# Evaluation of the Canoe Creek Bridge Abutments

*Effective repair of bridges experiencing structural distress requires a well-planned evaluation of the bridge based on site characteristics.*

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IN THE SUMMER of 1984, a geotechnical engineering consultant was asked by the Pennsylvania Department of Transportation (PennDOT), District 10, and its bridge consultant to participate in the evaluation of the Canoe Creek Bridges' apparent structural distress.[1] The bridges consisted of two similar, 833-foot long, six-span structures that carry the eastbound and westbound lanes of Interstate Highway 80 (I-80) across Canoe Creek (see Figures 1, 2 and 3). The bridges are located in Clarion County, Pennsylvania, approximately one mile east of Exit 7 on I-80 near the town of Knox. The first five spans on the west end of each bridge were supported by continuous steel girders, whereas the sixth span on the east end consisted of simply supported steel beams. All four bridge abutments, both Pier 5 supports and the left column of Pier 4 of the eastbound highway, were founded on H-piles bearing on rock. The remaining piers were supported on spread footings bearing on rock.

Prior to the geotechnical investigation, the bridge consultant had been involved in designing a remedial maintenance plan consisting of replacing the bridge decks, repairing erosional damage, bridge painting, *etc.* Upon thorough inspection, however, it was noted that both bridges had experienced, and were continuing to experience, significant structural distress. Backhoe excavations adjacent to the Pier 5 foundations revealed severe pile cap cracking (see Figure 4). The wingwalls of the two west abutments were severely cracked, and all of the bridge expansion dams were completely closed (see Figure 5). Both bridge Pier 5s exhibited considerable tilt (see Figure 6).

On the assumption that the observed distress was soils related, a geotechnical and engineering study plan was developed. This evaluation plan consisted of subsurface drilling and testing, site observations, laboratory soil testing, determination of the extent of distress, evaluation of alternative remedial measures, and structural and geotechnical recommendations.

## Subsurface Borings

The subsurface drilling program was conducted in two phases. A total of 16 boreholes were drilled at the locations shown in Figure 1. Twelve borings were drilled during the first phase — six borings along the eastbound highway and six borings along the westbound

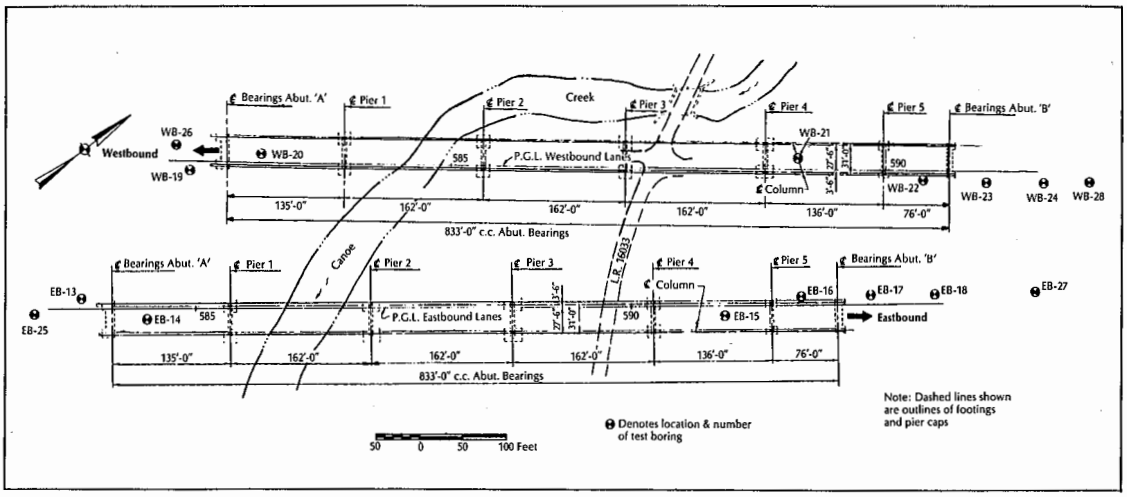


**FIGURE 1. Plan drawing of the original Canoe Creek Bridge design.**

highway. These Phase I borings were drilled to obtain geotechnical data to aid in determining the causes of the structural distress experienced by the dual bridges. Two borings were drilled at the west ends and four borings at the east ends of both the eastbound (Borings EB-13 through EB-18) and westbound (Borings WB-19 through WB-24) highways. Borehole depths ranged from 38 to 108 feet.

The purpose of the Phase II subsurface exploratory program was to help determine whether bridge abutments could be founded on spread footings. Four borings were drilled during Phase II (Borings EB-25 and 27, and WB-26 and 28). One boring was drilled at the approximate location of each of the four abutments. Phase II borings ranged in depth from 74 to 121 feet.

The borehole conditions encountered in Phase II were similar to those encountered in Phase I. The predominant soil type was a stiff to hard brown to gray silty clay with varying amounts of shale and sandstone rock fragments, coal and, to a lesser extent, sand. The second and third most predominant soil types consisted of shale and sandstone rock fragments, respectively, that had undergone varying amounts of weathering. The relative density of these materials was generally medium to very dense. Fill materials were encountered in the top portions of all 16 borings. Fill thicknesses ranged from approximately 12 to 74 feet. Underlying the fill materials were natural soils, ranging in thickness from approximately 3 to 17 feet. Because the fill materials were obtained from nearby borrow sources, they were very similar in composition to the *in situ* soils, making determination of the contact between the two strata difficult in some of the borings.

The rock underlying the natural soils consisted of shales, claystones, mudstones, siltstones, sandstones and some coal. Rock was cored in all of the borings except WB-28, which was terminated at the apparent top of rock due to the icy road conditions that made work conditions hazardous for the drill crew and inspector. Borehole coring lengths ranged from 10 to 20 feet. The rock surface was generally found to be in a considerably weathered condition. To illustrate the generally weak rock condition, the drill rig was able to auger through more than 20 feet of sandstone in Boring EB-25. In most of the borings, the condition of the rock improved with depth. Soft zones and clay intrusions were encountered at random locations in many of the boreholes.

Most of the soil samples recovered were in a moist condition. A considerably smaller number of samples were either dry or wet. Pockets of perched water were encountered in several of the boreholes at various elevations. The groundwater table was located below the fill. During the coring, water was usually lost through rock fractures and

