HIGH BRIDGE AREA ROADWAY SLIDE,

LETCHWORTH STATE PARK

PORTAGEVILLE, NEW YORK

INTRODUCTION

Letchworth State Park is located along the Genesee River about 35 miles south of Rochester, New York. The Genesee River has carved a gorge up to 550 feet deep through this area and plummets over three separate waterfalls as it flows northward towards Lake Ontario. Letchworth State Park is operated by the Office of Parks, Recreation and Historic Preservation (OPRHP).

Norfolk Southern operates a rail line that crosses about 230 feet above the Genesee River near the town of Portageville, the southern entrance to the park. The present bridge is a steel structure that replaced what at one time was the longest and highest wooden bridge in the country. This bridge is referred to as the High Bridge.

The Portageville park entrance provides important access for large trucks and busses that support park inns and restaurants. The entrance allows tourists access to two major waterfalls in that area. Figure 1 shows the location of the roadway, gorge, railroad and additional site features. The Portageville entrance road circles around and about 100 feet below the western abutment of the high bridge and has experienced subsidence for many years requiring numerous pavement repairs. By 2007 the slope conditions had deteriorated to the point where it was difficult to maintain traffic on the road. Vehicles rounding the curve beneath the bridge were encountering a pavement dip that was nearly 3 feet in depth. Approximately 150 feet of the road was severely cracked and showed signs of significant movement.

In response to the deteriorating slope and road conditions the OPRHP commissioned an engineering study to determine the cause of the subsidence and to formulate a plan to remediate slope movements. This paper describes the subsurface explorations conducted to evaluate the conditions and the subsequent design and construction to remediate the area.

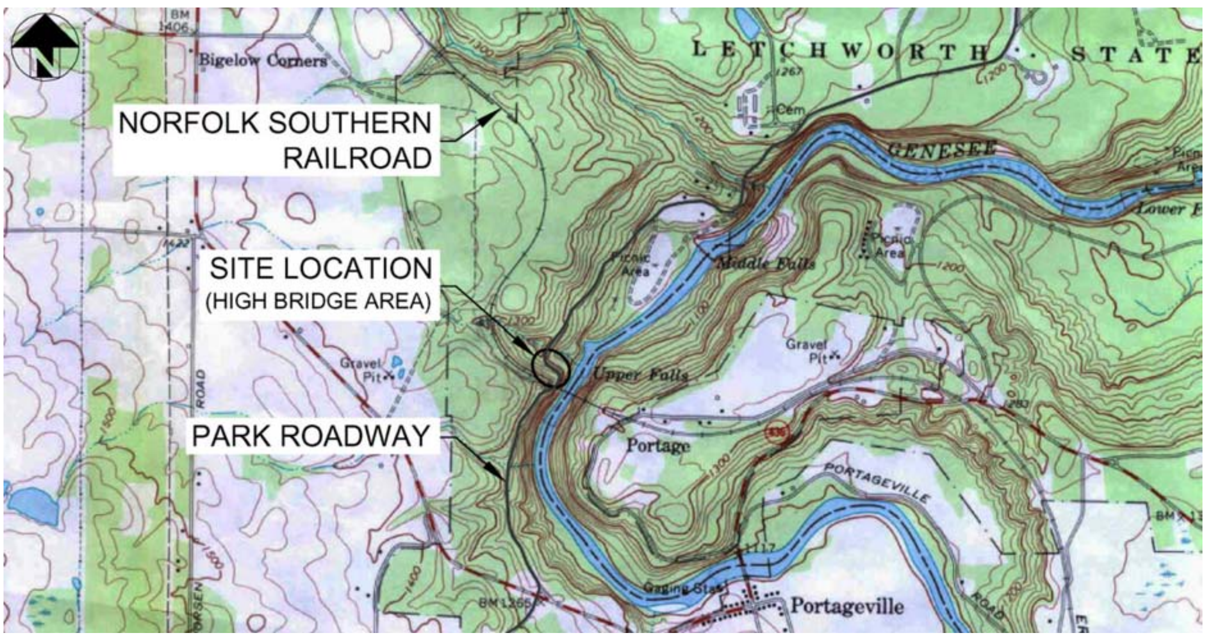

FIGURE 1

SUBSURFACE EXPLORATIONS

In 2007 Earth Dimensions, Inc. (EDI) drilled borings to rock and installed instrumentation to monitor slope movements and groundwater conditions. The site generally consists of glacially deposited surficial soils overlying sandstone and shale bedrock. A review of surficial geology maps indicates that the soils are either lacustrine or till in nature. Bedrock is of the Nunda Formation, which is part of the West Falls Group, formed during the Paleozoic Upper Devonian period.

