Bank Instability Problems Associated With the Riverside Construction

Gennaro G. Marino
Marino Engineering Associates, Inc., Urbana, IL

Abdolreza Osouli
Southern Illinois University Edwardsville, Edwardsville, IL

Commemorate the Legacy of Ralph B. Peck
Seventh International Conference on Case Histories in Geotechnical Engineering
April 29 - May 4, 2013 CHICAGO
and Symposium in Honor of Clyde Baker

BANK INSTABILITY PROBLEMS ASSOCIATED WITH THE RIVERSIDE CONSTRUCTION

Gennaro G. Marino, Ph.D., P.E.
Marino Engineering Associates, Inc.
Urbana, IL-USA 61801

Abdolreza Osouli, Ph.D.
Southern Illinois University Edwardsville
Edwardsville, IL-USA 62026

ABSTRACT

In this case study, the sliding of a riverbank during construction of a water intake facility in Tennessee was investigated and analyzed. The construction of the project involved the installation of 2-36 in. diameter intake pipes from the wet well to the river inlet which were 290 ft apart. An open cut excavation from the river inlet to the riverbank toe was used to connect the inlet to the tunnel-installed-intake pipes on the land side. During the excavation, a 25 ft wide slide, which 4 months later widened by another 15 ft, developed to the crest of the road on the riverbank. Consequently, a concern developed for the safety of the roadway. The geometry of the slopes and the cuts, pre- and post-construction geotechnical subsurface investigation, construction history, and sliding conditions were examined for the causes of the riverbank instabilities. The fundamental cause of the slides was the undermining of the latent bedrock surface from subaqueous excavation into the riverbank.

INTRODUCTION

This study is a geotechnical investigation of the bank instabilities associated with the riverside construction of the Water Intake Structure in Tennessee. A general plan of the water intake facilities along the river is shown in Fig. 1. This water is transported by pipeline to be treated and then distributed to neighboring communities. As can be seen from Fig. 1, the intake structure basically consisted of the wet well on the east side of a four lane roadway which is connected to two 36 in. diameter intake pipes that are at an approximate depth of 70 ft (i.e. planned invert elevation at 320 ft). The intake pipes are installed inside 5 ft diameter drilled-in steel casings from the shore well and below the road and a portion of the riverside embankment for the four-lane roadway.

The specified method of construction was to tunnel from inside the wet well to some point on the riverside of the road and install the 5 ft steel casing and subsequently the 3 ft intake pipes. For the remainder of the 290 ft from the wet well to the inlet in the river bottom, underwater construction or tunneling is specified. Consequently, underwater construction would involve trench excavation. Because of the specified invert elevation of 320 ft, this work would include drilling and blasting rock. Therefore, this project required sufficient subsurface geotechnical investigation to address tunneling and underwater excavation difficulties. Several design guidelines and text books are available for designing of such construction projects (Hoek, et al. 2000; Duncan and Wright 2005; Hung et al. 2009).

In the following sections of this paper, the history of the construction, associated ground conditions, and slide failure analyses are provided. The purpose of this study is focused on geotechnical investigation and field observations as two vital factors, recommended by Peck (1969) in conducting any construction project.

RIVERSIDE CONSTRUCTION HISTORY

“Production” drilling and blasting for the trench excavation was performed from July 9 to September 10, 2002. A cross-sectional view of the project conditions are shown in Fig. 2. It should be noted that the drill-and-blast work began from the inlet at Station 0+00 eastward towards the riverbank. The drilling and blasting continued to about Station 1+00 to September 10, 2002. The corresponding progress of the

