Long-Term Movements of Highway Bridge Approach Embankments and Pavements

Tommy C. Hopkins
University of Kentucky
September 25, 1986

Mr. Robert E. Johnson, P.E.
Division Administrator
Federal Highway Administration
330 West Broadway
Frankfort, Kentucky 40601

SUBJECT: Implementation Statement
Research Study KYHPR-77-80, "Settlement of Highway Bridge Approaches and Embankments".

Dear Mr. Johnson:

Several findings and recommendations contained in the final report as well as prior interim reports of this study have been incorporated into design practice and are contained in the Geotechnical Manual of the Division of Materials, Kentucky Department of Highways. Subsurface exploration and settlement and stability analyses are now routinely performed at bridge approach sites where the embankments are twenty (20) feet or more in height or where poor foundation conditions exist. This policy is a direct result of information gained during the course of the study. Hence, site specific investigations are conducted. As a result of this study, the long-term target factor of safety is 1.8 (geotechnical manual). When 1.8 cannot be obtained using available fill material, select granular material (Standard Drawings RGX - 100 and RGX - 105) is recommended. However, regardless of the results of the stability analyses, the geotechnical manual requires that select granular material be used when there are ample quantities of suitable granular material available from roadway excavation or when there is a likelihood of undesirable materials, such as soil-like shales, being used to construct the approach embankments. In situations where granular material must be processed or transported long distances, the geotechnical manual recommends that it may be more economical to flatten embankment slopes with a longer bridge structure.

Steps also have been taken to implement several other recommendations contained in the final report. For instance, some recommendations, which include specifying extra compactive effort of materials within a specified distance from the end of the abutment (especially when shales are used -- a special compaction provision has been written, adopted by the Kentucky Transportation Cabinet, and approved by the FHWA for state wide use), using the empirical approach for estimating creep settlement of embankments and for the design of approach embankments, and using the empirical method for estimating the rate of foundation consolidation have...
been presented and are being considered by geotechnical designers in the Division of Materials and Bridges of the Kentucky Department of Highways.

Information gained from this study has been valuable and designers are of the opinion the benefit - costs ratio of this project will be significantly high.

Very truly yours,

R. K. Capito, P.E.
State Highway Engineer
Research Report
UKTRP-85-12

LONG-TERM MOVEMENTS OF HIGHWAY BRIDGE
APPROACH EMBANKMENTS AND PAVEMENTS

by

Tommy C. Hopkins
Chief Research Engineer

Kentucky Transportation Research Program
College of Engineering
University of Kentucky
Lexington, Kentucky

in cooperation with
Kentucky Transportation Cabinet

and

Federal Highway Administration
U.S. Department of Transportation

The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Kentucky Transportation Cabinet, the Federal Highway Administration, nor the University of Kentucky. This report does not constitute a standard, specification, or regulation.

April 1985
**Study Title:** Settlement of Highway Bridge Approaches and Embankments

**Abstract:**
Long-term movements of highway bridge approach embankments and pavements are described and factors that may lead to settlement of highway bridge approach pavements are discussed. Six case histories are presented. The bridge approach sites were observed at different times during the period 1966 to 1985. At four sites, lateral movements, measured from slope inclinometers, of the approach embankments were monitored over time periods ranging from 8 to 14 years. Settlements of the highway bridge approaches were monitored some three to four years after paving. Estimated and observed settlements of the foundations at the study sites were obtained for a length of time sufficient to establish settlement patterns over long-term time periods. Subsurface explorations were conducted prior to and after construction. Triaxial tests were performed on foundation and embankment samples. Based on triaxial shear strengths and long-term seepage measurements, long-term factors of safety of the approach embankments were calculated. An empirical method of estimating the rate of foundation settlement is presented. This method appears to yield settlement estimates that agree better with observed settlements than estimated settlements based on laboratory parameters. An empirical method for estimating the long-term settlements of highway approach embankments is presented. This method may be useful in deciding whether permanent or temporary approach pavements should be installed. With regard to the design and construction of highway bridge approach embankments and pavements, several recommendations are presented.

**Key Words:**
- Highways
- Long-Term
- Pavements
- Design
- Bridge Approaches
- Embankments
- Movements
- Foundations
- Settlement

**Distribution Statement:**
Unlimited with approval of the Kentucky Transportation Cabinet
## CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIST OF FIGURES</td>
<td>iii</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>ix</td>
</tr>
<tr>
<td>INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>SCOPE AND OBJECTIVE</td>
<td>2</td>
</tr>
<tr>
<td>CASE HISTORIES</td>
<td></td>
</tr>
<tr>
<td>LEXINGTON RELIEF ROUTE -- FAYETTE COUNTY</td>
<td>3</td>
</tr>
<tr>
<td>I 24, EDDY CREEK -- LYON COUNTY</td>
<td>9</td>
</tr>
<tr>
<td>I 64, BULL FORK CREEK -- ROWAN COUNTY</td>
<td>35</td>
</tr>
<tr>
<td>I 71, KENTUCKY RIVER BASIN -- CARROLL COUNTY</td>
<td>59</td>
</tr>
<tr>
<td>I 64, SLATE CREEK -- BATH COUNTY</td>
<td>83</td>
</tr>
<tr>
<td>KY 30, NORTH FORK OF KENTUCKY RIVER</td>
<td>102</td>
</tr>
<tr>
<td>ANALYSIS AND DISCUSSION</td>
<td></td>
</tr>
<tr>
<td>SUMMARY OF OBSERVATIONS</td>
<td>107</td>
</tr>
<tr>
<td>PROPOSED EMPIRICAL METHOD OF ESTIMATING RATE OF PRIMARY SETTLEMENT OF FOUNDATIONS</td>
<td>110</td>
</tr>
<tr>
<td>RELATIONSHIPS</td>
<td>110</td>
</tr>
<tr>
<td>EXAMPLE CALCULATION</td>
<td>113</td>
</tr>
<tr>
<td>SECONDARY SETTLEMENTS OF FOUNDATIONS</td>
<td>116</td>
</tr>
<tr>
<td>PROPOSED EMPIRICAL METHOD OF PREDICTING SETTLEMENTS OF BRIDGE APPROACH EMBANKMENTS</td>
<td></td>
</tr>
<tr>
<td>RELATIONSHIPS</td>
<td>117</td>
</tr>
<tr>
<td>DESIGN EXAMPLE</td>
<td>120</td>
</tr>
<tr>
<td>COMPACTION</td>
<td>124</td>
</tr>
<tr>
<td>LATERAL MOVEMENTS</td>
<td>125</td>
</tr>
<tr>
<td>CONCLUSIONS AND DISCUSSION</td>
<td>126</td>
</tr>
<tr>
<td>RECOMMENDATIONS AND IMPLEMENTATION</td>
<td>130</td>
</tr>
<tr>
<td>BENEFITS OF IMPLEMENTATION</td>
<td>141</td>
</tr>
<tr>
<td>REFERENCES</td>
<td>143</td>
</tr>
<tr>
<td>APPENDIX</td>
<td>147</td>
</tr>
</tbody>
</table>
INTRODUCTION

Differential settlement between the highway pavement and bridge deck not only presents a hazard to rapidly moving traffic but creates a rough and uncomfortable ride. Additionally, these surface faults require costly maintenance that usually involves either mudjacking or patching the approach pavement; where a heavy traffic flow exists, this maintenance operation may tend to impede normal flow. Moreover, settlement of bridge approaches adversely affects the durability of road and structure.

Bridge abutments, and therefore bridge decks, usually are founded on relatively stable foundations such as rock or point-bearing piles driven to rock and, practically speaking, cannot settle. The highway pavement, however, is located on an embankment and foundation that are potentially free to settle. The extent to which either settlement of the embankment or foundation contributes to the approach settlement will obviously depend on the particular conditions existing at any given site. Based on previous studies (1-17), factors which may lead to differential settlement between the highway pavement and bridge deck may be summarized as follows:

1. primary consolidation of the embankment foundation,
2. secondary compression and shear strain of the foundation and embankment,
3. improper compaction of the approach embankment,
4. loss of material from and around the abutment and approach pavements,
5. lateral and vertical deformations, or creep, of the bridge approach embankment, and
6. lateral squeeze of the embankment foundation resulting from
settlement.

Based on results of a study completed in 1968 (1) and unpublished data obtained from instrumented sites after 1969, primary consolidation of embankment foundations at the study sites had frequently occurred before placement of the bridge approach pavements. In those cases, there generally was sufficient time for the completion of primary consolidation. However, long-term measurements at two sites, as described below, showed that secondary consolidation may contribute to the settlement of bridge approaches. Improper compaction of the approach embankment may lead to both short- and long-term embankment settlement, especially in embankments of relatively large heights. Poor compaction observed in studies conducted in 1968 (1, 2) oftentimes leads to the loss or internal erosion of soils from around the abutment and approach pavements and, consequently, a loss of pavement support. Finite element analyses of embankments showed that large vertical and lateral deformations may occur in relatively large approach embankments. To minimize bridge approach pavement settlements, each of the above six factors must be examined at a given site. In certain situations, all of the factors may be significant; in other cases, only some of the factors may be important. Generally, detailed subsurface and geotechnical studies will be required to determine the importance of each factor.

SCOPE AND OBJECTIVE

The objective of this study was to observe long-term movements of six bridge approach embankment sites. A variety of means and instruments, including optical surveys, visual inspections and photo-documentation, slope inclinometers, mercury-filled settlement gages (1), settlement platforms, and piezometers were utilized to monitor
movements. Not all sites were fully instrumented; nor were all sites fully observed. The selected sites were constructed at different times during the period 1966 to about 1978. The study period extended from 1966 to 1985. Data and observations obtained at each site illustrate the significance of the six factors listed above. Of primary interest was the magnitude and rate of lateral and vertical movements of bridge approach embankments over a period of several years. Although small movements might occur for one or two years (that is, movements on the order of about 0.2 to 0.5 inch per year), a major question is the cumulative effects of those movements over several years.

CASE HISTORIES

Case histories covering a period of several years are described below. Detailed descriptions and geotechnical data of several of these sites have been described elsewhere (1).

LEXINGTON RELIEF ROUTE -- FAYETTE COUNTY

The Lexington Relief Route Parkers Mill overpass is located on the Lexington Peneplain -- a mildly karst and gently rolling plain containing no prominent knobs or hills -- of the Inner Blue Grass Region of Central Kentucky. This site (Station 576+13 to Station 577+63) is situated approximately two miles southwest of downtown Lexington. In the vicinity of the crossing (Figure 1), the approach embankments are approximately 20 feet in height. Depth of the foundations is approximately 12 feet (Figure 2). The approaches slope downward from each end of the bridge on an approximate two-percent grade, and at approximately 250 feet from the ends of the bridge, embankment heights are about 6 or 8 feet. The approach embankment soils are silty clays
Figure 1. Lexington Relief Route Site.

Figure 2. Typical Cross Section of Embankment and Foundation, Station 575+65, Parkers Mill Road Overpass.
and were compacted in lifts measuring about 12 inches. A single-point mercury-filled settlement gage (1, 5) was installed at centerline of original ground, Station 575+65, on the northwestern approach foundation.

Exposed formations of the Inner Blue Grass Region are a series of sedimentary deposits composed chiefly of limestone with many interbedded shales and some clays. The outcrop pattern of this region is controlled by the Cincinnati Arch. The region can be considered as an old base plain that has been uplifted, and now is being dissected by the present erosional cycle. Consequently, the drainage is well developed throughout the region. Foundation soils at the site are residual chocolate-brown silty clays developed mainly from limestone or calcareous shales. Although those soils are relatively plastic, they are well drained because the joints, cracks, and solution channels of the bedrock allow water to escape rapidly. As a result, those soils develop a fragmentary structure that creates a highly permeable unit. Four borings made at the northwestern approach foundation penetrated limestone approximately 12 feet below ground elevation.

A comparison of predicted and observed settlements (Figure 3) of the northwestern approach foundation shows poor agreement (1). Observed settlements occurred rapidly, but were slower than the predicted rate. The observed magnitude of settlement was only 20 percent of the predicted settlement. Those exaggerated differences are due in part to the over-consolidated condition of the foundation soils. Measurements of settlement at this site show that the foundation has stabilized. The data also show that secondary consolidation of the foundation is not a contributing factor.
Figure 3. Observed and Predicted Time-Settlement Curves and Time-Loading Curves, Parkers Mill Road Overpass.
Approximately half of the northwestern approach embankment was completed between May 5-20, 1966, and the entire embankment was not completed until September 10, 1966. Approximately a month later, the abutments — placed on end-bent piles driven to bedrock — were completed. In the later part of November 1966, the approach pavements were placed. Thus, the approach foundation had roughly four months to consolidate under the load of the entire embankment.

Initial elevations of points on the northwestern approach pavements, obtained December 1, 1966, and a subsequent set of elevations of those points, obtained June 4, 1968, are shown in Figures 4 and 5. Those data show that the approach pavements have settled approximately 0.3 to 0.5 inch in the 18 months between readings. The "settlement cradle", as depicted in Figure 5, is about 75 feet in length for the exit approach. The settlement cradle of the entry approach (Figure 4) is about 175 feet in length. The observed time-settlement curve (Figure 4) reveals that the foundation settled approximately 0.2 inches after placement of the approach pavements. Approach settlement at this site is due mainly to settlement of the foundation and in part to settlement of the embankment. Although many bridge approaches on this route have been patched, total settlements are typically less than about 1 to 2 inches. Approaches at this site were patched on July 23-24, 1968.

In 1973 (17), soil samples were obtained from the northwestern approach embankment. Consolidated-undrained triaxial compression tests with pore pressure measurements were performed. As shown in Figure 6, the effective stress parameters $\phi'$ and $c'$ of the embankment soil were 28.5 degrees and 606 pounds per square foot, respectively. The effective stress parameters $\phi'$ and $c'$ of the foundation soil were 28.8
Figure 4. Settlement of the Entrance Approach of the Southbound Bridge, Parkers Mill Road Overpass.

Figure 5. Settlement of the Exit Approach of the Northbound Bridge, Parkers Mill Road Overpass.
degrees and 214 pounds per square foot, as shown in Figure 7. A
stability analysis of the approach embankment was performed using the
HOPK-I slope stability computer program (18). The long-term factor of
safety obtained from those analyses was 2.53. The critical circle is
shown in Figure 8.

Based on visual inspections conducted in 1985, relatively small
movements of the approach embankments have occurred at this site since
1966. Views of the exit and entry approach pavements, as they existed
in February 1985, are shown in Figures 9 and 10, respectively. A side
view illustrating the verticality of the exit approach abutment is shown
in Figure 11. Although the approaches have been patched, as shown in
Figures 9 and 10, the patching is very thin and extends about 50 feet
from the ends of the bridges. The abutments have remained very stable
over a period of 18 years, that is, they are still plumb and not tilted
or shifted horizontally. No secondary (or very nominal) settlement
occurred at the site. The approach embankments have high stability (FS
= 2.53). The approach embankments at the Parkers Mill site were
compacted very well and approach settlements are less than about 1 inch.

I 24 EDDY CREEK - LYON COUNTY

I 24 crosses the Eddy Creek basin in Western Kentucky approximately
6 miles southeast of Eddyville and 4 miles east of Barkley Lake.
Physiographically, the dominant feature of the area surrounding the site
is Barkley Lake, which was formed within the last decades by damming the
Cumberland River near Gilbertsville. Eddy Creek is situated in the
Western Pennyroyal, a division of the Mississippian Plateau, and it is a
prominent tributary of Barkley Lake. The area — "Land between the
Lakes" — positioned to the west of the site and Barkley Lake
Figure 6. Triaxial Test Results for the Embankment Soils, Parkers Mill Road Overpass.

Figure 7. Triaxial Test Results for the Foundation Soils, Parkers Mill Road Overpass.
Figure 8. Cross Section Used for Stability Analyses, Parkers Mill Road Overpass.
Figure 9. View of Northwestern Bridge Approach (Exit Side), Parkers Mill Road Overpass.

Figure 10. View of Northwestern Bridge Approach (Entry Side), Parkers Mill Road Overpass.
Figure 11. Side View of Northwestern Bridge Approach Abutment (Exit Side), Parkers Mill Road Overpass.
constitutes a transition in geology between Cretaceous and Tertiary sediments of the Jackson Purchase Region and outcrops of Mississippian limestone of the Western Pennyroyal Region. The mildly rolling karst topography of the area lying close to the site is dotted with limestone sinkholes and contains small local relief. The landscape reflects intense faulting. Near the basin, the topography is moderately rugged and more mature. In 1965 the basin was inundated by impounded waters of Barkley Lake.

In the immediate vicinity of the I-24 crossing, the basin is about 1,400 feet wide, extending from Station 3681+00 to Station 3695+00 (Figure 12). The depth of water in the basin measures 8 feet at normal pool elevation of 359 feet. The floor of the basin is level and lies near elevation 351 feet. From Station 3682+80 to Station 3680+70 (north side), the original ground line rises gently on a grade of about 13 percent; on the south side, beginning at Station 3693+25 and ending at Station 3695+50, the original ground line slopes moderately on a grade of 16 percent. Prior to the time of inundation by waters of Barkley Lake, Eddy Creek carved a channel roughly 80 feet wide, extending from about Station 3685+25 to Station 3686+20, and about 12 feet deep, elevation 339 feet to elevation 351 feet, in the basin floor. The foundation of the basin is composed of recent alluvium deposits ranging in thickness from 20 to 40 feet.