Seven test borings were advanced to rock to characterize the soils at this site at the locations shown on Figure 2. Asphalt thickness varied from 0.1 feet to 1.7 feet between boring locations. Soils consist of a shallow layer of sand and gravel fill, approximately 3 feet in depth. From approximately 3 feet to 10 feet a “water sorted” glacial deposit exists and consists of a mixture of sand, silt and clay and layers of sand and sandy silt. The density of this zone varies from loose to dense based on Standard Penetration Test (SPT) N-values that vary from 5 to 36. Below this layer from approximately 10 feet to 35 feet is a “clayey lake sediment” consisting of clayey silt or silty clay. The consistency of this layer ranges from stiff to hard based on SPT N-values that vary from 10 to 33. The clayey silt soil contains vertical, silt-filled desiccation cracks, extending to a depth of approximately 16 feet with horizontal silt lenses noted in the clayey silt deposit in Boreholes 1–07 and 2-07. The silt lenses are prevalent in Borehole 2-07 from a depth of 14 feet to 18 feet and in Borehole 1-07 from 16 to 22 feet deep, where they were observed to be greater than 1 inch thick. The silt seams are described as extremely moist and are subject to “liquefaction” when disturbed. A glacial till layer approximately 10 feet in thickness consisting of gravel, sand, silt and clay is below the “clayey lake sediment” and extends to the top of rock. The glacial till is hard based on SPT N-values that exceed 30. Borehole 4-07 was extended into the underlying bedrock, which was classified as a sandstone or siltstone, moderately soft to medium stiffness. The 10-foot rock core from Borehole 4-07 indicates excellent recovery percentages with a poor rock quality designation due to the soft and highly fractured bedrock.

EDI installed an inclinometer in Borehole 1-07 to monitor the amount and direction of slope movement. Data is first collected for two orthogonal axes, and then the vector sum of the measured displacements is calculated to determine the magnitude and direction of the movement. Data collected for over an approximately four-month period (Figure 3 below), indicated the slope moved approximately 0.5 inches and was occurring at a depth of approximately 16 feet. Laboratory tests on sample S9 from Borehole 1-07 (silt lens at 16 feet), indicates that the natural water content is 29.5 percent and the liquid limit is 26 percent.

EDI also installed standpipe piezometers in Boreholes 2-07 and 3-07. The piezometer in Borehole 3-07 screens the upper “water sorted” glacial deposit and the piezometer in Borehole 2-07 screens the lower “clayey lake sediment” intersecting the silt lenses. The groundwater level in both piezometers was lowest in late summer and in both cases was at or near the bottom of the piezometers. After approximately six months of monitoring the piezometers and with the lack of significant groundwater data, we recommended installing vibrating wire piezometers to attempt to measure pore pressure in the silt lenses. EDI installed vibrating wire piezometers in Boreholes 6-07 and 7-07 within zones of saturated silt lenses in the “clayey lake sediment”. Data from these piezometers indicates that the pore water pressure in the silt lenses at times was above the ground surface. We believe that the high pore pressure in the silt lenses reduces the soil strength of these layers providing a weak zone along which slope movement occurs.