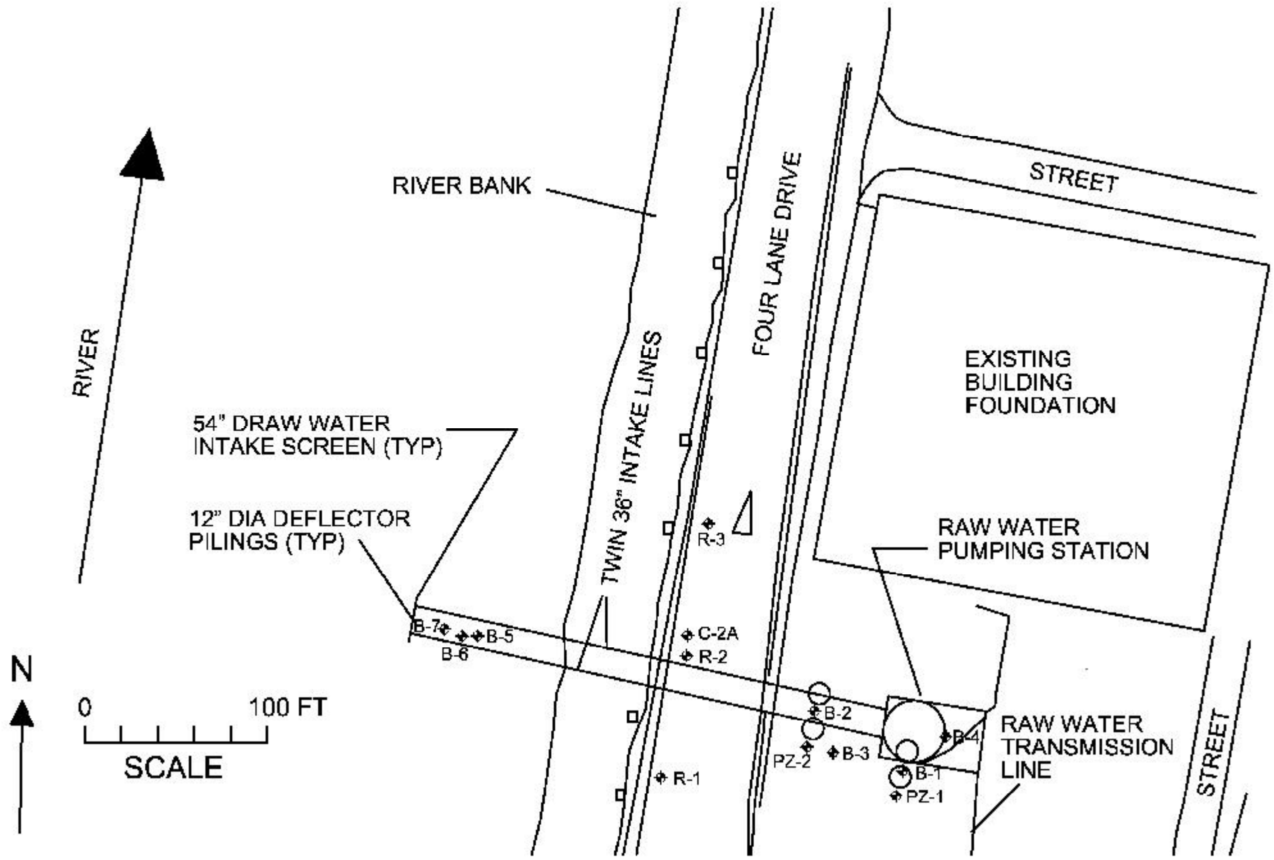

Fig. 1. Site plan of river intake area

trench excavation towards the river bank based on the available data is given in Fig. 2. During the excavation into the bank there were reports of minor subaqueous slides.

On September 11, 2002, river water began seeping into the intake pipe(s) through the still unexcavated portion of the riverbank. This water was sufficient in quantity that it was pumped during off-construction hours to serve the community needs. Pumping water was carried out during off-construction hours. From about September 11, 2002, the contractor intermittently conducted pin hammering, as well as more localized drilling and "small shot" blasting near the casings to break up the bedrock. The resulting broken rock and other material were removed either with a clam bucket or a long reach excavator. It should be noted that the ends of the 5 ft diameter steel casings are located at about Station 1+00 (see Fig. 2). To locate the exact position of the 5 ft steel casing, the contractor blew compressed air through the upstream intake pipe from the shore well and by October 10, 2002, reported air coming out "good" through "loose rock".

The first above-water slide occurred in the latter part of October 10, 2002. This slide was located about 5 ft upstream to 20 ft downstream of the planned centerline of the intake pipes up to near the crest of the embankment. A photograph of this slide is shown in Fig. 3. On October, 11, 2002, there were two large cracks reported by the guardrail. Later in the day, this cracked portion of earth also slid off and the slide reached to only 4 ft from the back of the guardrail. It should also be noted that the recorded change in water level in the adjacent river from the period of October 1 to October 10, 2002 was only 555 ft to 559 ft.

