# Landslide Stabilization at Missouri Route K Bridge over Blackwater River

Scott M. Olson
*URS Corporation, St. Louis, Missouri*

Alan Miller
*Missouri Department of Transportation, Jefferson City, Missouri*

Kenneth M. Berry
*URS Corporation, St. Louis, Missouri*

Jay Bestgen
*Missouri Department of Transportation, Jefferson City, Missouri*

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**Scott M. Olson**
URS Corporation
St. Louis, Missouri-USA-63110

**Alan Miller**
Missouri Department of Transportation
Jefferson City, Missouri-USA-65102

**Kenneth M. Berry**
URS Corporation
St. Louis, Missouri-USA-63110

**Jay Bestgen**
Missouri Department of Transportation
Jefferson City, Missouri-USA-65102

**ABSTRACT**

In May 2002, a landslide on the south bank of the Blackwater River damaged the Missouri Route K bridge that crosses it. A flood on the river triggered the landslide. Based on the field investigation and stability back-analysis, it appeared that the landslide actually consisted of two separate slides – a shallow slide triggered by rapid drawdown of the river and a deep slide triggered by artesian water pressures in a subsurface gravel layer. A rock berm that was keyed into the gravel was constructed to stabilize the slope. The rock toe berm was designed to resist both the shallow and deep slide by providing weight to the slope to prevent a rapid drawdown failure and providing a drainage outlet to relieve artesian pressures in the gravel layer.

## INTRODUCTION

The Missouri Route K bridge over the Blackwater River is a five-span steel girder structure near the town of Blackwater, Missouri (see Fig. 1). The bridge was built in the late 1960's and the bridge piers are founded on H-piles driven to bedrock or footings on bedrock. Figure 2 presents a design cross-section of the bridge.

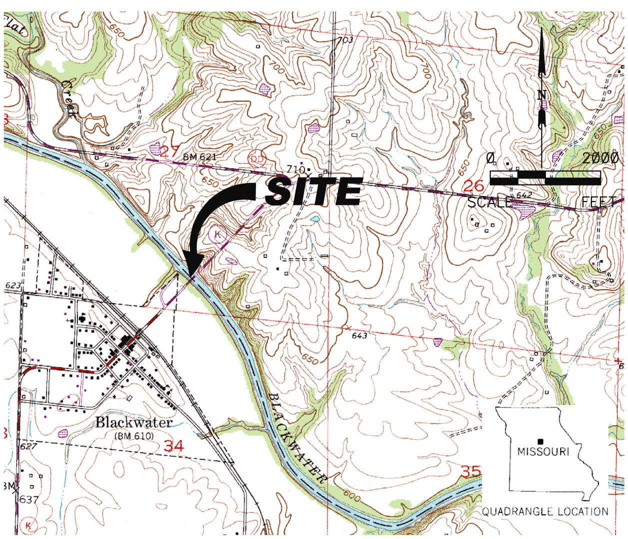


*Fig. 1. Site Vicinity Map.*

In May 2002 during a flood event on the Blackwater River, a landslide occurred on the south riverbank. The landslide caused bridge Pier 4 to move approximately 12 to 15 inches horizontally toward the river and 1 to 1.5 inches vertically upward. The pier movement caused the bridge girders to drop off the rocker bearings. The damage caused by the slide resulted in the Missouri Department of Transportation (MoDOT) closing the bridge to vehicular traffic.

This paper describes the landslide event; summarizes the landslide investigation and back-analysis; presents the various stabilization options that were considered; details the selected stabilization option; and describes the landslide and bridge repair.

## LANDSLIDE EVENT AND INVESTIGATION

In late May 2002, local residents of Blackwater and the surrounding area noticed a sag in the Route K bridge deck and railing. The residents alerted MoDOT. On May 29, 2002, MoDOT emergency personnel observed that the rockers supporting the main span girders had "rolled over" at all the bearings at Pier 4 of the bridge (see Fig. 3). MoDOT personnel suspected that a landslide had occurred and closed the bridge to vehicular traffic. MoDOT retained URS Corporation to investigate the landslide and recommend stabilization options.

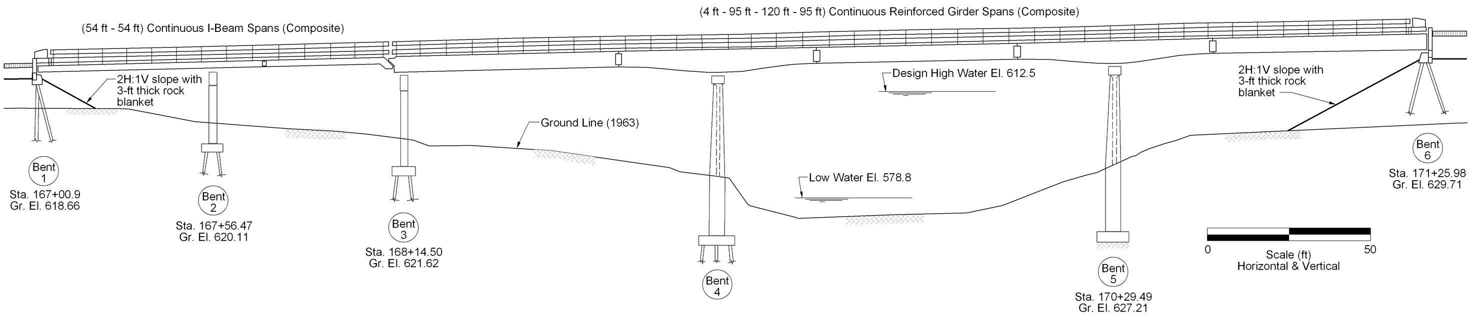


*Fig. 2. General Elevation of Missouri Route K Bridge over Blackwater River (Elevations in feet, NVGD).*

<u>Field Reconnaissance</u>

After the floodwaters receded, a site survey indicated that Pier 4 moved about 12 to 15 inches laterally toward the river and about 1 to 1.5 inches vertically upward. No other piers showed movement.

