SETTLEMENT OF HIGHWAY BRIDGE APPROACHES
AND EMBANKMENT FOUNDATIONS

by

Tommy C. Hopkins
Research Engineer Senior

February 1973
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BLUEGRASS PARKWAY BRIDGES OVER CHAPLIN RIVER

INTERIM REPORT
KYHPR-64-17; HPR-I(8), Part II

by

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Division of Research
DEPARTMENT OF HIGHWAYS
Commonwealth of Kentucky

in cooperation with the
U.S. Department of Transportation
Federal Highway Administration

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February 1973
### Abstract

The report examines the cause of a differential settlement between the highway approach embankments and bridge decks and abutment tilting at a bridge site located on the Bluegrass Parkway in Kentucky. The site contains several design, construction and maintenance features and soil types that are typical of many bridge sites in Kentucky. The approach embankments are side-hill fills. The approach pavements have settled several inches and have been patched on numerous occasions. Each of the four abutments give the appearance of having tilted backward toward the backfill. However, closer examination shows that the abutments have moved laterally toward the bridge ends and are bearing against the steel girder spans. Slope inclinometer readings obtained at each approach embankment over a two-year period shows that the approach fills have moved toward the bridge ends some three inches. The front portion of the western fill failed in 1969, some four years after construction, and exposed several feet of piling. Shear strength of the embankment and foundation soils were established from consolidated isotropic, undrained triaxial tests with pore pressure measurements and consolidated drained, direct shear tests. Slope stability analyses were performed using a computerized solution of Bishop's simplified method of slices. Study results show that the approaches settled as a result of embankment instability due to progressive failure. Based on peak shearing strength of the soils, the long-term safety factors of the approach embankments ranged from 0.96 to 1.03. Based on residual strength, the long-term safety factors were about 0.70. Consequently, the western embankment failed apparently as a result of a (time-conditioned) gradual decrease of the shear strength of the soils. Slope indicator data indicate that progressive failure is spreading; movement of the embankments is continuing. Stability of the approach falls, abutments and piles is precarious. Several remedial solutions are described.
MEMORANDUM TO:  J. R. Harbison
State Highway Engineer
Chairman, Research Committee

SUBJECT:  Chaplin River Bridges, Bluegrass Parkway; Interim Report on Study
KYHPR-64-17; HPR-1(8), Part II

In connection with the referenced study of settlement of embankments at bridges (i.e. The Bump at the End of the Bridge), we have made an in-depth study of the situation at the Chaplin River bridges on the Bluegrass Parkway. Our interest in this site goes back a few years - at least to the time when the abutments closed against the span and the piles under the west abutment became exposed. Unfortunately, in some respects perhaps, we made some false starts in the beginning in attempting to resolve expedient solutions to the problem. However, as time passed the symptoms become more fully revealed.

Now, having the advantage of well-slope histories and soil parameters, we find that the west embankment has developed into a landslide or fill-slip situation, which extends about 50 feet into the roadway on the west end. To complicate the matter further, recent well-slope measurements indicate that the movement into the bridge is accelerating. Failure of the west abutments seems imminent.

Whereas the planned Campground Reservoir was previously expected to submerge these bridges, I have been advised by Mr. Blankenship that current plans set the high-water elevation at 664 feet, which is 4 feet below the lowest point in the deck. The girders, of course, would be in water at times unless the bridge(s) are raised.

Several alternatives have been explored. Only one has been analyzed thoroughly from the standpoint of structural stability. It is the one discussed in the attached report. The berm configuration presented there (Figure 34) represents the maximum filling that seems possible. Structurally, it does not provide a safety factor which would assure a high degree of certainty or success. It would, therefore, be an expedient measure. The hydraulics of the resulting channel have not been analyzed; this would have to be done. The present channel velocity is in the order of 10 feet per second.

A retaining wall at the toe of each embankment should not be dismissed if the risks of excavating can be surmounted.

We have sketched a conceptual alternative involving low-water culverts and an underflow dam under the bridges to buttress the approach embankments on both sides of the channel. Overflow would be critical from the standpoint of erosion, and slope protection would be essential throughout. This scheme
would not interfere with traffic -- but neither would the berm as proposed in the report.

A lightweight backfill, involving excavation of the west embankment and refilling with lightweight material is also discussed in the report. This scheme could work in conjunction with the berm to attain a higher factor of safety. Of course, it would interfere with traffic.

Other alternatives might include the addition of one or more spans to the bridges at each end.

We have not determined whether piers are plumb or not.

Having realized the criticality of the problem very recently, we have expedited our report and submitted advance draft copies for early consideration to W. B. Drake, J. R. Spurrier, A. R. Romine, and A. B. Blankenship on February 1, 1973.

Respectfully submitted,

Jas. H. Havens
Director of Research

JHH:dw
Attachment
cc's: Research Committee
INTRODUCTION

Differential settlement between highway pavements and bridge decks, Figure 1, not only presents a hazardous condition to rapidly flowing traffic but creates a rough uncomfortable ride. Additionally, these roadway surface faults require costly maintenance which usually involves mudjacking and/or patching the approach pavement. Where heavy traffic movements exist, such maintenance operations tend to impede normal flow. Moreover, settlement of bridge approaches adversely affects durability of the road and structure. Previous studies (1, 2, 3, 4, 5) revealed that, statistically, 80 percent of bridge approach pavements constructed during the past decade in Kentucky eventually settled to such a degree that maintenance was required. Studies (6, 7) and inquiries by others have shown that the approach settlement problem is not necessarily confined to Kentucky; but rather, it occurs in many areas of the world.

The earlier studies showed that approach foundation settlement can be an important factor leading to development of faulted bridge approaches and emphasized the need to conduct settlement investigations at sites of proposed bridge construction projects to provide a basis for design to control or minimize the effect of approach settlement. Settlement of the approach embankment was recognized as a contributor to the settlement of bridge approaches, although the cause – whether due to a volume change or some form of instability, such as creep or progressive failure – was not recognized.

Results of informal observations made in 1969 at several bridge sites where the abutments were supported on piles strongly indicated that approach settlement was due to embankment instability. Abutments at these sites gave the appearance, as illustrated in Figure 2, of having tilted backward toward the backfill. However, closer examination revealed that the 4-inch spaces initially existing between the faces of the abutments and the bridge girder spans and the 2-inch spaces of the bridge expansion dams were, in many instances, closed or nearly closed. Consequently, the abutments had moved laterally toward the bridge and rotated about a horizontal axis. Movements of the type shown in Figure 2 are closely associated with the approach settlement problem. As the abutment moves toward the bridge, there is a tendency for the backfill to move laterally and vertically downward. In every case where such abutment movements have been observed, the approach pavements had settled excessively and required extensive maintenance. Furthermore, in many cases, additional and costly maintenance has been required to reset bridge rockers and replace bridge expansion dams. Possible structural damage, such as deck cracking, of the bridges at these sites after the abutments have borne against the bridge girder spans has not been assessed.