From older to younger, the geological formations near the site are the St. Louis and Ste. Genevieve limestones (Meramec Series) of the Mississippian System, loess (Pleistocene Series), and alluvium of the Quaternary System. The five borings apparently penetrated the lower member of the St. Louis limestone.
Construction consisted of an embankment approximately 35 feet high across the lake area and twin bridges across the Eddy Creek channel. After studying the site in 1965, the engineering firm responsible for final plans reported that the total calculated settlement of the foundation under the load of the proposed embankment would be on the order of 4 feet, and at least 3 years would be required for consolidation to reach 100 percent. Those values were apparently calculated from classification data and Terzaghi and Peck's liquid limit formula. The firm noted also that, without artificial drainage of the foundation, failure of the embankments would be imminent due to excessive pore pressures. Consequently, vertical sand drains with a sand blanket and berm embankments for accelerating foundation settlements were recommended. Moreover, a controlled rate of loading was specified.

Design of the embankment and berms, as recommended, is shown in Figures 12 and 13. The berms and main embankment were to be constructed of free draining granular material to a maximum elevation of 364 feet, 5 feet above normal pool elevation. High-water level was expected to be about 6 feet below the shoulder. The top portion of the embankment, from elevation 364 feet to grade elevation, was to be constructed of unclassified roadway material.

Vertical sand drains and a sand blanket were desirable from a standpoint of minimizing long-term, post-construction settlements. However, from an economical standpoint, the drains were objectionable, primarily because they would have to be installed under water. The cost of the drains as determined by the Division of Design was in excess of one million dollars.
Figure 13. Typical Cross Section of Approach Embankment and Foundation, Station 3688+00, I 24 over Eddy Creek.
In the summer of 1966, a study was initiated to investigate the site and recommend safe construction limitations. Primary objectives of the study were to seek a suitable alternate plan of construction that would not require vertical sand drains and to check the adequacy of the foundation to support the proposed embankment without failure or excessive post-construction settlements. Although the original schedule of the Eddy Creek project had not specified early construction, arrangements could be made to contract this section of I 24 well in advance of projects on either end.

Shelby-tube and split-spoon soil samples were obtained from five drill holes. Sampling was performed with a mobile drill rig mounted on a barge. Inspection of those samples revealed that the upper 40 feet of the foundation was composed of wet to saturated, soft, slightly organic silts and clays with some sand, apparently in lenses. Due to the limited number of borings, the horizontal extent of the sand lenses could not be determined. The silts and clays were underlain by what appeared to be a gravel hardpan having standard penetration values in excess of 100. Natural moisture contents of the foundation material ranged between 17 and 63 percent. Unconfined compression tests, consolidated-undrained triaxial tests with pore-pressure measurements, and one-dimensional consolidation tests were performed on specimens trimmed from Shelby tube samples. The average effective angle of friction, \( \phi' \), was 31 degrees, and the average effective cohesion, \( c' \), was 120 pounds per square foot. Fourteen one-dimensional consolidation tests were performed. Details of those tests and laboratory results are given elsewhere (1).
Stability analyses were made for various conditions of loading and foundation consolidation. Factors of safety are shown in Table 1. Shear strength of the embankment was assumed to be equal to that of the foundation in the analyses. A typical cross section of the embankment and foundation used in those analyses (and the consolidation analyses) is shown in Figure 13. The short-term factor of safety (no consolidation) was inadequate for all embankment and berm configurations. However, the long-term factor of safety (complete consolidation) was 2.84 for a berm 18 feet in height by 75 feet in width. Inasmuch as the short-term factor of safety for this berm was almost adequate (FS = 1.03) and the effective stress factor of safety for an average excess pore pressure as high as 15 pounds per square inch (head of water reaching 10 feet above the berm) was adequate (FS = 1.35), the analyses strongly indicated that the embankment could be constructed at a normal rate without sand drains with little risk of failure.

The calculated average ultimate settlement was 16 inches, which was some 32 inches less than the value obtained from index tests and Terzaghi and Peck's formula. The degree of consolidation as a function of time with and without vertical sand drains is shown in Figure 14 (both curves were corrected for an assumed loading period of 100 days). For the case of no sand drains the curve shows that 50 percent of ultimate settlement would be obtained in 240 days and 90 percent would require about 890 days. In the case of sand drains spaced 15 feet apart, assuming the permeability is the same in all directions, that is, \( C_v/C_h = 1.0 \), and assuming no smear, 50 percent of ultimate consolidation would be reached in about 80 days whereas 90 percent would require 170
<table>
<thead>
<tr>
<th>UNIT WEIGHT</th>
<th>BERM WIDTH</th>
<th>BERM HEIGHT</th>
<th>SHORT-TERM FACTOR OF SAFETY</th>
<th>LONG-TERM FACTOR OF SAFETY</th>
<th>PORE-PRESSURE CONDITION</th>
</tr>
</thead>
<tbody>
<tr>
<td>120 lb/ft³</td>
<td>0</td>
<td>0</td>
<td>.75</td>
<td></td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>50</td>
<td>15</td>
<td>.89</td>
<td></td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>75</td>
<td>18</td>
<td>1.03</td>
<td></td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>75</td>
<td>18</td>
<td>2.64</td>
<td>Equivalent to static water table at elevation of berm</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>75</td>
<td>18</td>
<td>2.37</td>
<td>Equivalent to static water table 10 feet above berm elevation</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>75</td>
<td>18</td>
<td>1.88</td>
<td>Equivalent to static water table 20 feet above berm elevation</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>75</td>
<td>18</td>
<td>1.35</td>
<td>Equivalent to static water table 30 feet above berm elevation</td>
<td></td>
</tr>
</tbody>
</table>
Figure 14. Degree of Consolidation-Time Curves, with and without Vertical Sand Drains, I 24 over Eddy Creek, Lyon County.

Figure 15. Observed and Predicted Time-Settlement Curves, Centerline, Station 3688+00; Eddy Creek.
days. The desirability of vertical sand drains was thus readily apparent. Time required for the embankment to set prior to constructing a permanent pavement, and to keep settlement of the bridge approaches within a tolerable range, was about 4 months with sand drains and 20 months without sand drains. Therefore, the consolidation analyses indicated that, without sand drains, an objectionable amount of settlement would occur if a permanent pavement was placed sooner than about 1.5 years after construction of the embankment.

Based on results of the stability and consolidation analyses, the following recommendations were made:

1. The embankment and berms should be constructed without sand drains. Piezometers should be installed along the centerline at about 200-foot spacings. A piezometer reading of 15 pounds per square inch on a gage at the elevation of the berm, 364 feet (an elevation head of 374 feet) would correspond to a long-term factor of safety of 1.35. This reading was considered to be the practical safe limit.

2. Either no pavement or only a temporary pavement should be constructed before foundation consolidation was at least 75 percent completed (approximately 12 inches) indicated by settlement platforms or gages along the centerline at 200-foot spacings.

3. Early construction of the project be specified to increase the consolidation period prior to paving.

4. The contractor be prepared, if necessary, to use a controlled loading rate.

A second alternate construction plan, which also did not involve vertical sand drains, consisted of displacing, either partially or completely, the soft foundation material with fill material. This plan
was never formally proposed, but it was considered more economical than sand drains.

In the latter part of April 1967, six settlement platforms were installed by the contractor at the following locations (see Figure 12):

1. Station 3683+50 on centerline,
2. Station 3688+00, 72 feet left of centerline,
3. Station 3688+00, 72 feet right of centerline,
4. Station 3688+00 on centerline,
5. Station 3691+00 on centerline, and
6. Station 3692+00 on centerline.

These platforms consisted of a 3- by 3-foot steel base and screwed extensions of 2.5-inch diameter steel pipe, approximately 5 feet in length.

Construction of the approach embankments across the lake area began in the early part of May 1968. The first stage of construction involved building the rockfill that formed the outer perimeter of the berms (see Figures 12 and 13). The rockfill was located approximately 170 feet right and left of centerline. Approximately 56,977 tons of limestone was trucked to the project for this purpose. The operation began by dumping rock at the lake edge; this rock was pushed into the lake with a tractor dozer to prepare a platform of sufficient size 1 foot above water. Upon completion of the first lift of rock, a second lift was immediately started to shape the rockfill according to the typical section shown in Figure 13.

The second stage of construction consisted of placing 131,731 cubic yards of a mixture of sand and gravel fill inside the rock embankment to elevation 364 feet. Approximately 54,847 cubic yards of this special
granular fill was delivered to the site by barges. This material was pumped into the barges from the bed of the Tennessee River at a point 10 miles from the site. Upon reaching the site, the barges were parked parallel to the rock embankment and unloaded by cranes operating from the top of the rock embankment. The remainder of the granular material was composed of a blend of locally available bank gravel and sand with a clay content of less than 10 percent. From the top of the berm (elevation 364 feet) to the grade elevation (about 386 feet), the remainder of the embankment was constructed of unclassified roadway soil.

Four double-tube piezometers were installed at the following locations (see Figure 12):

1. Station 3683+00 on centerline, piezometer tip at elevation 335 feet;
2. Station 3688+00 on centerline, piezometer tip at elevation 329 feet;
3. Station 3690+00 on centerline, piezometer tip at elevation 332 feet; and
4. Station 3692+00 on centerline, piezometer tip at elevation 329 feet.

On about October 8, 1968, the top portion of the southern approach embankment was completed, but the northern approach embankment was not completed until about December 30, 1968.

Observed and predicted time-settlement curves for the southern approach foundation are shown in Figures 15 through 19. Measurements obtained from the settlement platforms cover a period of approximately 2 to 3 years. This measurement period was sufficient to determine settlement patterns of the approach embankment foundations.
Figure 16. Observed and Predicted Time-Settlement Curves, 72 feet Right of Centerline, Station 3688+00; Eddy Creek.

Figure 17. Observed and Predicted Time-Settlement Curves, 72 feet Left of Centerline, Station 3688+00; Eddy Creek.
Figure 18. Observed and Predicted Time-Settlement Curves, Centerline, Station 3692+00; Eddy Creek.

Figure 19. Observed and Predicted Time-Settlement Curves, Centerline, Station 3691+00; Eddy Creek.
Based on results of the 1966 foundation study, construction procedures adopted and used for the I-24 Eddy Creek crossing were:

1. The embankment and berms were constructed without sand drains; piezometers were installed along the centerline at about 200-foot intervals to monitor foundation pore pressures and to insure that those pressures did not exceed predetermined critical values.

2. No pavement was to be constructed before the foundation consolidation was at least 75 percent completed as indicated by settlement platforms placed along the centerline at about 200-foot intervals. The consolidation analysis indicated it would take approximately 3 years for the completion of primary consolidation, based on the assumption of double drainage, that is, pore water in the foundation under the applied stress of the embankment would drain toward the top and bottom of the foundation. At about mid-depth of the foundation, sand was encountered in two of five drill holes. The horizontal extent of this material was indeterminate. If the sand extended horizontally throughout the foundation, the consolidation analysis showed that primary consolidation would have been completed in slightly less than 1 year.

3. Early construction of the project was specified to increase the time for consolidation prior to paving in an attempt to eliminate, or minimize, post-construction settlement.

From a standpoint of preventing an embankment failure and a large amount of post-construction foundation settlement, the adopted construction procedures were successful. Shortly after completion of the embankments, primary consolidation ceased and secondary consolidation began, as revealed by settlement measurements obtained
from May 1968, the start of construction, to July 1969, a time interval of 400 days. The rapid completion of primary consolidation was not expected, although not totally unexpected, since past experience at other bridge sites had shown generally that observed primary consolidation ended sooner than expected. The large discrepancy between the time at which primary consolidation ceased and the predicted time for completion has been attributed to difficulties in defining actual field drainage boundaries at the site.

In July 1969, it had been estimated that secondary consolidation would continue for several years and its magnitude would not exceed 3 inches. This prediction was based on the assumption that secondary compression, \( H_{sec} \), of the foundation would appear as a straight line in the plot of settlement as a function of logarithm time, or mathematically would obey the following expression:

\[
H_{sec} = C_s H_t \log_{10} \left( \frac{T_{sec}}{T_p} \right)
\]

(1)

where

- \( H_{sec} \) = settlement from secondary consolidation,
- \( C_s \) = coefficient of secondary consolidation determined from laboratory consolidation tests or field settlement measurements,
- \( H_t \) = initial thickness of compressible stratum or laboratory sample,
- \( T_{sec} \) = useful life of structure or time for which settlement is significant, and
- \( T_p \) = time to completion of primary consolidation.
No reliable method is available for estimating secondary settlements, and Equation 1 is only approximate. However, the equation gives an indication of possible continuing settlement. Observed settlement readings obtained from May 1968 to July 1969 were not sufficient to determine future secondary settlement, but they did indicate that secondary settlement would continue. Since July 1969, additional settlement readings were acquired and significant aspects of those readings are discussed below.

Measurements obtained from May 1968 to June 1971, a time interval of 3 years, on settlement platforms located at Station 3688+00, 72 feet left of Station 3688+00, 72 feet right of Station 3688+00, Station 3691+00 and Station 3692+00 (southern approach foundation) are shown in Figures 15 through 19. Settlement data are plotted as a function of logarithm of time. Data obtained from May 1968 to April 1970 from a settlement platform located on the northern approach foundation at Station 3683+50 are shown in Figure 20. The time, \( T_{100} \), shown in the figures is the end of primary consolidation and was determined by intersecting the straight-line portion of the secondary compression curve with the tangent at the point of contraflexure on the primary compression portion of the curve. It is an empirical procedure; however, it has been shown to yield a close approximation to the end of primary consolidation.

Observed secondary compression appears as a straight line in the plots, Figures 15 through 20, of settlement as a function of logarithm time and can be expressed mathematically by Equation 1. Therefore, the straight line in each plot was projected over an assumed design life of 27.4 years (10,000 days). The amount of secondary compression is
Figure 20. Observed and Predicted Time-Settlement Curves, Centerline, Station 3683+50; Eddy Creek.
constant for any given cycle of logarithm time. For instance, at Station 3688+00, on centerline, the secondary compression (3.2 inches) occurring during the cycle of 100 to 1,000 days is the same as for the logarithm time cycle of 1,000 to 10,000 days as well as for the cycle of 10,000 to 100,000 days (27.4 years to 274 years). Hence, it is apparent that secondary compression will continue for a long period of time, but the rate at which it occurs decreases rapidly with increasing time. Consequently, after 27.4 years the effect of secondary compression on settlement of bridge approach pavements can be considered insignificant.

Settlement of the approach foundations at this site is further illustrated in Table 2, a summary of measured primary and secondary settlements and projected secondary settlements from data in Figures 15 through 20. Primary compression of the southern approach foundation ranged from 12 to 18 inches; for the northern approach foundation, compression was 7 inches. At all locations, primary compression ended rapidly, requiring only 3 to 5 months from the start of construction.

In earlier time intervals, secondary compressions are relatively large, but decrease with increasing time; they become insignificant after 27.4 years. Hence, it is apparent from these data that, had the approach pavements at this site been constructed shortly after completion of the embankment, they would have required maintenance by 1971. Consequently, the benefit of specifying early construction at bridge sites of this type is evident.