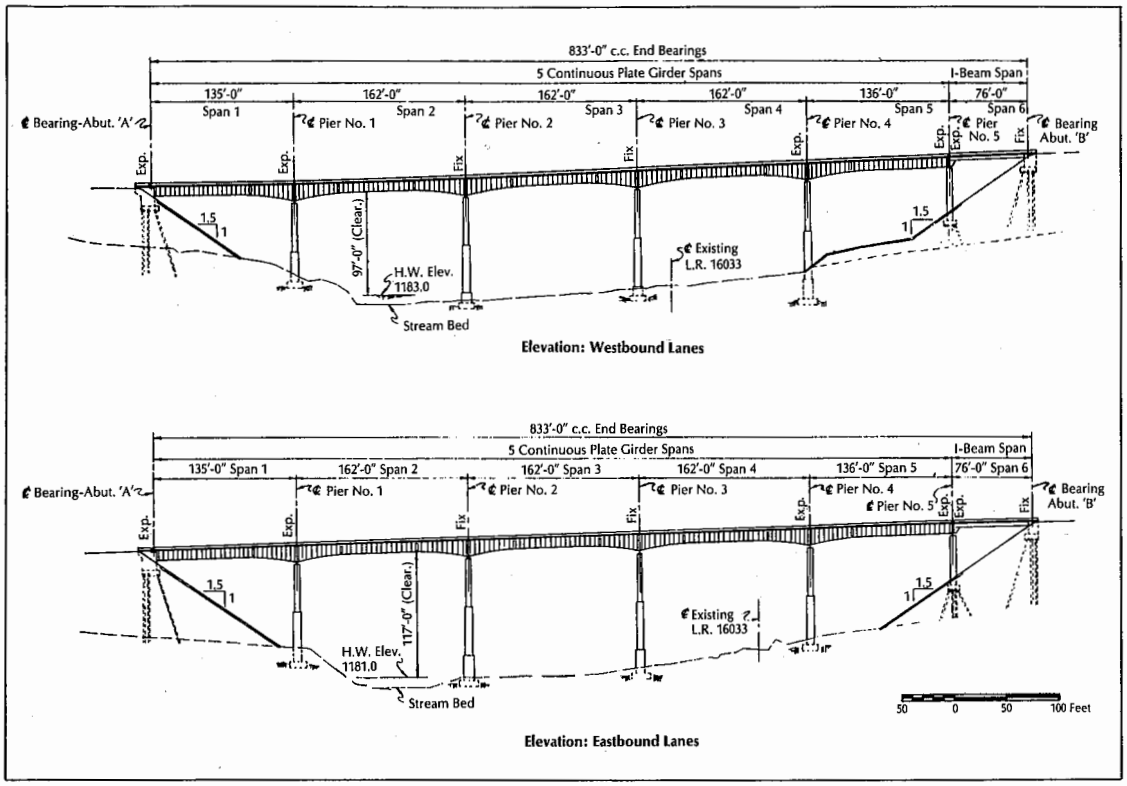


**FIGURE 2. Cross-section of the original Canoe Creek Bridge design.**

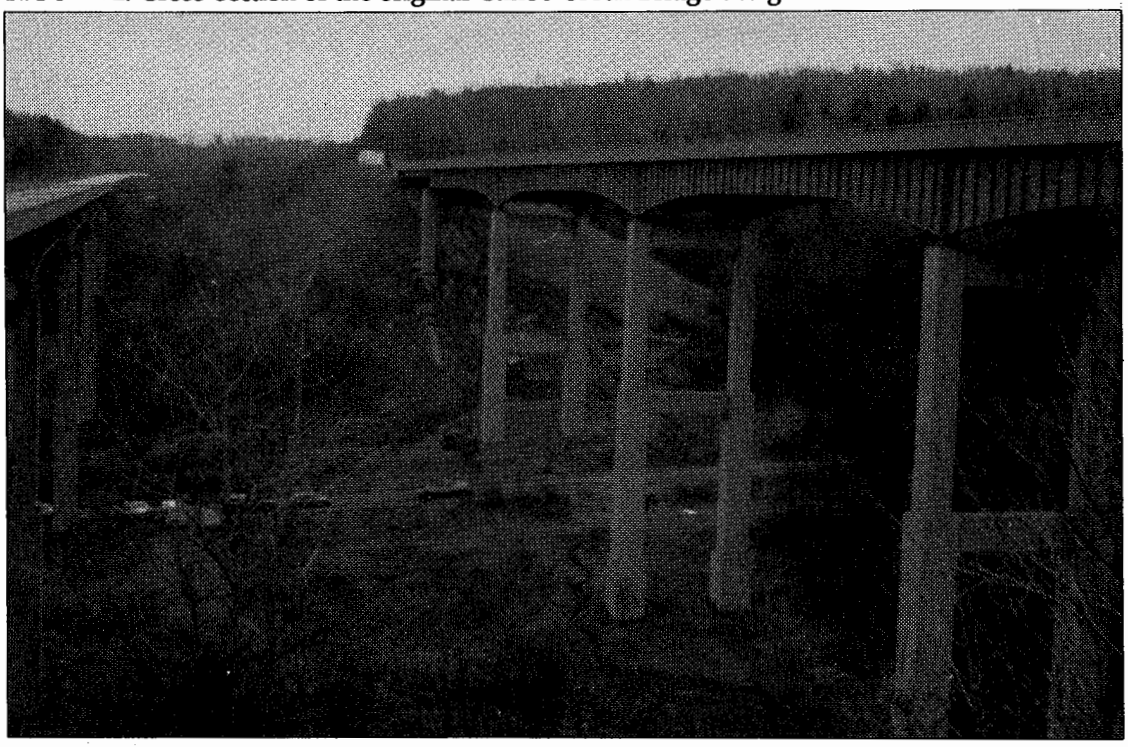

**FIGURE 3. Photo of the Canoe Creek Bridges.**

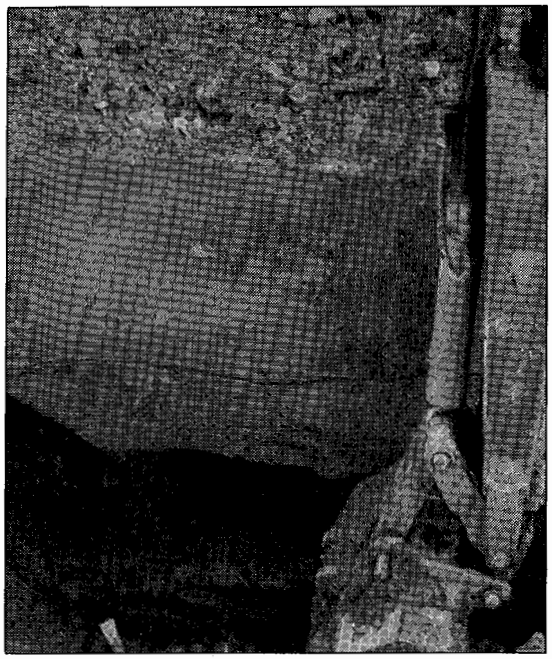

FIGURE 4. Cracked Pier 5 pile cap.

generally did not return to the surface. After the completion of boring, the water level was observed to be dropping in all boreholes.

## Site Observations

During the course of the drilling program, a site reconnaissance was conducted to augment the information obtained from the drilling program.

Certain sections of the two bridges appeared to have undergone considerable movement. This movement was evident from observations made at several locations. The rocker bearings at the two west abutments were in a severely tilted position (see Figures 7 and 8), indicating that the abutments had moved at least several inches towards the creek. The wingwalls of these two abutments had undergone considerable cracking. The two Pier 5s had tilted to such an extent that the tilts were noticeable to the eye. Because simply-supported spans were pinned to the east abutments, relative movements between these two structural components were not possible. The extent of lateral abutment movement was noticeable at both Pier 5s, however, where it was evident that the simply-supported beams had been pushed horizontally several inches toward the creek (see Figures 9 and 10).

