in accordance with the manufacturer's specifications. Larger rollers will be permitted as required to obtain density. Each tamping-foot on the static roller

shall project from the drum a minimum of seven (7) inches. Each tamping-foot on the vibratory tamping-foot roller shall project from the drum a minimum of four (4) inches. The surface area of the end of each foot on both tamping-foot rollers shall be no less than five (5) square inches.

Unless otherwise approved in writing, each embankment lift shall receive a minimum of three passes with the static roller and a minimum of two passes with a vibratory roller. The rollers shall not exceed three (3) MPH during these passes. Each embankment layer shall be compacted to a density of at least 95 percent of maximum dry density as determined by KM 64-511, moisture-density test. The number of passes will be adjusted upward if necessary to obtain 95% of maximum dry density. No additional compensation will be allowed for additional passes as specified herein, the cost of which shall be included in the contract unit prices for roadway excavation, borrow excavation, or embankment in place, whichever applies.

The in-place density will be determined by KM 64-512 (rubber balloon method) and by utilizing a nuclear density gauge. Tests will be conduced at such a frequency as deemed necessary to assure that an entire layer is compacted to the specified density. The layer shall not vary from the optimum moisture content as determined by KM 64-511 (moisture-density test) by more than -4.0 percent to +2.0 percent. This moisture content requirement shall have equal weight with the density requirement when determining the acceptability of a layer. Multiple determinations of moisture content will be required as with density measurements.

The upper surface of a layer shall be shaped so as to provide complete drainage of surface water.

The above described requirements are in addition to sections 207.05, 208.02, and 208.05 of the 1984 Kentucky Standard Specifications for Road and Bridge Construction.

Payment for all labor, machinery, materials, and any costs associated with the construction of shale embankments shall be included in the contract unit price for roadway excavation, borrow excavation, or embankment in place, whichever applies.

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