DOUBLE NICKEL MICROPILE LANDSLIDE STABILIZATION

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ABSTRACT

In July 2011, the Wyoming Department of Transportation (WYDOT) desired to stabilize a recurring landslide located on Highway 28 between Farson and Lander, Wyoming in Fremont County. The landslide referred to as “Double Nickel” is located at Milepost 55.5 and has affected approximately 1,500 lineal feet of roadway section. The slope had experienced repeated events of movement with the most recent occurring in the spring of 2010 following a heavy snowfall and rapid melt. Several repairs and various roadway alignments had been tried in the past dating back to 1992 in an attempt to mitigate the slide. Using a consultant design and an accelerated design schedule, WYDOT completed their subsurface investigation and landslide stabilization design documents within five months. After evaluation of several landslide mitigation alternatives over 500 large diameter (12”) micropile “shear piles” were selected as the most technically and economically feasible design and construction solution. Bid documents were then finalized for a January 2012 bid letting with construction anticipated in the spring of 2012.

This paper will present and discuss descriptions of the historical landslide movements and repairs, environmental and geotechnical assessments for design and construction, slope stability analysis, landslide mitigation alternatives, micropile shear pin design features, site challenges, micropile installation, monitoring and performance (to date) of the landslide stabilization system.

INTRODUCTION AND BACKGROUND

Project Description

The Wyoming Department of Transportation (WYDOT) desires to stabilize a recurring landslide located on Highway 28 between Farson and Lander, Wyoming in Fremont County. Refer to Figure 1 for a site vicinity map. The landslide is referred to as “Double Nickel” and is located at Milepost 55.5. The slope has experienced repeated events of movement with the most recent occurring in the spring of 2010 following a heavy snowfall and rapid melt. Several repairs and various roadway alignments have been tried in an attempt to mitigate the slide. WYDOT contracted with HNTB Corporation (HNTB) to evaluate and design the landslide stabilization on an accelerated schedule with intentions to let the project by January 2012 with substantial completion by Fall 2012. Design requirements established by WYDOT included the following:

- The selected stabilization option shall maintain two-way, two-lane traffic during construction.

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- Improvements shall be constructed from within the existing right-of-way limits.
- The landslide stabilization shall be designed to a static factor of safety equal to 1.3 with a target factor of safety during a seismic event of 1.0+.