The contractor attempted to stabilize the above slide by placing riprap on the failed embankment section assuming bedrock was present, however, without toe support it raveled off the slope. As an alternative, it was decided to merely cover the slope with a tarp to prevent or minimize precipitation for saturating the slope surface (see Fig. 4).

After "clamming" to remove the debris in the river bottom from the landslide, the contractor continued their intermittent pin hammering and localized drilling and blasting excavation method to the 5 ft steel casings. The broken rock and other material were removed with the long-reach excavator and clam bucket, as well as performing hand excavation close to

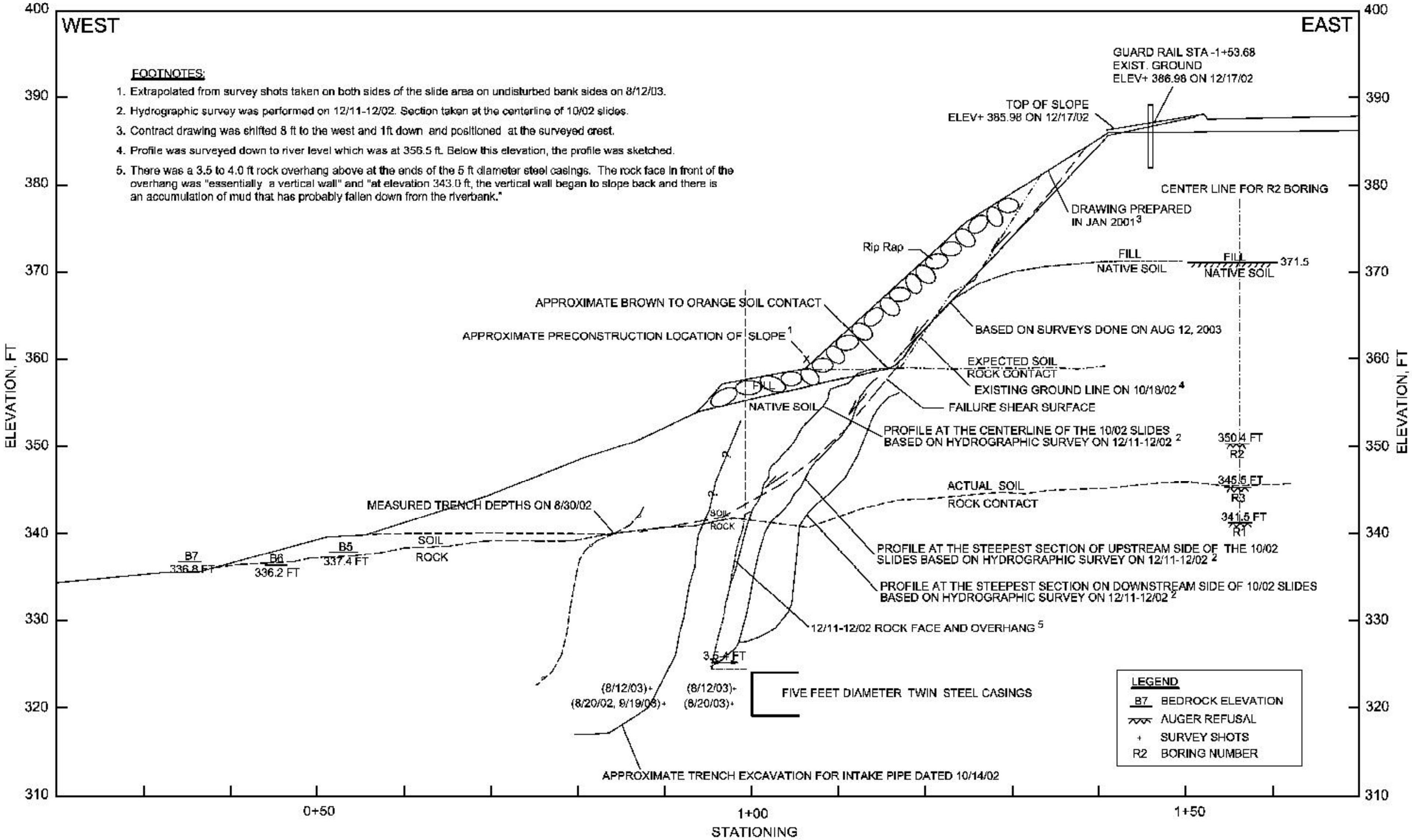

Fig. 2. Topographic profiles with ground conditions along the river bank during trench excavation

