A case study is presented of a bridge on Interstate 471 in Campbell County, Kentucky. The south approach to the bridge is a side-hill fill. During construction, the south intermediate piers tilted as a result of movement in the south approach foundation. Construction was temporarily halted on the south approach embankment, allowing pore pressures in the foundation to dissipate and slowing the movement. The piers were then reconstructed.

The south approach end bents were instrumented with earth pressure meters. The piles were instrumented with slope inclinometers. Most recent data show that movement is continuing and the present factor of safety is approximately 1.03.
ANALYSIS OF MOVEMENTS AND FORCES ON BRIDGE APPROACHES:
A CASE STUDY (BRIDGE OVER CHESAPEAKE AVENUE ON
INTERSTATE 471 IN CAMPBELL COUNTY, KENTUCKY)

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INTRODUCTION

For many years, movement of bridge abutments and the associated subsidence of roadway approaches have presented a problem for highway designers and maintenance personnel. Abutment movement and approach subsidence may be interrelated in that movement of the foundation or embankment material whether by consolidation or slippage could result in additional loading of the piles and/or abutment. Movement of the abutment (due to earth pressure) or bending of the supporting piles would result in a void to be filled by adjacent embankment material, producing additional approach subsidence. These factors eventually result in conditions requiring expensive corrective measures and maintenance. Settlements and movements of bridge approaches are widespread problems. From a survey (details to be published in a separate report), 27 states indicated major problems with bridge approaches.

Studies in New Jersey (1) and Canada (2) associate abutment movement with settlement of the foundation. In those cases, it was reported that the top of the abutment moved away from the bridge. In the New Jersey investigation, there was evidence of sufficient lateral forces to produce buckling of the piling; however, the instrumentation available precluded a complete analysis of pressure distributions. Other researchers (3, 4, 5, 6) have reported on the response of laterally loaded piles and the effects of lateral movements on bridge approaches.

In many cases studied in different parts of the world, negative skin friction was shown to be capable of inducing a significant force (7, 8). Van Weele (9) reported negative skin friction as being a common problem in Holland where there are many areas in which piles have been driven through fill on soft ground. Lambe (10) noted that tests and actual experiences in Holland emphasized the importance of including force induced by skin friction as part of the design load.

In many cases observed in Kentucky, abutments have moved toward the bridge with the bottom of the abutment moving further than the top.
Settlement of the foundation may tend to tilt the abutment away from the bridge; however, other movements in the embankment (unrelated to settlement) may eventually cause the abutment to translate toward the bridge. Slippage of the fill due to sloping bedrock (side-hill approach embankment), or simply creep of the embankment under its own weight, could eventually drag the abutment into the bridge as observed in some cases (11).

In January 1964, the Kentucky Department of Highways, Division of Research, initiated a study (11) concentrating on settlement of foundations. In conjunction with that settlement study, it was deemed desirable to investigate abutment movements and rotations, and that study was initiated in May 1972. The proposed objectives are to

1. provide an experimental analysis of lateral forces exerted upon piling used for support of a bridge end bent,
2. analyze present design procedures related to lateral loads on piles and recommend changes if necessary,
3. measure magnitude of settlements in fill and foundation,
4. measure and analyze forces exerted on the end bent and translational and rotational movements of the end bent, and
5. measure downdrag forces on piles.

This report documents the construction, instrumentation, data collection, and analysis of a particular bridge site.

SITE DESCRIPTION

Several years of literature review, instrumentation selection, and extensive searching were needed to choose an appropriate study site. Criteria for selection were a high side-hill approach embankment, deep foundation, long piles, and soils known for their poor engineering performance. A site having all of those conditions was located in northern Kentucky on Interstate 471 in Campbell County (Figure 1). The construction contract was awarded in October 1977.
At this site, Interstate 471 passes through a residential area, crossing Chesapeake Avenue and causing the relocation of Ohio Avenue (Figure 2). Approach fills reach a height of approximately 45 feet above original ground at the south approach and 60 feet at the north approach (Figure 3). The area of primary interest (south approach fill) extends from Station 228+15 to Station 232+90. Depth of foundation is irregular but is approximately 30 feet at the location of end bent Number 1. Final grade is in a shallow vertical curve in this area, but the average grade is -3.8 percent. As shown in Figure 3, the south approach fill was placed on a steeply sloping rock line.

The bridge spans 213 feet and is constructed of precast beams placed on spill-through piers and pile-supported end bents. All piling is point bearing with straight and battered placement. The bridge begins at Station 232+08 and extends to Station 234+21. The piers are located at Stations 232+89 and 233+43.