Terzaghi and Peck (8) in 1948 noted that an abutment located on a spread footing and underlain by a compressible layer will become inclined towards the central and deeper part of the settlement crater. In a study of seven underpass bridge structures, Stermac, Devata and Selby (9) attributed the unusual abutment movements to foundation settlement. They also noted that patching the approach pavements only contributes to the process of tilting and subsidence. However, in
a study of a bridge site in New Jersey where earth pressure cells and strain meters were attached to the abutments and top portion of the piles, respectively, Tsebotarioff (10) concluded that the settlement concept as the governing factor leading to the development of abutment tilting was inadequate. Measurements obtained at that site (11) showed that lateral pressures acting against the abutment decreased with time while the lateral pressures acting against the vertical H-piles under the heel of the abutment increased. These measurements suggested that embankment instability may be a likely cause of abutment tilting and approach settlement. Tsebotarioff concluded that shearing deformations in an embankment foundation are a factor causing abutment tilting.

There is other evidence that embankment instability is an important factor causing approach settlement. Bishop (cf. 12) concluded that local overstress can occur in certain areas of the slope when the safety factor, determined by a slip circle method, is below a value of 1.8. Long-term safety factors of the majority of existing approach embankment slopes are unlikely to have safety factors above this value. The majority of earth structures are designed for safety factors less than 1.8. Consequently, a state of plastic equilibrium must be considered to exist in a portion of the slope (12). Such a condition probably initiates embankment creep and may, in some cases, lead to failure.

The report submitted herein is a case history describing observations and in-depth analyses made at a bridge site located on the Bluegrass Parkway. It is a substantial effort to show evidence that embankment instability is a significant factor causing settlement of bridge approaches. The Bluegrass Parkway Bridges over Chaplin River were selected for study because the site contained several design, construction, and maintenance features and soil types that are typical of many bridge sites existing in Kentucky. Moreover, the large movements and conditions observed at the site in the latter part of 1972 provided a better understanding of the nature of many faulted bridge approaches. The report serves a dual purpose inasmuch as a portion of the study was devoted to developing an expedient remedial solution which would prevent the likely failure of the approach embankments, abutments and piles. Also, a portion of the investigation was devoted to the development of a computerized solution of Bishop's simplified method of slices (20) for use in the analysis of the Chaplin River approach slopes and in the design of approach embankments at future bridge sites.

PROJECT DESCRIPTION

The Bluegrass Parkway Bridges crossing over Chaplin River (Project No. CK-3-3-12, Station 2316+29) are located between the central and western portions of Kentucky and lie approximately 40 miles west of Lexington and about 25 miles east of Bardstown. The Parkway is a 4-traffic lane, concrete toll facility and each 2-lane roadway pavement is separated by a 35-foot median. The facility provides a major link between the central and western portions of Kentucky. A general view of the site is shown in Figure 3. Chaplin River, at the site, forms a natural boundary between Washington and Nelson Counties.

An areal plan and centerline profile of the site are shown in Figures 4 and 5, respectively. Both approach embankments are side-hill fills. The eastern approach embankment is approximately 320 feet in length and attains a maximum height in the vicinity of Station 2318+00 of about 50 feet. Slope of original groundline and the rockline under this fill is approximately 14 percent. The eastern foundation is about 10 to 15 feet thick. Grade of the approach pavements on the eastern side of the river is +2.6 percent. The western embankment attains a maximum height of about 25 feet.

Figure 3. General View of the Bluegrass Parkway Bridges over Chaplin River, Looking Westward.
Figure 4. Areal Plan of the Bluegrass Parkway Bridges over Chaplin River.
Figure 5. Centerline Profile Section of the Chaplin River Site.
near Station 2314+70 and is about 70 feet in length. Record plans show that the foundation under this fill was to have been benched in the vicinity of the latter station. Rockline under the western embankment slopes steeply; it has a grade of about 40 percent. Grade of the western approach pavement is -4.0 percent. Each approach embankment contains an earth core constructed to facilitate pile driving. Slopes of the front portion of each approach embankment are 1.5 horizontal to 1 vertical. Record plans specified that the front portion of each embankment was to have been constructed of rock.

The two bridges and four abutments are identical in design. The bridges are 330 feet in length, 30 feet in width and have span lengths of 100, 130 and 100 feet. Each abutment is 14 feet in height, 8 feet in width at the bottom, 2 feet in width at the top and 37 feet in length, and rests on two rows of H-piles (12 BP53) totaling 13. Wingwalls of the abutments are not supported on piles. Piles in both the front and back rows were battered according to recommendation by Stermac, Devata and Shelby (9) that provisions should be made in abutment design to resist a horizontal force, presumably acting in a direction away from the bridge. Batter of the front piles was 3 horizontal to 12 vertical; the back row was battered 2 to 12. The concept was utilized for a period of time at several bridge sites on the Bluegrass Parkway and in the Department’s design criteria, but it was later abandoned when experience indicated the reverse batter produced more likelihood of lateral movement toward the bridge (13).

Design records show that, in checking the stability of the abutments, the equivalent-fluid concept was used to compute the active horizontal thrust due to the weight of the backfill. In these computations, the unit weight of the backfill was assumed to have a value of 34 pounds per cubic foot. The equivalent fluid concept has been used extensively in Kentucky. Computations show that for the backfill to conform to the above assumption it would have an internal angle of friction, $\phi$, of about 35 degrees. In effect, the design backfill material is assumed to be sand or gravel. In actuality, few abutments in the state have been backfilled with such materials. Records show that construction of the embankments and bridges started in the spring of 1964; they were completed in the spring of 1965.

SURFACE INVESTIGATION

Topography. The site is situated in the western portion of the Bluegrass Physiographic Region, one of three sections occupying the Interior Low Plateaus Providence of mid-central, eastern United States. The Bluegrass area is limestone country and, typically, a rolling upland in the youthful stage of dissection. The area is characterized by Ordovician outcrop (14). The surface of the area is mildly karst and gently rolling. However, in the vicinity of large streams, the surface is geologically mature and distinctly rough.

Topography in the vicinity of the site is rugged, dissected and consists of long, narrow, steep-sided ridges and narrow, winding, V-shaped valleys (15). The drainage pattern of the area is dendritic. Such terrain is typical of the Eden Hill country, one of three distinguishable areas of the Bluegrass Region. Total variation in relief near the site is approximately 130 feet, ranging from an elevation of 595 feet in the valley bottom of Chaplin River to an elevation of about 725 feet at the crests of the adjoining hills.