Fig. 3. Photograph taken on 10/11/2002 of slide on 10/10/2002. Note pre-existing rip rap covering the slope and that the slide progressed up the slope by the end of the day

Fig. 4. Air photo taken on 1/22/2003 of tarp covered slide area with orange soil beneath tarp. Note river level on this day varied from Elevation 355.6 to 356.3 ft

the casings. This excavation methodology appears to have been used into February 2003. The extent of the trench excavation for December 10-17, 2002 is also shown on Fig. 2. It is worth noting that the riverside south bound lane had to be closed down in early February. By February 7, 2003, the upstream riverside pipe was installed to within a "1 to 2 inch push" to connect it to the existing landside 3 ft intake pipe.

On February 20, 2003, another 15 additional feet of the riverbank had slid down on the downstream side. This slide was connected to the original slide. The total slide width increased to about 40 ft. The recorded river had reached an elevation as high as 378.3 ft on February 17, 2003 and on the day of the slide the river had receded to elevation 370.2 ft. After this last slide event, TDOT required that the entire slope be stabilized with riprap, thus covering any pipe installation work. This work was completed to support the embankment by late March 2003 and performed during the period of severe river conditions.

The temporary backfill was removed when minimal river level fluctuations are expected. The failed section of the embankment was then properly backfilled with riprap to the river bottom after 40 ft of pipe sections were connected to the existing pipe below the embankment.

SLIDE AND ASSOCIATED GROUND CONDITIONS

The slope conditions on the east side of the river were studied upstream and downstream of the intake excavation. Conditions were observed from 1400 ft downstream (north) of the project site to a point 800 ft upstream (south) in a rented boat and by walking the slopes. Evidence of minor past slope failures at the river line were extensive along the 2,200 ft long section of the east bank and occur on both the upstream and downstream sides of the intake alignment. The geometry of the slopes consisted of three basic elements starting from the top down to the river edge as follows (see Fig. 5 and Fig. 6):

- Upper vegetated unfailed slope
  - Below (west of) roadway
  - Approximate inclination: 40 to 46°
- Immediate steep slope face (scarp)
  - Approximate inclination: 78 to 90°
  - Typical height: 4 to 7 ft
- Lower flatter slope made up of failed soil and displaced vegetation
  - Approximate inclination: 10 to 33°
  - Typical land width: 5 to 17 ft (east-west)
  - Typical land height: 2 to 9 ft

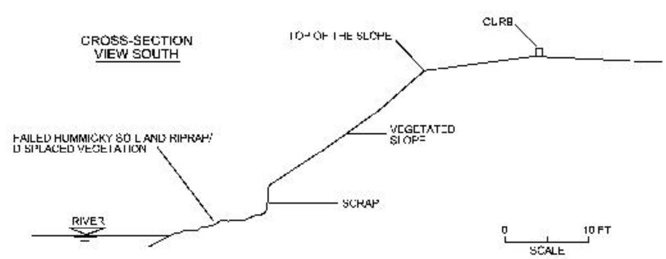

Fig. 5. Typical cross-section of failed slopes east of riverside drive in project location

At some locations, overhangs have developed along the tops of the immediate slopes as a result of bank failure/erosion and the presence of a thick root mat (see Fig. 7). It is important to note that the contact between the brown road embankment fill and the red-brown decomposed rock/colluvium could be seen at and above the water line at various locations. It was observed that riprap blocks up to 5 ft in size have been placed along the failed slopes in order to buttress and protect the embankment supporting the roadway. The riprap contains shot rock that has been placed on the toe of the river line slope and against the vertical to near-vertical scarps above the water line.

Fig. 6. Failed slope in east bank of river 210 ft south of intake pipeline

Two basic soil types were exposed in the failed and limited unfailed slopes along the east side of the river (see Fig. 7). The upper soil layer in the exposed slope is basically a light brown to dark brown silt or clay with man-made debris and limestone blocks. Based on the ASTM visual-manual classification, the matrix which makes up most of the soil unit is a low plasticity silt (ML). The upper layer is 10 to more than 30 ft thick and makes up the embankment that supports the roadway. The contact between the upper and lower soil units slopes toward the river (west) at an inclination of approximately 10° to 20° where visible. The contact varies in elevation by approximately 5 to 7 ft in an upstream/ downstream direction.