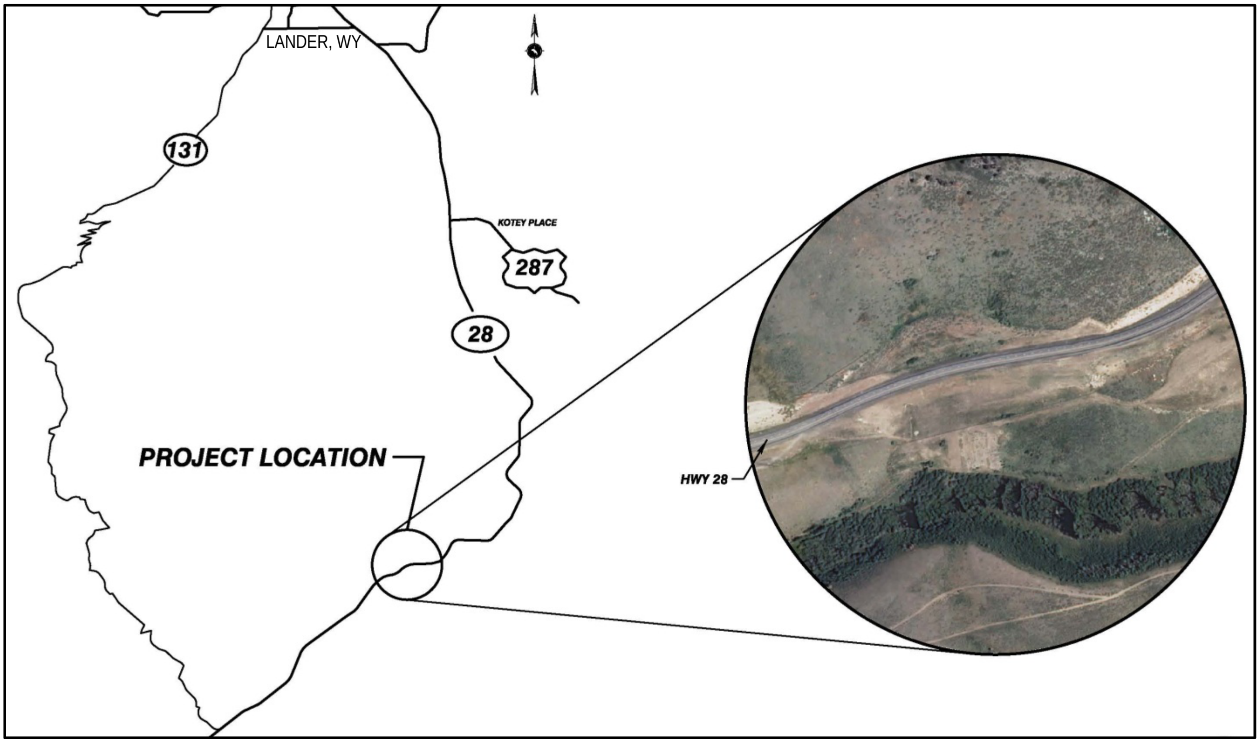

Figure 1. Double Nickel Site Vicinity Map.

LANDSLIDE HISTORY

Prior to 1985, Highway 28 was positioned further upslope from the current alignment. In order to improve geometrics, the roadway was realigned and "straightened". As part of the highway improvement design, WYDOT performed a typical roadway investigation consisting of numerous shallow borings drilled along the proposed alignment in 1981 and 1982. A significant landslide was documented to the west of the project site, occurring along a bentonitic clay layer at a depth of 3.7 to 4.2 m (12 to 14 ft). A spring was noted within the project limits about 4.6 m (15 ft) right of the proposed centerline. Roadway design included an underdrain to discharge the spring flow. Within the project limits, a significant amount of fill (12 m (40 ft)) was required to bring the proposed roadway to grade.

Movement was observed in the summer of 1987 and was originally attributed to excessive settlement of the fill. Several inclinometers were installed to monitor the movement. Further disturbance was noted in the summer of 1988 and additional inclinometers were installed. Continued movement indicated the development of a landslide rather than settlement of the fill. Additional investigations were conducted in August 1991 and July/August 1992.

Three toe trench drains were installed in 1992 immediately down slope of the spring location. The design length of the trenches was 36.6 m (120 ft) with a maximum footprint width of 15.2 m (50 ft) at the top. A drainage trench was installed at the base of the toe trenches. As-built documentation indicates that the slide plane was intercepted in the back slope of the trenches at the right-of-way limit. Backfill material consists of large diameter, iron ore rock from a nearby quarry.