There were numerous indications that the fill (originally placed at a slope of 1.5 horizontal:1 vertical) was in a state of movement. Slides were visually noted at many locations, varying from small localized sloughs to massive, although relatively shallow, failures. The largest such slide was noted at the south side of the west abutment embankment on the eastbound highway. In this area, virtually the entire face of the fill from the bridge to the embankment corner had slid away. The depth of the slide was approxi-

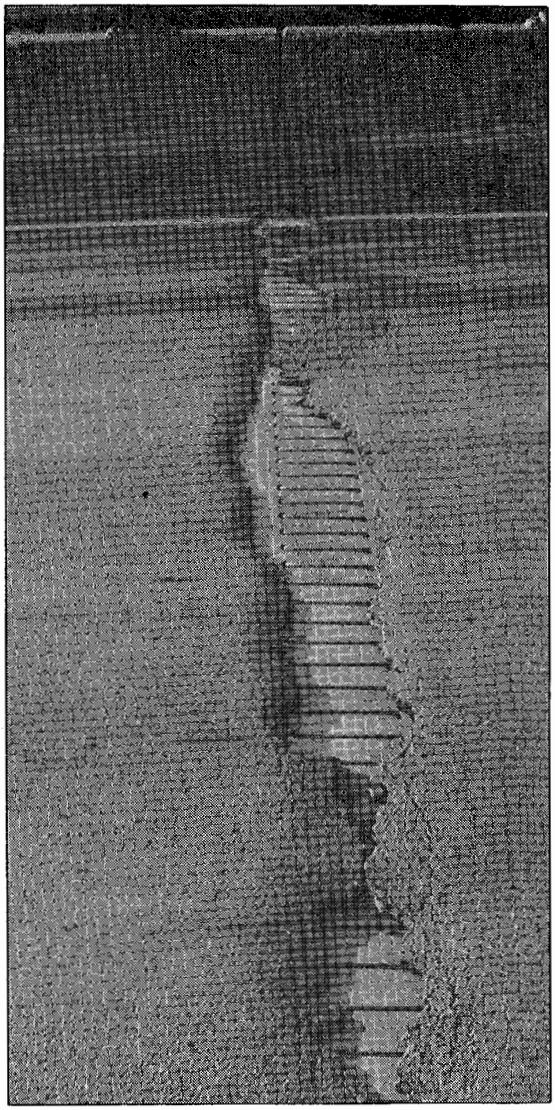

FIGURE 5. Closed expansion dam.

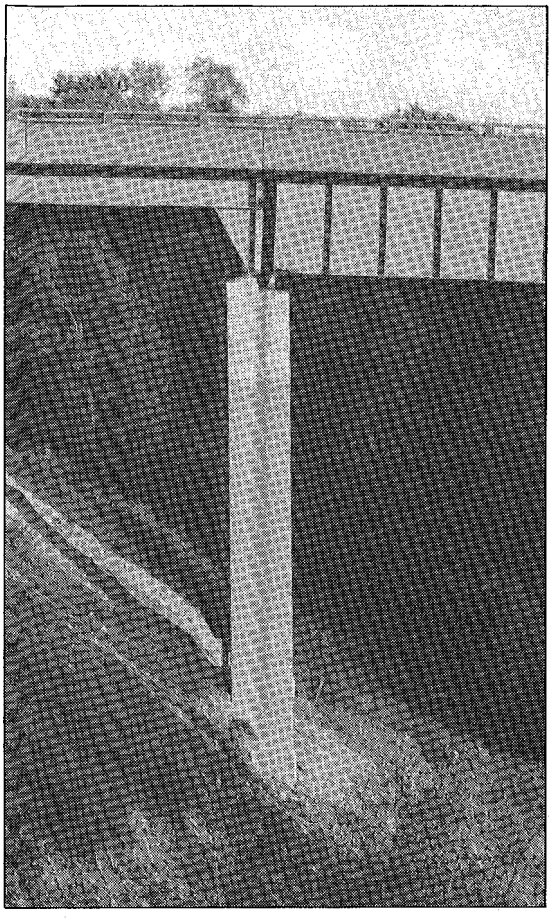

FIGURE 6. A view of the eastbound bridge's Pier 5.

mately 10 feet (see Figures 11 and 12). At some locations, the fence that had been installed near the toe of the fill was completely covered by slide material. Other areas where slides were noted included: the north side of the west abutment embankment below the westbound highway; the south side of the east abutment embankment below the eastbound highway; an area between the two bridges at the east side; an area north of the westbound highway on the east side; and areas adjacent to the two Pier 5s where some backhoe exploratory work recently had been conducted. The only area that appeared to be relatively stable occurred between the two highways on the west side, where the embankment was constructed at a slope of approximately 1.7:1.

The vegetative cover was lost in the areas

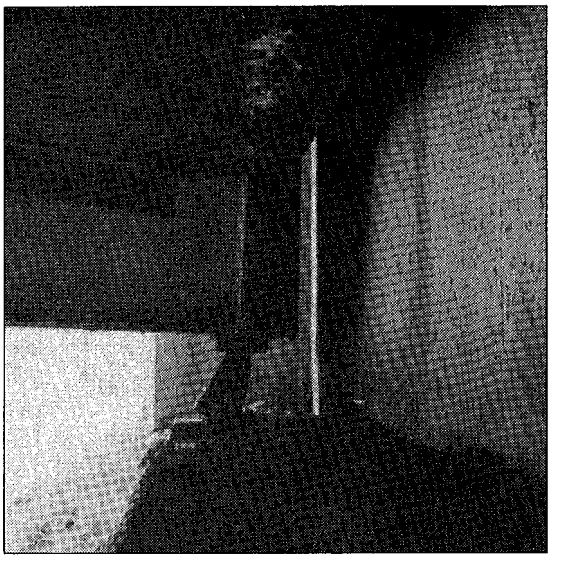

FIGURE 7. Eastbound bridge rocker expansion bearings at its west abutment.

where sloughing had occurred, leaving large portions of the embankment uncovered. Where vegetation still remained, the trunks of many trees exhibited significant curvature that was caused by the trees' attempts to remain vertical as their roots slowly crept down the embankment with the unstable surficial soil

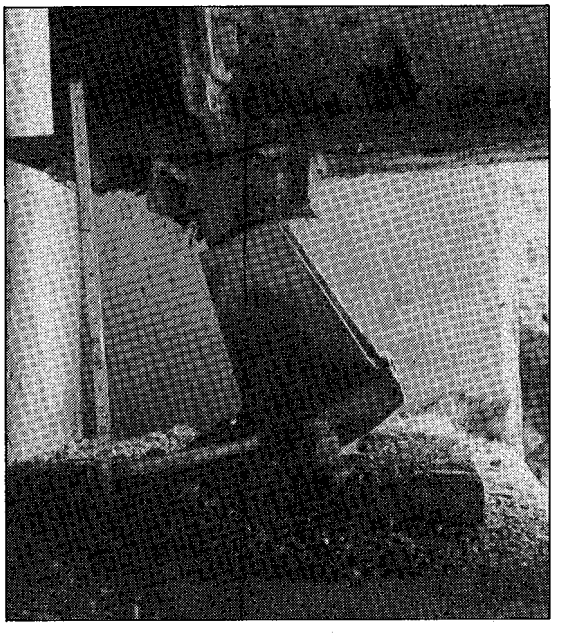

FIGURE 8. Eastbound bridge rocker expansion bearings located at its west abutment.