The exposed lower soil unit is a red-brown to yellow-brown clay with limestone fragments to boulder size material. Based on the visual-manual classification, the lower soil formation is a fat clay (CH). The soil has been developed by weathering of the underlying limestone and shale and by mass movement down the native slope.

The visible slope failures along the east bank of the river occur primarily in the silty brown fill that makes up the highway embankment. In local reaches, the failures include the upper portion of the weathered rock profile and/or colluvium. Bank instability is greatest during and shortly after periods of high river flow (such as in February 2003) when the second slide occurred. This slope failure mechanism involves over-steepening of the bank by erosion on the outside meander of the river and/or excess pore water pressures in the slope as the water level recedes. A submerged open cut excavation into the toe of a soil slope during placement of the intake pipeline would clearly reduce the stability of the bank similar to a natural river erosional mechanism.

LOCATION: 770 FT SOUTH OF PIPELINE (VIEW NORTH)

LOCATION: 430 FT NORTH OF PIPELINE (VIEW EAST)

Fig. 7. Overhangs above failed slopes related to bank instability and root mat

In local reaches, the failures have progressed further up the slope where stormwater pipelines emerge from the riverbank. Water flowing from the pipe discharges on the slope and erodes the soil beneath the pipe. Erosion progresses inward, steepens the slope and ultimately causes the bank to collapse. The resulting failures have contained fallen sections of the stormwater pipes.

At the time of the site visit, the slope above the intake pipeline was covered by rip rap from the crest down to some unknown distance below the water line. The general profile of the repaired project slope above the pipeline was mapped as well as the slopes immediately adjacent to the project alignment (see Fig. 8). The inclination of the rip rap slope varies between 27° and 42°. The steepest portion of the repaired failed slope is 4 ft below the crest. The vegetated slopes north and south of the pipeline alignment are inclined at angles ranging between 36° and 46°.

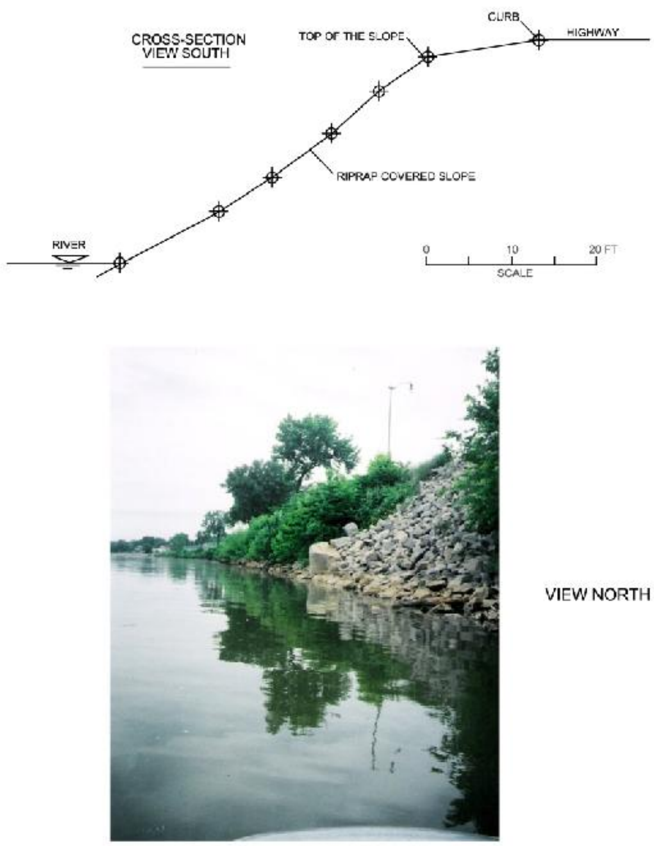

Fig. 8. Cross-section along riprap slope above intake pipeline

The 40 to 60 ft wide repaired slope is bounded by upstream and downstream banks that failed in the past and older riprap was visible above and below the water line. Clearly based on a simple site investigation, there was ample evidence of potentially unstable slopes at the river line near the intake pipeline alignment indicating the bedrock surface at some distance below the river level. Due to the close proximity of the roadway to the edge of the bank (approximately 13 ft) and the presence of visible slope failures upstream and downstream of the intake pipe alignment, protection of the roadway during tunnel/open cut construction would be a key design issue.