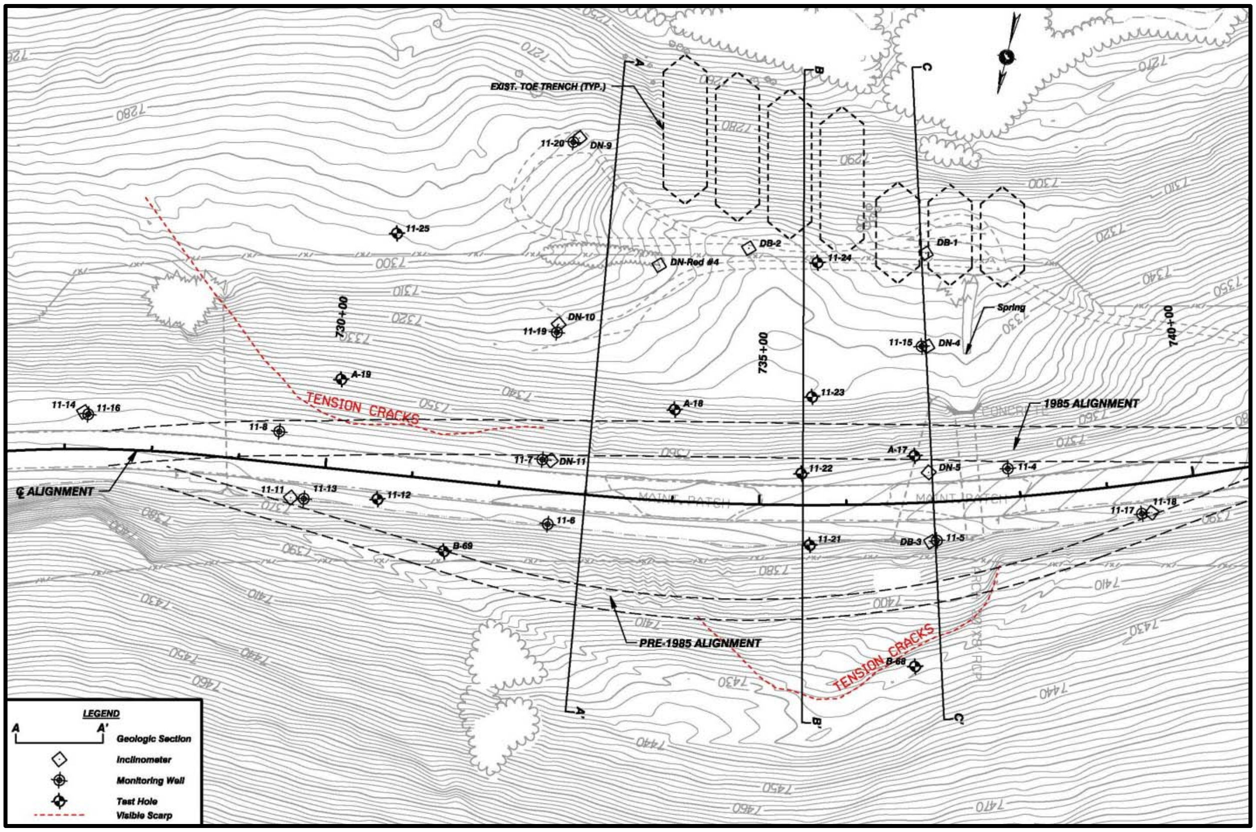

Figure 2. Site and Boring Location Plan.

Continued movement after the installation of the toe trench drains in 1992 necessitated a more extensive landslide repair. In 1994, WYDOT designed a landslide repair including realignment of the roadway to the north (i.e. upslope), placement of lightweight tire shred fill, and installation of four additional rock-filled toe berms. Furthermore, a three-sided box culvert was constructed to collect and discharge the spring water. Refer to Figure 2 for a plan view illustrating the various roadway alignments and location of previous improvements.

Movement of the slope requiring frequent maintenance is on-going. After the 1994 improvements, three inclinometers were installed in December 1995. Two more inclinometers were installed in April 2000. Heavy snowfall followed by a rapid snowmelt in the spring of 2010 aggravated the slide mass with new tension cracks forming 76 m (250 ft) east of the previously defined active landslide. In response, WYDOT installed additional inclinometers in June/July 2010. WYDOT conducted a more extensive investigation in the spring of 2011 including drilling 16 borings and installing open standpipe piezometers and inclinometers.

SUBSURFACE INVESTIGATIONS

As discussed above, WYDOT has completed numerous subsurface investigations directed primarily at installing instrumentation to monitor the site. Hence, limited samples of the soil and bedrock were collected and drilling methods were selected to achieve target depths for instrumentation installation. Characterization of the subsurface materials was based primarily on evaluation of cuttings and drill rig response. WYDOT installed casing for displacement monitoring in selected boreholes using Slope Indicator equipment. Several open standpipe piezometers were installed to monitor groundwater levels.

As part of the landslide stabilization design, HNTB and their sub-consultant Wyllie & Norrish Rock Engineers, Inc. (W&N) developed a subsurface exploration program in cooperation with WYDOT to achieve the following objectives:

- Investigate and characterize the engineering and geologic subsurface conditions;
- Install multiple-level vibrating wire piezometers to characterize pore water pressure;
- Identify potential shear planes; and,
- Characterize the structural integrity of the bedrock using an acoustic and optical televiewer instrument.

Boring locations were selected by WYDOT, HNTB, and W&N and staked in the field during a site reconnaissance on July 14, 2011. Four borings were located along the inferred axis of the landslide. A fifth boring location was selected to fill in an apparent gap in the existing boring information. Prior to drilling, WYDOT surveyed the location and elevation of the previously installed instrumentation (inclinometers and piezometers) and the proposed borings. The subsurface investigation was completed by WYDOT forces between August 2 and 12, 2011.

PROJECT GEOLOGY AND EXISTING SUBSURFACE CONDITIONS

The project site is underlain by soil overburden on top of the Tertiary age White River Formation. The White River consists of calcareous conglomerate and tuffaceous sandstone. An erosional unconformity is present between the younger White River Formation and the adjacent Pennsylvanian-aged Tensleep Sandstone and Amsden Formation to the north. Bedrock generally dips toward the northeast at 10 to 12 degrees. Significant structural features are not noted in the area.