Present Site Conditions. During inspections of the site (Figure 3) in late 1972, several visible failure signs were observed which indicated the stability of the approach embankment piles and abutments as being precarious. These are indicated on the areal plan of the site (Figure 3). As shown in Figure 6, a portion of the embankment located in front of the western abutments had slumped and moved downward several feet exposing some 8 to 10 feet of the H-piling supporting the westbound abutment and the top of the piling supporting the eastbound abutment. The lower portion of the failure was resting against the piers located on the west side of the river (Figure 7). The surface of the slump was S-shaped. Development of the slump started sometime in 1969. Efforts made at that time to correct the failure consisted of removing a portion of the material at the top of the slump, installing perforated pipe near an elevation of 650 feet at Station 2314+70 and rebuilding the top of the slump with stone to approximately the original grade. These efforts were based on the assumption that failure was caused by ground water seepage. Unfortunately, the reconstruction was unsuccessful, and since 1969 the slump continued to move downward as shown in Figure 6. Placement of rock at the top of the slump added additional driving moments.

The downward and lateral movement of the slump apparently affected other portions of the western embankment. As shown in Figure 8, transverse cracks occurred in the pavement some 56 feet behind the abutments near Station 2314+00. Both abutments had tilted backward (Figure 9); the bottom portions of each abutment were bearing against the bottoms of the bridge girder spans (Figure 10). A portion of the embankment (Figure 11) located between the wingwalls of the western abutments failed in late 1972. As shown in the bottom photograph of Figure 8 cracks occurred in the western approach pavements some 20 feet behind the abutment. Both approach pavements settled and required patching.
Present conditions at the eastern approach embankment are similar to those of the western embankment, although no portion of the eastern embankment has failed. Both abutments tilted, as typified in Figure 9, and the bottom portion of each abutment contacted the bottom edges of the steel girder spans. Approximately 76 feet behind the abutments, transverse pavement cracks appeared (Figure 12). Some slumping or settling of the embankment along the wingwalls is evident, as shown in Figure 13. Both approach pavements have settled excessively and required patching (Figure 14). The easterly abutment in the eastbound lanes is cracked.

SUBSURFACE INVESTIGATION

Engineering Geology. Bedrock (Clays Ferry Formation) of the hills adjoining the site is composed of sedimentary rocks of Upper Ordovician Age. A generalized geologic columnar section is shown in Figure 15. Rock strata at the site consist predominantly of olive-gray, lumpy, calcareous shale with some thin, medium to dark gray, evenly bedded, argillaceous limestone and dark-gray, calcareous siltstone layers. The limestone occurs as calcarenite and calcisilite in 1- to 3-inch thick beds (15). The siltstone, which forms a relatively small portion of the Clays Ferry Formation, occurs in 1- to 3-inch thick platy beds.

The hilly and geologically mature topography at the site was formed by weathering and erosion of the weaker, impervious shale layers. When soaked, the shales of the Clays Ferry Formation slake and fall apart. Downcutting action by Chaplin River created the present valley at the site. With the rapid erosion of the weak shales on the steep slopes, slabs of intercalated limestones were left scattered over the surface as shown in Figure 3, a view of the terrain and geology of the site.
Figure 8. View of Cracks Occurring in the Western Approach Pavements, near Station 2314 + 00.

Figure 9. View of Tilted Bridge Abutment, Westbound, West End.

Figure 10. View of the Westbound, West End Abutment Resting against the Steel Girder Span of the Bridge.
Figure 11. View of Slumped Material Located between the Wingwalls of the Western Abutments.

Figure 12. View of Cracks Occurring in the Eastern Approach Pavements, near Station 2318 + 70.

Figure 13. Settlement of the Embankment along the Eastern Abutment Wingwalls.

Figure 14. View of the Heavily Patched Eastern Approach Pavements.
Figure 15. A Generalized Geologic Columnar Section of the Chaplin River Area.
The surface of the sedimentary rock strata decomposed by weathering, forming a relatively thin zone of residual soils. The residual soils are highly plastic and are the most unstable regolith found in the area. These soils are always potential failure planes and several highway fill slips have occurred in the area. A thin layer of alluvium measuring about 20 feet in thickness covers the bedrock in the valley bottom of Chaplin River. The alluvium consists of clay, silt and gravel. Occurrence of these materials is of nominal significance to the stability of the approach embankments except as a drainage basin. The approach embankments are located mainly on the residual soils paralleling the underlying rock surface, although a relatively small portion of each of the embankment toes rests on the alluvial deposits in the bottom of Chaplin River.

Soil Exploration. Twenty-one undisturbed, Shelby tube soil samples were obtained from six borings. Locations of points from which the tube samples were extracted and the borings are shown in Figures 4 and 5, respectively. Four borings were drilled in the eastern approach embankment and foundation; two were drilled in the western approach embankment and foundation. Boring results are presented in Figure 16. Values of liquid limit, plasticity index, and natural water content are shown plotted adjacent to each boring. Attempts to obtain Dutch Cone penetration values for evaluating the in-situ shear strength of the approach soils were unsuccessful because of a failure of the Dutch Cone to penetrate the rocky soils at the site.

Field Instrumentation. In July 1970, slope inclinometer casing was installed in each approach embankment (see Figure 4) in an effort to determine the slip zone(s), rate and direction of movement, and mode of failure of the approach embankments, and to obtain subsurface water conditions. Perforated downspout was installed in each of the borings, Figure 4, to obtain additional groundwater information. Horizontally resultant movement and slope indicator dial changes as a function of depth, as well as resultant horizontal movement-time curves for selected depths, obtained at the eastern and western approach embankments are presented in Figures 17 and 18, respectively. Groundwater data obtained from the slope indicator wells are tabulated in Table 1. Very little useful data were obtained from the perforated downspout wells.

LABORATORY INVESTIGATION

Sample Preparation. Soil samples were extruded from the Shelby tubes, cut into 4-inch lengths, identified according to the visual-manual procedure (ASTM Designation: D 2488T), waxed and stored until ready for testing. Water content determinations were performed on each of the extruded samples. These results are showed in Figure 16. During trimming, a few samples were found to be unsuitable for strength testing purposes because of the presence of large stones and were discarded. Consequently, in a few cases there was a short supply of test specimens.

Index Properties of Soils. Soils located in both approach embankments and foundations generally consist of medium stiff to very stiff, moist, brown and gray plastic clay with some limestone and shale gravel. Index properties of the soils are tabulated in Table 2. These soils classify by the Unified, AASHO and Textural systems as CL, A-6 (06 to 16) and Clay. Liquid limit and plasticity index of the soils average about 37 percent and 17 percent and range from 32 to 41 percent and 13 to 20 percent, respectively. Natural water content is generally near or slightly below the plastic limit; liquidity index averages about -0.13 and ranges from 0.22 to -0.36. Hence, preconsolidation pressure of the soils is high. Activity of the soils (the ratio of plasticity index to percent by weight finer than 2 \( \mu \)), a measure of the ability of soil particles to attract water, is comparatively low ranging from 0.44 to 0.61. Permeability of the soils is very low. The soils are poorly graded. Materials located in the eastern approach embankment consist of about 10 percent gravel, 14 percent sand, 30 percent silt and 46 percent clay. Foundation soils of the eastern approach consist of approximately 12 percent gravel, 14 percent sand, 35 percent silt and 39 percent clay. Soils of the western approach consist of about 8 percent gravel, 15 percent sand, 39 percent silt and 38 percent clay. Values of the index properties of the soils located in the approach embankments and foundations vary only slightly.