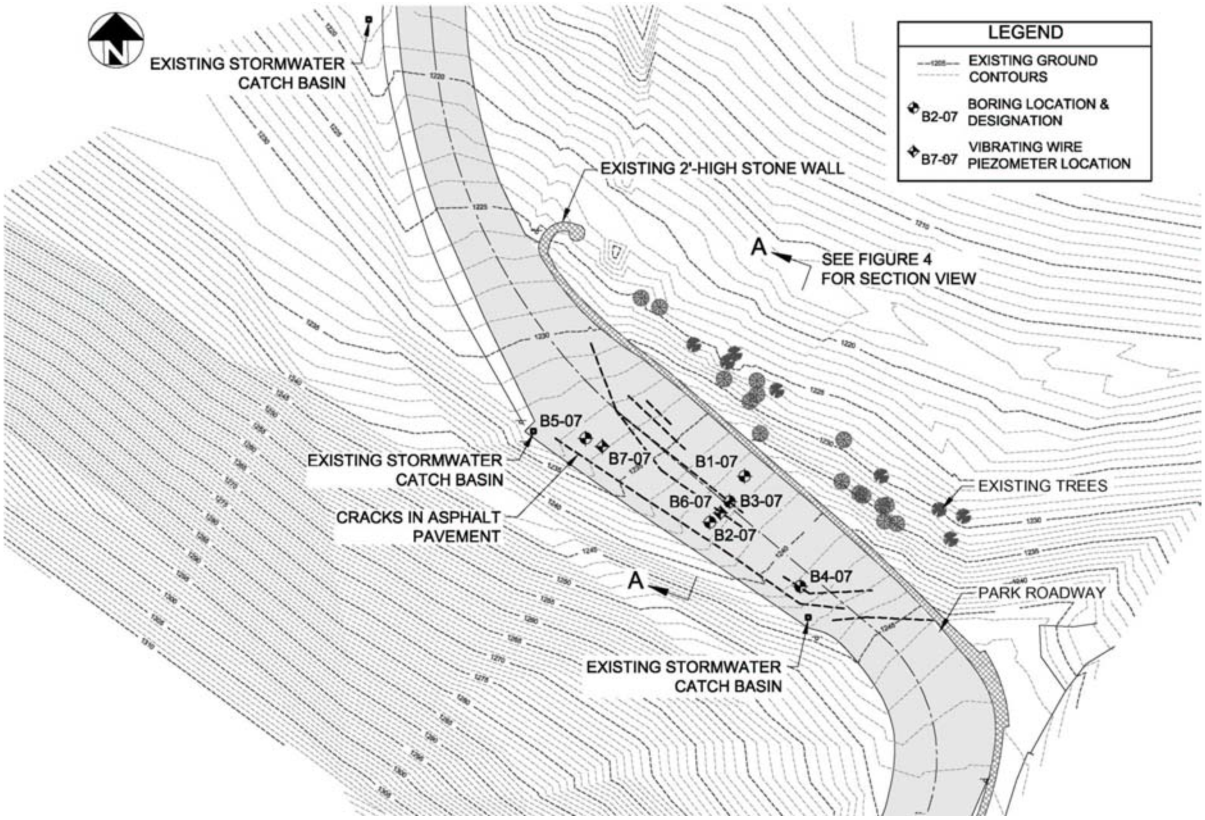
LEGEND
EXISTING GROUND CONTOURS
BORING LOCATION & DESIGNATION
B2-07
VIBRATING WIRE PIEZOMETER LOCATION
B7-07
EXISTING STORMWATER CATCH BASIN
EXISTING 2'-HIGH STONE WALL
A
SEE FIGURE 4 FOR SECTION VIEW
B5-07
B7-07
B1-07
B6-07
B3-07
B2-07
B4-07
EXISTING STORMWATER CATCH BASIN
CRACKS IN ASPHALT PAVEMENT
EXISTING TREES
PARK ROADWAY
A
EXISTING STORMWATER CATCH BASIN