Overburden materials encountered during drilling consist of fill, colluvium, and residual soils. Shredded tire fill from the 1994 repair was noted in several borings as was iron ore rock fill associated with the toe trench construction. The colluvium and residual materials vary considerably in composition but are generally described as stiff to hard, brown to reddish brown, gravelly clay and medium dense to dense, brown to gray, clayey gravel with sand, cobbles, and boulders. Moisture content of the overburden generally increases with depth. The fine-grained fraction is generally plastic.

Underlying the overburden is relatively soft, fine-grained bedrock consisting of shale, siltstone, and claystone. Harder sandstone and limestone bedrock is typically present in thin nodular seams. Due to the soft bedrock initially encountered in the borings at the site, the top of bedrock contact is not well defined and gradational as the weathering intensity decreases with increasing depth. The top of bedrock plan indicates a depression in the vicinity of Sta. 735+50 that corresponds to an erosional draw identified in a 1982 aerial photograph that was subsequently filled in for the 1985 roadway realignment. Overburden thickness ranges between approximately 3 and 23 m (10 and 75 ft) based on the available boring information. As indicated by the top of bedrock contours, the thickest overburden section is located near the axis of the landslide.

Groundwater was encountered during the subsurface investigation, and water levels were also measured after completion of drilling. Delayed groundwater levels were generally higher than the noted level of groundwater entry, indicating an upward hydraulic gradient (artesian conditions).

GEOLOGIC INTERPRETATION AND SLIDE GENESIS

In general, the suspected slide plane (determined from inclinometer readings and direct observation of disturbed material) is positioned at or near the top of bedrock. The maximum depth of the slide plane below the ground surface is approximately 24.5 m (80 ft) at the axis of the landslide at approximately Sta. 735+50 (Figure 3). The depth to the slide plane decreases to about 18.5 m (60 ft) on the side flanks of the slide.