Strength Tests. Soil strength parameters of the approach embankments and foundations were established from consolidated isotropic, undrained triaxial tests (CIU) with pore pressure measurements and consolidated drained, direct shear tests (CDS). The triaxial compression samples were subjected to a back pressure to completely saturate the test specimen and sheared undrained at a rate of 0.001 inches per minute. Pore pressure measurements were obtained from pore pressure transducers.
Figure 16. Boring Results.
Figure 17. Slope Indicator Results, Eastern Approach Embankment.
Figure 18. Slope Indicator Results, Western Approach Embankment.
Table 1.
Groundwater Elevations Observed at the Eastern and Western Approach Embankments.

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</tbody>
</table>

NOTE: Holes 2, 3, 4 and 7 drilled at Eastern Approach and Hole 6 drilled at Western Approach.
Results of the CIU tests performed on the eastern approach embankment, core, top and bottom foundation soils are shown in Figures 19, 20, 21 and 22, respectively. CIU test results for the western approach soils are shown in Figure 23. Three triaxial specimens are normally tested to determine shear strength parameters; however, this procedure could not be followed since several test specimens contained some large stones and had to be discarded. As a result, only two specimens from each soil zone, except the core, were found suitable for CIU testing. Since a shortage of CIU specimens had been anticipated, each specimen suitable for testing was strained beyond 25 percent, except test H-7 S-6A, to develop fully the $K_f$-failure envelope on a $p-q$ diagram. Shear strength parameters were obtained from relationships given by Lambe (16).

The CIU specimens were also strained to large values to observe the behavior of the effective stress paths and to, primarily, determine if the stress paths at large strain values would yield some indication of the residual strength of the approach soils. However, these efforts were unsuccessful, perhaps because triaxial compression tests cannot be extended to the large strains necessary to develop accurately the residual strength shear parameters. Large discontinuities in the curves shown in Figures 19 through 23 occurred when the CIU tests were cutoff at the end of the workday. Straining of the specimens was halted but the loads remained. With start-up the next day, the effective stress paths rose slightly above the $K_f$-failure envelop; but with increasing strain, the paths continued along the failure envelop.

Direct shear specimens were trimmed, placed in the shear apparatus, and consolidated to normal stresses (applied hydraulically) that were at least twice the effective in situ overburden pressures. The 2-inch diameter by 2-inch thick specimens were sheared in one direction to large displacements as suggested by Skempton (17). The average rate of strain was about $4.5 \times 10^{-4}$ inches per minute, some 1.5 to 2 times faster than the rate proposed by Lambe (18). Movement was slow, so that presumably excess pore pressures were not present in the clay specimens during shear. Generally six to eight days were required to consolidate and shear a specimen. During shear as the effective area of the test specimen decreased, the vertical load was decreased in such a manner that the applied normal stress was maintained essentially constant throughout the shearing operation. Generally, the load was reduced in one-pound increments about every 10 to 20 minutes.

Variation of the normal stress from the initial stress for all the tests was less than 0.5 percent. Volume decrease of each of the specimens during shear was primarily due to secondary compression and it was less than 0.5 percent. Approximately 90 to 95 percent of the volume decrease occurred before the peak shear stress was reached. The shear lips had sharp edges so that presumably adhesion between the failure surface of the clay and the lips of the shear rings was nominal and, therefore, the measured shear load was not significantly affected. Accurate measurements of vertical and horizontal deflections and load were obtained with dial gages having resolutions of 0.0001 inches.

Shear stress - horizontal deflection curves obtained from CDS tests performed on materials from the eastern approach embankment and foundation are presented in Figure 24. Fluctuations in the shear stresses after peak stresses had been reached were partly due to the presence of stones in the failure plane and partly due to cutoff and start-up of the shear test. Mohr’s failure envelopes for the various zones of soils were obtained from the data shown in Figure 24 by assuming that the measured stresses at failure are in the ratio

$$\frac{\tau}{\sigma} = \frac{\tau_{ff}}{\sigma_{ff}} = \tan \phi,$$

where $\tau_{ff}$ and $\sigma_{ff}$ are the shear and effective normal stresses at failure. In effect, the horizontal plane through the shear box and the theoretical plane were assumed to be identical. Peak and residual failure envelopes as well as failure envelopes obtained from triaxial tests are shown and compared in Figure 25.
Figure 19. Consolidated Isotropic, Undrained Triaxial Test Results, Eastern Embankment Soils.
Figure 20. Consolidated Isotropic, Undrained Triaxial Test Results, Eastern Embankment Core Soils.
Figure 21. Consolidated Isotropic, Undrained Triaxial Test Results, Eastern Foundation Soils, Top Layer.
BLUEGRASS PARKWAY
STATION 2318+09, ε
EASTERN APPROACH FOUNDATION SOILS (BOTTOM OF LAYER)
O HOLE 7 SAMPLE 6 55-57.5 FEET
\( \triangle \) HOLE 7 SAMPLE 6A 55-57.5 FEET
\( \sigma = 23.1^\circ \)
\( c = 4.26 \text{ PSI} \)

\[ q = 3.967 + 0.3920 \bar{p} \]
\[ \bar{p} = \frac{\sigma_1 - \sigma_3}{2} \text{ (POUNDS/INCH}^2) \]

Figure 22. Consolidated Isotropic, Undrained Triaxial Test Results, Eastern Foundation Soils, Bottom Layer.
Figure 23. Consolidated Isotropic, Undrained Triaxial Test Results, Western Approach Soils.
Figure 24. Shear Stress-Horizontal Deflection Curves, Eastern Approach Soils.
Figure 24. (Continued)
Figure 25. Comparison of Mohr's Failure Envelopes Obtained from CIU and CDS Tests.
Shear strength parameters obtained from CDS and CIU tests are tabulated and compared in Table 3. Specimens from the eastern embankment core designated for CDS testing were found, after trimming, unsuitable for testing and, consequently, CDS results were not obtained for the core materials. The same situation was encountered with specimens from the western embankment soils. Shear strength parameters, $\phi$ and $c$, obtained from CIU tests performed on the eastern embankment soils were 26.2° and 0, respectively, while the peak and residual parameters, as determined from CDS tests, were 28.2° and 0, and 25.3° and 0, respectively. Strength parameters determined from triaxial tests for the eastern embankment core were 24.1° and 267 pounds per square foot. CDS test results are not shown for reasons explained previously. The largest difference in peak and residual parameters occurred in the case of CDS tests performed on the eastern approach (top portion) foundation materials. The peak shear strength parameters, 27° and 321 pounds per square foot, decreased to residual values of 22.6° and 0. For the bottom portion of the eastern approach foundation, the peak and residual parameters were 31.1° and 108 pounds per square foot and 29.2° and 75 pounds per square foot, respectively. Triaxial parameters for the western embankment soils were 21.8° and 230 pounds per square foot.