FIGURE 2

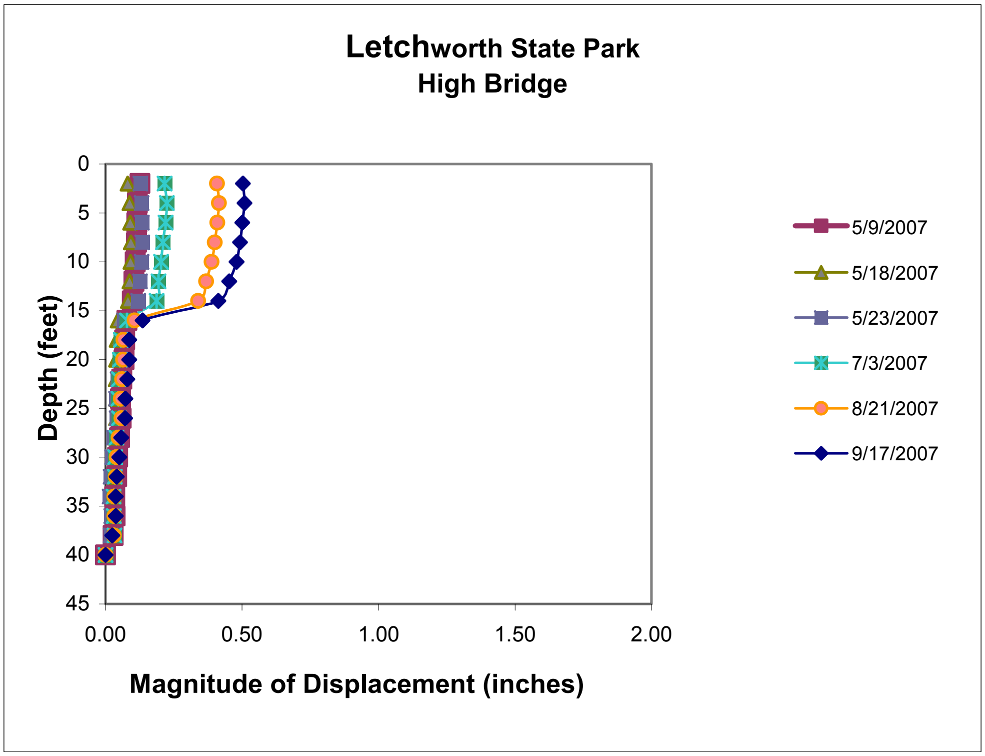

FIGURE 3

Figure 4 depicts data from Boreholes 1-07, 2-07 and 3-07 and the location of the theoretical failure plane in section view.

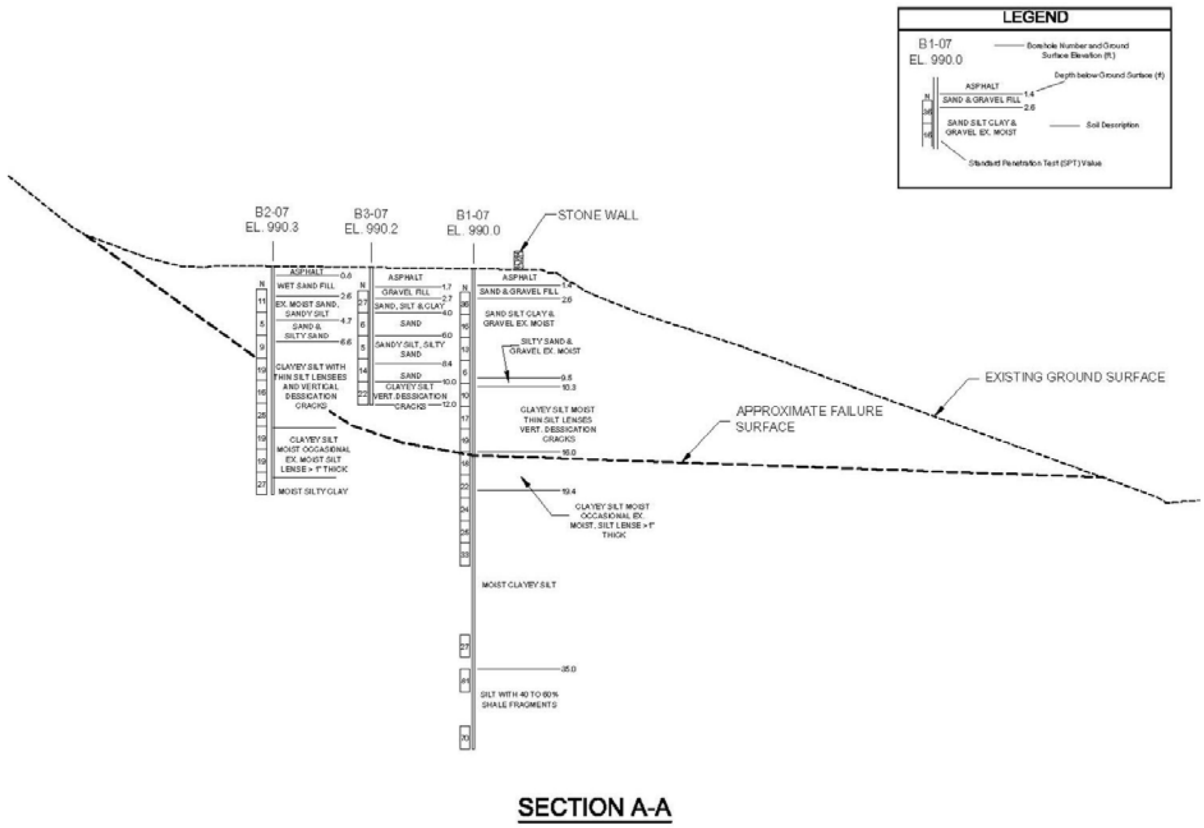

FIGURE 4

EVALUATIONS AND RECOMMENDATIONS

Traditional stabilization methods that involve lowering the top of the slope, flattening the slope or adding a buttress at the toe of the slope were not feasible here given that the road location and elevation cannot vary significantly from its current configuration. Furthermore, 20 feet of excavation to remove saturated silt lenses was also not considered a feasible option, as this could destabilize the Norfolk Southern Railroad track. Because of the site constraints, we recommended a two-step approach using anchored reaction blocks and drainage to improve slope stability and reduce the effect of future slope movements on the road.