GEOLOGY

Bedrock at this site is the Ordovician Age Kope Formation of the Eden Group (12). This formation is comprised of shale and limestone, with shale accounting for 75 to 80 percent of the unit. The shale is medium gray and light bluish-gray, weathering through greenish grays to dark yellowish-orange. Beds of shale up to 6 feet thick are layered with limestone up to 12 inches thick. The Kope shale is highly susceptible to slaking and has a very low slake durability index (13). Natural moisture contents range from eight to ten percent. This indicates the shale weathers rapidly to a clay.

Generally, bedding planes are horizontal, but this site lies within the Cincinnati Dome or Cincinnati Arch. The Dome is a result of arching of the strata after deposition and consolidation; therefore, the bedding at the site has a slight dip.
SUBSURFACE EXPLORATION

The subsurface exploration was conducted in 1969. That exploration covered the entire construction project, including the area under study. In October and November of 1977, a localized subsurface exploration in the area of the south approach was conducted.

The original subsurface report indicated the presence of a man-made fill having a maximum depth of 17 feet at Station 233+60 (60 feet left of centerline) in this area. Depth to rock varies, but was reported as approximately 20 to 40 feet. Rock cores taken in that exploration indicated fracturing of the limestone and a high degree of weathering of the shale. The shale was weathered to depths of 65 feet at Station 223+50 and 55 feet at Station 224+00.

The 1977 exploration was conducted primarily between Stations 231+50 and 232+00, and extended approximately 70 feet either side of centerline. It consisted of three soil borings with continuous Shelby-tube sampling, eight Dutch Cone penetrometer locations, and two rock cores (Figure 4). Rock was encountered at 30 feet in boring Number 1 and at 21 feet in borings Number 2 and 3. Rock cores were largely weathered shale having fractured limestone layers. The top 7 feet of soil contained brick, glass, and charred wood, indicating the presence of a man-made fill. From 7.0 feet to 17.5 feet, there was little resistance to Shelbytube sampling and the recovered material was very wet.

Results of the Dutch cone penetration tests (ASTM D-3441) and standard penetration tests (ASTM D-1586) indicated the presence of a zone of low shear strength in the foundation at approximately 10 to 15 feet in depth. This zone was confirmed by moisture contents determined from Shelby-tube samples. The moisture content and Dutch Cone data are plotted versus depth in Figures 5 and 6. Figure 7 is an illustration of the zone as determined from dutch cone and standard penetration data. Information acquired from conversations with local residents indicates the past existence of a farm pond in this area. This could account for the wet zone of low strength beneath the man-made fill.
INSTRUMENTATION

Instrumentation included multipoint, mercury-filled settlement gages, horizontal and vertical slope inclinometers, settlement platforms, and earth pressure meters. The location and identification of instruments are presented in Figure 8. Information acquired by the Kentucky Department of Highways, Division of Materials from additional instrumentation is also reported.

The horizontal slope inclinometer was a system devised specifically for this type application and supplied by the Terra Technology Company of Seattle, Washington. This system is a modified version of their inclinometer, which utilizes a sensor mounted in a stainless steel torpedo. The sensor is a thermally compensated, fluid damped, and flexure-servo accelerometer sensitive to inclination away from a horizontal plane.

By use of a cable that is left in place, the torpedo is drawn through a tube exiting on either side slope. Continuous readings taken through the tube result in a profile of the tube. Positioning of the sensor relative to the tube is maintained by a wheel configuration having two fixed wheels mounted at third points around the torpedo and a third wheel (spring loaded) that rides on the top of the tube. Settlement of the foundation or fill may be determined by establishing the elevation of the projecting ends of the tube.

Placement of the system was accomplished by permitting the fill construction to progress to an elevation approximately 1 foot higher than the desired location of the inclinometer. A trench was then cut and the tube was placed in it. A rope was pulled through all sections of tube, then the tubing was joined to make one continuous tube.

In an effort to work more quickly and avoid delays in fill construction, locking O-rings were not utilized at tube unions. Neither was a select fill around the tubing specified. These factors led to partial closure of the tube, which made monitoring of the system difficult. Both horizontal slope inclinometers were destroyed by the contractor in
August 1979; however, installation of the system in other research efforts (14) has proven the system's value.

Vertical slope inclinometers were located on the end-bent piling and at various locations in the south approach embankment. Steel tubing, 1.5 inches square, was welded to the web of selected piles (Figure 9). A protective cover was welded to the leading end of the tubing and the pile was driven. To permit access to the tubing, a cavity was formed in the end bent through which the tubing projected. A steel plate was bolted over the cavity for protection (Figure 10). A total of eight piles were instrumented (four piles on each south end bent). Two battered piles and two straight piles were instrumented on each end bent (Figure 11).