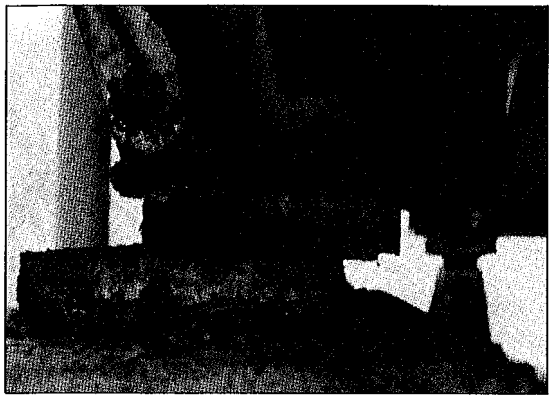

**FIGURE 9. Westbound bridge sliding expansion bearing at Pier 5.**

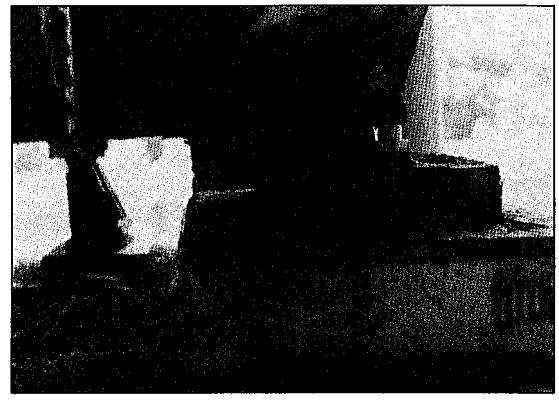

**FIGURE 10. Eastbound bridge sliding expansion bearing at Pier 5.**

(see Figures 13 and 14).

The embankment side slopes, which were at a slope of 2:1, showed no evidence of sloughing at any location although several small erosion scars were noted.

The section of the east side embankment located north of the westbound highway had undergone considerable erosional damage. This damage may have been caused, at least partially, by the underdrains at the abutment that allowed water to run down the face of the embankment in this area. The erosional damage at this location appeared to have contributed to the surface sloughing.

The highway pavements on all four sides of the two bridges had experienced severe cracking transverse to the roadway. The cracks extended from the abutments to locations ranging from 110 feet at the eastbound highway's west embankment to 395 feet at

**FIGURE 11. Slide scarp at the west side embankment of the eastbound highway.**

**FIGURE 12. Slide scarp at the west side of the eastbound highway.**

the eastbound highway's east embankment. These cracks indicated that the fill materials had undergone considerable sliding, creep and/or settlement. No longitudinal cracking was noted, indicating that the 2:1 side slopes were in a stable condition.

## Field Testing

Standard penetration tests (SPTs) were conducted in all of the boreholes at varying intervals. These data were used to assist in determining the consistency of the fill and the

**FIGURE 13. A curved tree trunk due to slope instability.**

**FIGURE 14. A tree recently toppled by slope instability.**

*in situ* soils. Whenever the recovered SPT samples were cohesive, pocket penetrometer readings were performed on them to estimate their unconfined strengths. Pressuremeter testing was conducted in three of the four boreholes drilled for investigation of the abutment foundation conditions. Rock quality designation (RQD) values were determined for all of the rock cores that were recovered. RQD values ranged from 0 to 75 percent and averaged 25 percent, indicating that the surface of the rock strata was relatively weathered. Results of the SPT and pocket penetrometer testing, as well as RQD values, were summarized on the boring logs, an example of which is shown in Figure 15.

## Laboratory Soil Testing

A number of laboratory tests were conducted to help ascertain the physical properties of the fill and the *in situ* soils.

*Triaxial Testing.* A three-point consolidated undrained triaxial test with pore pressure measurements ($\overline{CU}$) was performed on soil obtained from Boring EB-18, resulting in an effective friction angle of 27.7 degrees and an effective cohesive strength of 460 pounds per square foot (psf). As was typical for all of the undisturbed samples collected during the subsurface drilling program, the samples on which the triaxial testing was conducted contained significant amounts of rock fragments. Although the samples were predominantly composed of silty clay, the geotechnical consultant felt that the rock caused the test results to yield strengths higher than the clay matrix material itself. Based on this assumption, use of these strengths in a stability analysis would lead to unconservative results.

*Unconfined Compression.* Four samples were tested in unconfined compression. The test results yielded unconfined strengths ranging from 1,400 to 4,300 psf.

*Modified Proctor Compaction Tests.* Two bag samples were collected of the surface fill materials. Bag Sample 1 was obtained from the fill in an area located north of the westbound highway near the west abutment, approximately one-half of the distance between the toe and crest. Bag Sample 2 was obtained at a point located near the toe of the fill between the eastbound and westbound highways at the east side of the fill. Depths for both samples ranged from 0 to 1.0 foot. The Modified Proctor compaction test (ASTM D 1557) results were: a maximum dry density of 116.3 pounds per cubic foot (pcf) and an optimum water content of 11.3 percent for Bag Sample 1, and 116.5 pcf and 11.0 percent for Bag Sample 2.

*Specific Gravity.* Testing was conducted on the two surface bag samples, as well as the triaxial test sample. Specific gravity values of 2.57 and 2.64 were obtained for Bag Samples 1 and 2, respectively. A value of 2.65 was obtained for the triaxial test sample. The presence of coal fragments is believed to account for the relatively low values.

*Consolidation Tests.* Three consolidation tests were performed on samples obtained from borings drilled for the investigation of spread footing conditions at the location of proposed abutments. Test results revealed that the samples were overconsolidated and exhibited relatively low compressibility. The overconsolidation ratio appeared to decrease with sample depth as would be expected in a compacted fill.

*Water Content Testing.* Water content tests were performed on approximately one-half of the samples collected. Water content results, which were plotted on the boring logs, generally fell in the 5 to 20 percent range.

*In-Place Density Testing.* In-place density values were obtained for a number of the undisturbed samples collected in the drilling program. Dry density values ranged from 110 to 123 pcf. Wet densities ranged from 131 to 138 pcf. The tested samples generally contained a somewhat higher proportion of rock fragments than the samples on which compaction tests were performed. This factor is presumed to explain why the *in situ* densities often exceeded the maximum Modified Proctor density values. These results suggested that the fill materials were in a relatively well-compacted state.