On June 7, 2002, we observed a number of scarps striking roughly parallel to the river extending from the riverbank to approximately 10 feet north of Pier 3, as shown in Figs. 4 and 5. Lateral movements at the scarps were about 1 to 2 inches and some showed vertical displacement of up to 12 inches. By July 1, 2002 many of the scarps were more visible as a result of dry weather. We observed another major scarp (originally obscured by vegetation) that was located approximately 40 feet north of Pier 3 and extended approximately 300 ft to the west to a concrete boat ramp (see Fig. 5). The scarp appeared to be pre-existing and showed lateral and vertical displacements of about 3 inches and 3 ft, respectively.

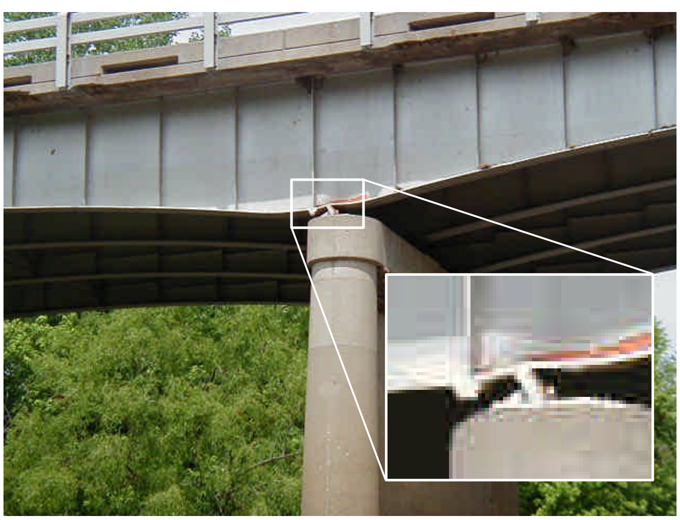

*Fig. 3. Damage to Rockers at Pier 4.*

<u>Field Investigation</u>

In June 2002, MoDOT drilled two borings through the slide mass and into bedrock. The boring locations are shown in Fig. 5. In one boring, MoDOT installed an open standpipe piezometer sealed into a gravel layer overlying bedrock. In the other boring, MoDOT installed an inclinometer socketed into bedrock. MoDOT personnel observed water flowing freely to the tops of the instrument casings (located 2 to 3 ft above existing grade).

In early July 2002, URS drilled four borings at the site as shown in Fig. 5. URS installed inclinometers (socketed into bedrock) in two borings to evaluate the extent of the existing slide and monitor slope movements during stabilization work at the bridge. URS also installed vibrating wire piezometers at three locations. One piezometer was installed to monitor groundwater conditions in the gravel layer overlying bedrock, and another was installed to monitor the surficial clayey and silty soils. URS installed the third piezometer in a 30-ft long steel casing that was lowered from the top of the bridge into the river to monitor river levels. One end of the casing was laid on the south riverbank while the other end extends about 25 feet into the river. All recent (2002) borings were drilled using rotary wash techniques. Samples were obtained using a split spoon sampler in accordance with the standard penetration test (SPT; ASTM D-1586) or a 3-inch O.D. thin-walled Shelby tube (ASTM D-1587).

In addition to the recent (2002) borings, borings logs from the original bridge borings drilled in 1963 were available. Twelve borings were drilled in 1963 – two borings at each bent. These borings are not shown in Fig. 5.

<u>Site Conditions</u>

Southwest of the river, the site consists of generally level ground at approximately El. 600. Approximately 75 to 100 feet southwest of the river, the grade begins to drop toward the river. The riverbank slope varies considerably upstream and downstream from the bridge, but it was about 2.5H:1V near the centerline of the bridge.

Bridge piers are numbered sequentially from 1 to 6, as indicated in Fig. 2. At the time of URS' field reconnaissance, Piers 4 and 5 were located within the river. A concrete boat ramp is located about 300 ft upstream of the bridge centerline and a 30- to 40-ft wide riprap berm that extends into the river is located about 30 ft downstream from the bridge centerline.

*Fig. 4. Northern Portion of "Shallow" Landslide.*

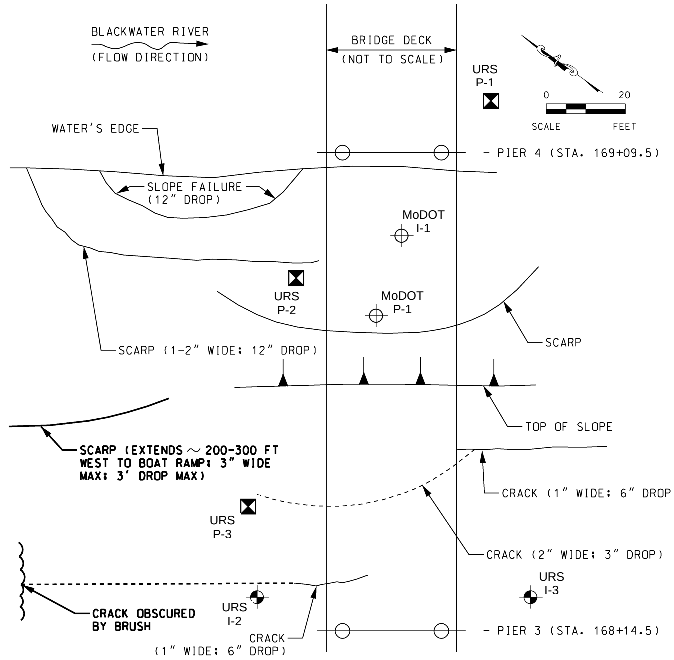


*Fig. 5. 2002 Boring Location Plan and Site Observations. (Note observations in bold made July 1, 2002.)*

## Subsurface Conditions

Four primary subsurface strata are present at the site: (1) interbedded silts and clays; (2) high plasticity clay; (3) gravel and sand; and (4) limestone bedrock. Figure 6 presents the subsurface profile and Fig. 7 summarizes the index and engineering properties of the materials.