We used PCSTABL6H, a slope stability program developed by Purdue University, to complete an analysis of the existing conditions and then to evaluate the benefit associated with adding normal force to the failure zone using reaction blocks and ground anchors. Both analyses were completed using the soil and groundwater data that had been collected, including a weak silt layer with high pore pressure to represent the silt lenses observed in the clayey lake sediment. The strength of the silt layer was estimated assuming a factor of safety of 1.0 for the existing conditions. The remediated conditions analysis was then completed using these same strength values and applying a normal force necessary to achieve a calculated factor of safety of 1.5.

We considered the factor of safety for the existing and remediated conditions to be benchmark values that show relative improvement to the slope offered by the selected remediation method. These values however do not address slope movements before or after remediation.

REMEDIAL DESIGN

Our design includes nine ground anchors and reaction blocks installed below the road. The blocks provide a stabilizing force that increases normal stress on the failure zone to resist future slope movement. Figure 5 shows the reaction blocks and ground anchors in plan and Figure 6 shows them in section view. The reaction blocks and ground anchors could be drilled and grouted into the hillside from a bench area below the road. We also recommended lowering the groundwater level using a series of subsurface drains to provide a pathway for the silt lenses to drain during and after the ground anchors are tensioned.

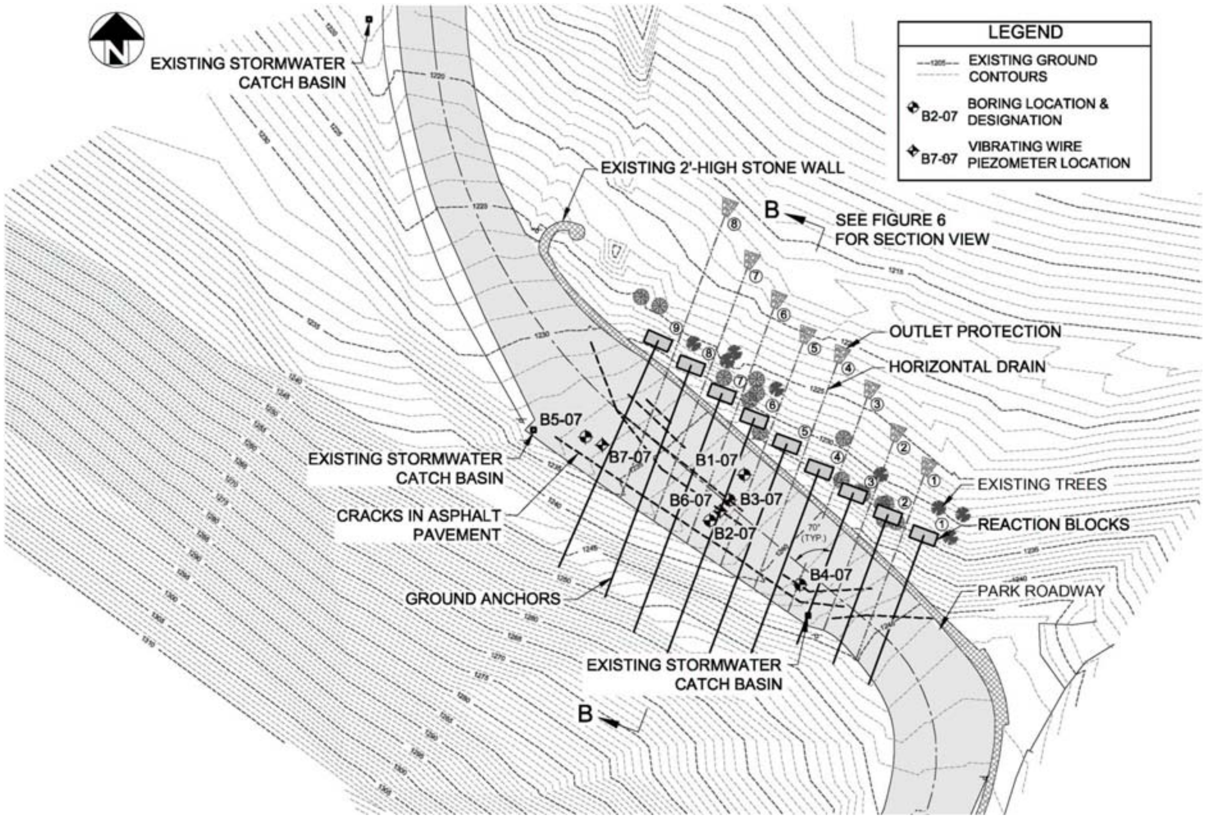

FIGURE 5

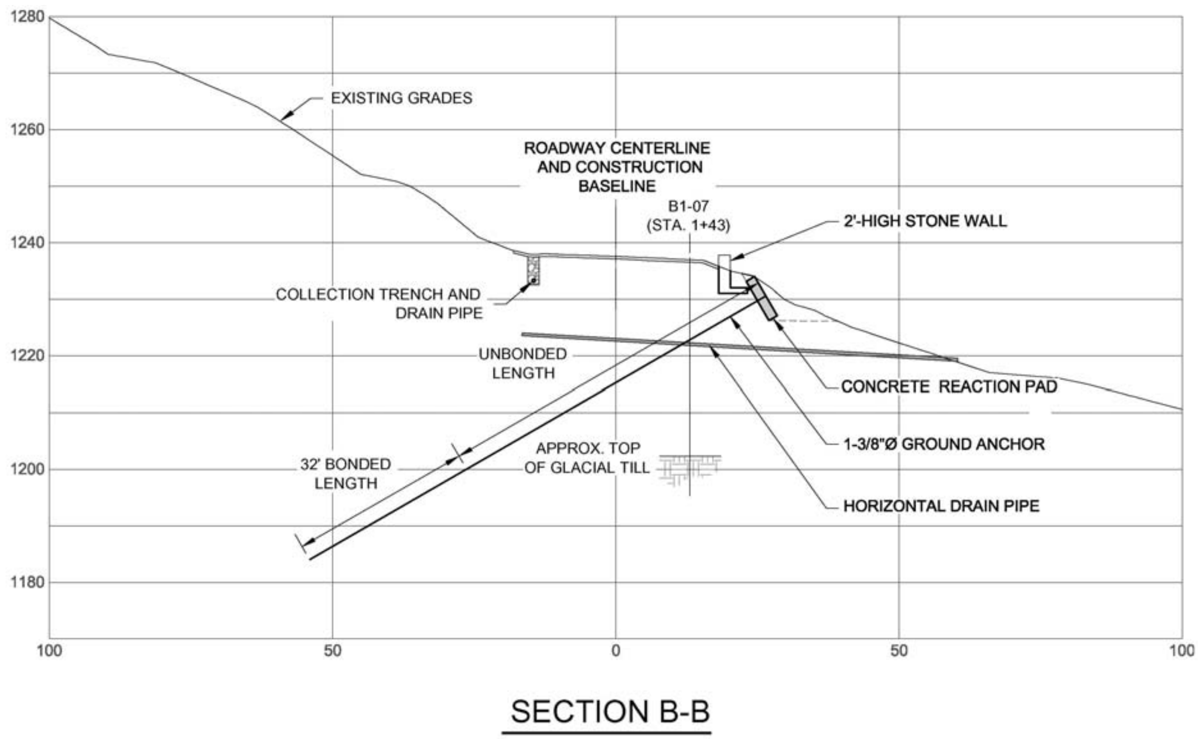

FIGURE 6