As shown in Table 2, total secondary compression of the approach foundations occurring between $t_{100}$ (end of primary compression) and 27.4 ranges from 2.1 to 6.2 inches and averages 4.4 inches. Predicted secondary compression varies between 0.7 to 2.3 inches. The field
### Table: Summary of Primary and Secondary Settlements

#### I 24 Across Eddy Creek, Lyon County

<table>
<thead>
<tr>
<th>TIME FROM TO (YEARS)</th>
<th>INTERVAL 3668+00</th>
<th>3668+00 72 RIGHT</th>
<th>3668+00 72 LEFT</th>
<th>3692+00</th>
<th>3691+00</th>
<th>3683+00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary Consolidation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0 T100</td>
<td>0.27</td>
<td>16.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0 T100</td>
<td>0.41</td>
<td>15.4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0 T100</td>
<td>0.34</td>
<td>2.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0 T100</td>
<td>0.34</td>
<td>17.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0 T100</td>
<td>0.41</td>
<td>17.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0 T100</td>
<td>0.27</td>
<td>7.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Primary Consolidation*

<table>
<thead>
<tr>
<th>TIME</th>
<th>INTERVAL</th>
<th>STA 3668+00</th>
<th>STA 3668+00 72 RIGHT</th>
<th>STA 3668+00 72 LEFT</th>
<th>STA 3692+00</th>
<th>STA 3691+00</th>
<th>STA 3683+00</th>
</tr>
</thead>
<tbody>
<tr>
<td>T100</td>
<td>1</td>
<td>1.8</td>
<td>1.2</td>
<td>0.5</td>
<td>1.3</td>
<td>0.9</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.4</td>
<td>1.4</td>
<td>0.5</td>
<td>1.4</td>
<td>1.0</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.8</td>
<td>0.7</td>
<td>0.3</td>
<td>0.6</td>
<td>0.6</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.9</td>
<td>0.9</td>
<td>0.3</td>
<td>0.8</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.6</td>
<td>0.6</td>
<td>0.2</td>
<td>0.6</td>
<td>0.4</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.4</td>
<td>0.4</td>
<td>0.2</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.3</td>
<td>0.3</td>
<td>0.1</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
</tr>
</tbody>
</table>

*Secondary Consolidation*

<table>
<thead>
<tr>
<th>TIME</th>
<th>INTERVAL</th>
<th>STA 3668+00</th>
<th>STA 3668+00 72 RIGHT</th>
<th>STA 3668+00 72 LEFT</th>
<th>STA 3692+00</th>
<th>STA 3691+00</th>
<th>STA 3683+00</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>6.2</td>
<td>5.5</td>
<td>2.1</td>
<td>5.2</td>
<td>4.0</td>
<td>3.6</td>
</tr>
</tbody>
</table>

*Secondary Consolidation*

<table>
<thead>
<tr>
<th>TIME</th>
<th>INTERVAL</th>
<th>STA 3668+00</th>
<th>STA 3668+00 72 RIGHT</th>
<th>STA 3668+00 72 LEFT</th>
<th>STA 3692+00</th>
<th>STA 3691+00</th>
<th>STA 3683+00</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2.3</td>
<td>1.7</td>
<td>1.3</td>
<td>0.7</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Predicted Primary Consolidation*

<table>
<thead>
<tr>
<th>TIME</th>
<th>INTERVAL</th>
<th>STA 3668+00</th>
<th>STA 3668+00 72 RIGHT</th>
<th>STA 3668+00 72 LEFT</th>
<th>STA 3692+00</th>
<th>STA 3691+00</th>
<th>STA 3683+00</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2.7</td>
<td>3.1</td>
<td>3.1</td>
<td>5.1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Secondary Consolidation to Predicted Secondary Consolidation*

<table>
<thead>
<tr>
<th>TIME</th>
<th>INTERVAL</th>
<th>STA 3668+00</th>
<th>STA 3668+00 72 RIGHT</th>
<th>STA 3668+00 72 LEFT</th>
<th>STA 3692+00</th>
<th>STA 3691+00</th>
<th>STA 3683+00</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>22.4</td>
<td>20.9</td>
<td>14.2</td>
<td>22.3</td>
<td>21.9</td>
<td>10.6</td>
</tr>
</tbody>
</table>

*Settlement (Primary Secondary)*

<table>
<thead>
<tr>
<th>TIME</th>
<th>INTERVAL</th>
<th>STA 3668+00</th>
<th>STA 3668+00 72 RIGHT</th>
<th>STA 3668+00 72 LEFT</th>
<th>STA 3692+00</th>
<th>STA 3691+00</th>
<th>STA 3683+00</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2.6</td>
<td>2.6</td>
<td>1.0</td>
<td>2.3</td>
<td>1.9</td>
<td>1.6</td>
</tr>
</tbody>
</table>

*Settlement: Proaches paved in mid-1972*

<table>
<thead>
<tr>
<th>TIME</th>
<th>INTERVAL</th>
<th>STA 3668+00</th>
<th>STA 3668+00 72 RIGHT</th>
<th>STA 3668+00 72 LEFT</th>
<th>STA 3692+00</th>
<th>STA 3691+00</th>
<th>STA 3683+00</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>2.2</td>
<td>2.3</td>
<td>0.8</td>
<td>2.0</td>
<td>1.6</td>
<td>1.4</td>
</tr>
</tbody>
</table>
Figure 21. View of Northern Entrance Approach Pavements, Eddy Creek, 1984.

Figure 22. View of Northern Exit Approach, Eddy Creek, 1984.
Figure 23. View of Southern Entrance Approach, Eddy Creek, 1984.

Figure 24. View of Southern Exit Approach, Eddy Creek, 1984.
coefficient of secondary consolidation, $C_s$, was approximately 0.007. The ratio of projected to predicted total secondary compression was approximately three at three locations and five at another location.

Approach pavements at this site were constructed in 1976, some 8 years after the start of construction of the approach embankments. Settlements of the approach pavements due to secondary compression is less than about 0.5 inches. Views of the approach pavement observed in 1985, 9 years after placement, are shown in Figures 21 through 24. The approach pavements have settled only slightly.

I 64, BULL FORK CREEK -- ROWAN COUNTY

I 64 crosses Bull Fork Creek and Bull Fork Road over twin bridges, Station 2396+56 to Station 2400+82 (Figure 25), in the Knobs Region of northeastern Kentucky. Bull Fork Creek basin is approximately 1,900 feet wide, beginning at Station 2384+00 and ending at Station 2403+00. The eastern approach embankment is approximately 75 feet high; the western approach is approximately 65 feet high. The embankment extends between bluffs (alternate layers of black shale and sandstone). The bluff nearest the bridge (east side) at Station 2403+00 rises 70 feet in a horizontal distance of 100 feet; the other bluff at Station 2384+00 rises approximately 60 feet in 100 feet. The floodplain on the western side of the bridge is relatively flat, whereas the eastern floodplain rises at an approximate grade of four percent. A large gas main, skewed 45 degrees right, crosses the highway centerline about 3 or 4 feet below ground surface near Station 2399+00, as shown in Figure 25.

Bull Fork basin is filled with water-lain transported soils (recent alluvium) -- black shale and yellow sandstone gravels and sand intermingled with silts and clays. Fluvial gravel and sand exhibit
Figure 25. I 64, Bull Fork Creek Bridge Site.
characteristically rounded shapes resulting from rolling by the swiftly flowing Bull Fork Creek. This creek usually flows most rapidly during the winter and spring seasons. A summary of laboratory test data showing some of the major characteristics of basin materials is presented elsewhere (1). Three borings of the west approach foundation and two borings of the east foundation penetrated black shale approximately 18 feet below ground elevation. The approach embankments, with the exception of the clay cores, were constructed of a greenish shale and sandstone obtained from the Waverly Formation.

Instrumentation included the installation of three double-tubed porous piezometers in the western approach foundation and a single point mercury-filled settlement gage located at ground elevation. A single-tube porous piezometer was installed in the eastern approach foundation, and a multiple-point (three points) mercury-filled settlement gage was located at ground elevation of the western approach embankment. Figure 26 shows typical sections of the eastern and western approach embankments and foundations and the locations and elevations of piezometers and mercury gages. Pore pressures were observed as the water level in vertical extensions of piezometer tubing.

Preliminary borings, performed in 1966, of the basin revealed that portions of the foundation material were relatively soft. Considering the large loads to be applied to the approach foundations and the presence of a gas main, a stability analysis was conducted for both the eastern and western approach embankments. Results of a total stress (short-term) stability analysis based on unconfined compressive strength and an effective stress (long-term) stability analysis based on assumed friction angle of 31 degrees, zero cohesion, and various levels of pore
Figure 26. Typical Cross Sections of the Western and Eastern Approach Embankments and Foundations, I-64 over Bull Fork Creek, Rowan County.

Figure 27. Observed and Predicted Time-Settlement Curves, Station 2396+00, Centerline, I-64 over Bull Fork Creek, Rowan County.
pressure are shown in Table 3. Long-term factors of safety for both embankments were approximately 1.20. Triaxial tests were not performed on soil specimens obtained from this site because most of these samples were saturated and sandy, or gravelly, and could not be handled without seriously disturbing them. However, it was assumed that the basin material would probably have high drained strengths and that an assumed friction angle of 31 degrees would be on the conservative side. The total stress analysis was conservative since the superior strength of the fill material was not considered and some increase in strength due to drainage would be expected. It was recommended that piezometer readings of 10 feet above ground elevation -- elevation 710 for the western embankment and 705 for the eastern embankment -- should be considered critical and that construction of the embankments be temporarily suspended or continued at such a slow rate that pore pressures did not increase further once the critical values were reached. If after construction of the embankment had commenced and critical pore pressures were reached immediately, a controlled loading rate would be formulated.

Consolidation test data indicated that ultimate settlements on the order of 1 foot or slightly greater could be expected and that the consolidation of basin material would proceed rather rapidly, probably faster than indicated by the test data, considering the granular nature of the basin materials. Difficulty was encountered in trimming some consolidation specimens, and no doubt this operation disturbed some of the "undisturbed" samples. Observed settlements and predicted settlements of the western approach foundation are shown in Figure 27. The predicted time-settlement curve was corrected for actual loading
### TABLE 3. SUMMARY OF SAFETY FACTORS, 164 OVER BULL FORK CREEK, ROWAN COUNTY

<table>
<thead>
<tr>
<th>UNIT (lbs/ft³)</th>
<th>WEIGHT (lbs/ft²)</th>
<th>COHESION</th>
<th>ANGLE (degrees)</th>
<th>TOTAL STRESS EFFECTIVE ANALYSIS</th>
<th>EFFECTIVE STRESS ANALYSIS</th>
<th>LONG-TERM</th>
<th>PORE-PRESSURE</th>
<th>SAFETY CONDITION</th>
</tr>
</thead>
<tbody>
<tr>
<td>130</td>
<td>835</td>
<td>0</td>
<td>0</td>
<td>0.55</td>
<td>Equivalent to static water table at ground surface</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>130</td>
<td>0</td>
<td>31</td>
<td>1.21</td>
<td></td>
<td>Equivalent to static water table at ground surface</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>130</td>
<td>0</td>
<td>31</td>
<td>1.21</td>
<td></td>
<td>Equivalent to static water table 10 feet above ground surface</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>130</td>
<td>0</td>
<td>31</td>
<td>1.11</td>
<td></td>
<td>Equivalent to static water table 20 feet above ground surface</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>130</td>
<td>0</td>
<td>31</td>
<td>0.97</td>
<td></td>
<td>Equivalent to static water table 30 feet above ground surface</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>130</td>
<td>0</td>
<td>31</td>
<td>0.62</td>
<td></td>
<td>Equivalent to static water table 40 feet above ground surface</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 28. Observed and Predicted Time-Settlement Curves, Point 1, Station 2401+25, 66 feet Right of Centerline, I 64 over Bull Fork Creek, Rowan County.

Figure 29. Observed and Predicted Time-Settlement Curves, Point 2, Station 2401+70, 142 feet Right of Centerline, I 64 over Bull Fork Creek, Rowan County.
Figure 30. Observed and Predicted Time-Settlement Curves, Point 3, Station 2402+65, 210 feet Right of Centerline, I 64 over Bull Fork Creek, Rowan County.

Figure 31. Pore Pressure Measurements, Western Approach Foundation, I 64 over Bull Fork Creek, Rowan County.
Figure 32. Pore Pressure Measurements, Eastern Approach Foundation, I 64 over Bull Fork Creek, Rowan County.
periods. As shown in this figure, the predicted and observed ultimate settlements are in fair agreement; but the observed rate of settlement proceeded much faster than the predicted rate. Subsequent readings indicated that consolidation of this embankment foundation has ceased.

Observed and predicted time-settlement curves of the eastern approach foundation are shown in Figures 28, 29, and 30 (Units 1, 2 and 3 of the multiple-point mercury-filled settlement gage, respectively). Predicted curves were corrected for the actual loading period. The observed settlements for Point 2 were discontinued due to a failure in the electrical system of that point when the contractor placed waste material on the south flank of the eastern embankment. This necessitated removal of the mercury gage monitoring site. During the rapid work to extend the wires and tubing, an improper connection was made with the wire associated with Point 2. However, proper connections were made for Points 1 and 3 and additional readings were obtained. As shown in these figures, predicted and observed ultimate settlements as well as rates of settlements are generally in poor to fair agreement. Consolidation of the foundation was essentially completed shortly after grade elevation of the embankment was reached. Secondary compression of the foundations was essentially zero, or very small.

Pore-pressure measurements are presented in Figure 31 for the western foundation and Figure 32 for the eastern foundation. Those data are commensurable with the observed settlement readings— that is, they show almost instantaneous dissipation just as the settlements were almost instantaneous. T-stakes placed around the toe of the eastern embankment showed no movement during or after construction.
Figure 33. Pavement Profiles, Westbound, East End Approach (Outside Pavement Edge), Bull Fork Creek.

Figure 34. Pavement Profiles, Westbound, East End Approach (Inside Pavement Edge), Bull Fork Creek.
Figure 35. Pavement Profiles, Eastbound, East End Approach (Outside Pavement Edge), Bull Fork Creek.

Figure 36. Pavement Profiles, Eastbound, East End Approach (Inside Pavement Edge), Bull Fork Creek.
Figure 37. Pavement Profiles, Eastbound, West End Approach (Outside Pavement Edge), Bull Fork Creek.

Figure 38. Pavement Profiles, Eastbound, West End Approach (Inside Pavement Edge), Bull Fork Creek.
Figure 39. Pavement Profiles, Westbound, West End Approach (Outside Pavement Edge), Bull Fork Creek.

Figure 40. Pavement Profiles, Westbound, West End Approach (Inside Pavement Edge), Bull Fork Creek.
Approach pavements at this site were placed in August 1968. Construction of the western embankment began January 6, 1967, and was completed July 20, 1967. Approximately 2 months later, pile-end-bent abutments were completed. Thus, this embankment and its foundation had approximately 1 year to consolidate before placement of the approach pavements. Construction of the eastern embankment began August 2, 1967, and was completed October 5, 1967. Pile-end-bent abutments were completed in November. This embankment and its foundation had approximately a year to consolidate before placement of the approach pavements.

In late October and early November of 1968, pavement elevations were obtained. Subsequent pavement elevations of all four approaches were obtained until December 1971, a period of approximately 3 years. Elevations were measured at the inside and outside edges of each approach pavement. Pavement settlements as a function of horizontal distance for the eastern approach embankments are shown in Figures 33 through 36. During the 3-year period, the eastern approaches settled approximately 4.5 inches (average value). The maximum settlement occurred about 50 feet from the ends of the bridges. However, settlements of 2 to 4 inches occurred within 300 feet of the ends of the approaches. At 300 feet, a transition occurs between the fill and a cut section. Pavement settlements as a function of horizontal distance from the ends of bridges for the western bridges are shown in Figures 37 through 40. Maximum average settlements obtained over a period of about 2.5 years was 3.4 inches. The maximum settlement occurred approximately 25 feet from the end of the bridge. Length of the settlement cradle was 130 feet. Settlement of about 1 inch occurred within 1,200 feet behind
the ends of the western abutments. Measurements of pavement profiles were discontinued after 1971 because the approaches were mudjacked and meaningful measurements could not be obtained. Additionally, since 1969 the approach pavements have been patched on numerous occasions.

In Figure 41, the maximum observed settlement from each set of profile measurements of the eastern approach pavement is plotted as a function of the logarithm of time. Those data were taken from Figure 33. The relationship in Figure 41 is linear and applies to the outside pavement edge of the westbound east end approach. If the linear relationship is projected, as shown in Figure 41, then the eastern approach embankment would settle about 12 inches in about 10,000 days (27.4 years) -- December 18, 1994. After that date, settlements would be almost insignificant. For example, 27.4 years after December 18, 1994, the approach would settle only an additional 1.8 inches. Maximum observed settlements of the western approach pavement are plotted as a function of the logarithm of time in Figure 42. Projection of the linear relationship shows that the western approach embankment (and pavement) will settle about 10.4 inches by December 18, 1994. Settlements after that date will be small and almost insignificant. Since primary and secondary consolidation of the eastern and western approach foundations ceased before construction of the approach pavements, the projected settlements of the approach pavements are entirely due to settlement of the approach embankments.

Profiles at the Bull Fork site show that settlements near the bridge ends are much larger than settlements at some distance. Such patterns show that settlements of the embankment are not only due to volume changes (secondary compression) of the embankment material but also
Figure 41. Settlement of the Eastern Approach Embankment, I 64, Bull Fork Creek.

Figure 42. Settlement of the Western Approach Embankment, I 64, Bull Fork Creek.
includes a shear strain deformation. Shear strain is more significant when the long-term factor of safety is less than about 1.5. To investigate lateral creep of the approach embankment, which could lead to vertical deformations, a slope inclinometer was installed in the eastern approach embankment in 1970 at the top of the slope. The inclinometer was installed near Station 2401+00 between the bridge abutments. This inclinometer was monitored over a 10-year period. Resultant lateral movements are shown as a function of depth in Figure 43. The largest lateral movement occurred in the top 40 feet of the embankment. The magnitude of that movement ranged from about 2.5 inches to 7 inches. Movement of the top 3 feet of the inclinometer casing was ignored since that portion of the casing protruded above the ground surface. From a depth of 40 feet to about 68 feet, the maximum movement was about 2 inches. At depths below 68 feet, the maximum movement was less than 1.5 inches. Lateral (resultant) movements as a function of time at selected depths are shown in Figure 44. The top 40 feet of the embankment has moved over the 10-year period (1970 to 1980) at a rate ranging from approximately 0.2 to 0.5 inch per year. Hence, for a short period of time, movements are relatively small. However, in the long term, movements become significant.