Interbedded Silts and Clays. Near surface soils consisted of an interbedded alluvial deposit of low plasticity silty clay and clayey silt with layers of high plasticity clay and sand to approximately El. 565. Water contents ranged from 22 to 50%, averaging about 33%. SPT blowcounts ranged from 3 to 10, averaging about 5. The undrained shear strength ($s_u$) determined from two unconfined compression (UC) and four unconsolidated-undrained (UU) triaxial tests ranged from 460 to 1400 psf, averaging about 900 psf. Drained peak friction angles ($\phi'$) ranged from 27 to 33°.

High Plasticity Clay. All recent (2002) borings encountered a 2 to 4-ft thick layer of high plasticity clay underlying the clay and silt layer. Water contents, liquid limits, and plasticity

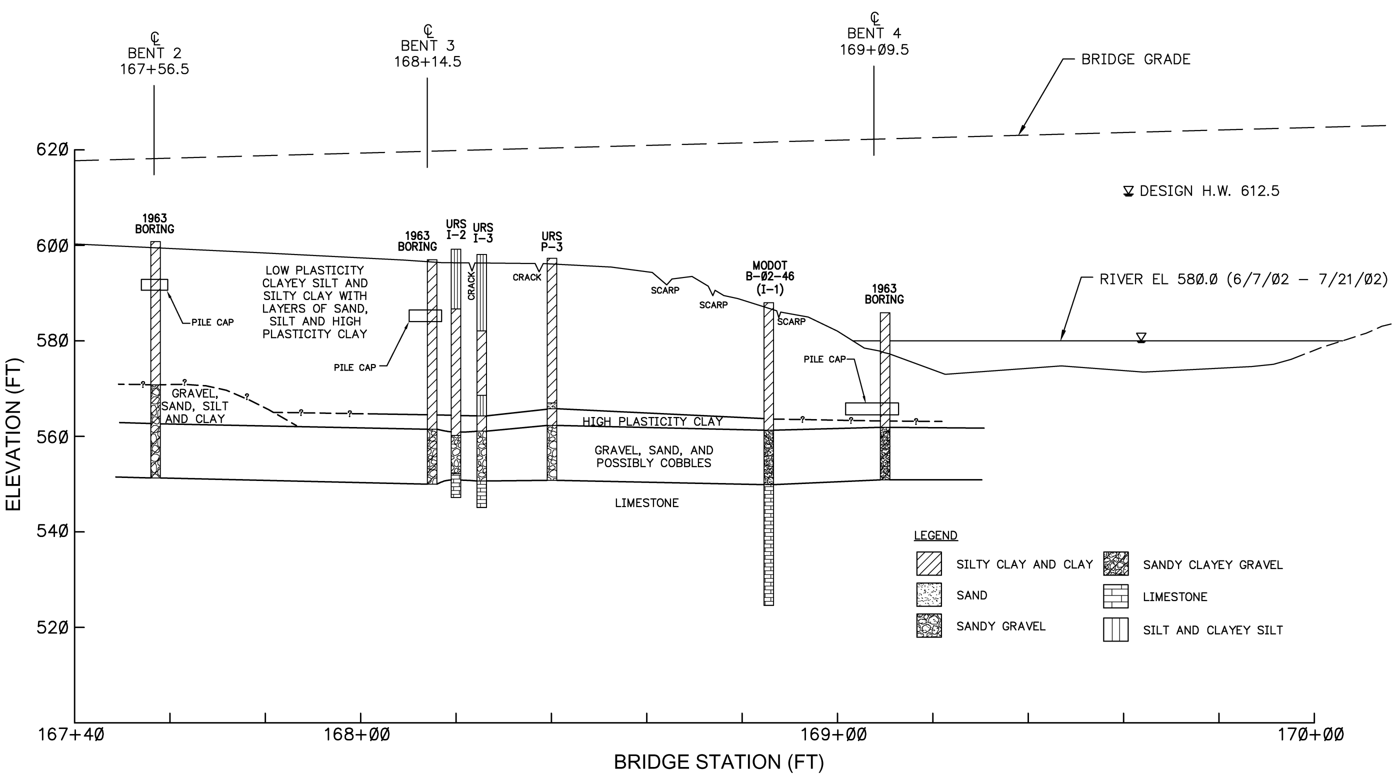


*Fig. 6. Subsurface Profile.*

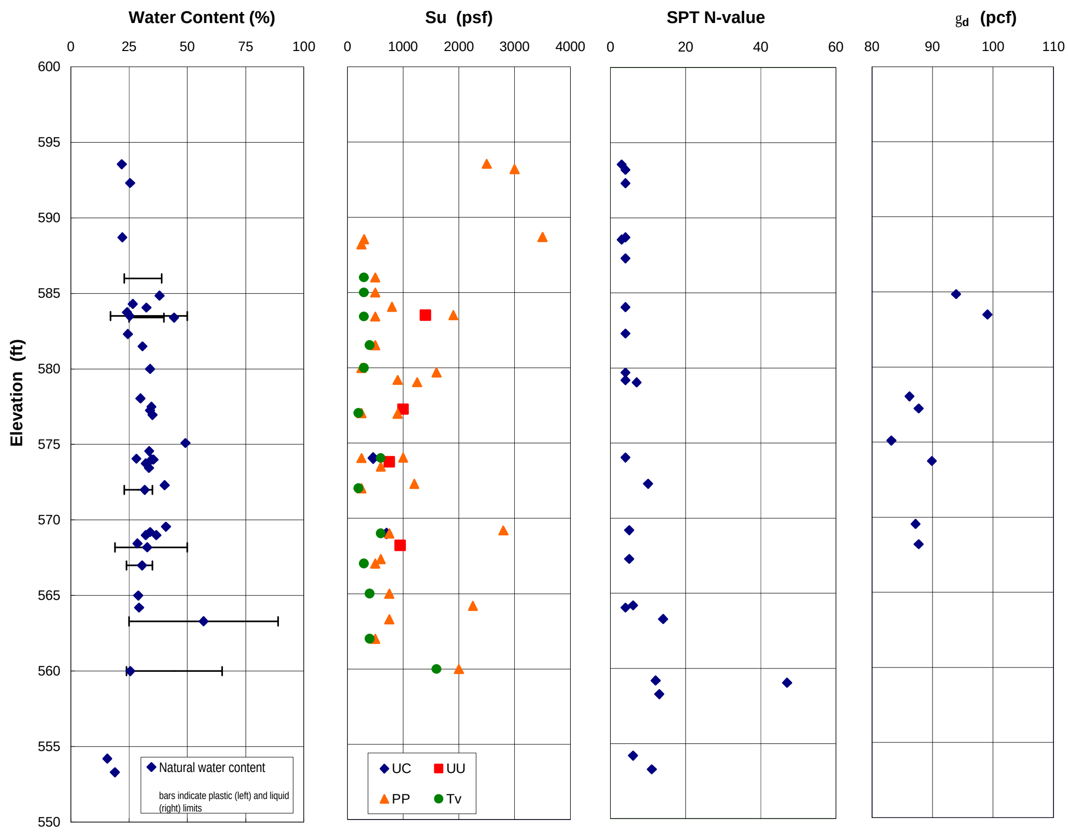


*Fig. 7. Index and Engineering Properties of Subsurface Materials.*

indices in this layer ranged from 26 to 57% (3 tests), 65 to 89 (2 tests), and 41 to 64 (2 tests). Measured drained fully-softened friction angle ($\phi'_{fs}$) ranged from 17 to 22°, while published correlations for $\phi'_{fs}$ ranged from 14 to 18°.

<u>Gravel and Sand.</u> Underlying the clayey soils, all recent borings encountered a 10-ft thick layer of gravel and sand. Original bridge borings show that this layer pinches out south of Pier 6. Generally, this layer is loose to medium dense with SPT blowcounts ranging from 6 to 13. Grain size analyses conducted on two split spoon samples indicated average $D_{50}$, $D_{10}$, Cc (coefficient of curvature), and Cu (coefficient of uniformity) values of approximately 10 mm, 3.5 mm, 1.3, and 4.0, respectively. Additionally, the possible presence of cobbles in this layer was inferred from the original bridge boring logs, drilling characteristics, and the one high SPT blowcount value (N = 47).

<u>Limestone.</u> Bedrock consists of the Burlington Limestone Formation. Limestone cores indicated that the depth of weathering is thin (typically less than 2 to 5 ft thick) and the unweathered limestone is of good quality, with RQD values ranging from 72 to 100%. Limestone outcrops near the bridge were heavily jointed, with both open vertical joints and open, closely spaced horizontal bedding joints.