1-3/8 inch high strength (150 kips per square inch) steel bars for the ground anchors, each designed to carry a design load of 142.2 kips were specified. The bonded zone was specified to be 32 feet long and to be installed completely beyond the theoretical failure zone. The bond would be developed in either glacial till or bedrock depending on location. The unbonded length for each ground anchor varied between 60 and 100 feet, based on its location and observed ground conditions. Dual corrosion protected (DCP) bars were specified to maximize service life of the anchors.

The reaction blocks are reinforced concrete and are 8 feet by 8 feet, 21 inches thick and spaced approximately 12 feet on center. The ground anchors extend through the reaction blocks and are locked off at the face using base plates and nuts. This system allows the ground anchor force to be transmitted into the reaction block, which in turn exerts a uniform pressure on the surface of the slope. This pressure is then translated into a stabilizing force on the failure surface.

The design includes two types of drainage systems. A drainage trench approximately 4 feet deep with a perforated pipe surrounded in clean stone is included along the upslope side of the roadway. The purpose of the trench is to collect groundwater from the upper zones of soil and reduce recharge to the silt lenses. Eight horizontal drains that are designed to intercept groundwater in the deeper silt lenses were also included to provide a drainage pathway for the silt lenses during and after tensioning of the ground anchors. The horizontal drains consist of 6-inch diameter slotted drainpipes placed directly in contact with the native site soils. See Figure 6 for the location of the drains in section view.