During installation of the slope inclinometer in 1970, undisturbed Shelby tube samples were obtained from the inclinometer boring and three other borings. The eastern embankment had been constructed of shale and sandstone, containing a large percentage of shale. At that time, specifications did not distinguish between durable rock and non-durable rock. Consequently, the embankment was constructed using lift thicknesses of about 2 to 3 feet. Natural water contents of the fill
Figure 43. Resultant Movements in Inches as a Function of Depth in Feet, Bull Fork Creek.

Figure 44. Resultant Movements at Selected Depths as a Function of Time, Bull Fork Creek.
ranged from about 8 percent up to 40 percent. The fill consisted of a mixture of weathered soft shale and hard durable rock. Consistency of specimens from the fill ranged from soft to medium firm. Liquid limits of the fill material ranged from 25 to 31 percent. Plasticity indices ranged from 4 to 11.

Consolidated-undrained triaxial compression tests with pore-pressure measurements were performed on material from the eastern fill. Results of those tests (p-q diagram) are shown in Figure 45. Effective stress parameters $\phi'$ and $c'$ were 29.8 degrees and 69 pounds per square foot, respectively. Stability analyses of the eastern approach embankment, Figure 46, based on an effective stress analysis and observed ground-water elevations obtained from the slope inclinometer casing, yielded a factor of safety of 1.25. A stability analysis of the western approach embankment, Figure 47, yielded a factor of safety of 1.33. Hence, the analyses indicated stability of both approaches is low.

Visual inspection of the site in 1984 showed that considerable settlements have occurred since 1968 when the concrete approach pavements were constructed. During the 16-year period, the approaches have been mudjacked and patched on several occasions. Views of the approach pavements located on the eastern embankment are shown in Figures 48 and 49. All abutments have rotated or tilted downward in the back. A view of the tilt of the eastbound, east end abutment is shown in Figure 50. As the abutments have tilted, which has caused a forward movement of the bottom of the front of the abutment, cracks have developed in the fill in front of the abutments. With a separation of berm materials and the front of the abutment, erosion due to runoff at the ends of the bridge has occurred (Figure 51). Views of the western
Figure 45. Triaxial Test Results (p-q Diagram), Bull Fork Creek, Eastern Approach Embankment.
Figure 46. Cross Section (along Centerline) of the Eastern Approach Embankment, Bull Fork Creek.

Figure 47. Cross Section (along Centerline) of the Western Approach Embankment, Bull Fork Creek.
Figure 48. View of Westbound, East End Bridge Approach, Bull Ford Creek.

Figure 49. View of Eastbound, East End Bridge Approach, Bull Fork Creek.
Figure 50. Tilted Bridge Abutment, Eastbound, East End, Bull Fork Creek.

Figure 51. View of Erosion in Front of Eastbound, East End Abutment, Bull Fork Creek.
approaches are shown in Figures 52 and 53. Tilt of the westbound, west end abutment is shown in Figure 54.

I 71, KENTUCKY RIVER BASIN -- CARROLL COUNTY

I 71 crosses the Kentucky River over twin bridges, Station 2111+85 to Station 2119+85, approximately 3-miles southwest of Carrollton and the junction of this river with the Ohio River (Figure 55). This site is located in the Outer Bluegrass Region of northern Kentucky. The Kentucky River basin at this crossing is approximately 1.5 miles wide. Elevations of original ground of the southwest and northeast sides of this broad flat flood plain are approximately 465 feet and 460 feet, respectively. Generally, normal pool elevation is 430 feet with a high water elevation of 480 feet recorded in 1937. On the southwest side of the river, near Station 2106+00, the highway embankment crosses at a right angle to a small drainage basin. From this intersection, the basin makes a 90 degree turn and continues parallel with the south bank of the highway embankment until it junctions with the river. The ground elevation of the bottom of this basin is approximately 445 feet and the width is on the order of 500 feet. Water from the Kentucky River often spills over into this basin and becomes trapped. Preliminary borings revealed that the basin contained saturated, very soft, sandy clay for an approximate depth of 8-10 feet. From Station 2106+00 to Station 2111+85, approximately one-third of the embankment lies within this basin. As shown in Figure 56, the southwestern approach embankment ranges in height from 35 to 55 feet and the thickness of the foundation is approximately 100 feet.

The southwestern approach foundation is composed of recent alluvial deposits of clay, sandy clay, loose and very fine sand, and some silt.
Figure 52. View of Westbound, West End Bridge Approach, Bull Fork Creek.

Figure 53. View of Eastbound, West End Bridge Approach, Bull Fork Creek.
Figure 54. Tilted Bridge Abutment, Westbound, West End, Bull Fork
Figure 56. Typical Cross Section of Embankment and Foundation, Near Station 2111+50, I 71 over Kentucky River, Carroll County.
Field and laboratory descriptions of soil samples obtained from three borings of the southwestern foundation and one boring of the northeastern foundation are presented elsewhere (1). The geology of this region was significantly influenced indirectly by the Illinoian and Wisconsin ice sheets. Both ice sheets reached Kentucky; however, only the Illinoian crossed over the present Ohio River, leaving some scattered drift in the Ohio River counties from Oldham to Bracken. This drift was not of sufficient thickness to materially influence the topography or the soils of this area. The presence of geologic erratics, or large boulders, at high elevations suggests there was a Pre-Illinoian ice invasion. More significantly, the heights of the Illinois (600 to 620 feet) and the Wisconsin (540 feet at Cincinnati) fills were sufficient to have ponded waters of the northward flowing Kentucky River. Then, supposedly at separate times, partial filling of this river area occurred by deposition of alluvium in temporary lakes.

Natural water contents of the southwestern foundation soils ranged from about 17.5 percent to 31 percent. Liquid limits ranged from about 23 to 36 percent for the sandy clay zones while the sandy layers ranged from non-plastic to 20 percent. Plasticity indices ranged from non-plastic to 14 percent for all layers.

Six single-tube porous-element remote-reading piezometers were installed at the following locations:

1. Station 2105+50 on centerline, Piezometer Number 1 at elevation 382 feet, Piezometer Number 2 at elevation 402 feet.

   2. Station 2110+75, 65 feet right of centerline, Piezometer Number 3 at elevation 397 feet and Piezometer Number 4 at elevation 425 feet.
3. Station 2111+70, 42 feet left of centerline, Piezometer Number 5 at elevation 395 feet and Piezometer Number 6 at elevation 426.

Two single-point mercury-filled settlement gages were installed at Station 2111+50, one 42 feet left and the other 65 feet right of centerline. The relative locations of all instrumentation are shown in Figures 55 and 56.

Triaxial and unconfined compression tests were performed to define the shear strength of the embankment foundation. Summary data are shown in Table 4. The average effective angle of internal friction was 28 degrees, and the average effective cohesion was about 1.3 pounds per square inch. The average unconfined compressive strength was approximately 2,300 pounds per square foot. Unconsolidated-undrained triaxial tests were performed on some saturated sandy specimens; results were included in the unconfined compressive strength average.

Unconfined compressive strength data were used in a computer analysis to determine the factor of safety for stability of the embankment for the short-term case (assuming no strength gain due to consolidation). Analyses were made for the embankment as designed as well as with added berms of various sizes. Also, the stability was determined for each berm configuration assuming both equal and different strengths for the foundation and embankment. For the homogeneous case, the strength of both the embankment and foundation were assumed to be represented by the average of the unconfined compressive strength of the foundation material. Four test results more than 50 percent higher than the average were neglected in determining that average. For the layered case, the foundation strength determined as above was used; the embankment strength was assumed to be represented by the higher values.
<table>
<thead>
<tr>
<th>LOCATION</th>
<th>SAMPLE NO.</th>
<th>DESCRIPTION</th>
<th>DEPTH (feet)</th>
<th>MOISTURE CONTENT (%)</th>
<th>UNCONFINED COMPRESSION TEST MAX STRESS (psi)</th>
<th>CONSOLIDATION PRESSURE (psi)</th>
<th>COHESION (psi)</th>
<th>FRICTION ANGLE (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>42°12'17&quot; N 211+50</td>
<td>H-1 S-3C</td>
<td>Wet, brown, sandy clay</td>
<td>15-18</td>
<td>19.3</td>
<td>15.0</td>
<td>5.0</td>
<td>25.0</td>
<td>32.0</td>
</tr>
<tr>
<td></td>
<td>H-1 S-3E</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.0</td>
<td>25.0</td>
</tr>
<tr>
<td></td>
<td>H-1 S-9A</td>
<td>Saturated, brown, sandy clay</td>
<td>25-28</td>
<td></td>
<td>35.0</td>
<td>2.5</td>
<td>25.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-1 S-9D</td>
<td></td>
<td></td>
<td></td>
<td>15.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-1 S-14R</td>
<td>Saturated, gray, soft clay</td>
<td>95-98</td>
<td></td>
<td>15.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-1 S-14B</td>
<td></td>
<td></td>
<td></td>
<td>35.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-1 S-14C</td>
<td></td>
<td></td>
<td></td>
<td>55.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sta 2106+85,</td>
<td>H-2 S-12A</td>
<td>Moist, gray clay</td>
<td>80-83</td>
<td></td>
<td>25.0</td>
<td>0.0</td>
<td>32.0</td>
<td></td>
</tr>
<tr>
<td>Centerline</td>
<td>H-2 S-12B</td>
<td></td>
<td></td>
<td></td>
<td>65.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-2 S-3C</td>
<td>Moist, brown clay</td>
<td>15-18</td>
<td>27.1</td>
<td>45.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-2 S-3D</td>
<td></td>
<td></td>
<td></td>
<td>25.0</td>
<td>1.5</td>
<td>25.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-2 S-3E</td>
<td></td>
<td></td>
<td></td>
<td>5.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-2 S-3F</td>
<td></td>
<td></td>
<td></td>
<td>65.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>42°12'17&quot; N 211+50</td>
<td>H-1 S-1</td>
<td>Moist, brown clay</td>
<td>5-8</td>
<td>26.8</td>
<td>28.73</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-1 S-2A</td>
<td>Moist, brown clay</td>
<td>10-13</td>
<td></td>
<td>28.03</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-1 S-3B</td>
<td>Wet, brown, sandy clay</td>
<td>15-18</td>
<td></td>
<td>10.59</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-1 S-4B</td>
<td>Wet, brown, sandy clay</td>
<td>20-23</td>
<td>30.3</td>
<td>14.91</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-1 S-9E</td>
<td>Saturated, brown, sandy clay</td>
<td>25-28</td>
<td>42.5</td>
<td>10.61</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-1 S-6B</td>
<td>Moist, gray sand</td>
<td>40-43</td>
<td>38.4</td>
<td>8.58</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-1 S-9</td>
<td>Moist, gray sand</td>
<td>45-48</td>
<td>20.9</td>
<td>15.72</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sta 2106+85,</td>
<td>H-2 S-1B</td>
<td>Moist, brown clay</td>
<td>5-8</td>
<td></td>
<td>34.49</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Centerline</td>
<td>H-2 S-2</td>
<td>Moist, brown clay</td>
<td>10-13</td>
<td></td>
<td>22.94</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-2 S-3B</td>
<td>Moist, brown clay</td>
<td>15-18</td>
<td>27.1</td>
<td>14.49</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-2 S-4</td>
<td>Wet, brown, sandy clay</td>
<td>20-23</td>
<td>30.5</td>
<td>7.88</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-2 S-12D</td>
<td>Moist, gray clay</td>
<td>80-83</td>
<td></td>
<td>6.19</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>H-2 S-13B</td>
<td>Wet, gray clay</td>
<td>85-88</td>
<td>26.8</td>
<td>7.28</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
that were ignored previously. Factors of safety determined in this manner were all low (less than one -- see Table 5).

Since the foundation is roughly one-half sand, significant consolidation and strength gain were anticipated during the construction period. Therefore, triaxial tests were performed to determine the long-term strength parameters (the unconfined compression tests were performed and analyzed before triaxial testing). If the total stress analysis based on the unconfined compression test data had produced adequate factors of safety, it would not have been necessary to do the consolidated-undrained (with pore-pressure measurements) triaxial testing. However, as shown in Table 5, the total stress factors of safety were inadequate and triaxial tests were necessary to define effective stress shear strengths. Long-term factors of safety -- after the strength gain due to consolidation -- also are shown in Table 5. Water-table heights were measured from the toe of the fill at the critical sections, that is, the elevation of the slough right of centerline (elevation 446 feet).

The minimum calculated factor of safety for pore pressures equivalent to a static water table at the elevation of the toe of the fill was 1.28, and the minimum calculated factor of safety corresponding to the water table at 20 feet above the toe was 1.18. Considering that an error of plus or minus 15 percent is to be expected in a stability analysis of this type, it was recommended that embankment construction be temporarily suspended if piezometer readings (corrected to the slough elevation) reached 10 psi or 20 feet of head, a critical water table elevation of 466 feet. The maximum observed pore pressures, presented in terms of equivalent water table elevations, are shown below:

Piezometer 1 = below ground elevation
### TABLE 5: SUMMARY OF SAFETY FACTORS, I 71 OVER KENTUCKY RIVER, CARROLL COUNTY

<table>
<thead>
<tr>
<th>FOUNDATION</th>
<th>EMBANKMENT</th>
<th>WATER TABLE</th>
<th>SHORT-TERM SAFETY FACTOR</th>
<th>LONG-TERM SAFETY FACTOR</th>
<th>DIMENSIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>C(lbs/ft²)</td>
<td>tan $\theta$</td>
<td>C(lbs/ft²)</td>
<td>tan $\theta$</td>
<td>Elevation</td>
<td>(feet)</td>
</tr>
<tr>
<td>1077</td>
<td>0.000</td>
<td>3172</td>
<td>0.000</td>
<td>446</td>
<td>0.82</td>
</tr>
<tr>
<td>1077</td>
<td>0.000</td>
<td>1077</td>
<td>0.000</td>
<td>446</td>
<td>0.82</td>
</tr>
<tr>
<td>1077</td>
<td>0.000</td>
<td>3172</td>
<td>0.000</td>
<td>446</td>
<td>0.93</td>
</tr>
<tr>
<td>1077</td>
<td>0.000</td>
<td>3172</td>
<td>0.000</td>
<td>446</td>
<td>1.04</td>
</tr>
<tr>
<td>1077</td>
<td>0.000</td>
<td>3172</td>
<td>0.000</td>
<td>446</td>
<td>0.84</td>
</tr>
<tr>
<td>1077</td>
<td>0.000</td>
<td>1077</td>
<td>0.000</td>
<td>446</td>
<td>0.91</td>
</tr>
<tr>
<td>1077</td>
<td>0.000</td>
<td>1077</td>
<td>0.000</td>
<td>446</td>
<td>1.03</td>
</tr>
<tr>
<td>1077</td>
<td>0.000</td>
<td>1077</td>
<td>0.000</td>
<td>446</td>
<td>0.84</td>
</tr>
<tr>
<td>144</td>
<td>0.543</td>
<td>144</td>
<td>0.543</td>
<td>446</td>
<td>1.94</td>
</tr>
<tr>
<td>144</td>
<td>0.543</td>
<td>144</td>
<td>0.543</td>
<td>456</td>
<td>1.26</td>
</tr>
<tr>
<td>144</td>
<td>0.543</td>
<td>144</td>
<td>0.543</td>
<td>466</td>
<td>1.22</td>
</tr>
<tr>
<td>144</td>
<td>0.543</td>
<td>144</td>
<td>0.543</td>
<td>476</td>
<td>1.24</td>
</tr>
<tr>
<td>0</td>
<td>0.625</td>
<td>288</td>
<td>0.466</td>
<td>446</td>
<td>1.28</td>
</tr>
<tr>
<td>0</td>
<td>0.625</td>
<td>288</td>
<td>0.466</td>
<td>456</td>
<td>1.21</td>
</tr>
<tr>
<td>0</td>
<td>0.625</td>
<td>288</td>
<td>0.466</td>
<td>466</td>
<td>1.18</td>
</tr>
</tbody>
</table>
Piezometer 2 = 466.2 feet
Piezometer 3 = 455.2 feet
Piezometer 4 = 454.0 feet
Piezometer 5 = below ground elevation
Piezometer 6 = 468.0 feet

At Piezometer 6, the water level slightly exceeded the recommended elevation. However, this reading was observed immediately after the embankment had been completed to within 5 feet of the final grade elevation of 501 feet. Although the computed factor of safety was still somewhat above one, no more fill was placed until after there was a substantial decrease in the observed pore pressures.