<u>Groundwater.</u> Although initial observations indicated an artesian head in the gravel layer (i.e., observed from water flowing from instrument casings), piezometer readings indicated that the artesian pressure had dissipated by early July 2002. Groundwater levels in the slope stabilized at El. 587 to 590 and the river level receded to El. 580. The river level remained fairly constant during the investigation and repair.

## LANDSLIDE BACK-ANALYSIS

We conducted a back-analysis of the slope failure to estimate the groundwater conditions required to trigger failure. This section describes the key observations, discusses our assumptions and back-analysis of the slope failure, and provides the results of the back-analysis.

### <u>Key Observations</u>

On approximately May 20, 2002, a large storm triggered flooding on the Blackwater River. Flooding typically occurs every spring on the river. The Lamine and Missouri Rivers (which are downstream of the Blackwater) were near normal river stages at the time of the flood. The precise high water elevation on the Blackwater during the flood is not known, but watermarks visible in photographs of the bridge suggest a high water elevation of at least El. 612.5 to 615.

On May 29, 2002, MoDOT conducted an emergency bridge inspection. The floodwaters were at approximately El. 599. MoDOT observed damage at Pier 4 and closed the bridge.

Divers probed the riverbed and found that the riverbed soils were relatively stiff and intact and no scour had occurred.

While installing instrumentation in the slide mass in early June 2002, MoDOT personnel observed water flowing freely to the tops of the instrument casings (located 2 to 3 ft above existing grade). Inclinometer measurements in the slide mass indicated two zones of soil movement near Pier 4 – from approximately 0 to 8 ft below grade (within the clay and silt layer) and 24 to 28 ft below grade (within the high plasticity clay layer). By June 25, the slope movement essentially had stopped. By July 1, inclinometer measurements indicated that essentially no movements were occurring and that water levels had stabilized at El. 587 to 590 in the slope and at El. 580 in the river.