Although protected at the leading end, the slope inclinometer casing suffered some damage during pile driving. Table 1 illustrates the length of each instrumented pile and the depth to which it was possible to monitor the slope inclinometer casing. Closure of the casing was probably due in part to bending of the piling during placement. The deviations of Piles 23, 26, 34, and 37 from planned positions are shown in Figures 12 through 15, respectively.

Five inclinometers were placed in the south approach embankment after completion of construction. They were located at Station 231+90 (approximately 20 feet and 70 feet left of centerline), on centerline at Station 233+00, 125 feet left of centerline at Station 233+00, and 125 feet right of centerline at Station 233+80. All casings were set in rock to stabilize the bottoms.

Thirteen Carlson earth-pressure meters were installed at the earth-pile cap interface and the earth-end-bent interface (Figures 8 and 16). Two were placed under the pile cap and five were placed on the vertical earth-end-bent interface of the southbound lanes. The remaining six were placed on the vertical earth-end-bent interface on the northbound lanes.

Four settlement platforms were placed at the earth-pile cap interface to monitor potential fill settlement away from the end bent (Figure 17). Two were placed on the northbound lanes and two on the southbound lanes.
Nine mercury-filled settlement gages were installed. One was installed at the interface between the original ground and the embankment. The remaining eight gages were installed at various locations in the embankment (Figure 18).

CONSTRUCTION

Construction began in February 1978 with some excavation in the area of the south approach and the driving of piling for the bridge piers. On May 30 and 31, Settlement Gages 1-A and 2-A were installed. Gage 1-A was installed 15 feet south of end-bent Number 1. Gage 2-A was installed 75 feet south of end-bent Number 1. Both gages extended across all driving lanes of Interstate 471 at an approximate gage elevation of 541.0 feet.

Excavation for a 72-inch structural plate pipe at Station 231+34 (see Figure 2) triggered a landslide that destroyed Gage 2-A. Gage 1-A was destroyed by a dozer and was replaced by Gages 1 and 2. Those gages were basically duplicates; therefore, Gage 2 was considered a back-up and was not monitored.

Embankment construction for the south approach began near the end of June 1978. By September 1, fill construction had progressed to an elevation permitting installation of Settlement Gages 3 through 6. Those gages were installed at an approximate elevation of 568.0 feet for Gages 3 and 4 and 570.0 feet for Gages 5 and 6 (Figure 18).

In September 1978, at an approximate elevation difference of 33.0 feet between the north and south approach embankments, movement occurred in the foundation of the south approach embankment. As a result of that movement, the recently constructed piers at Station 232+89 tilted (approximately 8 inches out of plumb) toward the south (Figure 19). However, the piers at Station 233+43 were not affected. Construction of the south approach embankment was immediately halted. Slope inclinometers installed in the south approach embankment indicated movement was in a north-to-northwest
direction. It was decided to begin construction of the north approach embankment in an attempt to stabilize the foundation and, hopefully, stop movement in the foundation. Therefore, during November 1978, 15 feet of fill was placed on the north approach embankment. At the end of November, construction on the north approach was halted for the winter. Construction of the north approach was continued in April 1979 and was essentially completed in October 1979.

Slope inclinometers indicated that movement of the south approach appeared to have stopped; therefore, construction of the south approach resumed in August 1979 and was essentially complete by mid-September. During that time, Settlement Gages 4 and 6 and the horizontal slope inclinometers were destroyed by cutting a haul road into the south approach side slope. Also, during that time, Settlement Gages 7 and 8 were installed at an elevation of 578 feet and Gage 9 was installed at an elevation of 582 feet (Figure 18).

Removal and reconstruction of the piers at Station 232+89 began in June 1980 and was completed by October of that year. Pile driving for the south end bent began in October, and construction of the end bent was completed in November 1980. During construction of the end bent, slope inclinometer casings were installed on selected piles as previously described (Figure 9). Earth pressure cells and settlement platforms also were installed in the end bent at that time (see Figure 20). The structure was essentially in place by the end of 1980 and the only significant activity remaining was drilling and placement of slope inclinometers (Numbers 9 through 13) in September and October 1981. A chronology of the construction sequence is shown in Figure 21.
SOILS DATA

Table 2 summarizes data from laboratory tests on Shelby-tube samples. Natural moisture contents (ASTM D 2216-80) were obtained on most tube samples. Atterberg limits (ASTM D 423-66 and ASTM D 424-59), and specific gravity (ASTM D 854-58) were performed on a select number of tube samples. Most soils classified as a clayey material having low plasticities (CL) using the Unified Classification System. However, the soil in the approximate area of the old farm pond classified as a silty material having low plasticity (ML) and as a mixture of clay and silt having low plasticity (CL-ML). Most of the silty materials had liquidity indices greater than 1.0, indicating a normally consolidated soil. Table 2 also shows that the samples from near the bottom of the borings had lower liquidity indices. That indicates the samples were overconsolidated (liquidity index < 0.4).