## Cause of Structural Distress

Data collected by the PennDOT and its bridge consultant indicated that the continuous span sections, as well as Piers 1 through 4, for both

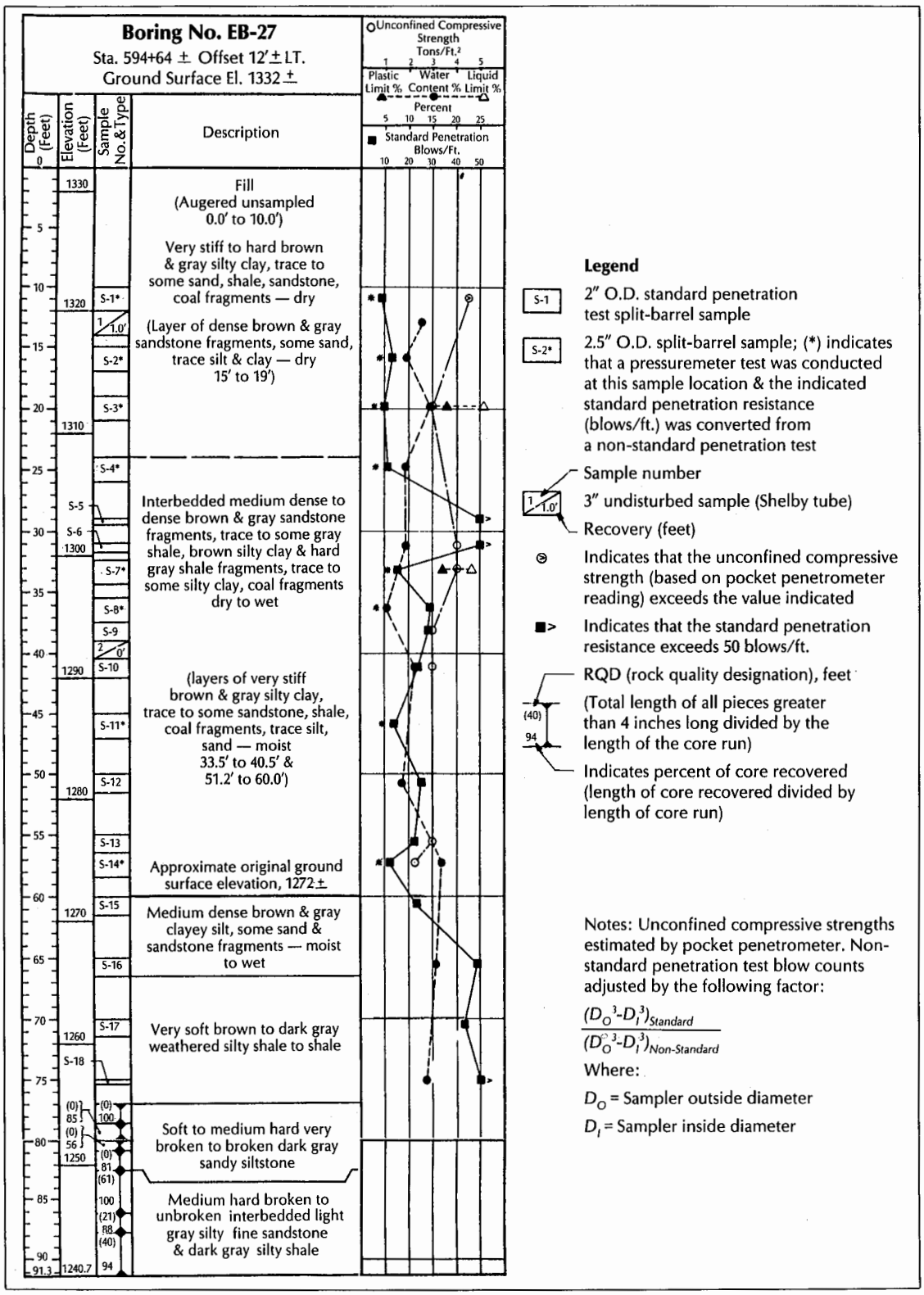


**FIGURE 15. A typical boring log from the Canoe Creek Bridge evaluation.**

eastbound and westbound bridges had not been displaced laterally to any significant degree. These bridge elements appeared to be in approximately the same position as when they were originally constructed. The structural components that had experienced distress included the four abutments and the two Pier 5s of each bridge. The bottoms of the westbound and eastbound Pier 5s had displaced laterally 4 inches and 5.5 inches, respectively. The top of each pier appeared to retain, approximately, its original construction position. The four abutments also appeared to have moved towards the creek by a significant amount, as evidenced by the closed expansion dams. A review of historical correspondence revealed that the distress had been repaired at least one time in the past, shortly after construction of the bridge. After repair, continued movement was still noted. Total lateral movements at each abutment since initial construction may have been on the order of 1 foot. There was no evidence that measurements had been undertaken to evaluate vertical movements.

The bridge was founded on 24 individual structural elements, consisting of the 4 abutments and 5 piers each with two columns for each of the two bridges. Of these 24 elements, 8 had experienced measurable lateral movements and 16 remained in their approximate original positions. The 8 elements that had experienced distress consisted of the 4 abutments and the 4 columns that comprised the two bridge Pier 5s. All 8 of these structural components shared the common characteristic of being founded in the steeply sloping fill materials. The foundations of all of the 16 stable elements were located beyond the fill sections. This commonality suggested that the cause of the structural distress was related to the stability of the fill, as opposed to other candidate causes such as construction errors, frost heave or corrosion. In addition, the directions of observed displacement were consistent with the belief that the distress was caused by slope instability.

Observations made during the field investigation supported this conclusion. Numerous instances of surface sloughing had been noted. Slope indicator data recorded by PennDOT over several years revealed that two types of movements were occurring. Up to a depth of ten feet, movements averaging up to one inch per year were recorded. Beneath this depth the rate of movement decreased by approximately an order of magnitude. The shape of the deflection curves indicated that slip circle type failures were occurring at shallow depths. At greater depths, the deflections appeared to be related to creep type movements and showed no evidence of any shear planes. This collection of information supported the contention that the existing embankment stability could have been no better than marginal. Therefore, review of the evaluation program data concluded that the structural distress was caused by movement of the fill materials in a downhill direction. This fill movement had caused the abutments, as well as the bottoms of Pier 5s, to move in the same direction. Based on these observations, the stability of the embankments had to be improved to remove the cause of the structural distress.

Another characteristic that the 8 distressed elements shared was that each was founded on H-piles. However, an examination of the pile driving records, and the fact that at least one other column (the left column of Pier 4 for the eastbound highway) had also been placed on piles and showed no evidence of significant movement, suggested that the piles had probably been installed properly and were not the cause of the observed distress.

## Remedial Scheme Recommendations

The search for an optimal remedial scheme concentrated on six alternative measures:

1. Use anchor tiebacks to support abutment and Pier 5 foundations.
2. Underpin the abutment and Pier 5 foundations.
3. Excavate fill to flatten the slope and extend the bridges.
4. Flatten the slope by adding fill.
5. Use vertical retaining structures in conjunction with adding fill to flatten the slope.
6. Flatten the slope by adding fill to

**TABLE 1**

**Stability Analyses Results**

| $\bar{c}$ = 0 psf | | $\bar{c}$ = 200 psf | | $\bar{c}$ = 500 psf | | $\bar{c}$ = 1000 psf | |
|---|---|---|---|---|---|---|---|
| $\bar{\phi}$ | FS | $\bar{\phi}$ | FS. | $\bar{\phi}$ | FS | $\bar{\phi}$ | FS |
| 31° | 0.95 | 24° | 0.97 | 17° | 0.96 | 10° | 0.98 |
| 35° | 1.06 | 26° | 1.05 | 20° | 1.07 | 12° | 1.07 |
| | | 28° | 1.12 | 28° | 1.39 | | |

**TABLE 2**

**Effective Cohesive Strengths & Friction Angles**

| $\bar{c}$ | $\bar{\phi}$ |
|---|---|
| 0 psf | 32.5° |
| 200 psf | 24.6° |
| 500 psf | 17.9° |
| 1000 psf | 10.2° |

the toe areas while excavating soil from the top portions of the embankments.