## Assumptions for Back-Analysis

Based on the field investigation, we anticipated that two separate slides occurred at the site – a "shallow" and a "deep" slide. The shallow slide likely was triggered by rapid drawdown of the river, i.e., high water pressure in the silt and clay attempting to exit the slope with no equalizing water pressure from the river against the slope. This slide presumably occurred after the deep slide as the Blackwater River receded. This type of slide occurs regularly along the Blackwater and evidence of rapid drawdown slides is visible at numerous locations upstream and downstream of the bridge.

We suspected that artesian water pressures in the gravel layer triggered the deep slide. Based on observations of limestone outcrops along the northern bank, it seems likely that the gravel layer is hydraulically connected to the river via jointing in the limestone bedrock (and possibly local scour of the riverbed). While the floodwaters started to recede, we believe sufficient artesian head was present in the gravel layer to cause a quick condition below the streambed. This caused the clayey streambed soils (and toe of the slope) to fracture and "blowout." When the toe of the slope was lost, a slide occurred within the high plasticity clay near the interface with the gravel layer or along the upper surface of the gravel and sand layer. These circumstances are rather unique because similar failures did not occur during major floods in 1993 and 1995. In this case, we presume that the floodwaters receded more quickly than the artesian pressure dissipated due to the relatively low levels of the Lamine and Missouri Rivers.

Initial stability back-analyses appeared to substantiate the potential failure mechanisms and separate slides described above. However, a number of assumptions were required to conduct the back-analysis. Our assumptions are shown below. Table 1 provides the input soil parameters used for the various materials.

- The upper clay and silt layer was undrained during the deep failure and was assigned undrained strength parameters. During the shallow, rapid drawdown failure, the clay and silt were partially drained. We assigned this layer a bi-linear envelope proposed by the Corps' of Engineers (1970) for rapid drawdown analysis. We used this simplified strength envelope because insufficient test data were available to define more detailed failure criteria such as those proposed by Lowe and Karafiath (1960) or Duncan et al. (1990).
- At least the lower portion of the high plasticity clay was drained during all cases due to its proximity to the gravel layer. For modeling purposes, we assigned the entire layer its drained shear strength and assumed that it was hydraulically connected to the artesian water pressures.
- The gravel layer was modeled as a limit boundary to allow a composite/block slip surface through the high plasticity clay, the weakest layer.
- Due to modeling limitations of the computer software, we iteratively determined the extent of quick conditions in the gravel for various artesian pressures for the deep slide. Where the gravel layer was quick, we assumed that this would cause an uplift failure, i.e., "blowout," of the clayey soils above the gravel. We assumed that the boundary of this uplifted block (along the fracture) would temporarily have zero shear strength. For modeling purposes, we assigned zero shear strength to the entire uplifted zone. However, we restricted the deep failure surfaces to those that passed through the high plasticity clay such that unrealistic surfaces that passed through the middle of the uplifted block (with zero shear strength) were not calculated to be critical.

## Back-Analysis Results

The results of the back-analyses are presented below. In addition, we evaluated the current stability of the shallow and

Table 1. Soil Properties used for Slope Stability Analyses.

| | | Analysis Case | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | **Back-Analysis of Shallow Slide** | | | **Back-Analysis of Deep Slide** | | | **Rock Toe Berm Final Configuration** | | | **Interim Construction Case** | | |
| **Soil Layer** | **Unit Weight (pcf)** | **Drainage** | **c (psf)** | **f' (°)** | **Drainage** | **c (psf)** | **f' (°)** | **Drainage** | **c (psf)** | **f' (°)** | **Drainage** | **c (psf)** | **f' (°)** |
| Silt and Clay | 115 | Partial | bilinear envelope[a] | | Undrained | 300[b] | 16[b] | Undrained | 300[b] | 16[b] | Undrained | 300[b] | 16[b] |
| "Quick" Silt and Clay[c] | 115 | -- | -- | -- | Undrained | 0 | 0 | -- | -- | -- | -- | -- | -- |
| High Plasticity Clay | 115 | Drained | 0 | 17 | Drained | 0 | 17 | Drained | 0 | 17 | Drained | 0 | 17 |
| Gravel and Sand | 115 | Drained | 0 | 35 | Drained | 0 | 35 | Drained | 0 | 35 | Drained | 0 | 35 |
| Rock Blanket Fill | 120 | -- | -- | -- | -- | -- | -- | Drained | 0 | 34 | Drained | 0 | 34 |

[a]Bilinear envelope recommended by Corps' of Engineers (1970): $\phi' = 28^o$, $c' = 0$ to $\sigma'_n = 1250$ psf; then $\phi = 16^o$, $c = 300$ psf.

[b]Combination of c and $\phi$ used to model slightly overconsolidated soil with $s_u = 300$ psf at ground surface and $s_u/\sigma'_v = 0.29$ (silt and clay primarily in triaxial compression).

[c]Soil located in streambed at toe of slope.

deep slide to provide an additional check on the input soil strength parameters. We conducted the slope stability analyses using Spencer's (1967) method as coded in the software program SLOPE/W (Geo-Slope 2002).

Shallow Slide. Figure 8 presents the back-analysis results for the shallow, rapid drawdown failure. As indicated in the figure, the head of the critical slip surface corresponds to the large scarp at the edge of the bank. Furthermore, the depth of the critical slip surface corresponds to the upper zone of movement measured in MoDOT I-1. We believe that this slide occurred after the deep failure as the floodwaters had receded, and that this slide did not influence the movement of Pier 4.