Consolidated-isotropic-undrained triaxial tests were performed on a number of Shelby-tube samples. Figure 22 shows the stress paths for the samples from the foundation in the area of the old pond. The angle of internal friction, \( \phi' \), was 26.0 degrees and the cohesion, \( c' \), was 288 pounds per square foot. Those parameters were used in the stability analysis (to be discussed later).

FIELD DATA

Settlements of the approach fill and of the end bent were monitored by multipoint settlement gages and settlement platforms. The settlement platforms were installed to monitor potential differential settlement of the end bent and underlying fill. Settlement platform data, as shown in Table 3, indicate no significant differential settlement (the small amount of change in the data is due to the precision of the method of reading).

Approach fill and foundation settlements were monitored by the multipoint mercury-filled settlement gages. Foundation settlement at points monitored by Gage 1 averaged 9.3 inches approximately 700 days after
initiation of fill construction (Figure 23). As seen in Figure 24, data obtained by the Division of Materials from a settlement platform placed in the same area as Gage 1 confirm the data from Gage 1.

Gages 3, 4, 7, and 8 were placed to monitor the fill near the south end bent (Figure 8). Gages 3 and 4 were placed at approximately the same elevation in the fill (Figure 18) and indicate similar magnitudes of settlement. At 700 days, both gages indicated an average point settlement of approximately 7 inches (Figures 25 and 26). At that time, Gage 4 was destroyed. However, Gage 3 was monitored through 1,300 days. Final settlement data on Gage 3 indicated an average point settlement of 16 inches.

Gages 7 and 8 were placed higher in the fill (Figure 18) and at approximately the same time as the destruction of Gages 4 and 6. Those gages were at the same location and should indicate similar settlements. As may be seen in Figures 27 and 28, the settlement plots differ greatly, with Gage 8 probably being erroneous. However, final average point settlements for the gages are relatively close at 14 inches for Gage 7 and 16 inches for Gage 8.

Settlement of the fill further south was monitored by Gages 5, 6, and 9 (Figure 8). Gages 5 and 6 were placed lower in the fill and Gage 9 was placed above them (Figure 18). Average point settlement on Gages 5 and 6 was slightly greater than 4 inches at 700 days (Figures 29 and 30).

At approximately 700 days from the start of the project (August 1979), Gage 6 was destroyed and Gage 9 was installed. Monitoring of Gage 5 continued through 1,800 days, with a final average point settlement of approximately 9 inches. Average point settlement of Gage 9 at 1,400 days was 5.5 inches (Figure 31).

Monitoring of slope inclinometers on the piling (Slope Inclinometers 1 through 8) began in June 1981. Data obtained from those inclinometers are difficult to interpret. This is due to several factors including lateral earth movement, earth settlement, and subsequent downward drag on the
piling. Also, as stated previously, the inclinometers could not be monitored as far as the seated pile point. That necessitated referencing all movements to the top of the piling. However, the top of the piling alos will move as the end bent moves. The initial position of the end bent was not determined by an optical survey.

Apparent movements of the instrumented pilings are shown in Figures 32 through 39. It may be noted there was movement in the order of 4 inches, with battered piling moving more than straight piling.

Slope inclinometers not on piling (Numbers 9 through 13) are seated in rock and produce more consistent data. As shown in Figures 40 through 44, most of the slope inclinometers indicate a maximum movement of approximately 1.2 inches. However, Slope Inclinometer 10 indicates a movement of 2.5 inches.

Of the 13 earth pressure meters installed, three were apparently damaged during installation or construction of the end bent. Those three are Meters 106 and 107 in the southbound lanes and Meter 101 in the northbound lanes. Earth pressure trends revealed by those meters are similar to other meters, but the initial data and pressure magnitudes would imply a possibly damaged meter. Pressure data for the northbound lanes are plotted versus time in Figure 45 and for the southbound lanes in Figure 46.