GEOTECHNICAL INVESTIGATION

As part of the geotechnical design investigation, a total of 8 borings were drilled: 3 around the Pump Station Building (B-1, B-4, and PZ-1), 3 immediately west of the wet well shaft and east of the highway (B-2, B-3, and PZ-2), and 3 in the river near the intake structure (B-5, B-6, and B-7) (see Fig. 1). No design borings were drilled in or immediately above the riverbank slope.

The B-series borings were augered to the top of the rock without undisturbed sampling and then cored down to depths of 2 to 14 ft below the specified tunnel invert (320 ft). The core borings show a consistent top of rock (hard gray limestone) elevation ranging between 357 to 359.2 ft (see Table 1). The logs also reveal an upper weathered zone in Borings B-2 and B-3 down to elevations of 341.4 and 348.5 ft, respectively.

Table 1. Summary of Design Borehole Data in and near Pump Station

Boring Number	Top of Rock Elevation	Bottom Elevation of Upper Weathered Zone	Bottom Elevation of Borehole
B-1	357 ft	347 ft	317.9 ft
B-2	358.5 ft	341.4 ft	310 ft
B-3	359.2 ft	348.5 ft	305.9 ft
B-4	357.5 ft	NONE	309.2 ft

The PZ-series borings (PZ-1 and PZ-2) were augered to refusal at depths of 25 ft (Elevation 365 ft) and 42.5 ft (Elevation 347.5 ft), respectively. Split spoon and auger soil samples were taken down to depths of 25 ft (PZ-1) and 16 ft (PZ-2) below the original ground surface (Elevation 390 ft). Two piezometers were installed in the PZ soil borings.

The boreholes (B-5 to B-7) were collared at depths of 17.8 to 22 ft below the water line. The bottoms of the borings were drilled to depths of 3.6 ft above to 4.9 ft below tunnel invert. The logs for the river borings confirmed “top of bedrock” at a consistent elevation ranging between 336.2 and 337.4 ft. A layer of boulders, riprap, mud-filled voids and fractured/weathered limestone 0.2 to 7.1 ft thick is shown above the bedrock surface. The materials above the bedrock surface are weathered limestone/shale similar to the solutioned rock below the soil contact beneath the Pump Station. Moreover, these materials are not the same ML and MH soils encountered above the limestone at the Pump Station Building site.

Between November 19 and 20, 2004, three additional auger borings (R-1, R-2, and R-3) were drilled in the roadway on the west side (see Fig. 1). The borings were located within about 20 ft of the crest of the slope on the north and south sides of the alignment. The holes were drilled approximately 9 months after the February 2003 landslide. The borings were advanced using a hollow stem auger and were drilled to refusal. Split spoon soil samples were taken every 2.5 ft in the upper 20 ft of the hole and then at 5 ft intervals to refusal. No rock cores were taken in the post-construction boreholes.

The post-construction boring logs show 15.5 to 18 ft of fill above a colluvial/residual soil. The soil thicknesses and elevations of the contacts in the actual soil profile near the top of the slope are summarized in Table 2.

Table 2. Summary of Soil Thickness/Contact Elevations Encountered in Post-Construction Borings Located Near the Top of the Slope

Boring Number	Fill Thickness	Residual Soil/ Colluvium Thickness	Elevation Fill/Soil Contact	Elevation Auger Refusal
R-1	18 ft	27.5 ft	369 ft	341.5 ft
R-2	15.5 ft	20.9 ft	371.5 ft	350.6 ft
R-3	18 ft	23.9 ft	369 ft	345.5 ft

The fill below the highway is primarily lean clay (CL) with clean sand (SP) layers. Standard Penetration Test values range between 4 and 15 blows/ft. The fill also contains rock fragments and organic/man-made materials. The residual soil/colluvium is typically an orange, red and/or brown fat clay (CH in the Unified Soil Classification System). Clays are sensitive to moisture and undergo large reductions in strength when exposed to water. A 5 ft thick layer of clean sand was encountered in Post-Construction Boring R-1. In addition, a soft, gravelly fat clay was encountered in Boring R-3 just above the top of rock.