CONSTRUCTION

OPRHP contracted with Keeler Construction Company, Inc. (Keeler) in the spring of 2008 to reconstruct the roadway and remediate the slope remediation discussed above. Keeler subcontracted with Berkel and Company Contractors, Inc. (Berkel) to construct and install the ground anchors and horizontal drains. BVR Construction Company, Inc. (BVR) was subcontracted to install the reaction blocks. Figure 7 shows the condition of the roadway prior to construction.

FIGURE 7

Keeler began construction by removing the existing road surface and fill materials that had been placed over the years as the road had continued to move. The upslope subsurface drainage was installed along with new surface water catch basins and outlet pipes. The road sub base was then regraded and compacted using a 2-inch minus crushed stone material. The work progressed down slope of the road by clearing and removing the existing trees and vegetation to allow for installation of the ground anchors and horizontal drains.

Berkel was then mobilized to the site to install the ground anchors and horizontal drains. The subcontractor utilized a horizontal rotary air powered drill rig to drill boreholes to the design lengths shown and advanced steel casing pipe to create a 5-1/4 inch diameter hole to bedrock. Boreholes were advanced without the casing into the bedrock resulting in a 4-1/2-inch diameter hole. Prior to drilling, a cross section at each ground anchor was plotted and the previously collected boring data superimposed. Based on these sections, it was expected that some of the bonded lengths might be partially in glacial till and bedrock. During ground anchor installation, it was found that bedrock was present at shallower depths than had been expected, which allowed for the bonded zone of all the ground anchors to be in bedrock. Upon completing each borehole, the DCP bar was inserted and grouted into place. Figure 8 is a photo of the drill rig installing the ground anchors.

FIGURE 8

Upon completing the ground anchors, Berkel proceeded down slope to the horizontal drain outlet locations. The drain holes were installed using the same drill rig. Upon finishing each borehole, the 6 inch slotted pipe was inserted to the design length and the casing was then removed. The last 10 feet of each pipe was installed without slots and was completed with the installation of an animal guard outlet and riprap apron. Figure 9 is a photo of the drill rig installing the horizontal drains.

FIGURE 9

BVR was then mobilized to the site to construct the reaction blocks. The design had specified that the reaction blocks be cast in place concrete formed by using shotcrete. This method was chosen to enable the concrete to be placed in intimate contact with the ground surface and constructed without the use of forms.

Prior to placing the shotcrete, BVR installed two mats of reinforcing steel and a protective pipe sleeve to allow the ground anchor to pass through the reaction block. The shotcrete was then installed to the specified thickness with the surface of the blocks troweled to allow for placement of the ground anchor base plate and nut. Figure 10 is a photo of the shotcrete installation.

FIGURE 10

After the reaction block concrete reached its specified compressive strength, Berkel returned to test and tension the ground anchors. Each anchor was then tested to 133 percent of the design load. The first ground anchor was performance tested, which is a cyclic test of loading and unloading, followed by all of the other ground anchors being proof tested, which is a single cycle test. The results show that all the anchors exhibited the expected elongations during testing. After completing the testing, all nine of the ground anchors were locked off at the design load. Berkel installed load cells on three of the ground anchors for future monitoring.

The load cells are an open steel cylinder installed between two base plates, which are locked off with a nut. The load cells each contain three vibrating wire strain gauges that are activated when read. During ground anchor lock off, the load cells and jack pressure were compared to check that the correct design load was applied. The intent of installing the load cells was to enable a recheck of the ground anchors over time to watch for loss in ground anchor tension.

Once the ground anchors were tested and locked off, Keeler installed protective caps over the ground anchor head assemblies and backfilled around the reaction blocks. The backfilling was to create a gradual down slope area adjacent to the road that could be revegetated. Keeler then repaved the roadway and applied topsoil and seed to the completed slope. Figure 11 shows the condition of the roadway at the end of construction.

FIGURE 11

MONITORING

Instrumentation included an inclinometer, two vibrating wire piezometers, two monitoring wells and three load cells.

The vibrating wire piezometers and both monitoring wells were destroyed during removal of the pavement. The inclinometer survived the construction phase and continues to be monitored to check the effectiveness of the remediation. Inclinometer measurements taken shortly after pavement removal showed large deflections near the surface, probably associated with the construction. We collected a new baseline measurement after construction was completed and have made several additional measurements. To date, approximately one year after construction, inclinometer measurements have shown little to no movements.