**ANALYSIS OF DATA**

**Settlement**

Figures 47 through 54 show the rates of settlement for Gages 1 through 9, respectively. It is apparent the majority of settlement occurred within a relatively short period of time (150 to 250 days). Gages 1 and 3 show an increase in rate at 700 days. That corresponds to the time that reconstruction began on the south approach fill. Figures 23 and 25 also show the settlements accelerating at those two gages at 700 days.
Figures 23 and 25 also show small amounts of heave occurring around October and November 1978, and again in April 1979 and continuing through June of that year. Those periods correspond with construction of the north approach embankment. Therefore, it appears that construction of the north approach embankment did cause some movement deep within the foundation of the south approach embankment, although that movement apparently did not affect the stability or location of the failure surface of the south approach embankment (to be discussed more fully in the section on stability analysis). This conclusion appears to be supported by the fact there were no observed rises in pore pressure in the south approach foundation during those periods (Figure 55).

Examination of the plots of the individual gages indicates the point closest to the side slope of the embankment (Point No. 2) generally settles more than the interior points. For the gages at lower elevations in the embankment (Gages 3 through 6), this phenomenon begins shortly after installation of the gage and, in general, continues through the date of the last reading. For Gages 7, 8, and 9 (higher elevations), 600 to 1,000 days elapse before this trend develops. The cause for this phenomenon appears to be slumping of the embankment under its own weight. An event similar to this was previously reported by Allen and Meade (14).

Stability Analysis

In reviewing the field data, it appears the foundation in the area of the south end bent was suspect. The underlying rock was composed of fragmented limestone layers and weathered Kope shale. This resulted in competent rock being at a lower elevation than expected. Also, the foundation material immediately below the man-made fill in the area exhibited a low shear strength and high water content. This is possibly the result of the former existence of the farm pond at the site.
Because of the conditions in the foundation, a problem arose with the placement of fill material. Construction of the fill proceeded rapidly with approximately 20 feet of fill being placed in the first 25 days. This appears to have been too fast, as indicated by the significant rise in pore pressure shown in Figure 55.

Those conditions were the probable causes leading to the slope failure. The failure tilted the piers that were in place at the toe of the south approach embankment. That necessitated removal and replacement of the piers and caused a delay of several months.

Failure of the embankment due to those conditions was substantiated by a laboratory slope stability analysis using a program developed by Hopkins (15). Using the engineering properties listed in Table 4, the analysis predicts a factor of safety of 0.95. After dissipation of the pore pressure, construction of the fill was completed with a factor of safety of 1.18. The predicted failure circle and fill heights at various times are shown in Figure 56.

At the time of failure, engineers from the Kentucky Department of Highways and the original consulting firm concluded that construction of the south approach embankment should be halted. Slope inclinometers at the site indicated movement at the toe of the embankment near the piers was in a north-northwest direction. Furthermore, it was decided (at the same time) that construction should begin on the north approach embankment, hopefully to balance the driving forces on the south embankment and stop the movement. (Figure 56 illustrates existing conditions at the time of failure.) However, it should be noted that the north piers (Station 233+43) had not moved -- an indication that the movement did not extend beyond Chesapeake Avenue. The theoretical failure surface from the stability analysis (shown in Figure 56) appears to verify quite well the observed movements in the field. The failure surface intersects the groundline just north of the toe of the south piers and does not cross Chesapeake Avenue. Therefore, the conclusion is that construction of the
north approach embankment did not stop the movement of the south approach embankment. However, after construction of the south embankment was halted, pore pressures in the foundation dropped rapidly (Figure 55), undoubtedly increasing the factor of safety, and the movement decreased significantly.

However, the most recent data indicate continuing movement of the south approach fill, with a current factor of safety of 1.03. The question of future stability of the embankment becomes one of significance. One of the more important factors influencing the situation is the elevation of the water table in the embankment.

Experience has shown that the water table in side-hill embankments saturates increasingly larger portions of the embankment over an extended time. Data obtained at the site indicate the water table fluctuates somewhat with intensity of precipitation, but is generally rising (Figure 57). From the slope stability analysis, a projected water table elevation of 570 feet would yield a factor of safety of 1.00 (Figure 58). Unless remedial action is taken, the water table elevation may approach 570 feet. If that occurs, the slope could fail again, possibly within two to five years (assuming the water table continues to rise at the present rate).

Slope Inclinometers and Earth Pressure Meters

Slope inclinometer data at the site are of two types. Slope inclinometers installed on piling (Slope Inclinometers 1 through 8) were in place soon after fill construction was complete. Data obtained by those instruments will primarily reflect the condition and changes in the piling. Slope inclinometers drilled into the fill and foundation were placed approximately one year later and reflect movements of the fill and foundation materials.