Deep Slide. Figure 9 presents the back-analysis results for the deep failure related to artesian pressures. As indicated in the figure, an artesian water level in the gravel layer of El. 620 is required to yield a factor of safety of about unity. This implies that the flood levels were higher than El. 612.5 and/or joints in the limestone charged the artesian pressures. This artesian head triggered quick conditions over a wide zone in the gravel layer. The soil labeled "quick" clay and silt indicates the extent of quick conditions in the gravel layer. In addition, a tension crack to the depth of the more permanent watertable (about El. 590) was added to reduce tensile forces in the slices. Figure 9 indicates that the head of the critical slip surface falls between the cracks observed just north of Pier 3 and the bottom of the critical slip surface corresponds to the lower zone of movement measured in the inclinometer.

Because the stability analyses provided factors of safety close to unity, these conditions (i.e., an approximate head difference of about 20 feet between the river and gravel layer) were judged to reasonably approximate the critical conditions that triggered failure. Therefore, we used these conditions to design stabilization options.

Current Conditions

For evaluating the post-failure stability of the slope, we assumed that water levels in the slope and the river were consistent with porewater pressures measured in early July 2002 and no artesian pressures were present in the gravel layer.

The analyses indicated that the shallow slide was marginally stable, with a factor of safety (FS) of about 1.1. This FS is consistent with the minor creep movements observed in MoDOT I-1. As such, the drained friction angle of 28° assumed for the clay and silt layer appeared reasonable.

The analyses also indicated that the deep slide had stabilized, with a FS of about 1.3. This FS was consistent with the inclinometer data in MoDOT I-1 and survey measurements of Pier 4 that indicated essentially no continuing movement of the deep slide or Pier 4.

LANDSLIDE STABILIZATION DESIGN

We developed a number of design options to stabilize the slope failure. These options included:

- A rock toe berm;