SLIDE FAILURE ANALYSIS

The post-construction geotechnical data revealed a lower soil-rock contact than expected and fill/residual soil on the slope below the Normal Pool Elevation (359 ft). According to Boring R-2 the rock surface is 8 to 17.5 ft lower than the indicated rock elevation based on the pre-construction information (see Fig. 2). In addition, the actual pre-construction ground surface was discovered to be 8 ft west of that shown on construction drawings. Therefore, the October sliding was obviously more deep-seated than assumed by the contractor. Thus, the slope below the water line at Elevation 359 ft contained soils that would be and were highly vulnerable to sliding as a result of the specified open cut excavation. Placement of a single boring on Riverside Drive would have at least alerted the Engineer and Contractor that the elevation of the rock surface decreases toward the bank and the soils might be present below the Normal Pool water line. Moreover, boring(s) drilled at the edge of the river would have more definitely outlined the presence of fill and native soil in the slope and reveal that the toe of the soil slope would be undercut by the specified subaqueous open cut excavation.

Figure 2 shows the subaqueous trench excavation limits as the work approached the riverbank. Available slope surveys performed above the water are on October 18, 2002 after the October 10-11, 2002 sliding and on August 12, 2003 after the February 20, 2003 adjacent downstream slide; thus these profiles approximate the sliding surface.

Also, superimposed on Fig. 2 is data from surveys performed on December 11-12, 2002. The profiles which were taken from the survey are located at the approximate centerline of the October, 2002 sliding and the greatest undercuts on the upstream and downstream sides.

The ground condition information presented on Fig. 2 consists of the approximate fill/native soil and soil/rock contacts. The soil/rock interface was determined from the “B” and “R” borings, and is consistent with the sliding surface contour map, project photographs, and field mapping.

As can be seen in Fig. 2, the soil portion of the slope was undercut as the trench excavation preceded towards the bank. On October 10, 2002 the approximate location of the cut was at about Station 0+95. The “clamming” which was done by the contractor on this same day resulted in the nearly 12 ft soil undercut closer to the bank. Clearly this undercutting resulted in the bank instability as the sliding commenced later that same day. Based on the above data, a more representative depiction of the slide configuration is shown on Fig. 2. Consequently, the unstable slope condition resulted in the February 2003 embankment failure triggered by the rapid drawdown of the river.

The riverbank intake slope discussed herein was highly susceptible to slope failure given the specified subaqueous excavation. The subaqueous excavation in the plans and specifications allowed for significant undercutting of the toe of the soil slope. Consequently, this excavation methodology was doomed for failure.

The project plans and specifications established extensive requirements to protect existing structures that could be affected by intake shaft/tunnel construction, yet no such provisions were required to excavate to the specified stationing into the riverbank. The feasibility of an allowable cut into the riverside bank of a height up to 35 ft to the pipe invert at Station 1+00 should have been certainly investigated.

LESSONS LEARNED

A reasonable effort should be made to understand the overall design concept of the project in order to identify critical geoconstruction activity issues that could be associated with that design. The subaqueous excavation to Station 1+00 was carried out because the bedrock surface was considered present at approximate Elevation 359 ft at the riverside slope. Per design drawings, the top of the rock, east of the river, is between Elevations 357 and 359 ft and is at the same elevation as the ledge at the water edge at approximate Station 0+90 and at the Normal Pool Elevation (359 ft).

With the interpretation that the soil slope is supported at the riverside bedrock surface in the vicinity of Elevation 359 ft, the open cut could be allowed to about Station 1+15 without the toe of the soil slope being undermined. This interpretation also assumed that the excavation face would be in solutioned rock above sound rock and the rock face would be near vertical because of the steeply inclined joints in the horizontally bedded limestone and vertically drilled blast holes from the available geologic data. There were no design borings available at the edge of the river or at the top of the slope, however, to verify the top of rock elevation prior to the slide.