Alternatives 1 and 2 had been suggested during the early phases of the project. At that time, the cause of the structural distress was still not known and it was considered possible that improperly designed foundations could have caused the distress. When the investigation had progressed sufficiently to conclude that the unstable fill was the source of the problem, it was concluded that neither foundation underpinning nor the use of tiebacks could halt additional movements. Alternatives 4 and 6 also proved to be infeasible due to topographic constraints, right-of-way restrictions, the requirement that Canoe Creek would have to be relocated, and the fact that the required additional fill would have imposed additional loads on Piers 1 and 4.

Only Alternatives 3 and 5 appeared feasible. Preliminary cost estimates were prepared for both remedial schemes and presented to PennDOT. Because the cost of implementing Alternative 5 was significantly higher than that of Alternative 3, PennDOT decided to eliminate Alternative 5 from further consideration.

Based on the conclusion that the embankments were in a state of failure, it was assumed that the factor of safety of the embankments was less than 1.00. However, because the rate of movement was very slow, it was assumed that the factor of safety (FS) was very near 1.00. Therefore, the estimated factor of safety for a model fill cross-section was taken to be 0.99.

Four values of effective cohesion, $\bar{c}$, covering the range of probable actual strength, were selected for use in stability analysis. The selected effective cohesive strength values were 0, 200, 500 and 1,000 psf. Several stability analyses were performed for each cohesion value while varying the effective friction angle, $\bar{\phi}$. The stability analyses yielded the results shown in Table 1.

The results of the stability analyses were plotted on an effective friction angle versus factor of safety graph, as shown in the top portion of Figure 16. Individual values of effective friction angles that would yield a factor of safety of 0.99 for each of the four effective cohesive strengths were obtained by linear interpolation as shown in Figure 16. The interpolation procedure yielded the combination of effective cohesive strengths and friction angles as shown in Table 2, each of which resulted in a factor of safety of 0.99 for the model embankment.

These four sets of calculated strength values were plotted and a curve was drawn through the data points as shown in the lower graph of Figure 16. This curve represented all the possible combinations of effective cohesive strength and friction angle that would produce a factor of safety of 0.99 for the model embankment configuration. Since the factor of safety of the four highway abutment embankments was previously estimated to be approximately 0.99, it was, therefore, postulated that the actual overall average strength parameters for the embankments would fall very near or on this curve.

An effective angle of internal friction of 25 degrees and an effective cohesive strength of 200 psf were selected as being representative

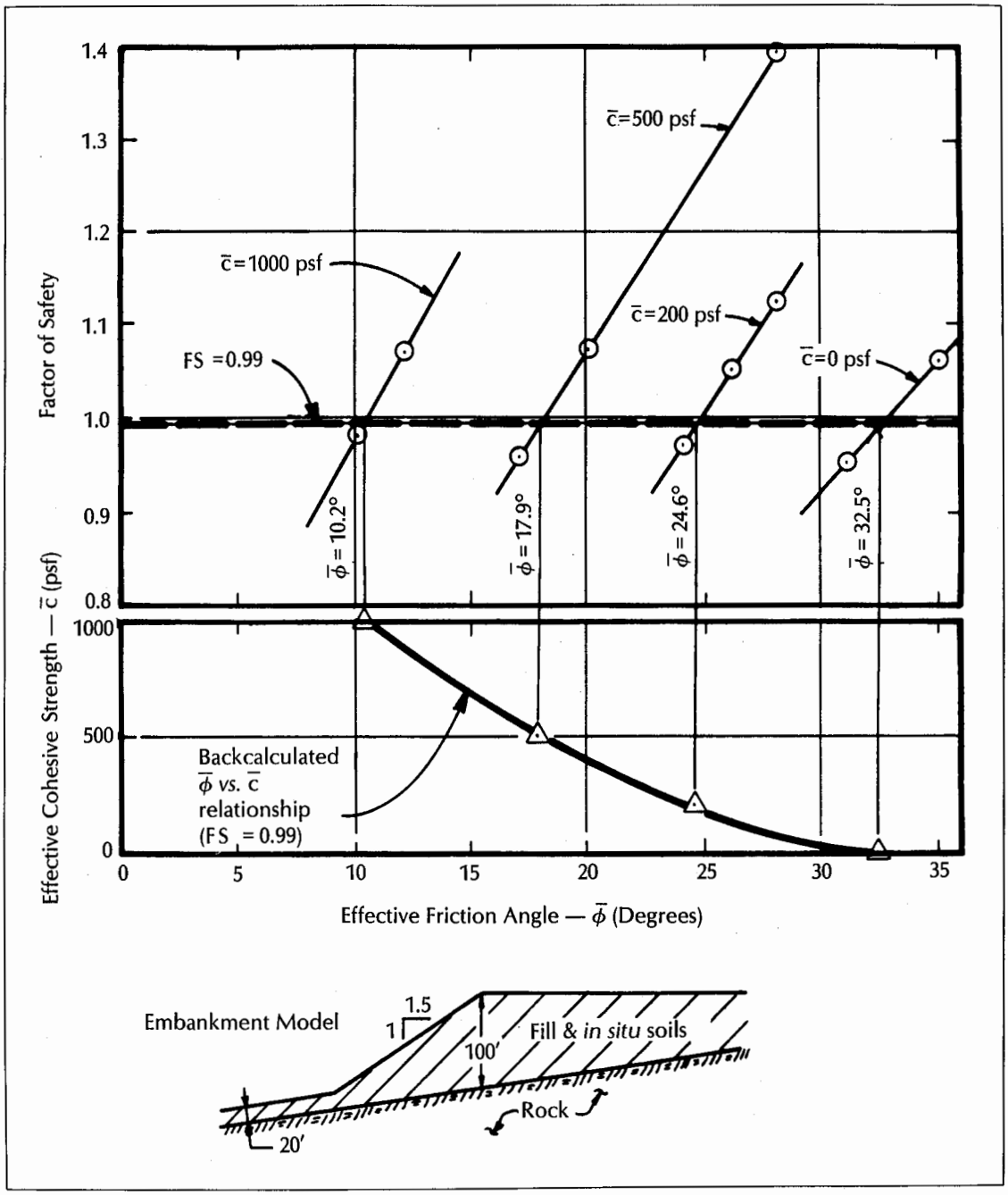


**FIGURE 16. Stability analysis for the bridges.**

of field conditions. Based on these strength parameters, factors of safety were obtained for regraded embankment configurations as shown in Table 3. Because of the vital significance of Highway I-80 as a transportation corridor, PennDOT deemed it prudent to select the 2.5:1 slope.