According to Skempton (18), overconsolidated plastic clays that have had an opportunity to weather -- thereby developing a jointed or fissured structure -- will generally exhibit the greatest difference in peak and residual strength, while the difference in peak and residual strength of reworked compacted clays may be small and insignificant, depending however on the clay content of the soils. With an increase in clay content, the difference in the two strengths increases; conversely with an increase of gravel, sand and silt, the difference decreases. Shear strength results presented in Table 3 generally reflect these concepts. Soils located in the top of the eastern approach foundation were weathered and exhibited the greatest reduction in strength. The bottom portion of this layer, which was less susceptible to weathering than the top portion, showed a much smaller decrease in shear strength. The slight reduction in shear strength of the eastern embankment may have been due to the clay retaining part of its fissured structure after deposition and a failure to completely rework the soils embankment and foundation, centerline of roadway, there are two principal failure zones and several shear planes present (see Figure 17). The zones occur at depths ranging from approximately 0 to 23 feet and from about 40 to 52 feet. At a depth of 45 feet, a distinct shear plane occurs as shown by the data in the left portion of Figure 17. Total recorded horizontal resultant movement for the upper portion of the embankment since installation of the slope inclinometer in July 1970 ranges from about 3.5 inches to 1.5 inches and for the bottom portion of the fill the recorded movement is slightly over 1 inch.

Movement of the eastern approach fill is in a westerly direction, toward the river. As shown in Figure 17, these movements (for selected depths) from June 1970 to June 1972 are linear with time. During this period, the upper portion of the fill moved at a rate of about 1 inch per year while the lower portion moved at a rate of approximately 0.5 inch per year. However, after March 1972, movement of the upper portion of the embankment from about 0 to a depth of 40 feet accelerated. Assuming the relationship between the movement and time was linear during the period 1965 to March 1972, the total movement of the upper portion of the eastern fill is computed to be 7 inches and 3.5 inches for the lower portion. The validity of measurements obtained from the eastern slope indicator is indirectly confirmed by the fact that the spaces between the faces of the abutments near the rockers and the bottom edges of the steel girder spans, originally 4 inches, were nearly closed in 1969, four years after completion of construction (see Figure 10).

Progressive failure of the eastern approach embankment apparently started shortly after completion of construction, and, initially, probably involved the sloping portion of the embankment (Blocks I and Ia) as visualized in Figure 26. Failure mode of this embankment appears to conform to concepts presented by Bjerrum (19). This mass appears to have moved laterally, toward the river, and downward. Consequently, the abutments moved laterally until they were restrained by the ends of the steel girder spans. Once horizontal movement of the abutments was halted by the girder spans, the upper portion of the soil mass (Block 1a) was constrained in the horizontal direction, although downward movement of the mass was not necessarily restricted. However, the upper portion of the

**ANALYSIS**

**Failure Mode of Unstable Approach Embankments.**

Based on measurements obtained from the slope indicator casing located in the eastern approach
Table 3. Shear Strength Parameters and Summarized Index Properties of CIU and CDS Test Specimens Obtained from Eastern Approach Soils.

<table>
<thead>
<tr>
<th>SOIL ZONE</th>
<th>CONSOLIDATED-UNDRained TRIAXIAL COMPRESSION</th>
<th>CONSOLIDATED-DRAINED, DIRECT SHEAR</th>
<th>INDEX PROPERTIES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \delta ) (DEG) ( e ) (PSF)</td>
<td>( \delta_p ) (DEG) ( e_p ) (PSF)</td>
<td>( \delta_r ) (DEG) ( e_r ) (PSF)</td>
</tr>
<tr>
<td>Embankment</td>
<td>26.2 0</td>
<td>28.2 0</td>
<td>0 0</td>
</tr>
<tr>
<td>Embankment Core</td>
<td>24.1 267</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation Soils (Top of Layer)</td>
<td>22.0 364</td>
<td>27.0 321</td>
<td>22.6 0</td>
</tr>
<tr>
<td>Foundation Soils (Bottom of Layer)</td>
<td>23.1 613</td>
<td>31.1 108</td>
<td>29.2 75</td>
</tr>
<tr>
<td>Embankment Soils</td>
<td>21.8 230</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>( *34 ) ( 15 ) ( 0.22 ) 30 CL 0.50</td>
<td>( *32 ) ( 13 ) ( -0.16 ) ( 0.29 ) 27 - 35 CL 0.44</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( *37 ) ( 17 ) ( 0.07 ) 39 CL 0.59</td>
<td>( *34 ) ( 15 ) ( -0.02 ) ( 0.12 ) 35 - 42 CL 0.57</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( *37 ) ( 17 ) ( 0.07 ) 29 CL 0.59</td>
<td>( *33 ) ( 16 ) ( 0.31 ) ( 0.16 ) 21 38 CL 0.61</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( *37 ) ( 17 ) ( -0.01 ) 30 CL 0.57</td>
<td>( *35 ) ( 15 ) ( 0.01 ) ( 0.22 ) 28 32 CL 0.61</td>
<td></td>
</tr>
</tbody>
</table>

*Average Values
*Range of Values
Figure 26. Visualized Failure Patterns of the Approach Embankments.
January 1966. The stability analysis of constructed embankments by Bishop was programmed for the 370 IBM computer in order to investigate rapidly practically all avenues of failure in a given slope problem and so that the minimum safety factor and the most critical failure circle could be obtained from a grid type, search operation. Accuracy of the computer program was verified by comparing safety factors obtained by others with safety factors obtained from the computer program. Additionally, safety factors determined from hand

Shear stresses along the leading edges of each of the failure masses were apparently greater than the peak strengths of the soils in those zones. The condition was largely fostered by the intrusion of groundwater into the bottom portion of the embankment. With a rise in the water table and an increase in pore pressures, shear strengths of the soils decreased. Additionally, the availability of water also may have resulted in an increase in shear stresses above those normally induced by the force of gravity. These supplementary shear stresses can be caused by the tendency of over-consolidated plastic clays, such as located in the bottom portion of the fill at this site, to swell or expand in the presence of water. This tendency to expand would, in effect, induce lateral stresses parallel to the foundation. With downward and outward movement of Block 1, lateral stresses between the two blocks gradually decreased, mobilizing resistance of the battered piles and the shear strength of the soils along the second plane of failure (Block 2). The accelerated movements of the embankment observed after March 1972 and the observed pavement cracks at about the same time indicate the yield stress of the steel piles may have been reached. Consequently, stability of the eastern approach embankment is precarious, and complete failure of the embankment, piles and abutment is a likely possibility.