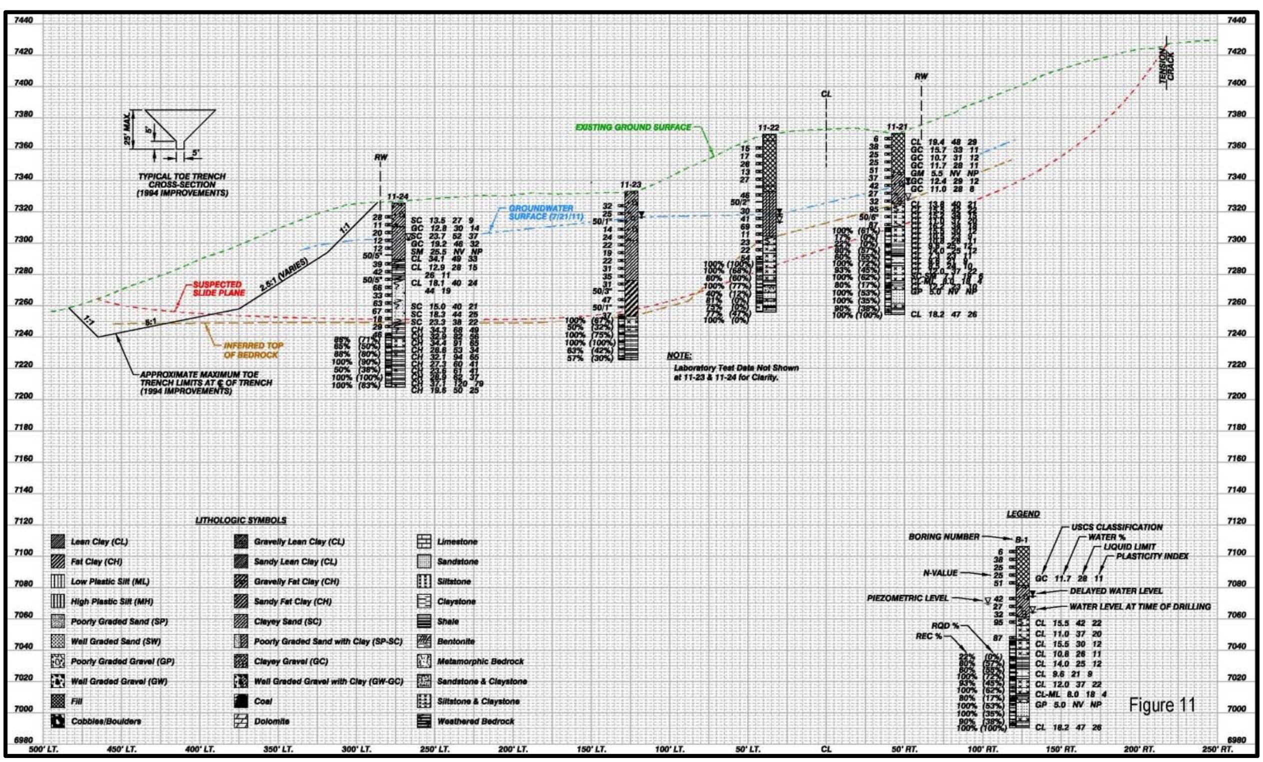

Figure 3. Cross-Section at Slide Axis (Sta. 735+50).

The slide is most influenced by the White River Formation, consisting of Tertiary-aged, slightly indurated, tuffaceous sandstone and shale. The White River is described in the literature as being ash rich. Thus, it is believed to contain numerous bentonitic layers and consist of an overall weak material. The Tertiary period (30 million years before present (ybp)) is characterized by the Laramide Orogeny and filling of the structural basins by sediments eroded from the mountains. The White River Formation is underlain and flanked by stronger Paleozoic-aged (300 to 400 million ybp) bedrock of limestone, shale, and sandstone. The Tensleep Formation is described as a cliff forming sandstone, is geometrically parallel to the strike, and may have formed detached blocks. These blocks may have detached laterally from the formation and moved down slope prior to Tertiary time. These blocks are not believed to be the cause of or contributing to the failure; however, the slide itself may be made up of en echelon blocks.

Groundwater is likely transmitted down dip with considerable head from southwest to northeast, downward from the continental divide to the site. When examined from along the strike, the Madison Formation could be directly contributing water laterally in the elevation range of 7250 to 7300 feet, further weakening the White River materials and providing a mechanism for slide movement.

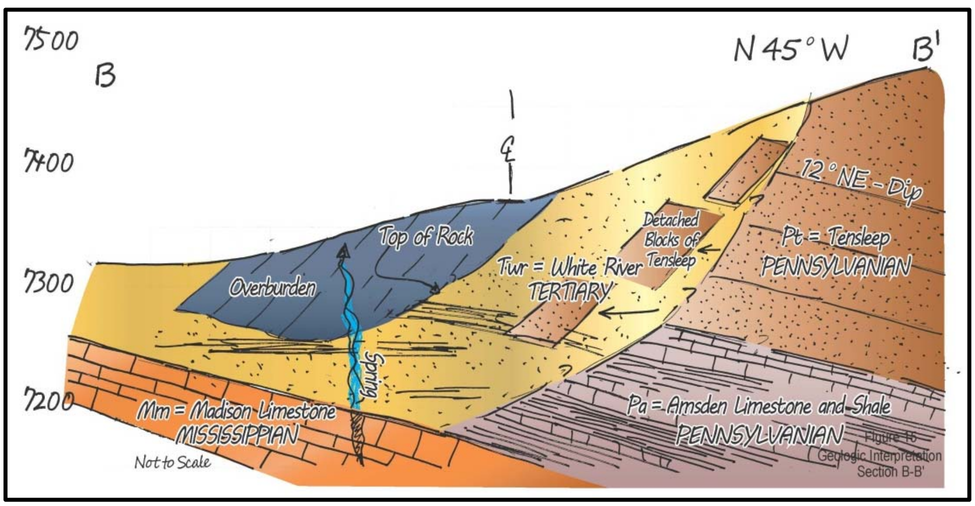

Figure 4. Geologic Interpretation of Slide Genesis

In summary, it is believed the slide is occurring in overall weak Tertiary materials aided by bentonitic zones and weathering of the shale layers to fat clay. The axis of the slide appears to be positioned within a bedrock depression from a past erosional event. The mechanism is triggered by the addition of groundwater from the Madison Formation as well as along the contact between the Tertiary and Paleozoic formations. Surface water is also contributed near the present head scarp of the slide.

SLOPE STABILITY EVALUATION

An assessment of the landslide was performed to evaluate potential landslide movement under measured or inferred engineering parameters for the site. These parameters include slide geometry, measured groundwater levels, and material properties. Stability model(s) were developed to evaluate the ratio of resisting forces to driving forces expressed as a factor of safety (FS). A calculated FS slightly less than unity would be consistent with landslide movement. Slope stability evaluations were conducted using Slide6.0 (version 6.012) software developed by RocScience.