To implement slope regrading and replace the damaged bridge components, the following bridge modifications were required (see Figure 17):

- Replace all four abutments
- Replace Pier 5 for both bridges

- Remove the steel beams that compromised the sixth span on the east end of each bridge because they had deteriorated to a point that warranted their replacement
- Extend the bridges by adding the following simply-supported spans: one 90-foot span for the west end of both bridges; two 118-foot spans for the east end of the westbound structure; and three 100-foot spans for the east end of the eastbound structure
- Add Piers 1-A and 6 for both bridges, and also a Pier 7 for the eastbound structure to support the new spans

## Abutment Foundation Recommendations

PennDOT requested that the feasibility of founding the four abutments on spread footings be investigated in order to decrease the overall construction costs. PennDOT bridges are normally supported by piles because of the generally held belief that they provide better support. This procedure is not unique to Pennsylvania. A recent survey conducted by the Federal Highway Administration (FHWA) found that 42 states require bridge superstructures to be either founded on rock or supported by piles.[2] Other studies have discovered that this requirement may lead to unnecessarily expensive designs without achieving the desired effect. Observations of the vertical and horizontal movements of previously constructed bridges provided evidence that on the average, the more expensive pile-supported structures performed no better than structures supported on spread footings.[3,4]

To determine the appropriate geotechnical parameters for the spread footing design, Borings EB-25, WB-26, EB-27, and WB-28 were drilled to determine the type of materials

**TABLE 3**

**Factors of Safety for Regraded Embankment Configurations**

| Regraded Configuration | Factor of Safety |
|---|---|
| 2:1 slope | 1.3 |
| 2:1 slope with 20-foot bench | 1.4 |
| 2.5:1 slope | 1.5 |

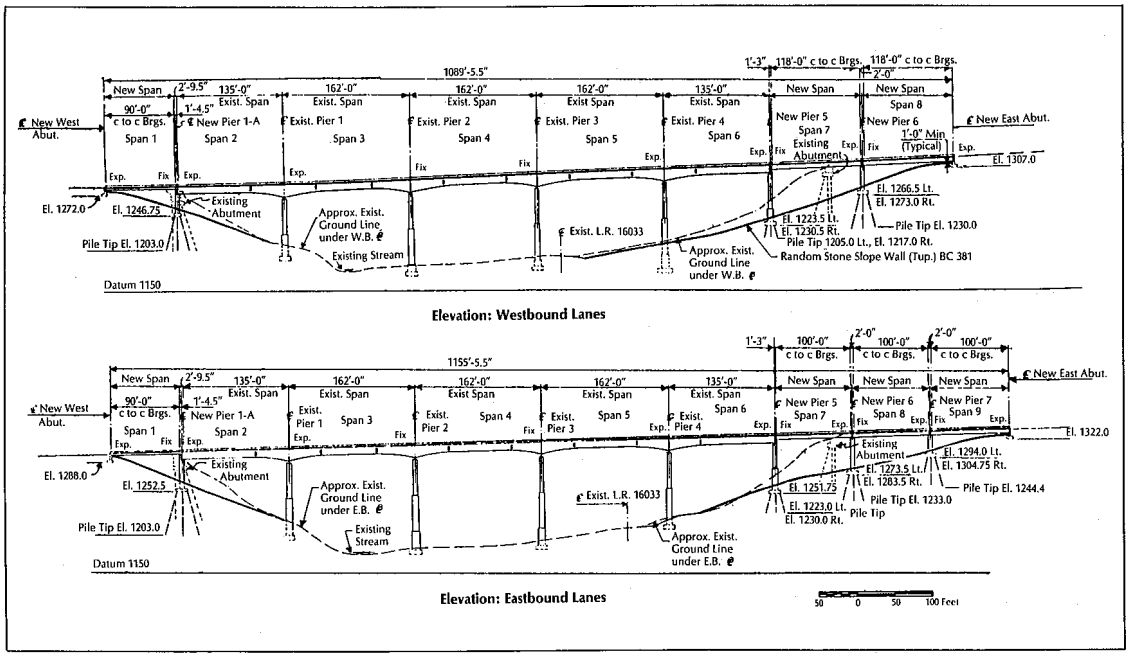


**FIGURE 17. The Canoe Creek Bridge cross-sections depicting redesigned configurations.**

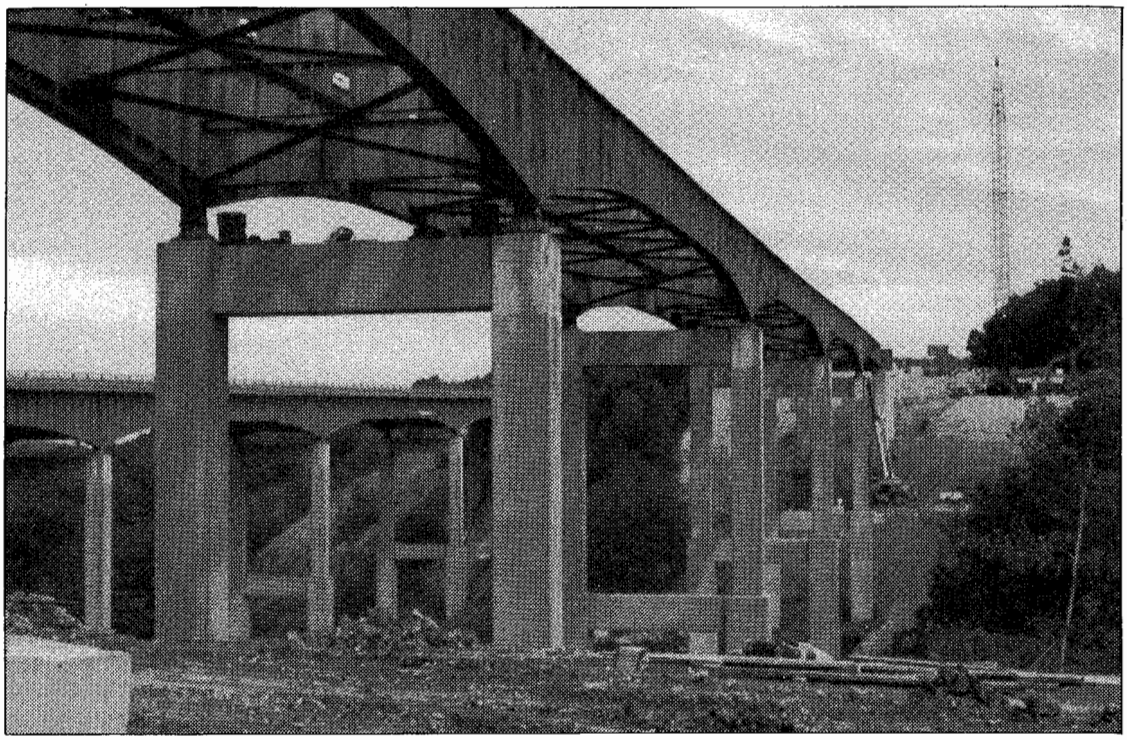

FIGURE 18. A view of the eastbound bridge after deck removal.