Preliminary estimates of ultimate settlement for the drainage basin foundation, based on Terzaghi and Peck's liquid limit formula and soils data contained on the plan sheets, ranged from 4 to 6 feet. However, the extent of the sand layers was unknown at that time, and it was assumed that the foundation was composed entirely of clay. As a result, those estimates were exaggerated. More refined estimates based on consolidation tests performed on undisturbed soil samples and more detailed information of the foundation conditions were considerably less -- approximately 2 feet. Several consolidation tests were performed on soil samples collected from each layer of the foundation, and average void ratio-log pressure curves were obtained for each layer. Those curves are shown elsewhere (1). Using those average curves, ultimate settlements were calculated. Coefficient of consolidation-log pressure curves, from which an average curve was derived for the uppermost clay layers and the clay layers located nearest to rock, are presented elsewhere (1). An average coefficient of consolidation was selected for
each of the separate clay systems and predicted time-settlement curves were calculated for the foundation at Station 2111+50, 65 feet right and 42 feet left of centerline. The average coefficient of consolidation used for obtaining the time-settlement curve, Station 2111+50, 65 feet right of centerline, did not include the coefficient of consolidation—log pressure curves of the uppermost clay layer, elevation 445 feet and 465 feet.

Predicted and observed logarithm of time-settlement curves for Station 2111+50, 42 feet left of centerline and 65 feet right of centerline, are shown in Figures 57 and 58, respectively. The observed rates of settlements progressively decrease with increasing time. Primary consolidation occurred in about 150 days. Secondary consolidation appears as a straight line in the settlement-logarithm of time plot. Construction of the southwestern approach embankment began August 26, 1966, and was completed October 15, 1966. Approximately a month later, excavation for the pile-end-bent abutments was completed and a month later the abutments were constructed. Initial pavement elevations were obtained in September 1968, shortly after placement of the approach pavements. Hence, there had been a time lapsed of approximately 2 years between the time the embankment was started and the time the approach pavements were constructed. This was sufficient for the completion of initial and primary consolidation of the foundation, but it was not sufficient for the completion of secondary consolidation, as shown in Figures 57 and 58. Primary consolidation was completed at or near the time of completion of the approach embankment. Approximately 2.7 inches of secondary consolidation of the foundation occurred between October 1968 (placement of the approaches) and December
Figure 57. Predicted and Observed Logarithm of Time-Settlement Curves, Station 2111+50, 42 feet Left of Centerline, I 71 over Kentucky River, Carroll County.

Figure 58. Predicted and Observed Logarithm of Time-Settlement Curves, Station 2111+50, 65 feet Right of Centerline, I 71 over Kentucky River, Carroll County.
1984 at Station 2111+50, 42 feet right of centerline. At Station 2111+50, 65 feet right of centerline (ravine), about 3.7 inches of secondary consolidation of the foundation occurred between placement of the bridge approach pavement and December 1984. Hence, the magnitude of the foundation secondary consolidation is significant with regard to settlement of the approach pavements.

To study the behavior of the southwestern approach pavements, elevations were obtained at the time of construction of the approaches and at subsequent times. Differential settlements at various times as a function of horizontal distance behind the abutments are shown in Figures 59, 60, 61, and 62. Pavement elevations were obtained at various times from October 1968 to June 1972. Measurements were discontinued after that date when the approaches required patching. Differential settlements of the outside and inside exit pavement edges are shown in Figures 59 and 60, respectively. Settlement of the left pavement edge, between about 500 to 900 feet, is in the vicinity of the drainage basin. Maximum settlement of the exit approach pavement, which occurred during the period August 24, 1968, to June 27, 1972, within 50 feet of the abutment, was about 1.6 inches (outside edge). Maximum differential settlements of the exit approach, obtained from profile measurements at various times, and which occurred at a distance of 35 feet from the end of the bridge, are plotted in the upper right-hand corner of Figure 57 as a function of logarithm of time. The relationship appears to be linear; settlements, therefore, were projected to 10,000 days (January 1994). Secondary consolidation of the foundation during that period was about 1.3 inches. Hence, a portion of the settlement of the exit approach was due to secondary consolidation of the foundation.
Figure 59. Settlement of Outside Edge of the Exit Approach Pavement, Southwestern End of Bridge, I 71 over Kentucky River, Carroll County.

Figure 60. Settlement of Inside Edge of the Exit Approach Pavement, Southwestern End of Bridge, I 71 over Kentucky River, Carroll County.
Figure 61. Settlement of Outside Edge of the Entry Approach Pavement, Southwestern End of Bridge, I 71 over Kentucky River, Carroll County.

Figure 62. Settlement of Inside Edge of the Entry Approach Pavement, Southwestern End of Bridge, I 71 over Kentucky River, Carroll County.
Part of the approach settlement was in the embankment, a result of some secondary compression and shear strain at the end of the approach fill. Projected secondary consolidation during the period September 9, 1968, to December 1984 is 1.4 inches. By January 1994 (10,000 days after the start of construction), the exit approach pavement, based on projections of the data, Figure 58, will have settled about 4 inches. About 3.1 inches of this settlement is due to secondary compression of the foundation while 0.9 inches is due to secondary compression and shear strain of the embankment.

Differential settlements of the outside and inside edges of the entry approach pavements are shown in Figures 61 and 62, respectively. As shown in Figure 56, the fill height of the embankment under the entry approach is about 55 feet, or some 20 feet thicker than the fill under the exit approach. The maximum settlement of about 2.8 inches (secondary compression), which occurred during the period September 24, 1968, to June 27, 1972, occurred at a distance of 150 feet from the end of the bridge. Apparently, the embankment settled about 0.7 inches during that same period. Maximum differential settlements of the entry approach obtained from profile measurements, and which occurred at a distance of 150 feet from the end of the bridge, are plotted in the upper right-hand corner of Figure 58 as a function of the logarithm of time. Projected settlement of the entry approach from June 27, 1972, to December 1, 1984, is about 2.3 inches. By January 1994 (10,000 days after start of construction), the entry approach pavement is predicted to settle a total of 7 inches. About 4.3 inches of that settlement will be due to secondary compression of the foundation and 2.7 inches will be due to secondary compression and shear strain of the embankment. Hence,
based on these data, secondary consolidation of the foundation is significant with regard to the approach settlement.

In June 1970, approximately 2 years after the start of embankment construction, two slope inclinometers were installed at locations shown in Figures 55 and 56. Horizontal movements for Well Number 1 were monitored from July 9, 1970, to June 5, 1978 — approximately 8 years. Resultant horizontal movements for Well Number 1, located at Station 2111+20 next to the northbound exit approach, is shown in Figure 63. Resultant horizontal movements obtained during a 14-year period (1970 to 1984) from Well Number 2, which is located at Station 2111+20 next to the southbound entry approach, is shown in Figure 64. Resultant movements of selected depths are plotted as a function of time in Figures 65 and 66. Lateral movements (Figures 63 through 66) of the southwestern approach embankment observed during the 14-year period are less than about 1.0 inch. Hence, the small lateral movements of the embankment appear to be insignificant at this site.

During installation of the slope inclinometers, "undisturbed" soil samples were obtained from the embankment and foundation. Consolidated-undrained triaxial compression tests with pore-pressure measurements were performed on the soils (brown sandy clay) obtained from an elevation interval of 432.5 to 418.5 feet (Figure 56). Effective stress parameters (Figure 67) $\phi'$ and $c'$ of the embankment soils were 24.8 degrees and 250 pounds per square foot, respectively. The parameters of the brown sandy clay (elevation 446 to about 406 feet) (Figure 68) were 29.6 degrees and 103 pounds per square foot, respectively. Stability analyses were performed using a cross section parallel to roadway centerline (Figure 69) and a cross section perpendicular to centerline.
Figure 63. Resultant Horizontal Movement, as a Function of Depth, Well Number 1, Station 2111+20, Southbound Exit Approach, I 71, Kentucky River.
Figure 64. Resultant Horizontal Movement, Well Number 2, Station 2111+20, Northbound Entry Approach.
CARROLL CO. #1

- 7.5
- 16.0
- 26.0
- 39.0
- 47.5

**Figure 65. Resultant Horizontal Movement, at Selected Depths as a Function of Time, I 71, Southbound Exit Approach.**
Figure 66. Resultant Horizontal Movement, at Selected Depths as a Function of Time, I 71, Northbound Entry Approach.
Figure 67. Effective Stress Parameters of Embankment Soils, I 71, Southwestern Approach.

Figure 68. Effective Stress Parameters of Foundation Soils (Brown Sandy Clay), I 71, Southwestern Approach.
Figure 69. Stability Analysis of Approach Embankment Section along Roadway Centerline, I 71, Kentucky River.

Figure 70. Stability Analysis of Approach Embankment Section Perpendicular to Roadway Centerline, I 71, Kentucky River.
(Figure 70) to determine the long-term factors of safety. Using a slope
stability computer program (18), the observed water table obtained from
the two slope inclinometers over an 8-year period, and the search grid
of the computer program, the most critical circles of the two sections
having minimum factors of safety were determined. The factor of safety
of the section in Figure 69 was 1.52. For the section in Figure 70, the
minimum factor of safety was 1.37. The critical circles for the two
sections are shown in Figures 69 and 70.

Conditions existing at the site in February 1985, some 16 years
after the pavements were constructed, are shown in Figures 71 through
75. Views of the approach pavements are shown in Figures 71 and 72.
Views of the side of the abutments are shown in Figures 73 through 75.
During the summer of 1984, the roadway along this section of I 71 was
paved (overlaid).

I 64, SLATE CREEK -- BATH COUNTY

I 64 crosses Slate Creek over twin bridges near Station 621+00
(Figure 76). This site is located in the Knobs Region of Kentucky,
approximately 2 miles south of Owingeville. In the vicinity of the
crossing, the basin of Slate Creek is roughly 800 feet wide. The flood
plain on the eastern side of the river near the crossing is relatively
flat and wide, and the original ground elevation is near 705 feet. A
bluff is situated on the western side of the river with the top of this
bluff approximately at elevation 745 feet. The western approach
embankment is about 40 feet in height, while the eastern embankment is
55 feet in height.

Both embankments are composed mainly of a greenish nondurable shale
with some limestone rocks. The approach embankments were constructed
Figure 71. View of Entry Approach, Southwestern End of Bridge, I 71.

Figure 72. View of Exit Approach, Southwestern End of Bridge, I 71.
Figure 73. View of Face of Abutment, Entry Approach, I 71.

Figure 74. View of Face of Abutment, Exit Approach, I 71.
Figure 75. View of Side of Abutment, Entry Approach, I 71.
using lift thicknesses of 2 to 3 feet. Shale in the embankment was obtained from the Crab Orchard Formation (Silurian period). Borings in the eastern approach foundation apparently penetrated the lower portion of the Maysville limestone at about an elevation of 692 feet. The eastern foundation (approximately 12 feet thick, Figure 76) is composed of water-deposited transported soils (recent alluvium of the Pleistocene period) — slightly sandy silty clays with a few small stones present. Natural moisture contents of the eastern approach foundation soils are low and range from 20 to 25 percent. A summary of field and laboratory data is given elsewhere (1). These soils classify as A-4 by the AASHTO Classification System and ML by the Unified System. A single-point mercury-filled settlement gage (1, 5) was installed at Station 622+97, 42 feet right of centerline and 42 feet from the east end of the bridge. The bridge abutments rest on end-bent piles driven through constructed earth cores.

Consolidation data indicated that settlements on the order of 8 inches could be anticipated for the eastern approach foundation and the rate of consolidation would be fairly rapid. Void ratio-log pressure curves and coefficient of consolidation-log pressure curves are presented elsewhere (1). Observed and predicted time-settlement curves for the eastern approach foundation are shown in Figure 77. Construction of the embankment began November 23, 1965. However, only a small amount of fill was placed in the first 125 days; most of the fill was placed during the next 80 days. The predicted rate of settlement was adjusted to the next 80 days and a linear loading rate was used to approximate the actual loading rate. The early portion of the predicted curve was drawn to reflect the probable settlement for the first 125
Figure 77. Observed and Predicted Time-Settlement Curves, 42 feet Right of Centerline, Station 622+97, I 64 over Slate Creek, Bath County.
days. Unfortunately, only a few observed settlement readings were obtained at this site because the mercury gage has not always been operative. However, the observed readings shown in Figure 77 indicate that settlement of the eastern approach foundation had ceased before placement of the approach pavements on July 14, 1967, 554 days after the start of construction of the embankment.

Settlements of the eastern approach pavements from August 1967 to May 1970 — about 2.8 years — are shown in Figures 78 through 83. Settlements of the inside pavement, centerline, and outside pavement edge of the eastbound east end approach are shown in Figures 78 through 80, respectively. Maximum settlement ranged from 3 to almost 4.7 inches and occurred at a distance of about 30 feet from the end of the bridge. The width of the settlement cradle, as shown in Figure 74, ranged from about 75 to 200 feet. Maximum settlements of the inside pavement edge, centerline, and outside pavement edge of the westbound east end approach pavements are shown in Figures 81 through 83, respectively. Maximum settlement was about 2 inches. During the period May 1969 to May 1970, both east and west approach pavements were mudjacked; the westbound east end approach was heavily mudjacked. That approach apparently was raised by the mudjacking some 6 to 7 inches between a distance behind the abutment ranging from 100 to 200 feet. Settlements of the eastern approach pavement (eastbound lanes, east end) as a function of the logarithm of time are shown in Figure 84. Settlements in this figure are maximum values observed from profile measurements of the inside and outside pavement edges of the eastbound lanes, east of the bridge. Maximum values occurred at distances behind the bridge of 35 and 50 feet, respectively. The relationship between differential settlement and
Figure 78. Settlement of Eastbound, East End, Inside Approach Pavement Edge, I 64, Slate Creek.

Figure 79. Settlement of Eastbound, East End, Centerline of Approach Pavement, I 64, Slate Creek.
Figure 80. Settlement of Eastbound, East End, Outside Edge of Approach Pavement, I 64, Slate Creek.
Figure 81. Settlement of Westbound, East End, Inside Approach Pavement Edge, I 64, Slate Creek.
Figure 82. Settlement of Westbound, East End, Centerline of Approach Pavement, I-64, Slate Creek.
Figure 83. Settlement of Westbound, East End, Outside Approach Pavement Edge, I 64, Slate Creek.
Figure 84. Settlement of Eastern Approach Embankment as a Function of Logarithm of Time, I 64, Slate Creek.

Figure 85. Resultant Horizontal Movement as a Function of Depth, I 64, Slate Creek.
the logarithm of time is linear. Based on a projection of this linear relationship, the approach pavements (and the eastern embankment) will have settled from 8 to 14 inches by April 10, 1994 (10,000 days or 27.4 years after start of construction). Settlements after that date will be almost insignificant.

On August 18, 1970, a slope inclinometer was installed in the eastern approach embankment at about Station 1622+80, between the eastern bridge abutments (top of the slope). The slope inclinometer was monitored from August 18, 1970, to July 2, 1979 -- a period of almost 9 years. Results of these measurements are shown in Figure 85. Maximum horizontal movement of the eastern embankment over the 9-year period was less than 1.2 inches, and such a small movement has apparently not influenced approach pavement settlements. Resultant horizontal movements as a function of time at selected depths are shown in Figure 86.

Estimated factor of safety of the eastern approach embankment (laboratory compacted samples of the Crab Orchard) based on assumed shear strength values of $\phi^*$ and $c^*$ of 28 degrees and 0.0, respectively, was 1.12. A factor of safety of 1.10 was obtained from the ICES LEASE-I program (19). The stability analysis is shown in Figure 87.

The condition of the eastern approach pavements and abutments in February 1985 -- some 17 years after placement -- are shown in Figures 88 through 92. Both approaches were mudjacked during the period May 1969 to May 1970, about 2 years after construction of the approach pavements. Both approach pavements have been mudjacked and patched extensively. In 1984, the approaches and roadway pavement were paved. Based on the projection in Figure 84, the eastern approach pavements will settle about 0.8 to 2 inches during the period December 12, 1984,
Figure 86. Resultant Horizontal Movement as a Function of Time, at Selected Depths, I 64, Slate Creek.

Figure 87. Stability Analysis of Eastern Approach Embankment Section along Roadway Centerline, I 64, Slate Creek.
Figure 88. View of Eastbound, East End Bridge Approach, I 64, Slate Creek.

Figure 89. View of Westbound, East End Bridge Approach, I 64, Slate Creek.
Figure 90. Side View of Westbound, East End Bridge Abutment.

Figure 91. View of Erosion of Face of Slope of Eastern Approach Abutment, I 64, Slate Creek.
Figure 92. View of Erosion Next to Face of West Bound, East End Bridge Abutment, I 64, Slate Creek.
to April 10, 1994 (8.3 years). After 1994, the settlements should be insignificant.

KY 30, NORTH FORK OF KENTUCKY RIVER

KY 30 crosses the North Fork of the Kentucky River near Jackson, Kentucky, between Stations 120+00 and 131+00. The site is located in the Eastern Coal Region. Bedrock in the vicinity of the site consists of layers of shales and sandstones. Overburden soils, except in floodplains, are relatively thin, ranging from 2 to about 6 feet in thickness. The southern bridge abutment is situated on bedrock. The northern bridge abutment (on a spread footer) is located on a 52-foot high approach embankment.