As previously stated, interpretation of data from piling inclinometers is difficult. Due to factors previously mentioned in the section entitled
Field Data, the pile movement may only be considered relative to a point on top of the piling itself. However, that point is probably moving also. Therefore, the true magnitude of movement is difficult to determine.

Data obtained from Slope Inclinometers 9 through 13 reveal movement of the entire fill and foundation in the area of the south end bent. Inclinometers 9, 11, and 13 are moving in a north-northwest direction (see Figure 8 for location of inclinometers). Inclinometer 9 shows greatest movement in the top 10 feet (Figure 40). This would locate the movement in the top portion of the soft layer previously discussed. Also, that corresponds closely to the critical failure surface, as determined from the stability analysis. Inclinometer 13 was installed at approximately the same elevation as Inclinometer 9 and is located in the northwest toe of the south approach embankment. From 23 feet deep to the top of the inclinometer, there is a dramatic increase in movement (Figure 44). From 7 feet to 23 feet, the movement appears to be in the soft foundation layer. However, from 7 feet to the top of the inclinometer, the movement is probably due to slumping of the toe of the embankment, as that portion of the inclinometer was installed through the toe of the embankment.

Inclinometer 11 was installed at the top of the embankment slope and south of end bent No. 1 on the southbound lanes. Figure 42 shows a zone of movement from 57 feet to 60 feet. Again, movement appears to be occurring in the soft foundation layer. From 8 feet to 20 feet, there is a second zone of movement. That portion of the embankment is moving almost due west. It would appear movement is due to slumping of the embankment — a conclusion substantiated by the previously discussed settlement data.

Inclinometer 10 is located immediately south of the end bent of the southbound lanes (near the median). At the 60-foot depth, there is a distinct failure plane (Figure 41). The direction of movement at that depth is generally northwest. Movement is also in the soft foundation layer. From 20 feet to the surface, a second zone of movement may be noted. The direction of movement is to the northeast. That indicates the
embankment is attempting to "spill through" the space between the two end bents.

All inclinometers on the piling (Inclinometers 1 through 8) indicate two general movements. There appears to be soil movement in the top 15 feet to 20 feet. Movement is generally north-northwest, along the centerline of the roadway. The movement is from an apparent slumping of the soil behind the end bent and a continuing rotation of that soil down and under the end bent with a consequent bulging on the front face of the embankment. That phenomenon has been observed in other cases, as reported by Hopkins (16). Also, the piling inclinometers show a definite northwesterly movement in the foundation, as indicated by inclinometers installed in the fill. The greatest movement appears to be 2.5 inches at Slope Inclinometer 10, but as seen in Figure 59, all slope inclinometers reveal a continuing movement.

When summarizing the slope inclinometer data, a probable explanation of events emerges. Again, it appears there is a general foundation movement in the northwesterly direction. The end bent and its supporting piling presents some resistance to movement, thereby resulting in some degree of "spill through" between the end bents of the northbound and southbound lanes (as indicated by Inclinometer 10).

As a result of earth movement, two things appear to be happening to the end bent and its supporting piling. There is bending or sliding of the piling that tends to drag the end bent down and rotate it about a horizontal axis (Figure 60). Then, because of the "spill through", the end bent tends to rotate around a vertical axis (Figure 61).

The hypothetical rotation about a horizontal axis is supported by earth pressure data. If the end bent is rotating as illustrated in Figure 60, the earth pressure would probably be the greatest at the top of the vertical earth-abutment interface (Figure 62) and on the bottom face of the pad (Figure 46). If the end bent were not being rotated or dragged downward, the piling would support the end bent and earth pressures on the bottom face would tend to be very low.
CONCLUSIONS

The south approach embankment apparently failed because of rapid pore pressure build-up in the foundation. Failure occurred in the soft layer in the foundation.

Halting construction on the south approach embankment allowed pore pressures in the foundation to dissipate, which helped to stabilize the embankment. Construction of the north approach embankment apparently did little to stabilize the south embankment.

Installing slope inclinometers on piling is an effective means of determining initial alignment of driven piles, as well as movement of the piles with time.

Two of the four instrumented piles on the end bent of the south approach on the southbound lanes are badly misaligned. This could indicate the problem may be more prevalent than realized.

Settlement rate data show that most of primary consolidation occurred within 150 to 250 days.

The embankment appears to be slumping or creeping under its own weight.

The end bent on the south approach of the southbound lanes appears to be rotating about both a horizontal axis and a vertical axis.

The foundation soils are still moving in a north-northwest direction.