The failure mode of the western approach embankment is similar in almost every respect to the failure pattern, as described above, of the eastern approach embankment. However, in the former case, the failure pattern was more explicit and developed at a faster rate. Progressive failure apparently began immediately after construction and has continued. Referring to Figure 26, the front portion of the embankment, Block 1, completely failed in early 1969, some four years after construction, moving downward and outward toward the river and exposing, initially, about 5 feet of the battered piles of the westbound, west abutment (see Figure 6). Complete failure of the soil mass between the abutments (a portion of Block 1a), which contained the upper portion of the slope indicator casing, was delayed partly perhaps by the effect of arching between the abutments until mid-1972. The recorded movement of that mass from mid-1970 to mid-1972 was about 9 inches (Figure 18). From mid-1970 to mid-1971, the mass moved at a rate of about 0.5 inch per year; after mid-1971, the movements accelerated and in August 1972 the mass collapsed. Assuming a linear relationship between movement and time for the period mid-1965 to mid-1969, the computed total movement of Block 1a located behind the abutments was 2 inches.

With the downward and outward movement of Block 1, lateral stresses between Blocks 1 and 2 gradually decreased, mobilizing the shear strength of soils along the leading edges of Block 2 and the resistance of the battered piles. Consequently, the active thrust of Blocks 2 and 1a and the piles was large enough to push the bottom of the abutments near the rockers against the steel girder spans – an event which occurred during the period 1965 to 1969. Once the abutments contacted the ends of the bridge girder spans, portions of Block 1a were constrained to some extent in a horizontal and vertical direction by the piles and abutments. However, slope indicator data and the appearance of pavement, shoulder, and ground cracks indicate that failure of Block 2 is in progress. Slope indicator data show that a distinct failure zone has developed at a depth of above 22 feet, near the rock line. Block 2 is moving at a rate of about 0.3 inch per year. Hence, stability of the western approach embankment is extremely precarious.

**Stability Analysis of Constructed Embankments.**

Slope stability of the bridge approach embankments was analyzed using the simplified version of Bishop's method of slices (20). Slope stability computations were based on effective stress methods. Bjerrum (21) and Sevaldson (23) have shown that the application of total stress methods to long-term stability of existing slopes is invalid. The accuracy and justifiable use of the Bishop approach in slope stability problems has been well documented and established (21, 22, 23). Equations of the simplified Bishop method and the general equations necessary to describe the geometry of a given slope were programmed for the 370 IBM computer in order to investigate rapidly practically all avenues of failure in a given slope problem and so that the minimum safety factor and the most critical failure circle could be obtained from a grid type, search operation. Accuracy of the computer program was verified by comparing safety factors obtained by others with safety factors obtained from the computer program. Additionally, safety factors determined from hand
computations of several slope stability example problems were found to be the same as those obtained from the computer program.

Efforts to determine the short-term and long-term stability of the constructed approach embankments were influenced and complicated by three factors and involved certain simplifying assumptions. The first factor involved the treatment in the analyses of the influence of the external resisting forces due to the soil-abutment-bridge girder spans interaction, piles, and piers on the stability of the approach embankments. Inclusion of these forces in the analyses would have required an extension of the Bishop equations and the computer program. Furthermore, an accurate determination of these forces appeared formidable. Consequently, by ignoring these forces, the stability analyses were simplified. Therefore, safety factors computed on the basis of this simplifying assumption represented minimum values; that is, the actual safety factors were indeterminately higher.

The second factor influencing the results of the stability analyses involved an accurate determination of pore pressures and an assessment of their effect on the shear strength of soils located in the approach embankments and foundations. Accurate determinations of the short-term stability of the embankments were not made because the initial foundation pore pressures were unknown. No attempt was made to estimate these pore pressures. Nor was an attempt made to evaluate the short-term stability of the embankments based on a total stress analysis using undrained strength, C_u, since the initial strength of the foundation materials were unknown. However, an upper bound of short-term safety factors was determined based on the assumption that seepage lines at each approach were located initially in the lower portion of each approach foundation. Hence, in these computations excess hydrostatic pore pressures were ignored; resulting values of short-term safety factors gave some indication of the short-term stability of the embankments. Furthermore, the results also were useful in assessing benefits that might be derived from lowering the water table.

Long-term stability of the embankments was determined using pore pressures obtained from flow nets. Seepage lines at each embankment were established from the highest observed water table elevations occurring during a two-year period. In performing the long-term analyses, excess pore pressures due to foundation consolidation were assumed to have disappeared, and pore pressures corresponding to the highest observed seepage lines were used. Changes in the highest observed seepage lines were assumed to be limited to relatively small seasonal variations; that is, these seepage lines were assumed to be in a state of equilibrium.

The third factor having a great effect on the slope stability results involved a proper selection of shear strength parameters. Four different cases of shear strength were used in analyzing the eastern approach embankment; two of those cases were used in the western approach embankment analyses. The various effective stress parameters (apparent values) and the shear strength cases considered in the analyses are tabulated in Table 4. Case 1 and 2 strength parameters

Table 4. Strength Parameters and Shear Strength Cases Considered in Stability Analyses of Approach Embankments.

<table>
<thead>
<tr>
<th>SLOPE</th>
<th>SOIL</th>
<th>CASE NUMBER</th>
<th>1 PEAK (CIU)</th>
<th>2 PEAK (CDS)</th>
<th>3 RESIDUAL (CDS)</th>
<th>4 c = 0 (CIU)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>c (PSF) (DEG)</td>
<td>c (PSF) (DEG)</td>
<td>c (PSF) (DEG)</td>
<td>c (PSF) (DEG)</td>
</tr>
<tr>
<td>Embankment</td>
<td>Core</td>
<td>γ_t = 129 PCF</td>
<td>0 26.2</td>
<td>0 28.2</td>
<td>0 25.3</td>
<td>0 26.2</td>
</tr>
<tr>
<td></td>
<td>Top Foundation</td>
<td>γ_t = 129 PCF</td>
<td>267.3 24.1</td>
<td>0 28.2</td>
<td>0 25.3</td>
<td>0 24.1</td>
</tr>
<tr>
<td></td>
<td>Bot. Foundation</td>
<td>γ_t = 129 PCF</td>
<td>364.3 22</td>
<td>364.3 27</td>
<td>0 22</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>γ_t = 129 PCF</td>
<td>613.4 23</td>
<td>108 31.1</td>
<td>75 29.2</td>
<td>0 23</td>
</tr>
<tr>
<td>Eastern</td>
<td>Embankment</td>
<td>γ_t = 130 PCF</td>
<td>230 21.8</td>
<td>0 21.8</td>
<td>0 35</td>
<td></td>
</tr>
<tr>
<td>Western</td>
<td>Rock Berm</td>
<td>γ_t = 130 PCF</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

29
represent peak shear strengths observed in the CIU and CDS tests, respectively, while Case 3 parameters represent residual shear strength determined from the CDS tests. Case 4 parameters were obtained from the CIU tests, but the cohesive values determined in those tests were set equal to zero. The primary purpose of using peak and residual shear strengths was to obtain a bound of maximum and minimum values of safety factors that may exist at the site today, considering that present values of true cohesion and internal angles of friction (cf. 16) of soils in the shear zones may not necessarily be the same as those determined by tests, and also considering that portions of the slip planes may contain soils having peak strengths while other portions may consist of soils having residual strengths. Another objective was to compare safety factors obtained using Case 1 strength data with those determined from Case 2 strengths and to make comparisons of safety factors determined on the basis of Case 3 and 4 shear strengths.