As shown in Figure 93, foundation soils (alluvial deposits) consist of a layer of silty clay resting on a layer of silty sand. The silty clay layer is about 8 to 10 feet in thickness and classifies as ML-CL. The natural water content of that soil is about 23 percent. Liquid and plastic limits are 25 and 20 percent, respectively. That soil layer consists of about 31 percent (−No. 40 to +No. 200 sieve) of fine sand and 45 percent (−No. 200 to +0.005 mm) of silt. About 22 percent was finer than 0.005 mm. The silty clay layer rests on a sand layer approximately 20 feet thick. The sand is non-plastic. Natural water content is about 12 percent. The sand classifies as SM. This soil layer consists of 67 percent (−No. 40 to + No. 200 sieve) of fine sand and 21 percent (−No. 200 to +0.005 mm) of silt. About 12 percent of the soil is finer than the 0.005-mm size.

Consolidation tests were performed on the silty clay layer. Those tests showed that foundation settlement would occur almost instantly as the fill was constructed. Predicted settlement of the foundation under
Figure 93. Cross Section along Roadway Centerline, KY 30, Boonesville-Jackson Highway.

Figure 94. Settlement of Left Side of Abutment as a Function of Logarithm of Time, KY 30, Boonesville-Jackson Highway.
A 52-foot embankment was 1.9 inches. Time rate calculations predicted that 90 percent of the settlement would occur in 15 days. A long-term factor of safety of 2.05 was reported (20).

A mercury-filled settlement gage (five points) was installed on the northern approach foundation. However, the gage was destroyed by the contractor during initial stages of construction. Consequently, foundation settlements could not be monitored. However, eleven points were established on the spread abutment shortly after construction. Elevations of those points were obtained over a 4-year period. Settlements of the left side of the abutment of two points (Points 1 and 2) on the abutment are shown in Figure 94. Settlements of the right side of the abutment (Points 10 and 11) are shown in Figure 95. In both figures, the relationship between settlement and the logarithm of time is linear. Settlements for the seven points are essentially the same. The abutment settled about 3.5 to 4.5 inches after about 8.8 years (1985). Assuming a linear relationship between settlement and logarithm of time will continue, the maximum projected settlement of the abutment ranges from 4.4 inches (Figure 94) to 5.7 inches (Figure 95) at 27.4 years. After 27.4 years, the settlements are essentially insignificant. Since the foundation is essentially fine-grained sand, most of the settlement of the foundation occurred almost immediately during construction of the embankment. Consequently, most of the settlement of the abutment is due primarily to some secondary compression and shear strain of the 52-foot high embankment. Views of the northern approach pavements and the spread abutment are shown in Figures 96 and 97, respectively.
Figure 95. Settlement of Right Side of Abutment as a Function of Logarithm of Time, KY 30, Boonesville-Jackson Highway.
Figure 96. View of Northern Approach Pavement, KY 30, Boonesville-Jackson Highway.

Figure 97. View of Spread Abutment, KY 30, Boonesville-Jackson Highway.
ANALYSIS AND DISCUSSION

SUMMARY OF OBSERVATIONS

A summary of observations at the six selected bridge sites are presented in Tables 6a and 6b. Although the characteristics of the six sites differ, initial and primary consolidation in all cases was completed before placement of the bridge approach pavements. The observed rate of primary consolidation usually was much faster than the predicted rate. As shown in Table 6b, the ratio of predicted t₁₀₀ (the time for 100 percent completion of primary consolidation) to the observed t₁₀₀ ranged from 1.0 to 9.1 and generally averaged about 4.0. The ratio of predicted magnitude of primary settlement to observed settlement at t₁₀₀ averaged about 1.2, if data from the Parkers-Mill site are excluded. In those cases, foundation deposits were mainly alluvium. At the Parkers-Mill site, the foundation soils were residual and highly overconsolidated. The ratio of predicted to observed settlement at that site was only 4.8 (observed settlement only about 20 percent of the predicted settlement).

As shown in Table 6a, lengths of settlement cradles ranged from 70 feet to 300 feet. The maximum approach settlement occurred at a distance ranging from 15 to 150 feet from the end of the bridge, averaging about 50 to 60 feet. The ratio of the length of the settlement cradle to the sum of the embankment height and depth of foundation ranged from about 1 to 4.
<table>
<thead>
<tr>
<th>SITE</th>
<th>TYPE</th>
<th>HEIGHT (feet)</th>
<th>GENERAL COMPOSITION</th>
<th>APPROACH END</th>
<th>FOUNDATION</th>
<th>GENERAL COMPOSITION</th>
<th>WIDTH OF BRIDGE CRADLE (feet)</th>
<th>MAXIMUM SETTLEMENT FROM END OF BRIDGE (inches)</th>
<th>DISTANCE OF BRIDGE SETTLEMENT (feet)</th>
<th>SETTLEMENT RATIO (172.8 YEARS)</th>
<th>PROJECTED MAXIMUM SETTLEMENT (inches) AFTER START OF CONSTRUCTION</th>
<th>OTHERS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Route 4, Eddy Creek</td>
<td>Built-up, over lake</td>
<td>50</td>
<td>Special granular till and rock (lower part) unclassified till (upper part)</td>
<td>Northern</td>
<td>Silty clays</td>
<td>Mainly clays and slits, some sand and gravel</td>
<td>22</td>
<td>1,0</td>
<td>15-25</td>
<td>1.1-2.5</td>
<td>1.0</td>
<td>0</td>
</tr>
<tr>
<td>I 65, Fork Creek</td>
<td>Built-up, over small stream and road — groundline at end of bridge slopes toward stream</td>
<td>27</td>
<td>Non-durables shales and durables rock (sandstones)</td>
<td>Eastern</td>
<td>Conglomerate of gravel, sand, silt and clay</td>
<td>23</td>
<td>4.2-4.9</td>
<td>29</td>
<td>3.0</td>
<td>12</td>
<td>0.0097</td>
<td></td>
</tr>
<tr>
<td>I 71, Ky River</td>
<td>Built-up, over Ky River (major river)</td>
<td>55</td>
<td>Silty clays</td>
<td>Southern</td>
<td>Layers of clay, silty clay, clayey sands, sand</td>
<td>100</td>
<td>1.9-2.0</td>
<td>30</td>
<td>4.0</td>
<td>0.0</td>
<td>0.0017</td>
<td></td>
</tr>
<tr>
<td>I 64, State Creek</td>
<td>Built-up, over slate creek (major stream) — floodplain</td>
<td>55</td>
<td>Non-durables shales</td>
<td>Eastern</td>
<td>12</td>
<td>19-200</td>
<td>1.9-4.7</td>
<td>50</td>
<td>1.1</td>
<td>0-14</td>
<td>0.0009 to 0.0173</td>
<td>3.0</td>
</tr>
<tr>
<td>Ry 30</td>
<td>Built-up, over North Fork of Ky River (major stream)</td>
<td>52</td>
<td>Sand placed in 2-foot lifts</td>
<td>Northern</td>
<td>Clayey silts and fine sands</td>
<td>30</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>1.7</td>
<td>0.0022</td>
</tr>
</tbody>
</table>

*Wc* = width of settlement cradle  
*h* = height of fill  
*df* = depth of foundation  
*cs* = coefficient of secondary compression and shear strain — applies only to embankment
<table>
<thead>
<tr>
<th>SITE</th>
<th>RATIO OF PREDICTED PRIMARY CONсолIDATION TO OBSERVED PRIMARY CONсолIDATION</th>
<th>RATIO OF PREDICTED SECONDARY CONсолIDATION TO OBSERVED SECONDARY CONсолIDATION</th>
<th>RATIO OF TOTAL CONсолIDATION TO OBSERVED CONсолIDATION</th>
<th>BRIDGE APPROACH SETTLEMENT FACTORS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(FOUNDATION)</td>
<td>($)</td>
<td>($1)</td>
<td></td>
</tr>
<tr>
<td>Route 4</td>
<td>1.0</td>
<td>19.7</td>
<td>—</td>
<td>19.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1000.0</td>
<td>2.7</td>
<td>83.7</td>
</tr>
<tr>
<td></td>
<td>(Southern)</td>
<td>97.0</td>
<td>2.7</td>
<td>97.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>96.0</td>
<td>2.7</td>
<td>94.3</td>
</tr>
<tr>
<td></td>
<td>(Northern)</td>
<td>—</td>
<td>—</td>
<td>9.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>181.0</td>
<td>—</td>
<td>181.0</td>
</tr>
<tr>
<td></td>
<td>(Eastern)</td>
<td>111.5</td>
<td>—</td>
<td>111.5</td>
</tr>
<tr>
<td></td>
<td>(Western)</td>
<td>146.3</td>
<td>—</td>
<td>146.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>125.0</td>
<td>—</td>
<td>125.0</td>
</tr>
<tr>
<td></td>
<td>(River)</td>
<td>151.0</td>
<td>—</td>
<td>151.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200.0</td>
<td>—</td>
<td>200.0</td>
</tr>
</tbody>
</table>
PROPOSED EMPIRICAL METHOD OF ESTIMATING RATE OF PRIMARY SETTLEMENT OF FOUNDATIONS

Relationships

Generally, the predicted rate of settlement was slower than the actual rate. Perhaps one means of constructing a more realistic settlement curve, rather than using $C_v$-values from laboratory consolidation tests, is to use the relationship between the time of the end of embankment construction, $T_C$, and observed values of $t_{100}$ from several sites. In Figure 98, observed or actual values of $t_{100}$ are shown as a function of the times required to reach final grade elevation of several approach embankments. The observed values of $t_{100}$ were obtained from the six study sites described above and unpublished data obtained from records of the Kentucky Department of Highways. These values were obtained from mercury-filled settlement gages and/or settlement platforms. Based on a linear regression analysis, the time (in days) for completion of primary compression, $t_{100}$, may be estimated from

$$t_{100} = 2.5 + 1.25 T_C. \quad (2)$$

If the time $T_C$ is known, then the time for completion of primary compression may be estimated. If the time of completion of construction is not available, then the value of $T_C$ may be estimated from Figure 99. In that figure, the logarithm of time (in days) required to reach grade elevation, $H_e$ (feet), is shown as a function of the logarithm of the height of embankment. Data were obtained from the six study sites and unpublished data from files of the Kentucky Department of Highways. Although considerable data scatter occurs in the $\log T_C - \log H_e$ relationship, $T_C$ (days) may be approximated by

110
Figure 98. Values of Observed \( t_{100} \) as a Function of Time, \( T_c \), Required to Reach Embankment Grade Elevation.

Figure 99. Time, \( T_c \), Required to Reach Embankment Grade Elevation as a Function of Embankment Height, \( H_e \)
Substituting the expression for $T_c$ into Equation 2, $t_{100}$ may be approximated by

$$t_{100} = 2.5 + (1.25) 10^{(1.2376 \log_{10} H_e + 0.1122)}.$$  

(4)

To estimate a field value of the coefficient of consolidation, $C_{v \text{field}}$, the expression for $t_{100}$, Equation 4, is inserted into the following equation:

$$C_{v \text{field}} = \frac{TH^2}{t_{100}}$$

$$= \frac{1.6H^2/(2.5 + (1.25)10^{(1.2376 \log_{10} H_e + 0.1122)})}{t_{100}}.$$  

(5)

where $T$ = dimensionless time factor (a value of 1.6 was used in the above equation and corresponds to about 100 percent consolidation),

$H$ = thickness of compressible stratum, and

$H_e$ = height of approach embankment.

To calculate the primary compression, consolidation tests are performed and an estimate of total settlement, $S$, is made. The average consolidation ratio, $U$, is given by

$$U = \text{compression at time } T/\text{compression at end of primary consolidation}.$$  

(6)

An expression for $U$ as a function of $T$ is given elsewhere (21).

To obtain settlement as a function of time, values of percentages of total settlement are assumed. A value of $T$ corresponding to an assumed value of $U$ is obtained from a theoretical relationship between $U$ and $T$. 

112
(21). To obtain the time, \( t_1 \), for a given percentage of total settlement, \( t_1 \)-values are computed from

\[
t_1 = \frac{TH^2}{C_v}
\]

(7)

Settlement, \( S_1 \), corresponding to a value of \( t_1 \) is computed from

\[
S_1 = U S.
\]

(8)

The coefficient of consolidation, \( C_v \), is obtained from laboratory tests or may be estimated from Equation 5. Equation 5 is empirical and is not intended as a substitute for laboratory values of \( C_v \). It is only an approximate means of obtaining a value of \( C_v \). However, it is suggested that the equation be used in conjunction with laboratory values to estimate a time-settlement curve of a given approach foundation.

Example Calculation

As a check of the empirical method of constructing a time-settlement curve and to illustrate the empirical procedure, the profile of the embankment at the Eddy Creek Site, I 24 (Station 3988+00, 72 feet right of centerline), was analyzed. The embankment height was 35 feet. The estimated time to construct the embankment is obtained from

\[
T_c = 10(1.2376 \log_{10} H_e + 0.1122)
\]

\[
= 105.5 \text{ days.}
\]

To estimate the time at which primary compression will be completed, Equation 2 is used:

\[
t_{100} = 2.5 + 1.25 T_c
\]

\[
= 2.5 + 1.25 (105.5) \text{ days} = 134 \text{ days.}
\]
At this site, the observed $t_{100}$-value was 150 days.

Based on a consolidation analysis, the estimated total primary compression was 15.2 inches. Using the coordinates 106 days and 15.2 inches (the end of primary compression), the predicted $t_{100}$ point is plotted as shown in Figure 100. The coefficient of primary compression is obtained from the empirical expression

$$C_v = \frac{TH^2}{t_{100}}$$

$$= 1.6 H^2 / (2.5 + (1.25)10(1.2376 \log_{10} H_e + 0.1122))$$

$$= 14.59 \text{ ft}^2/\text{day.}$$

Using the observed $t_{100}$ obtained from settlement measurements, the observed value of $C_v$-field is

$$C_v\text{-field} = 13.07 \text{ ft}^2/\text{day.}$$

Estimated values of settlement during construction were obtained using time factors, $T$, from Reference 21 (NAFACS, page 7-6-14, middle panel). Observed and predicted curves are compared in Figure 100. The instantaneous curve was corrected for the gradual loading period, $T_c$, using time factors in Reference 21. A linear loading rate of the fill was assumed for the gradual loading case. The predicted settlement curve as constructed by the empirical method, the predicted curve based on a laboratory $C_v$-value, and the observed settlement curve are compared in Figure 100.

To completely construct the time-settlement curve, secondary compression of the foundation is estimated from Equation 1. A value of the secondary coefficient of consolidation, $C_a$, is obtained from laboratory tests. Based on data from the six study sites, the time,
Figure 100. Comparison of the Observed Settlement Curve, the Settlement Curve Predicted from an Empirical Method, and the Settlement Curve Predicted from Laboratory Consolidation Test Results.

Figure 101. Projected Settlements of Highway Bridge Approach Embankments as a Function of Long-Term Factors of Safety at Six Study Sites.
\( t_{\text{sec}} \), over which significant settlement may occur should generally be taken to be 27.4 years or 10,000 days. The secondary compression curve is linear when settlement is plotted as a function of the logarithm of time from \( t_{100} \) to \( t_{\text{sec}} \).

SECONDARY SETTLEMENTS OF FOUNDATIONS

Secondary consolidation of the foundation was a significant factor contributing to the settlement of the approach pavements at two bridge sites — Kentucky River and Eddy Creek. At the Eddy Creek site, secondary consolidation was not as much a factor affecting the approach pavement as at the Kentucky River site. At the Eddy Creek site, early construction had been specified and some 10 years elapsed between the time construction started and the time of placement of the approach pavements. However, at both sites the major portion of the settlement of the approach pavements was apparently due to secondary compression of the foundation. At the Parkers-Mill site, foundation soils were residual and highly overconsolidated. Moreover, foundation soils were relatively thin. Secondary compression was nominal; that is, it was small and could not be measured. At the Bull Fork site, secondary compression of the foundation was nominal and could not be measured. Foundation soils at that site were very sandy and gravelly. Such soils, as expected, did not exhibit appreciable secondary compression. Foundation soils at the Slate Creek site were relatively thin and did not appear to exhibit damaging amounts of secondary compression. Coefficients of secondary compression, \( C_s \), obtained from observed settlement curves at the Eddy Creek site and the Kentucky River site were approximately 0.007.
REPROPOSED EMPIRICAL METHOD OF PREDICTING SETTLEMENTS
OF BRIDGE APPROACH EMBANKMENTS

Relationships

In Figure 101, the projected settlements of highway bridge approach embankments are shown as a function of the long-term factors of safety of the highway approach embankments at the six study sites. The projected embankment settlements are those that will occur at the end of 27.4 years provided the linear relationship between approach settlement and the logarithm of time is valid. As shown in Figure 101, there is a distinctive difference between settlements of approach embankments having factors of safety less than about 1.50 and those of approach embankments having factors of safety larger than approximately 1.50. The relationship between factor of safety and approach embankment settlement in Figure 101 strongly indicates that the settlement of approach embankments can be minimized by designing the approach embankment using a minimum long-term factor of safety of 1.50. Apparently, the settlement of the approach embankment that occurs within the settlement cradle is due partly to secondary compression and partly to shear strain. As the factor of safety decreases below a value of 1.50, the shear strain component of the total approach embankment settlement is much larger than the secondary compression component. When the long-term factor of safety of the approach embankment is larger than 1.50, the shear strain component is a small percentage of the total embankment settlement; the secondary compression is a more significant portion of the total embankment settlement. Using a factor of safety equal to or larger than 1.50, the shear strain is made small.