The present factor of safety for the south approach embankment is 1.03. Because of continuing movement in the foundation and a continuing rise in the elevation of the water table in the embankment, it is expected the south approach embankment may fail again in the future.
REFERENCES


12. Gibbons, A. B.: Geologic Map of Parts of Newport and Withamsville Quadrangles, GQ-1072, Department of the


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<td>S-1B</td>
<td>2.5-5.0</td>
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<td>17.1</td>
<td>27.0</td>
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<tr>
<td>S-2B</td>
<td>5.0-7.5</td>
<td>16.5</td>
<td>22.2</td>
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<td>ML</td>
<td>1.84</td>
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| HOLE 2 - SURFACE ELEVATION 541.04 ft |
| S-1A   | 2.5-5.0      | 48.6     | 29.5| 19.2| 10.3| 2.69| CL     | 0.48|
| S-1B   | 2.5-5.0      | 18.3     | 26.4| 22.2| 4.2 | 2.67| ML     | 1.33|
| S-2A   | 5.0-7.5      | 25.8     | 47.3| 25.2| 22.1| 2.74| CL     | 0.31|

Table 2. SUMMARY OF LABORATORY TEST DATA
<table>
<thead>
<tr>
<th>HOLE 3 - SURFACE ELEVATION 541.00 ft</th>
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<tbody>
<tr>
<td>S-1A 2.5-5.0</td>
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<td>S-1B 2.5-5.0</td>
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<tr>
<td>S-2A 5.0-7.5</td>
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<tr>
<td>S-2B 5.0-7.5</td>
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<td>S-2C 5.0-7.5</td>
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<tr>
<td>S-4D 10.0-12.5</td>
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<td>S-7A 17.5-20.0</td>
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<td>S-8A 20.0-21.0</td>
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<td>S-8B 20.0-21.0</td>
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<p>| S-5B 12.5-15.0 | 12.5-15.0 | 30.5 |
| S-5C 12.5-15.0 | 25.7      | 23.4 | 19.9 | 3.5 | 2.71 | ML | 1.66 |
| S-5D 12.5-15.0 | 27.1      | 33.2 | 21.4 | 11.8| 2.75 | CL | -0.40 |
| S-6A 15.0-17.5 | 30.5      | 25.2 |
| S-6B 15.0-17.5 | 21.1      | 37.3 | 21.7 | 15.6| 2.76 | CL | -0.04 |
| S-6C 15.0-17.5 | 37.4      | 41.8 | 22.7 | 19.1| 2.76 | CL | -0.05 |
| S-6D 15.0-17.5 | 39.2      | 41.8 | 22.7 | 19.1| 2.76 | CL | -0.05 |
| S-7A 17.5-20.0 | 30.5      | 35.9 | 19.0 | 16.9| 2.76 | CL | 0.42 |
| S-7B 17.5-20.0 | 41.1      | 22.0 | 19.1 | 2.77| CL  | -0.02 |
| S-7C 17.5-20.0 | 39.2      | 22.0 | 17.2 | 2.75| CL  | 0.44 |
| S-7D 17.5-20.0 | 39.7      | 22.5 | 17.2 | 2.72| CL  | 0.08 |
| S-7E 17.5-20.0 | 39.2      | 22.9 | 16.3 | 2.76| CL  | -0.03 |
| S-7F 17.5-20.0 | 39.2      | 23.0 | 16.7 | 2.74| CL  | -0.05 |
| S-8A 20.0-21.0 | 36.7      | 22.3 | 14.4 | 2.77| CL  | -0.40 |</p>
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<th>PLATFORM 1</th>
<th>PLATFORM 2</th>
<th>PLATFORM 3</th>
<th>PLATFORM 4</th>
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Table 4. Summary of Stability Analysis

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<tr>
<th>Case Number</th>
<th>Approach</th>
<th>Effective Stress Parameters</th>
<th>Pore Pressures (Foundation Only)</th>
<th>Pore Pressure Ratio (Foundation)</th>
<th>Coordinates** of Critical Circle</th>
<th>Minimum Factor of Safety</th>
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<td>Foundation</td>
<td>Head</td>
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<td>o c' (deg) (pcf)</td>
<td>P1 Head (ft)</td>
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<td>780. 1.03</td>
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</table>

Note: Tip of Pneumatic Piezometer No. 1 = 44.2 ft
Tip of Pneumatic Piezometer No. 2 = 37.2 ft

* Pore Pressures from observed water table Dec 1984 in slope inclinometers.

** For Coordinate Grid refer to Figure 56.
Figure 1. Study Site – Interstate 471 Bridge over Chesapeake Avenue in Campbell County, Kentucky.
Figure 2. Plan View of Study Site.
Figure 3. Centerline Profile of Study Site.
Figure 4. Location of Subsurface Testing and Sampling Performed by Research Team.
FIGURE 5. MOISTURE CONTENT OF FOUNDATION BORINGS.