Additionally, the horizontal drainpipes could be monitored visually to view if they were actively draining water from the subsurface soils. Upon completing the installation, the westerly drains (drains 5 through 8 shown on Figure 5) began draining water immediately and have continued following ground anchor tensioning. The easterly drains (drains 1 through 4 shown on Figure 5) have not been observed to drain water to date.

The load cells were installed on three of the anchors near the end of construction and also been monitored since the design load was applied. To date, the cells have lost approximately 12 to 25 percent of their pretension load. Figure 12 is a plot of the load cell data.

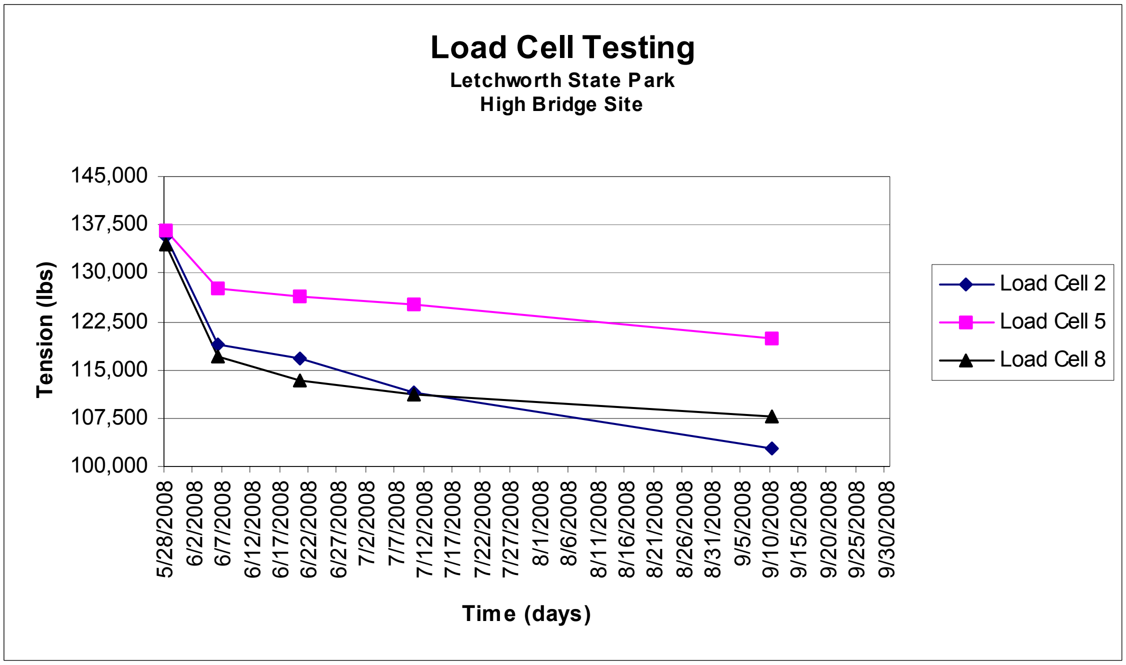

FIGURE 12

SUMMARY

The Portageville entrance road to Letchworth State Park had experienced subsidence for many years requiring numerous pavement repairs. By 2007 approximately 150 feet of the road was severely cracked and showed signs of significant movement. The slope conditions had deteriorated to the point where it was difficult to maintain traffic on the road.

An engineering study commissioned by OPRHP identified movement occurring at a depth of approximately 16 feet below the road. The movement was occurring through a zone of saturated silt lenses confined between layers of clay.

Traditional methods to stabilize the slope were considered but found impractical when considering the site conditions. A non-traditional approach utilizing reaction blocks and ground anchors was selected to improve the stability of the road and slope. The design included nine reaction blocks and ground anchors to provide a stabilizing force to the moving soil mass. Additionally, a trench drain on the uphill side of the road and horizontal drains below the road were added to the design. The upslope drain would minimize the water entering the soils below the road and the lateral drains would remove existing water trapped within the silt lenses and allow for water to be released during reaction block loading.

Inclinometer readings indicate that no detectable movement has occurred since construction was completed in the spring of 2008. The ground anchor load cell monitoring data shows some loss of pretension, however the data indicates that the loss of pretension is slowing. Visual observations of the lateral drains indicate that water is slowly draining from the slope. It is anticipated that monitoring of the inclinometer, load cells and lateral drains will continue for several more years.

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