Since the relationship of approach embankment settlement as a function of the logarithm of time appears to be linear, then the
coefficient of secondary compression and shear strain, $C_{ss}$, might be estimated from

$$C_{ss} = \frac{H_{ss}}{H_e \log_{10}(t_{ss} / t_c)}, \quad (9)$$

where $H_{ss} =$ settlement (inches) of the approach embankment due to secondary compression and shear strain,

$H_e =$ height of embankment (inches),

t$_c =$ time (days) of placement of approach pavement (the time between the start of embankment construction and the placement of the approach pavement), and

t$_{ss} =$ time (days) at the end of significant secondary compression and shear strain of the approach embankment.

Using Equation 9, values of $C_{ss}$ were computed for five of the six study sites. If the logarithm of the reciprocal of $C_{ss}$ (or $1/C_{ss}$) is plotted as a function of the logarithm of the ratio, $F_T$, of the approach embankment height, $H_e$, to the long-term factor of safety, $F$, then an approximate linear relationship is obtained (Figure 102):

$$\log \left( \frac{1}{C_{ss}} \right) = -1.5013 \log_{10}(H_e/F) + 4.6755 \quad (10)$$

and

$$\frac{1}{C_{ss}} = 10 \left( -1.5013 \log_{10} F_T + 4.6755 \right) \quad (11)$$

so that

$$C_{ss} = 10 \left( 1.5013 \log_{10} F_T - 4.6755 \right) \quad (12)$$

Using the value of $C_{ss}$ estimated from Equation 12, the settlement of the approach embankment due to secondary settlement and shear strain, $H_{ss}$, is estimated from
Figure 102. Relationship between the Reciprocal of the Coefficient of Secondary Compression and Shear Strain ($1/C_{sec}$) and the Ratio, $F_a$, of Bridge Approach Embankment Height to the Long-Term Factor of Safety.
Design Example

To illustrate the use of equations 12 through 14 as a means of estimating the secondary and shear strain settlement within the settlement cradle of an approach embankment, the following example is described. Configuration of the embankment and foundation are shown in Figure 46. The approach embankment is 75 feet in height on an alluvial floodplain. Detailed subsurface exploration of the approach embankment foundation showed that the foundation was only about 12 feet in thickness and consisted of clayey silts and sands with some gravel. Conventional consolidation tests were performed on undisturbed soil samples recovered from the foundation. Results from the consolidation tests were used to predict the magnitude and rate of foundation settlement. Based on the consolidation analysis, approximately 14 inches of initial and primary consolidation would occur in about 100 days after the start of embankment construction. The approach pavements will be placed some 800 days after start of construction. Hence, sufficient time is available for primary consolidation to occur before placement of the approach pavements. To check the analysis, settlement gages were to be installed on the foundation to track the settlement of the foundation during construction.

Additionally, two rows of concrete settlement monuments were to be installed on top of the embankment immediately after grade elevation of the embankment was reached. The monuments will be spaced on 20-foot
intervals over a distance of approximately 500 feet — 5 times (height of fill + foundation thickness) — behind the abutments. To avoid the influence of frost heave, the monuments are to extend to a minimum depth of about 3 to 4 feet below grade elevation. Elevations of the monuments will be obtained during construction. Settlements of the concrete monuments will be plotted as a function of the logarithm of time. Hence, the actual settlement of the embankment and foundation may be observed and the trend of the approach settlement can be established. Provided the relationship between settlement, as obtained from the monuments, and logarithm of time is linear, the settlement behavior of the embankment and foundation can be predicted at some future date. Hence, these data will be useful in deciding whether temporary approach pavements should be installed, with the permanent approach pavements to be installed at some future time or whether permanent approach pavements may be installed immediately. If the projected settlement-logarithm of time relationship shows that small amounts of settlement will occur at some future time, and provided early construction is specified, then abutments on spread footings could be considered. The coefficient of secondary compression for foundation soils is obtained from the consolidation tests. Using Equation 11, secondary compression and shear strain was estimated to be very small.

Since settlement of the foundation is fairly insignificant, then most of the approach settlement will occur in the approach embankment. Consolidated-undrained triaxial tests with pore-pressure measurements were performed on undisturbed samples from the foundation. The effective stress parameters, $\phi^c$ and $c^c$, of the foundation were determined to be 31 degrees and 0, respectively. Fill materials for the
approach embankment will be obtained from an adjacent cut section. Rock core samples showed that the borrow materials consisted of interbedded shales and sandstones. Slate-durability tests showed a large portion of the material was non-durable shales. To estimate effective stress parameters of the fill materials, consolidated-undrained triaxial tests with pore-pressure measurements were performed on remolded samples of a conglomerate mix of non-durable shales and sandstones. A moisture-density test (ASTM D 698) was performed on the mixture to establish remolding conditions. Triaxial specimens were remolded to 98 percent of maximum dry density and optimum water content. Triaxial tests yielded a $\phi$ of 29.8 degrees and a small cohesion, $c^\prime$, of 70 pounds per square foot. It was estimated that the ground water level would probably rise into the fill to a level about 10 feet above the original groundline. Based on effective stress strength parameters and estimates of long-term pore pressures, a stability analysis of the approach embankment yielded a factor of safety of 1.27. To check the settlement of the approach fill within the settlement cradle, Equations 12 through 14 were used.

The approach pavements were to be constructed some 800 days ($t_c$) after start of construction. The designer checks the settlement using an assumed value of 10,000 days for $t_{ss}$. From equation 12, an estimate of the coefficient of secondary compression and shear strain is computed:

$$c_{ss} = 10 \left(1.5013 \log_{10}(75/1.27) - 4.6755\right)$$

$$= 0.00963.$$  

From Equation 13, the settlement at the end of 27.4 years was estimated:

$$H_{ss} = c_{ss} H_e \log_{10}(t_{ss}/t_c)$$

$$= (0.00963)(75 \text{ ft})(12 \text{ in./ft})(\log(10,000 \text{ days}/800 \text{ days}))$$
This amount of settlement is not acceptable. Hence, options to increase the factor of safety of the fill were considered. The first feature consisted of a shear key and a small berm at the toe of the fill. The second feature consists of constructing the lower 10 feet of the fill of rock to minimize the build-up of pore pressures in the lower portion of the embankment and therefore ensure that the factor of safety will not be reduced at some future date. Using these design alternatives, another stability analysis was performed. A long-term factor of safety of 1.65 was obtained. From Equation 13,

\[ C_{ss} = 10(1.5013 \log_{10} (75/1.65) - 4.6755) \]

\[ = 0.00612, \]

and the estimated settlement is

\[ H_{ss} = C_{ss} H_a \log_{10} \left( t_{ss}/t_c \right) \]

\[ = (0.0065)(75 \text{ ft})(12\text{ in/ft})(\log (10,000 \text{ days}/800 \text{ days})) \]

\[ = 6.4 \text{ inches.} \]

Although 6.4 inches of settlement is large, it was acceptable. Other options to increase the factor of safety involved lengthening the bridge to decrease the slope of the approach embankment and increase the long-term factor of safety. To reduce the settlement to a tolerable range (2 to 3 inches), the factor of safety would have to be increased to about 3.0. That was uneconomical. To determine the time at which a permanent pavement should be constructed, Equation 13 was solved for \( t_{ss} \) assuming that when the settlement reached 2.5 inches, the approach would require
some form of maintenance — either patching or mudjacking. From Equation 13,

\[ \log \left( \frac{t_{ss}}{t_c} \right) = \frac{H_{ss}}{(C_{ss})(H_e)} \]  

(15)

or

\[ t_{ss} = t_c 10^{\left( \frac{H_{ss}}{(C_{ss})(H_e)} \right)} \]  

(16)

Then

\[ t_{ss} = (800 \text{ days}) 10^{\left( \frac{2.5}{(0.0065)(75 \text{ ft})(12 \text{ in./ft})} \right)} \]

= 2,140 days or 5.9 years after the start of construction.

Hence, approach pavements would require maintenance about 3.7 years (5.5 yra. - 2.2 yra.) after placement. Additionally, approach pavements would require maintenance again after 15.7 years. Hence, after 13.5 years in service, about 5 inches of settlement would occur. Only 1.4 inches of settlement would occur in the time span from 13.5 years to 27.2 years. Hence, permanent pavements could be installed after 13.5 years.

**Compaction**

The lack of proper compaction of approach embankments appeared to be a major factor leading to settlement of the approach pavements at the Bull Fork Creek site and the Slate Creek site. Approach embankments at those two sites were constructed of a mixture of broken non-durable shales and hard durable rocks (sandstone and limestone). Under compaction specifications (22) existing at the time (1965) these approach embankments were constructed, lift thicknesses of 2 to 3 feet were permitted. At that time, no distinction was made between durable
and nondurable shales. Recent studies (23-26) have shown that compaction of mixtures of non-durable shales and durable hard rocks is extremely difficult using conventional compaction equipment. Difficulties arise because durable rocks are not broken down during compaction and, with the passage of time, non-durable shales tend to weather and degrade, leaving voids in the fill. Such condition may lead to volume changes in the fill and hence leads to settlement of bridge approaches. Moreover, relatively large lift thicknesses tend to aggravate this condition.

Secondary compression of the foundation at the Bull Fork and Slate Creek sites was only nominal. Consequently, settlement of the approach pavements was mainly due to a volume change in the embankment. Approach pavement settlements, as shown in Table 6, at both sites were much more pronounced than at the Parkers Mill, Eddy Creek, and Kentucky River sites. At the latter three sites, approach embankments were compacted in about 1-foot lifts. Those embankments were constructed of granular materials and silty clays. Generally, compaction of those embankments posed no major problems.

LATERAL MOVEMENTS

Relatively large lateral movements, as measured by a slope inclinometer, occurred at the Bull Fork Creek site. Those approach embankments were situated on natural ground sloping toward the bridge ends. Lateral movements of the eastern approach embankment were 4 to 6 inches. That approach embankment settled at least 5 inches. Lateral movements recorded by slope inclinometers at the Eddy Creek, Kentucky River, and Slate Creek sites were only nominal (<0.5 - 1.0 inch). Hence, lateral movements at those sites were not a factor leading to
settlement of the bridge approach pavements. At the Bull Fork Creek site, lateral movements were a major factor, as was poor compaction of the approach embankments. Based on case histories at other sites (13-16), large lateral movement was a prominent factor leading to the settlement of the approach pavements. Lateral movements were most prominent where the approach embankments were located on foundations that sloped toward the ends of bridges.

CONCLUSIONS AND DISCUSSION

Based on information compiled from observations at six selected bridge sites, the following conclusions are presented (these findings are also supported from data accumulated from other observations published elsewhere (1-5 and 14-17):

1. Settlement of bridge approach foundations contributes significantly to settlements of approach pavements. The contribution is dependent on the time at which the approach pavements are placed during the construction process. As shown by measurements at the six sites, sufficient time existed for completion of initial and primary consolidation of the foundation. However, it should not be concluded primary compression of the foundation is not a factor causing approach pavement settlement. If bridge approach pavements were constructed during the primary phase, then large approach settlements would occur. In all cases, except at the Bull Fork site (western approach), observed rates of primary consolidation of the foundation proceeded much faster than predicted rates of primary consolidation. The observed rates were some 1 to 9 (average of 4) times faster than predicted rates. In four of the six cases, primary consolidation was essentially completed very shortly after final grade elevation of the approach embankment was
reached. At two sites, secondary compression of the foundation was a significant factor causing settlement of approach pavements. When observed secondary compression was plotted as a function of logarithm of time, a linear relationship was obtained.

2. Improper compaction of approach embankments may lead to settlement of approach pavements. Improper compaction may be the result of improper identification of the materials used to construct the approach embankments. At two sites (Slate Creek and Bull Fork Creek) where approaches were constructed of a mixture of durable rocks and non-durable shales and in 2- to 3-foot lifts, large approach pavement settlements (>4 inches) occurred. Other case histories (14-17) similar to the Bull Fork and Slate Creek sites support this finding. Approach pavements at the Slate Creek and Bull Fork Creek sites were mudjacked and patched on several occasions. Secondary compression of the foundation was nominal at either site. At the Slate Creek site, only nominal lateral movements were observed. Hence, pavement settlements were a result of volume change of the approach embankment. At the Bull Fork site, both large lateral movements and volume changes of the approach embankment occurred.

3. Bridge approach embankments and foundations that exhibit small post-construction settlement and that cause nominal approach pavement settlements may be constructed. Small pavement settlements (initial, primary, and secondary compression of the foundation) at the Parkers Hill site were completed before placement of approach pavements. Stability of the approach embankment was very high (factor of safety of 2.53). No lateral movements of the approach embankment were evident -- the abutments are still vertical. The approach embankment was compacted
in approximately 1-foot lifts. No serious erosion around the abutment was observed. Although the approach pavements have been patched, the patching is very thin and pavement settlements are less than about 1 inch.

Using appropriate construction materials, such as aeolact granular material, and specifying early construction of approach embankments, post-construction settlement of approach pavements may be reduced significantly, as illustrated by settlement data from the Eddy Creek site. At that site, some 8 years elapsed between the completion of the approach embankments and placement of the approach pavements.

4. Lateral movements of approach embankments due to shear strain may lead to settlement of approach pavements. At the Bull Fork Creek site, large lateral movements (which cause shear strain settlement), occurring over a period of several years in combination with deformations of embankment soils due to poor compaction, lead to large approach settlements. That observation was supported by other case histories (14–17).

5. Although no direct evidence was obtained, erosion of materials from around the abutment and internal erosion can aggravate the settlement of approach pavements. This observation is supported by studies conducted in 1964 and 1968 (1, 2). As an example, the face of one approach embankment was badly eroded (gullies up to 3 feet deep were observed). Also, materials were washed away from under and around the abutments, leading to loss of approach pavement support.

6. Generally, if the embankment had a large factor of safety (FS > 1.5), settlement of the approach pavement was smaller than at those sites where the factor of safety was relatively low.
7. Secondary compression and shear strain settlement of approach embankments and foundations is a major factor leading to the settlement of bridge approach embankments. Effects of secondary compression and shear strain settlement appear years after construction and are linear with logarithm of time.

8. Predicted primary rates of foundation settlements, based on laboratory consolidation test parameters, seldom agree with observed rates of primary settlements. The proposed empirical method of predicting the rate of primary settlement may lead to better agreement between predicted and observed settlement rates.

9. Total estimated settlements generally were in reasonable agreement with observed total settlements.

10. Reinforced concrete bridge approach pavements have been proposed, and are currently being used by several states, as one solution to the "bump" at the end of the bridge. However, this technique does not eliminate differential settlement between the bridge deck and approach pavement. The technique does provide a smoother transition, perhaps, when used in place of asphaltic concretes. As shown by profile measurements at the study sites reported herein, the length of the settlement zone ranges from 50 to 300 feet behind the end of the bridge. Consequently, it would be uneconomical to attempt to bridge the settlement zone or cradle at many sites. The reinforced bridge approach slab would be too lengthy and would, essentially, have to be designed in the same manner as bridge spans. Reinforced bridge approach pavements may be applicable for relatively small embankments and in cases where asphaltic concrete pavements are used. In the latter case, and to be effective, a reinforced concrete approach would replace...
the asphaltic concrete approach pavement. This does not eliminate the "bump", however, but lessens and relocates the bump to a distance further back from the end of the bridge. For relatively large embankments, benefits of using reinforced bridge approach pavements are questionable, unless means are adopted to minimize the large settlements that are generally associated with large embankments. Use of reinforced bridge approaches does not address the factors that cause approach settlement and is not a substitute for making careful analyses and design and construction procedures to minimize settlements of the embankment and foundation. Hence, this technique should be used only after careful settlement and stability analyses have been performed, and it should be used selectively. The technique is not a total solution to the problem of the bump at the end of the bridge. At locations where reinforced concrete approach pavements are used, profile measurements are needed to determine the effectiveness of this technique.

RECOMMENDATIONS AND IMPLEMENTATION

To prevent, or minimize, the effects of settlements of highway bridge approach pavements, detailed attention must be given to the factors -- primary and secondary compression, shear strains, compaction, lateral movements, and erosion -- that may contribute to the phenomenon. Consequently, a detailed subsurface and geotechnical investigation (with a comprehensive report) must be conducted. With regard to factors that may cause settlement of bridge approach pavements, the following recommendations are presented and discussed below.

Where approach embankments are located on compressible foundations composed of silts, silty clays, or clays, design and construction techniques must be employed to eliminate or minimize post-construction settlements. These methods are as follows:
1. Preconsolidate the foundation with a surcharged fill. This procedure would be useful in cases where sufficient time is available for consolidation under the surcharge load. Since the approach surcharge embankment would be higher than grade elevation, the surcharge would have to be removed before placement of approach pavements. Detailed stability analyses would be required to insure the safety of the surcharged embankment. Moreover, a detailed settlement analysis of the foundation would be required. In particular, great emphasis should be placed on eliminating secondary consolidation of the foundation. Data presented herein show that foundation soils having a coefficient of secondary compression as low as 0.007 can produce foundation settlements over long periods that significantly affect settlements of approach pavements.