FIGURE 6. DUTCH CONE PENETRATION -- FOUNDATION (AVERAGE OF EIGHT TESTS).
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Figure 10. Cross Section of End Bent Showing Typical Piling and Slope Inclinometer Placement.
Figure 11. Location of Instrumented Piling in End Bent Number 1.
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Figure 13. Initial position of Slope Inclinometer Number 2 (Pile Number 36).
Figure 14. Initial Position of Slope Inclinometer Number 3 (Pile Number 26).
Figure 15. Initial Position of Slope Inclinometer Number 4 (Pile Number 23).
Figure 16. Cross-section of End Bent with Location of Earth Pressure Meters.
Figure 17. Cross-section of End Bent with Typical Settlement Platform Placement.
Figure 18. Cross-section of South Bridge Approach with Settlement Gage Location.
Figure 19. Piers at the Toe of the South Approach Fill after Embankment Failure.
Figure 20. Placement of Earth Pressure Meters and Settlement Platforms during Forming the South End Bent.
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FIGURE 24. FILL HEIGHT AND SETTLEMENT VERSUS TIME.
FIGURE 26. SETTLEMENT (GAGE 4) VERSUS TIME.
FIGURE 27. SETTLEMENT (GAGE 7) VERSUS TIME.
Figure 30. Settlement (gage 6) versus time.
Figure 32. Movement versus depth as of 8-16-84 (slope inclinometer II).
FIGURE 33. MOVEMENT VERSUS DEPTH AS OF 10-20-81 (SLOPE INCLINOMETER 2).
FIGURE 34. MOVEMENT VERSUS DEPTH AS OF 8-16-84 (SLOPE INCLINOMETER 31).
FIGURE 35. MOVEMENT VERSUS DEPTH AS OF 8-16-84
(SLOPE INCLINOMETER 4).
FIGURE 35. MOVEMENT VERSUS DEPTH AS OF 8-16-84
(SLOPE INCLINOMETER 5).
FIGURE 37.  MOVEMENT VERSUS DEPTH AS OF 8-16-84
(SLOPE INCLINOMETER 6).
FIGURE 38. MOVEMENT VERSUS DEPTH AS OF 8-16-84 (SLOPE INCLINOMETER 7).
FIGURE 39. MOVEMENT VERSUS DEPTH AS OF 6-2-83
(SLOPE INCLINOMETER 8).
FIGURE 40. MOVEMENT VERSUS DEPTH (SLOPE INCLINOMETER 9).
RESULTANT MOVEMENT (INCHES)

DEPTH (FEET)

0.0 0.5 1.0 1.5 2.0 2.5

0 5 10 15 20

08-15-84
05-31-83
05-26-82
01-21-82
10-30-81
10-25-81
10-20-81

FIGURE 41. MOVEMENT VERSUS DEPTH (SLOPE INCLINOMETER 10).
FIGURE 42. MOVEMENT VERSUS DEPTH (SLOPE INCLINOMETER III).
Figure 43. Movement versus depth (slope inclinometer 12).
FIGURE 44. MOVEMENT VERSUS DEPTH ISLOPE INCLINOMETER 131.
FIGURE 45. EARTH PRESSURE ON END BENT FOR NORTHBOUND LANE.
FIGURE 46. EARTH PRESSURE ON END BENT FOR SOUTHBOUND LANE.
FIGURE 47. RATE OF SETTLEMENT (CAGE 1 - POINTS AVERAGED).
FIGURE 48. RATE OF SETTLEMENT (GAGE 3 - POINTS AVERAGED).
FIGURE 49. RATE OF SETTLEMENT (GAGE 4 - POINTS AVERAGED).
FIGURE 50. RATE OF SETTLEMENT (GAGE 5 - POINTS AVERAGED).
Figure 51. Rate of settlement (gage 5 - points averaged).
Figure 52. Rate of settlement (gage 7 - points averaged).
FIGURE 53. RATE OF SETTLEMENT (GAGE 8 - POINTS AVERAGED).
Figure 54. Rate of settlement (Cage 9 - Points Averaged).
Figure 55. Fill elevation and piezometric head versus time.
Figure 58. Profile of slope stability analysis section.
FIGURE 57. WATER TABLE ELEVATION VERSUS TIME.
FIGURE 59. MAXIMUM SLOPE INCLINOMETER MOVEMENT VERSUS TIME.
FIGURE 61. ROTATION OF END BENT ABOUT A HYPOTHETICAL VERTICAL AXIS.