Long-term safety factors of the approach embankments based on the highest observed pore pressures and the different cases of shear strength are presented in Figure 27. Critical failure circles corresponding to those safety factors are compared graphically in that figure with slope indicator data and the failure profile of the slump located in the front portion of the western approach embankment. At both embankments and for all cases of shear strength considered in the analyses, the long-term safety factors are near or below a value of 1.0, and the corresponding critical circles pass slightly under the abutments near the uppermost shear zones detected by the slope indicator. Long-term safety factors based on peak strengths (Case 1, CIU strength, and Case 2, CDS strength) of the eastern slope are slightly below 1.0 and have values of 0.95 and 0.91, respectively. Safety factors associated with Case 3 (residual strength) and Case 4 (c = 0) of the later slope were approximately the same and have values of 0.75 and 0.68, respectively. Based on peak strength (Case 1), the long-term safety factor of the western slope is 1.03; for Case 4 strength, the safety factor is 0.73. Both critical failure circles corresponding to these safety factors pass near failure points of the existing profile of the western slope.

In the series of analyses shown in Figure 28, seepage lines were lowered and assumed to be located near the rock line under each approach soil mass. The safety factors shown represent conservative estimates of the initial stability of the approach slopes. Safety factors of the eastern slope ranged from about 0.96 (peak strength) to 0.78 (residual strength); for the western slope, they ranged from 1.06 to 0.75. Results of these analyses are about the same as those shown in Figure 27; the safety factors are only slightly higher. Hence, lowering the water table has little effect on the stability of the front portions of the approach embankments.

The analyses shown in Figures 28 and 29 were performed to determine the stability of the slopes for failure circles passing near the pavement cracks and the lower shear zones detected by the slope indicators. At the western approach, the circles also passed near the indicated failure point located near the toe of the slump. Safety factors of the eastern slope based on the high water table (Figure 29) ranged from 1.27 to 0.90, and for the lower seepage line, the safety factors varied between 1.48 and 1.07 (Figure 30). Safety factors of the western slope were approximately the same for a given shear strength and for both considered positions of the water table. Using peak strength, the safety factors were 1.17 and 1.21 for the high and low water tables, respectively. Safety factors of the slope based on Case 4 shear strength were 0.86 for a high water table and 0.90 for the low water table. With regard to the deeper failure circles and a lowering of the water table, stability of the eastern slope would improve moderately; for the western slope, the stability would increase only slightly.

Remedial Stability Analysis. Three different schemes were investigated for purposes of developing a remedial solution that would improve the present stability of the approach slopes and likely prevent failure. As shown in Figure 31, benefits of using lightweight material, such as cinders, were investigated. The material was assumed to have a $\phi$ of 35$^\circ$ and a unit weight of 70 pounds per cubic foot. To determine the maximum benefits of such a plan, the entire fill located behind the abutments was replaced with the lightweight material. Circles passing slightly under the abutments were critical; safety factors of these circles were compared with circles previously obtained using the fill material. These results show the safety factors of the front portion of the slopes are increased slightly (about 0.03 for the western slope and 0.08 for the eastern slope). However, a more significant gain in stability is achieved when failure circles passing through the lower shear zones and near the pavement cracks are considered. The gain in safety factor is 0.15 for the eastern slope and 0.08 for the western slope. Peak strengths were used in all analyses shown in Figure 31.

The gain in stability, if the bridges were extended and the backfill behind the bridge abutments were excavated leaving a void space, is shown in Figure 32. For the critical circles passing slightly under the abutments, the safety factor gain is 0.12 for the eastern slope and 0.07 for the western slope. For the deeper circles, the increases are 0.07 (eastern slope) and 0.25 (western slope).
WESTERN APPROACH

EASTERN APPROACH

Case 1 (CTU Peak Strength)
Case 2 (CD5 Peak Strength)
Case 3 (CD5 Residual Strength)
Case 4 (CTU Strength, C = 0)

Figure 27. Long-term Stability Results Based on Highest Observed Seepage Lines.

Figure 28. Conservative Estimates of the Short-term Stability of Approach Slopes Assuming Position of Watertable is Near Rock line.
Figure 29. Long-term Stability Results Based on Highest Observed Seepage Lines and Failure Circles Passing through Lower Shear Zones Detected by Slope Indicators and near Pavement Cracks.

Figure 30. Stability of Approach Slopes Based on Lowered Seepage Lines and Failure Circles Passing through Lower Shear Zones Detected by Slope Indicators and near Pavement Cracks.
Figure 31. Stability Results Using Lightweight Aggregate Fill.

Figure 32. Stability Results Using Unloading and Bridge Extension Scheme.
In the third remedial scheme, several berm, slope, different shear strength situations, and water table configurations were considered as shown in Figures 33 through 35. However, construction of berms or slope changes at the site may seriously constrict the flow of Chaplin River. Furthermore, whether such berms or slope changes could withstand forces due to the large velocities of the river flow is questionable. Hence, hydraulic changes in the river channel due to such construction should be investigated. As shown in Figure 33, decreasing the slopes to 2.25 horizontal to 1.0 vertical is about the maximum slope decrease that might be tolerated without seriously constricting the channel. Based on peak strength and observed pore pressures, the safety factors are 1.18 (western slope) and 1.39 (eastern slope). Materials of the slope berms were assumed to be sound rock and to have a \( \phi \) of 35°. Comparative values based on residual strengths is 1.01 and 1.11, respectively. The uppermost circles passing through the slopes indicate the portions of the slopes would be stable. Safety factors for those circles are based on the lower shear strengths.

In Figure 35, the effect of rapid drawdown was investigated. Normal flow depth conditions were assumed to exist and a computerized solution of Manning's equation was used to obtain approximate values of flow depths. Computations were based on 50-year flood data. As shown in Figure 35, rapid drawdown lowers the safety factor from 1.18 to 0.95 (western slope) and from 1.39 to 1.05 (eastern slope). Peak strength data were used in the latter analysis. Hence, for rapid drawdown and a 50-year flood, the analysis shows the change in slopes would be inadequate and construction of the sloping berms involves a calculated risk.

From a viewpoint of soil mechanics, the berms shown in Figure 34 are more desirable than the slope changes of Figure 33. Based on peak strength, safety factors are 1.43 for the western slope and 1.41 for the eastern slope. Comparative values based on lower shear strengths are 1.21 and 1.26. If rapid drawdown is considered, comparative values based on the higher shear strengths are 1.11 and 1.24. However, the berms would largely restrict the flow of Chaplin River, and from a hydraulics standpoint may be objectionable.

CONCLUSIONS

Stability of the eastern and western bridge approach slopes is precarious. Slope indicator data and results of slope stability analyses and surface observations show that eventual failure of the approach embankments, piles, and abutments is a likely possibility. Slope indicator data show both embankments are moving and that shear zones are located in the upper and lower portions of each embankment. Movements of the eastern embankments have accelerated, indicating the yield stress of the piles supporting the eastern abutments may have been reached.