In surcharging, the approach embankment is built from about 3 to 10 feet above grade elevation. The bridge approach embankment and surcharge loading should be constructed as early as possible during construction so as much time as practicable be allowed before the surcharge must be removed. It is recommended that the length of the surcharge, SL, be equal to approximately five times the sum of the height of embankment, \( H_e \), and depth of foundation, \( d_f \):

\[
SL = 5(H_e + d_f).
\]

Economics and benefits must be investigated for each bridge site. It may not be beneficial nor economical to surcharge approach embankments exceeding heights of approximately 50 feet (11). When embankment heights exceed about 50 feet, a 10-foot high surcharge load may be somewhat insignificant in relation to the total load of the approach embankment.
When performing settlement analyses, the rate of foundation settlement often proceeds much faster than the predicted rate. As one means of constructing a more realistic, predicted settlement-time curve (mainly for alluvial deposits), the end of primary consolidation, $t_{100}$, is assumed equal to, or nearly equal to, the time of completion of the embankment or the surcharged embankment. The magnitude of primary settlement is estimated using conventional incremental-load consolidation test data. Estimated time for completion of an embankment of a given height is shown in Figure 99. Estimated secondary compression of the foundation is obtained from Equation 1. The coefficient of secondary compression, $C_s$, is obtained from conventional incrementally loaded consolidation tests. In that equation, the ratio of $t_{100}$ to $t_{sec}$ is taken as 10, or the value obtained from the ratio of $t_{100}$ (estimated empirically or from laboratory tests) and $t_{sec}$ (assumed). For practical purposes, $t_{sec}$ should be at least equal to about 25 years. To obtain the primary compression curve, the estimated field coefficient of consolidation is estimated from Equation 5, and Equations 7 and 8 are used to calculate settlements as a function of time. Using the empirical value estimated from Equation 5, the primary compression is calculated in the usual manner. These analyses are then repeated for the surcharged fill. The amount of settlement under the surcharged fill should be of a sufficient amount to eliminate the total settlement of the final embankment that would occur in 25 years (see Reference 21). Whether this amount of foundation settlement can be eliminated in a reasonable time will depend on the actual rate of settlement and surcharging period.
2. If analyses show that the use of a surcharged fill is not beneficial or economical and the consolidation analyses indicate slow rates of foundation settlement, wick drains or sand drains may be required to increase the rate of settlement. It is recommended that such drains be installed in the foundation within a distance, SL, of the bridge end given by Equation 17. In some cases, the use of wick or sand drains in combination with surcharging may provide a good means of accelerating foundation settlement.

3. Completely or partially remove soft compressible material in the foundation and replace with rock or a suitable well-compacted material. This procedure may not be practical or economical at depths greater than 10 or 15 feet or where seepage into the excavation may pose a problem. In certain situations, removal of a small portion of the foundation may be beneficial in minimizing effects of secondary compression. It is recommended that the benefits and costs of total or partial removal of the foundation be analyzed.

4. Allow sufficient time for consolidation of the foundation under the load of the planned embankment. In many instances, for the case of pile end-bent abutments, the time between construction of the embankment and approach paving may be sufficient to complete primary consolidation of the foundation. However, in other instances, it may be necessary to extend this period, whenever feasible. For the case of the open-column abutment, the embankment should be constructed before the abutment to allow as much time as possible before pavement construction. In performing settlement analyses, Figure 99 (heights of various embankments plotted as a function of completion times) may be useful in estimating the completion time of a given embankment. The time between
the start of construction and the time the approach pavements will be placed should be determined or estimated. This period of time must be known so that the various settlement analyses as described above may be accomplished. See Item 1 above and the sections entitled PROPOSED EMPIRICAL METHOD OF ESTIMATING RATE OF PRIMARY SETTLEMENT OF FOUNDATIONS and PROPOSED EMPIRICAL METHOD PREDICTING SETTLEMENTS OF APPROACH EMBANKMENTS.

5. Use of lightweight fill, such as furnace slag, lightweight concrete, expanded shale (when available), coal waste refuse, coal, or other materials having relatively small unit weights may be helpful in reducing foundation settlements. Such materials should be encapsulated in a blanket of clay to avoid or minimize the acidity and corrosive effects.

6. When possible, approach embankments should be constructed of materials that exhibit small secondary compression and shear strain settlements. Granular materials, when well-compactad, will generally exhibit smaller secondary compression and shear strain settlement than fine-grained soils. However, secondary compression and shear strain settlement will occur even in well-compactad, granular embankments.

It is recommended that special compaction provisions be used in the design and construction of approach embankments. Some key elements of recommended provisions follow:

1. The recommended maximum length, \( L_c \), of the approach embankment where special compaction provisions would be applicable and where the magnitude of settlements warrant attention is as follows:

\[
L_c = 5(H_e + d_f)
\]  

(18)

where \( H_e \) = height of approach embankment and
\( d_f \) = depth of foundation.

2. The approach embankment should be compacted to 98 percent of maximum dry density (ASTM D 698). The recommended tolerable range of moisture content about optimum moisture content (ASTM D 698) is &plusmn;2 percent.

3. Recommended compaction equipment and methods applicable to fine- and coarse-grained soils are contained in Reference 21 (NAVFAC DM-7.2, May 1982), Table 5, pages 7.2-48 and 7.2-49 (also see Reference 22). However, when durable rock is used, it is recommended that the compacted lift thickness be 2 feet instead of 3 feet. This recommendation is not applicable to shales.

4. When durable shales have a slake-durability index (as determined from KH-64-513(79), 27) greater than 95 percent, the compacted lift thickness shall not be greater than 1.5 feet. See Table 5 in Reference 20 for recommended compaction equipment and methods. Tests should be performed on crushed specimens to estimate gradation characteristics of durable shales or rock. These characteristics will determine compaction methods and equipment to specify.

5. When nondurable shales having a slake-durability less than 95 percent but greater than 60 percent or when mixtures of non-durable shales and durable rocks are used to construct the approach embankment, the recommended loose lift thickness should not be greater than 12 inches. For non-durable shales having a slake-durability index less than 60 percent, the recommended loose lift thickness is 8 inches.

6. To compact shales having a slake-durability index less than 95 percent, the following items are recommended:
A. Oversized material — any rock material measuring 6 inches in
thickness and 1.5 feet in any other direction should — not exceed 20
percent of the compacted mass; that is, not more than seven 6-inch or
larger pieces in a square yard. Blasting techniques should be employed
to shoot the rock into pieces as small as possible.

B. Recommended loose lift thicknesses are described in Item 5
above.

C. Since the natural water contents of most non-durable shales are
below optimum as determined from ASTM D 698, the addition of water to
the shale will be necessary to obtain proper compaction. It is
recommended that water be added using only a spray bar attachment. The
moisture content range should be +2 percent about optimum water content.
Loose shale and water should be mixed using a heavy-duty 24-inch disks.

D. Shales should be compacted using static and vibratory
compactors. A static foot roller having a minimum weight of 53,000
pounds should be used to break down the shales after mixing with a
24-inch disk. The static tamping roller feet should measure a minimum
of 6 inches in length. Each foot should have a small area. Recommended
number of passes is two to four. When a vibratory pad drum roller is
used, the minimum weight should be 55,000 pounds. When a pneumatic
tire, vibratory roller is used, the minimum weight should be 100,000
pounds. Recommended number of passes using the vibratory roller is four
to six. If 98 percent of dry density is not achieved in six passes,
then the number of passes of the vibratory compactor should be increased
until such result is obtained.

E. Use of nuclear density equipment is permissible only if it is
checked (calibrated) against the sand cone technique.
F. Laboratory maximum dry density and optimum moisture content should be adjusted for oversize particles according to techniques and equations given elsewhere (28, 29) before comparing to field values.

Secondary compression and shear strain may occur even for well-compacted fills. These settlements may amount to 0.3 to 0.6 percent of the fill height in 15 to 20 years (21). As shown in Figure 103, estimated settlement of the embankment due to secondary compression and shear strain for embankments constructed of fine-grained plastic soils (CL, OL, MN, CB) become significant for embankment heights greater than about 30 feet. For other materials (GW, GP, GM, GC, SW, SP, SM, SC, and ML), settlements become significant for embankment heights greater than about 55 feet. Hence, in the construction of approach embankments, it is recommended that priority be given to selecting materials that exhibit little secondary compression. Moreover, consolidation tests using compacted specimens of embankment materials should be performed to estimate secondary compressions that may occur over a 25-year period. Efforts should be made in the design to limit the amount of secondary compression. If it is uneconomical to use select materials or if select materials are not available, and the approach embankment must be constructed to a height greater than about 30 feet of plastic soils, then consideration should be given to using temporary asphaltic concrete approaches that may easily be overlayed. Estimates of settlements of approach embankments due to shear strain and secondary compression may be obtained from the empirical method presented above (Equations 9 through 14). The technique also may be used to estimate a maintenance schedule for a newly constructed bridge approach pavement.
Figure 103. Secondary Compression and Shear Strain as a Function of the Height of Fill (21).

Figure 104. Typical Cross Section of a Side Hill Bridge Approach Embankment Where Lateral Movements May Occur.
Alternately, profile measurements of the approach should be obtained periodically for a 4- or 5-year period. The maximum settlement occurring within the settlement cradle should be in the case studies above. Provided the relationship of settlement and the logarithm of time is linear, a maintenance schedule could be established from projected settlements.

It is recommended that drainage measures around bridge abutments and under approach embankment pavements be given particular attention to prevent surface and subsurface erosion. The following items are proposed:

1. Select granular backfill should be used behind, under, and in front of the abutment. The select material should have less than five percent passing the No.-200 sieve.

2. The select backfill should be completely encapsulated with a geotextile filter fabric. Protective filter criteria should be satisfied.

3. Perforated pipe, or PVC, should be installed in the select backfill. The pipes should run to collection points outside the confines of the abutment. Water from these pipes should not be allowed to drain onto unprotected slopes of the approach embankment.

4. Some type of permanent slope protection should be installed on the face of the slopes of both river and dry crossings that lie directly under the bridge. Experience (1) has shown that sodding or plants frequently do not thrive on this area of the slope, especially in the upper reaches. The face of the slope should be covered with a geotextile drainage fabric or sand layer. A thin layer of select rock is placed on top of the fabric or sand. Protective filter criteria
should be met (21). Where stream velocities are large, rip rap should be placed over the smaller-sized select granular material. Based on past experience (1), concrete revetments oftentimes have been unsuccessful because of internal erosion under the revetment. However, this type of slope protection would probably perform properly if the concrete revetment was placed on top of geotextile drainage filter, or sand layer. The fabric would minimize the loss of soil due to erosion and seepage.

5. When drainage pipes of the bridge deck are placed directly over the face of the approach embankment, splash blocks should be placed directly under the drainage pipes to dissipate the energy of the falling water and to prevent erosion of the fill. Drains should not be discharged directly on the faces of approach slopes.

Based on observations reported above and other past experiences (1, 2, and 14-17), a typical situation where lateral movements of the approach embankment frequently occurs is illustrated in Figure 104. Generally, the approach embankment is a side-hill fill. The original groundline slopes toward the river. The approach embankment is located on relatively thin overburden soils. When the bedrock is composed of clayey shales, the overburden soils oftentimes contain a zone of highly weathered shale. For situations of this type, the following design items are recommended:

1. If the overburden soils are relatively thin, consideration should be given to removal of the overburden soils and weathered clayey shale zone. Benches should be constructed in the shale.

2. Select granular drainage material should be placed on the benches to prevent encroachment of groundwater into the approach fill at
a later date. Perforated pipe should be placed in the drainage material.

3. In situations as shown in Figure 104, consideration should be given to using small rock berms, or rock shear keys, at the toe of the approach embankment so the factor of safety can be maintained at a high level. Alternately, the bridge(s) may need to be lengthened.

Longitudinal camber (parabolical curve) of the bridge approach embankment should be considered (1). The amount to be used at any given site would depend on the thickness and compressibility of the embankment and foundation.

BENEFITS OF IMPLEMENTATION

Past studies of bridge approach settlements (1, 2, and 14-17) as well as the present research study have brought about several benefits in Kentucky. Subsurface investigations are presently conducted at all bridge approach embankments greater than 20 feet in height. Undisturbed samples are collected and tested. Triaxial and consolidation tests are performed and both settlement and stability analyses are made. At sites where large settlements may be encountered or where the short-term stability may be questionable, field instrumentation frequently is installed to monitor embankment construction and to insure safe construction. By performing settlement and stability analyses, problem embankments may be identified and the number of bridge approach embankment failures may be potentially reduced. Additionally, improved drainage techniques, as shown in the Appendix, are being used to minimize internal erosion under bridge approach pavements. Hence, the quality of design and construction of highway bridge approach embankments in Kentucky has been improved.
Recognition of the factors that lead to the settlement of highway bridge approach pavements should lead to improvements in the design and construction of approach pavements and embankments. Long-term lateral movements recorded at four sites show that in certain cases lateral embankment movements may be an important factor. Typically, lateral movements frequently occur in side-hill fill situations. Prevention of lateral movements may eliminate or help minimize costly future abutment and approach maintenance. Better compaction of bridge approach embankments may help minimize approach settlement and minimize future maintenance. This study also shows that the long-term effects of secondary compression of the foundation and embankment is a major factor causing the settlement of highway bridge approaches. If measures are taken during design, the effects of secondary compression on bridge approach settlement may be minimized. Hence, future maintenance may be minimized.

The study proposes empirical techniques for estimating the rate of foundation settlement and the settlement of approach embankments. Use of the proposed method of estimating the rate of foundation settlement may lead to making better settlement-rate forecasts than those obtained from laboratory parameters. Consequently, the technique may lead to improvements in scheduling construction stages of bridge approach embankments. The proposed empirical technique for forecasting settlement of approach embankments provides the designer with a decision-making procedure whereby choices may be made concerning acceptable settlements and factors of safety. As a result, the economies of these choices may be studied. Logical choices may be made with regard to available funding. This technique will be useful in
helping decide whether the abutment must be located on piles or may be placed on spread footings. The study strongly indicates that, in certain situations, abutments may be safely located on spread footings provided careful study, analysis, and design are performed. Hence, pile foundations may not be needed in those situations. However, extreme caution must be exercised by the designer. In particular, it is potentially dangerous to place abutments on spread footings on relatively large embankments (heights greater than 50 feet) unless the bridge can be designed to withstand large settlements. The success of a spread footing design will require great cooperation and coordination among the geotechnical designer, the bridge designer, and inspection and maintenance personnel.

REFERENCES


17. Gorman, C. T.; Hopkins, T. C.; and Drnevich, V. P.; "In Situ Shear Strength Parameters by Dutch Cone Penetration Tests," Division of Research, Kentucky Department of Transportation, September 1973 (also presented to the European Symposium on Penetration Testing, Stockholm, Sweden, June 1974).


27. Kentucky Methods, Division of Materials, Department of Transportation, Commonwealth of Kentucky, Frankfort, Kentucky, 1976.


APPENDIX

TREATMENT OF EMBANKMENTS AT BRIDGE END-BENT STRUCTURES
CONSTRUCTION SEQUENCE "A"

1. Construct Embankment to slopes A, B, F, and G such that no uncompacted or loose material shall remain.
2. Excavate for End-Bent to C, D, E, and F.
3. Drive Piling.
4. Place 2" Mortar Bed or Class "A" Concrete.
5. Construct Concrete End-Bent.
6. Backfill to C, D, E, F, G, H, and J.
7. Install 4" Perforated Underdrain Pipe and backfill.

CONSTRUCTION SEQUENCE "B" ①

1. Construct Embankment to temporary Slope ④
2. Drive Piling.
3. Place 2" Mortar Bed or Class "A" Concrete.
4. Construct Concrete End-Bent.
5. Backfill to finished Grade.

NOTES

① Construction Sequence "B" is a permitted alternate by the Contractor only when Select Granular Embankment Case II is required.
② 2" Mortar Bed or Class "A" Concrete.
③ 4" Perforated Underdrain Pipe wrapped with filter fabric for draining the excavated Trench and Select Granular Backfill.
④ Acceptable alternatives for temporary slopes (Construction Sequence "B").
⑤ H = Embankment Height measured from subgrade elevation of point ④ to the lowest elevation of the toe of the slope.
⑥ Shaded portion ④④④ and ④④④ represents Limits of Non-Erodible Select Granular Embankment.
⑦ Slopes are equal.
⑧ See current Special Provision No. 69 for Construction and Material Requirements, Method of Measurement and Basis of Payment.
⑨ Case I material shall be placed as a complete separate operation after construction of all other Embankment.
⑩ INDIVIDUAL FRAGMENTS LARGER THAN 4 INCHES IN ANY DIMENSION SHALL NOT BE PERMITTED WITHIN 3 INCHES OF THE STRUCTURE.