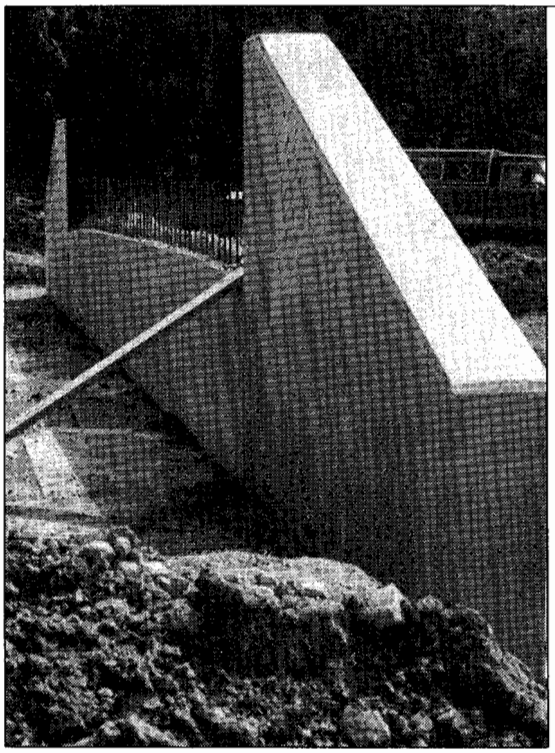

FIGURE 19. Newly constructed eastbound highway east end abutment and wing walls.

present in the embankment beneath the locations of the abutments (see Figure 1 for locations). Because of the difficulty in obtaining undisturbed samples caused by the rocky nature of the fill, and because of the questionable validity of using SPT settlement prediction methods for the existing fill materials, pressuremeter testing was conducted in three of these four borings to facilitate the estimation of probable settlements. Boring WB-26 was not tested with the pressuremeter due to logistical constraints.

Review of the pressuremeter data revealed that the settlement of the abutment foundations would be limited to approximately two inches. The previously mentioned bridge performance studies indicated that movements of this magnitude would not be detrimental to the Canoe Creek Bridges. The analyses were developed assuming footing pressures would not exceed 3,000 psf. Use of this allowable bearing pressure resulted in footing widths varying from 11 to 15 feet for the four abutments. A footing length of 37 feet and 10 inches was selected for each of the

FIGURE 20. A view of the reconstructed Pier 5.

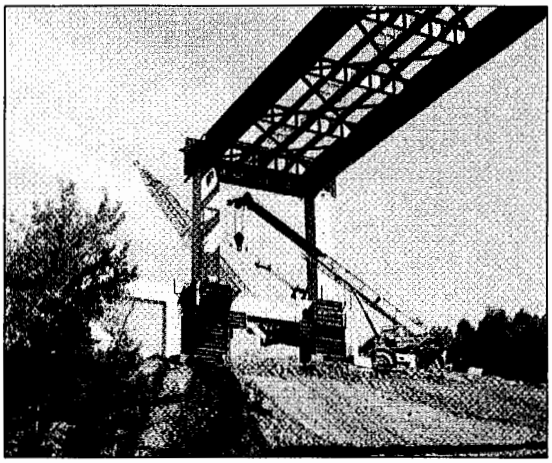

FIGURE 21. Reconstructed Pier 5.

abutments, based on the width of the highway.

In addition to the pressuremeter method of settlement analysis, settlements were also estimated using consolidation test results and blow count data. Analysis of the consolidation test data from undisturbed samples yielded a settlement estimate of approximately three inches, or approximately 50 percent more than that obtained using the pressuremeter method. However, this approach contained an obvious bias because the samples tested for consolidation were not truly representative of the embankment materials. Collection of undisturbed samples was possible only in the softer and more compressible portions of the fill. Therefore, samples could not be obtained from the very granular, very dense, stiff to hard, as well as rocky zones encountered during the drilling program. Accordingly, use of consolidation parameters for this particular situation yielded overly conservative estimates of anticipated settlement. If one were to make a reasonable assumption that the materials tested for consolidation were 50 percent more compressible than the typical embankment material, use of consolidation data would have

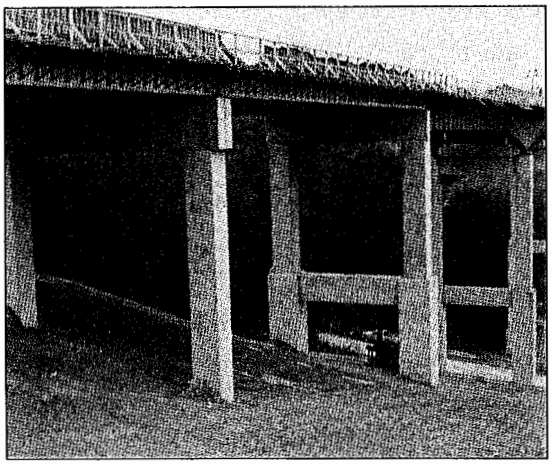

FIGURE 22. Recently added Pier 6, reconstructed Pier 5 and original Pier 4 (shown left to right). Note the new, simply supported prestressed concrete box beams and regraded slope.

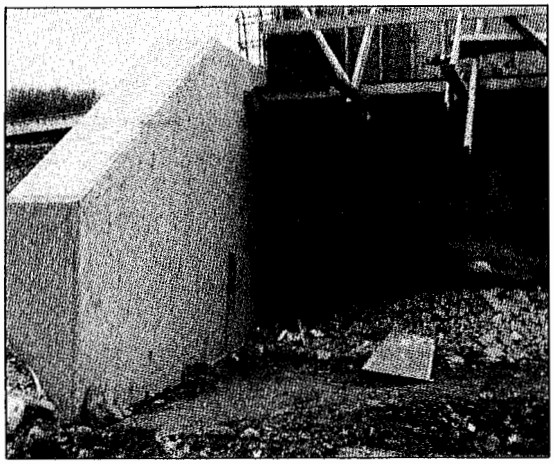

FIGURE 23. Newly constructed eastbound highway east end abutment and new box beams.

resulted in an estimated settlement of two inches.

Use of the blow count data also resulted in maximum estimated settlements of approximately two inches. It should be noted that the materials encountered in the fill were not well suited for use of the settlement-blow count relationship criteria that better suits estimates of elastic compression. The presence of significant amounts of gravel and cobble-sized particles added to the questionable validity of this technique. Use of this method alone was not sufficient for confidently predicting settlements at this site. Although the accuracy of this method may be somewhat limited in this case, it lent additional credibility to the results obtained using the other two techniques.

The stability of the embankments was re-evaluated to determine the impact of the fill-supported abutment loads on the factor of safety against embankment instability. These analyses indicated that the effect of the additional loading to be relatively minor, reducing the factor safety by only 0.02.

## Cost Savings

In evaluating the cost savings to be achieved with the selection of the spread footing foundations, it was noted that concrete costs for the two alternatives would have been similar. The reduction in cost would therefore accrue from the elimination of the piles.

Selection of an H-pile system, HP 12 x 53 section, would have required the purchase and installation of approximately 4,500 feet of piling. By eliminating the piles, construction costs were lowered by approximately $90,000, assuming a pile cost of $20.00/foot.

## Current Project Status

As of the spring of 1987, reconstruction of the Canoe Creek Bridges has been completed. Figures 18 through 23 depict recent construction activities. A monitoring program measuring settlement of the eastbound bridge abutments has been implemented. To date, the east and west abutments have settled 0.36 and 0.24 inches, respectively. The abutment foundations are performing well, and it appears that total settlements will be within the 2-inch design criteria.

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