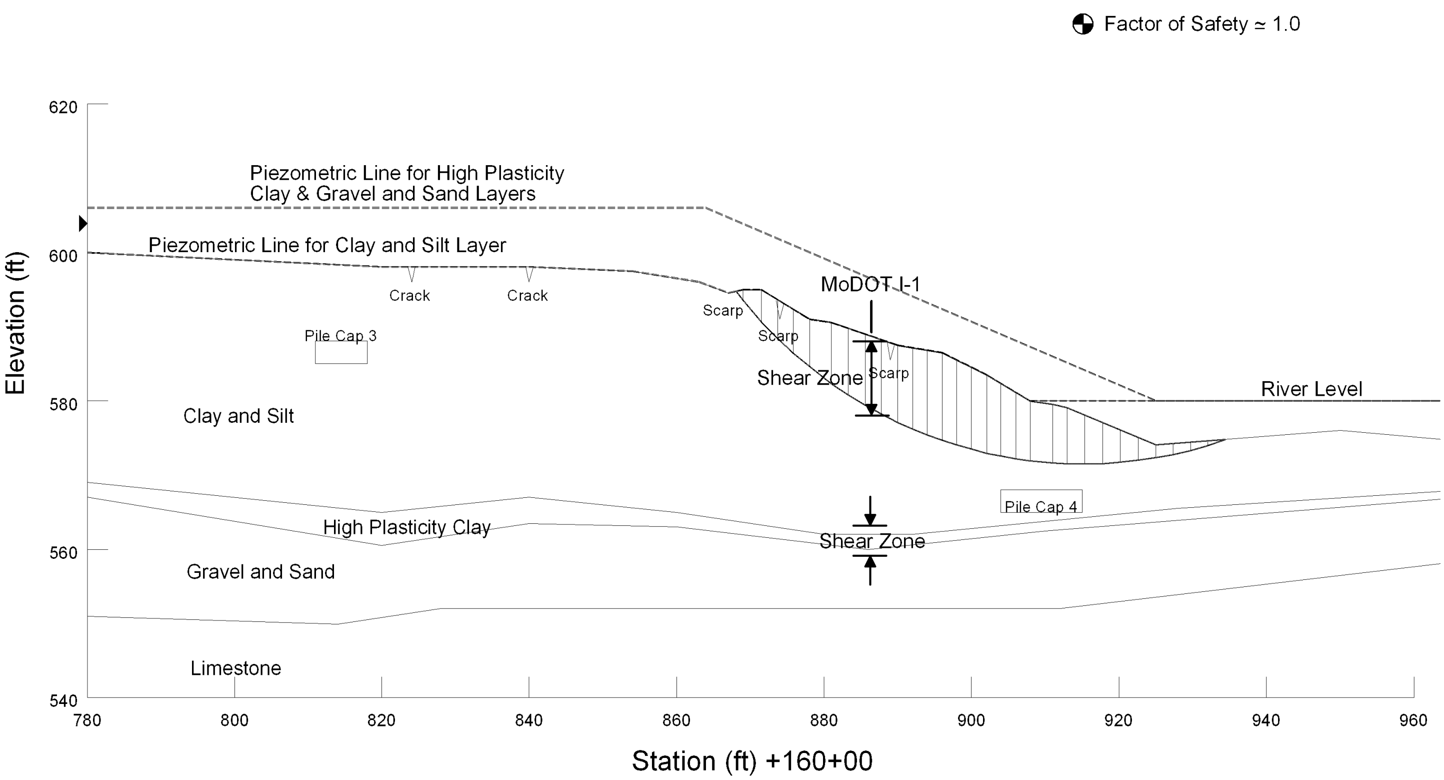


*Fig. 8. Back-analysis of "Shallow" Slide.*

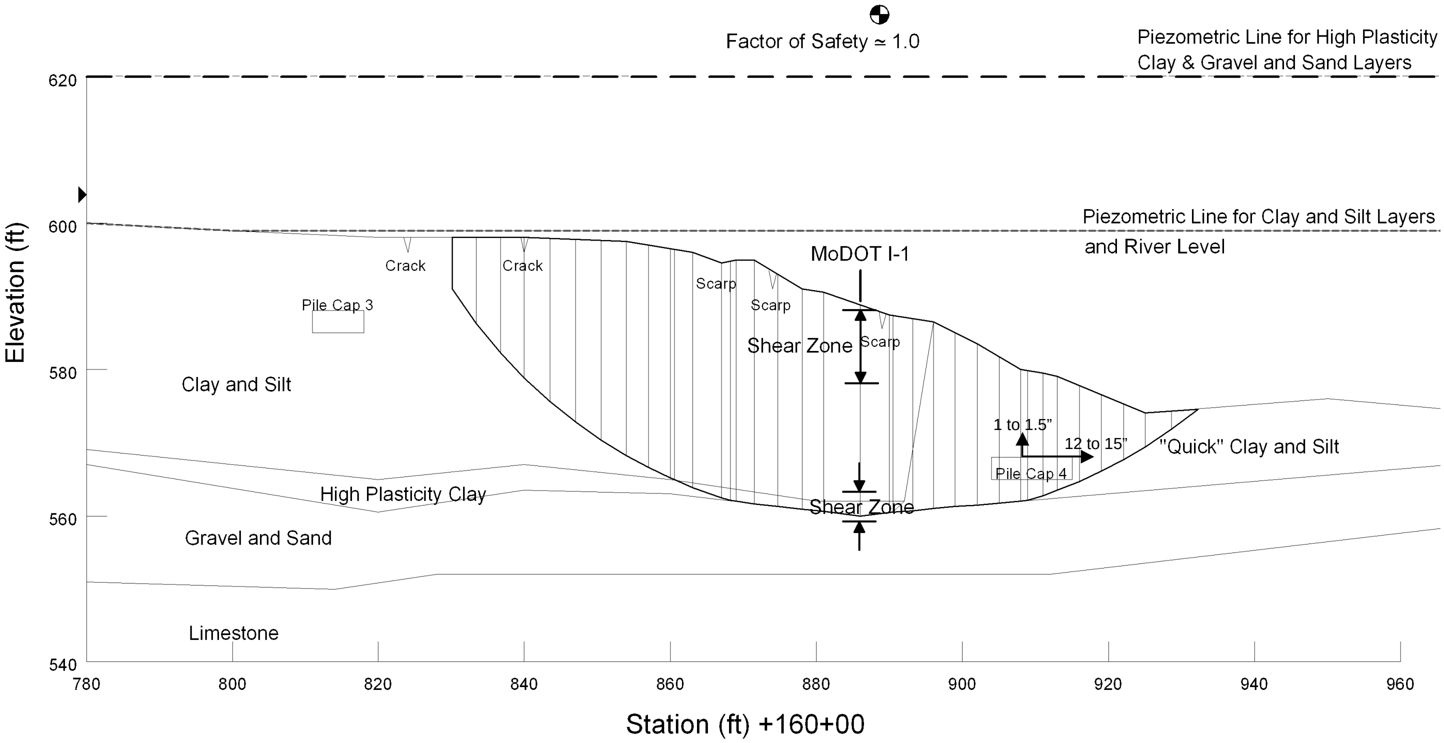


*Fig. 9. Back-analysis of "Deep" Slide. Note that Pile Cap 4 was Moved 12 to 15 inches Laterally and 1 to 1.5 inches Vertically.*

- drainage of the gravel layer via pressure relief wells;
- a rock shear key; and
- a "do nothing" approach for slide stabilization while designing a replacement bridge pier to accommodate the lateral soil load due to slope failure.

MoDOT selected the rock toe berm option on the basis of cost and its ability to prevent both the deep and shallow slides. Furthermore, we considered it unlikely that the relief well option would receive necessary maintenance and cleaning on a regular basis.

Rock Toe Berm Design

Figure 10 shows the design configuration of the rock toe berm. The only restriction for the toe berm was that the final berm configuration was limited to the geometric limits of the riverbank at the time of bridge construction in 1963. The final slope of the toe berm is 1.5H:1V and the top of the berm is at El. 590. This configuration was developed to provide a factor of safety against slope failure of at least 1.3 throughout construction and during a repeat of the water conditions that caused the failure in Spring 2002 (i.e., river level at El. 599, artesian head at El. 620). To prevent scour and erosion of the toe berm material, it was decided to utilize MoDOT Type 2 Rock Blanket material (about 3-ft diameter rock).

The rock toe trench provides a drainage path for artesian pressures during flood events. This element of the repair will prevent a quick condition from developing in the gravel layer and prevent a "blowout" near the berm toe. Furthermore, the toe trench protects the berm from being undercut by local erosion or scour of streambed soils at the toe of the berm. Lastly, the weight of the berm prevents a shallow rapid drawdown failure of the riverbank clays and silts.

Replacement Pier Design

As a result of the landslide-induced damage to Pier 4, it was necessary to replace the existing pier. To achieve a redundant design, MoDOT decided to design the foundation for the replacement pier to withstand the lateral soil pressures resulting from the landslide assuming that no slide stabilization was constructed. We estimated that the lateral soil load on the shafts would be uniformly distributed with a magnitude of 10.8 ksf. This value corresponds to $12s_u$, where the design undrained shear strength, $s_u$, was taken as 900 psf.