In addition to design, there should be sufficient subsurface data to determine the feasibility of specified elements of construction. For example, for this intake facility, borings and test data were used to determine the shallowest elevation that the tunnel could be placed without significant construction difficulties. Similar feasibility design analyses would be helpful for geotechnical design of the critical bank cuts.

The specified subaqueous excavation of the riverbank was also particularly concerning because of the vulnerability to sliding. Subaqueous excavation of the riverbank-embankment of soil or mixed face (soil and rock) conditions did not provide sufficient inspection or monitoring to adequately understand the consequences of what is being done. More specifically, it was not possible to conduct adequate visual inspection of the most critical bottom portion of the slope for rock or soil type, slope movements, or signs of distress and does not allow adequate control of the excavation as it proceeds inward into the bank. Therefore, subaqueous excavation is more appropriate for bottom trenching and probably not into an embankment slope protecting public work structures.

The bank sliding configuration in October 10-11, 2002 was assumed “bottoming out” at about 361 ft Elevation, which is consistent with the presumption that the bedrock surface is at this elevation. Therefore, the contractor attempted to place riprap on the slide surface to stabilize it.

Because the bedrock surface was actually over 17 ft deeper, the riprap would not stay on the slope and raveled down to the river bottom. After this attempt failed, the construction team concluded that the best solution was to tarp the slide surface until the intake pipe connections were made. After the bank had steepened for the October 2002 event, the slope became more susceptible to future failure from sudden drawdown conditions.

After the first slide, a full investigation should have been undertaken to assess the ground profile conditions. Proactive support of the slope should be undertaken. A sheetpile wall with optional tiebacks could have been designed and installed which would have adequately supported the adjacent highway before or even after the February 2003 sliding event. Other means to improve stability of the bank to about the level or greater than that which existed prior to riverside construction would have been to install: 1. A concrete secant retaining wall with upslope riprap cover; or 2. a tie-back soldier pile and lagging system (or a soil-nail supported wall in lieu of tie-backs) west of the highway guard rail near the crest of the slope.

These retaining systems would be designed based on an adjacent construction subsurface exploration program. Once installed, any of these methods of slope stabilization would have allowed riverside construction to continue unabated.

SUMMARY AND CONCLUSIONS

This investigation study was conducted into the cause(s) of the sliding of the riverbank during the construction of the water intake structure in Tennessee. The water intake facilities included the construction of a pump station, a wet well and 2-36 in. diameter intake pipes into the river.

The horizontal distance between the well and the inlet was 290 ft. The plans and specifications allowed for trench excavation from the inlet at Station 0+00 to into the riverbank at about Station 1+00. Subaqueous excavation methods used by the

Contractor included: drill and blast, "clamming", hand excavation, by excavator and pin hammering. Subaqueous excavation continued without any concern until the sliding event which occurred on October 10-11, 2002. The sliding surface was covered with tarps and excavation was continued for over 4 months until another slide occurred on February 20, 2003. This slide was about 15 ft wide making the cumulative slide width equal to 40 ft. This event appeared to be triggered by a drop in the river level. The fundamental cause of the October 2002 and February 2003 slides was the subaqueous excavation into the riverbank. The cut into the riverbank toe resulted in making the slope vulnerable to instability. Because of incomplete geotechnical information a higher rock and soil contact elevation was assumed, and therefore, a less favorable slope stability condition existed than envisioned. The open cut to about Station 1+00 resulted in significant undercut or undermining of the toe of soil portion of the slope. Moreover, the subaqueous method of pipe installation does not provide sufficient control of the excavation based on underwater inspection and monitoring of the slope.

REFERENCES

Duncan, J.M., and Wright, S.G. [2005]. *"Soil Strength and Slope Stability"*. Wiley, Hoboken, New Jersey, 312 p.

Hoek, E., Kaiser, P.K., and Bawden, W.F. [2000]. *"Support of Underground Excavations in Hard Rock"*. Taylor & Francis, Oxford, UK, 300 p.

Hung, C.J., Monsees, J., Munfah, N., and Wisniewski, J. [2009]. *"Technical Manual for Design and Construction of Road Tunnels"*. Civil Engineers, Technical Report, FHWA-NHI-10-034, 702 p.

Peck, R.B. [1969]. *"Advantages and Limitations of the Observational Method in Applied Soil Mechanics"*. Geotechnique, Vol. 19, No. 2, pp. 171-187.