Slope stability analyses of the slopes, as constructed, show the front portions of each approach embankment have been near failure since construction. Based on peak strength data, safety factors of the front portions of the eastern and western slopes were 0.95 and 1.03, respectively. Failure of a portion of the embankment located in the front of the western abutments corroborates results of the stability computations. This failure was a result of a gradual (time-conditioned) decrease in shear strength of the approach soils. The slope failed as the shear strength dropped from peak to residual values. Safety factors of the front portions of both embankments computed on the basis of residual strength (and CIU strength with \( c \) equal to zero) ranged from 0.68 to 0.75, some 25 percent below 1.0.

The lower shear zones detected by the slope indicators, the fact that the embankments are continuing to move as recorded by the slope indicators, and the appearance of pavement cracks located several feet behind each abutment strongly indicate that progressive failure of the embankments is spreading. Safety factors determined for failure circles passing through the cracks and the lower shear zones at the eastern and western embankments were 1.27 and 1.17 (peak strength) and 0.90 and 0.86 (residual strength). Therefore, the analyses indicate the larger soil masses may eventually fail once the soils reach residual strength values. The time of failure cannot be predicted; however, the appearance of cracks indicates it may be imminent. These analyses did not consider external forces due to the piles and abutment-bridge interaction. However, it is questionable whether such forces are sufficient to prevent the larger failures.
Figure 33. Stability Results Using Slope Changes and Highest Observed Seepage Lines.

Figure 34. Stability Results Using Berns and Highest Observed Seepage Lines.
Figure 35. Stability of Slope Changes and Berms Assuming Rapid Drawdown Conditions.
Settlement of the bridge approaches and the tilting of the abutments at this site were a result of embankment instability. Analyses showed that the most critical failure circles pass slightly under the bottoms of the bridge abutments and near the toe of the embankment. Stability analyses indicate this region of the approach embankments is the most vulnerable to movement and failure. Previous studies (3, 4) have shown that approach settlement is generally confined to a distance of 100 feet behind the abutments; maximum settlement is normally located at a distance ranging from 15 to 50 feet behind the abutment -- a point on the pavement, as shown in Figure 27, where the critical failure circle tends to intersect the ground surface and a region of the pavement which receives the most maintenance. Soils located in the approach embankments and foundations at the Chaplin River site are typical of materials commonly found at many bridge sites in Kentucky. Hence, embankment instability is considered a significant factor causing settlement of bridge approaches.

Safety factors obtained using peak strength data from consolidated isotropic, undrained triaxial tests with pore pressure measurements and those determined using strength data from the consolidated-drained, direct shear tests were about the same. Safety factors associated with the direct shear test were slightly lower than those obtained when using the consolidated-undrained effective stress data. There was good agreement between safety factors obtained using residual strength from the direct shear tests and the safety factors based on the consolidated-undrained effective stress parameters with cohesion set equal to zero.

Slope instability due to progressive failure should be considered in the design of approach embankments at sites of proposed bridge construction projects to prevent or minimize settlement of approach pavements. Where approach embankments are constructed on foundations consisting of soils which exhibit a significant drop in shear strength after peak shear strength is reached, the possibility of progressive failure increases and safety factors based on peak shear strengths are unconservative. In the stability analysis of the Chaplin River slopes, a drop of about 2° to 4° between the peak and residual angles of shearing resistance produced a difference of some 20 to 25 percent in the safety factor. In particular, approach foundations containing brittle, highly plastic, or overconsolidated clays or clay shales -- soils that are more likely to exhibit significant decreases in shear strengths -- should be thoroughly investigated and tested. When such soils are encountered, consideration should be given in design to increasing the long-term factor of safety above the normally accepted value (1.5), using the residual shear strengths, or alternately, using shear strength intermediate between peak and residual strengths.

Stability of the bridge approach embankments could be improved, as indicated by the stability analysis, by the following alternate plans:

1. Decrease the slopes of each embankment to 2.25 horizontal to 1 vertical as shown in Figure 33. Channel soils located under each toe of the sloping berms and in the front of each pier should be dredged to bedrock before start of construction of the slope changes. The sloping berms should be constructed of a free-draining material (sound sandstone or limestone) having a minimum angle of shearing resistance, \( \phi \), of 35°. The lower faces of the berm should be protected by a rip-rap blanket measuring at least 2 feet thick. On the western slope, the sloping berm should be constructed to an elevation near the toe of the slump before leveling excess material at the toe of the slump. However, only small sections of that material should be leveled at a time; rock should be placed immediately in the leveled sections.

2. Construct berms to the dimensions shown in Figure 34 using materials and the construction sequence specified under Plan 1.

3. Excavate material located behind and between the abutments approximately as shown in Figure 32 and extend each end of the bridges.

4. Excavate materials behind and between the abutments as indicated in Figure 31 and backfill with lightweight material. Reconstruct the pavements.

From a viewpoint of increasing the stability of the approach embankments to an acceptable limit, construction of the berms (Plan 2) is more desirable. However, hydraulics of the channel would be appreciably affected. Decreasing the slopes of the approach embankments (Plan 1) would increase stability of the slopes to a marginal limit. If progressive failure continued, or if rapid drawdown occurred (based on a 50-year flood) after construction, then the slopes would be near a state of failure. The effect of the rapid drawdown on the stability of the sloping berms might be largely decreased by a liberal use of perforated pipe in the bottom portion of the decreased slopes. Provided rapid drawdown could be avoided, the sloping berms would provide marginal stability in the near future; but
with increasing time, the slopes would eventually approach an unstable condition. Changes in the stream hydraulics resulting from the construction of the berms should be investigated.

Unloading the top of the approach slopes (Plans 3 or 4) would not prevent eventual failure of the slopes. The front portion of each embankment would continue to move. The bridge extension (Plan 3) would increase the safety of the western slope 0.07 and 0.12 for the eastern slope. Comparable increases in the safety factors of Plan 4 were 0.03 and 0.08. However, the unloading schemes could delay embankment failures for a time and might serve as a short-term remedial solution.

A fifth alternate which was not mentioned above consists of constructing piers at the present location of the abutments, extending the ends of the bridge and excavating large portions of the approach embankments.

Although a dewatering plan involving horizontal drains had been considered as a means of increasing stability of the slopes, the analyses showed that lowering the water table would yield only nominal benefits. Moreover, because of the low permeability of the approach soils, a considerable time would be required to lower the water tables using a dewatering scheme.

IMPLEMENTATION

Both settlement and stability investigations are being conducted at sites of proposed bridge construction projects involving significant fill heights in an attempt to control bridge approach settlement. The computer program developed in 1966 and the program (Bishop's simplified method) developed as a result of this study are being used to design the approach embankments at those sites.
REFERENCES

Conceptual Plan for Buttressing Pile End-bent Abutments
Chaplin River Bridges, Bluegrass Parkway