To withstand these lateral pressures, two 9-ft diameter drilled shafts were required for the Pier 4 replacement foundation. The shafts required 8.5-ft diameter sockets drilled 17 ft into limestone bedrock. Because of an accelerated construction schedule, construction of the shafts was to be done concurrently with slide stabilization work.

## SLOPE STABILIZATION AND PIER REPLACEMENT

Stabilization and pier replacement work for the bridge started in October 2002. During excavation of the bank to a slope of 3H:1V, a crack opened approximately 6 ft upstation of Pier 3. The crack had a maximum horizontal displacement of 4 to 5

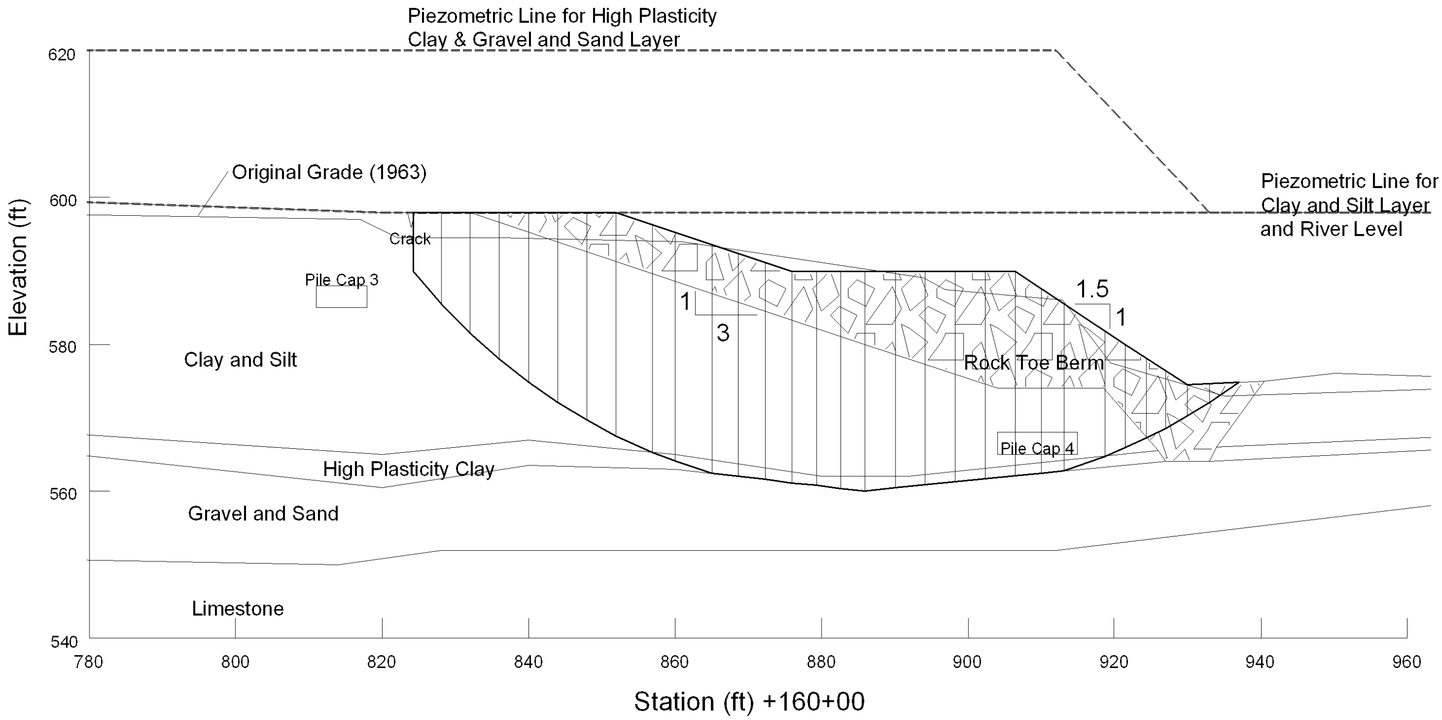


*Fig. 10. Rock Toe Berm Design Section and Example Trial Sliding Surface.*

inches and zero vertical displacement; however, ground surface measurements at Pier 3 indicated that the ground had subsided about 1 inch. We anticipate that slope movement occurred because the contractor began excavating the riverbank slope from the "bottom up" (rather than "top down"), thereby removing confining pressure at the toe before removing driving stress at the top of the slope. To prevent further movement, rock was dumped along the lower portion of the slope. Following rock placement, no additional movements were measured at Piers 3 or 4, at the surface survey markers, or in the inclinometers near Pier 3.

Prior to excavating the rock toe trench, the drilled shafts were excavated and cased using temporary steel casings. A work pad consisting of rock fill was placed around the temporary casings to about 1-ft above the river level to allow access to the toe trench location. The sides of the toe trench remained nearly vertical until the gravel layer was encountered. Rock fill was immediately dumped into the toe trench excavation. We anticipated that the rock fill would become infilled with river sediments over time. However, we also expect that any infilling will be "blown out" by minor buildups of artesian pressure in the gravel, allowing dissipation of artesian pressures prior to any significant slope displacement.

## CLOSURE

The remainder of the rock toe berm was constructed without incident. The drilled shafts were poured on November 29, 2002 and December 11, 2002. Demolition of the existing pier and construction of the replacement pier started on December 12, 2002. Following completion of the pier replacement, the Route K Bridge over the Blackwater River was re-opened to traffic on January 9, 2003, a little over six months after bridge closure.

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