Two geotechnical models for stability analyses were developed representing a cross section through the axis of the observed slide and another section on the side flank. Initially, a simple two-layer model was analyzed consisting of soil overburden and bedrock. A path search for non-circular surfaces was conducted and the critical slip surface originates at the toe of the slide with a calculated FS of 1.25.

Based on direct observation of disturbed materials along the inferred slide plane, depths of recorded inclinometer movements, and laboratory testing, a "pre-sheared" layer was incorporated into the model. The residual friction angle of this material was varied until the global FS was near unity. The bedrock was assigned an anisotropic strength function to incorporate weakness along the bentonitic bedding planes dipping into the hillside. Overburden material above the pre-sheared layer that may contain relict structure was also assigned the anisotropic strength function to account for potential bentonitic planes of weakness. Finally, the toe trenches present at the base of the slope (1992 and 1994 improvements) were included in the model with composite material properties from the overburden and rock fill. The following figure illustrates the anisotropic strength functions used in the analysis.

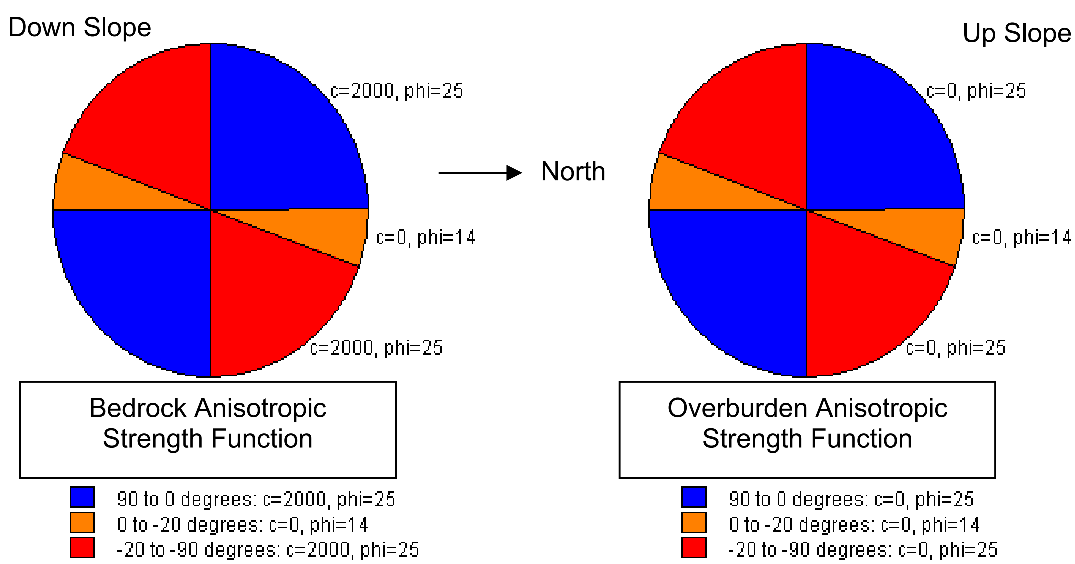

Figure 5. Anisotropic Strength Function.

In order to achieve a FS of unity (i.e. on the verge of failure), the residual friction angle of the pre-sheared layer and along potential bedding planes in the bedrock was lowered to 14 degrees. This is in good agreement with available residual direct shear test results on samples at or near the inferred slide plane and WYDOT's experience. A residual friction angle of 14 degrees also correlates well with published relationships based on Plasticity Index (PI). The same material parameters were then used in the stability model along the side flank of the slide.

CONCEPTUAL STABILIZATION ALTERNATIVES

General methods for landslide stabilization include slope grading, groundwater control, and structural reinforcement such as micropiles, drilled shafts, ground anchors, and driven piles. For the Double Nickel site, slope grading would likely have required the acquisition of additional right-of-way. Groundwater control through the installation of horizontal drains would also have required additional right-of-way and would be difficult to install due to the length of the drains and the subsurface conditions. Other active dewatering options such as a pump station would require utility improvements to the site and on-going maintenance. In order to meet the design criteria and schedule established by WYDOT, the preferred stabilization technique was structural reinforcement.

Based on the existing subsurface conditions and the high potential to encounter cobbles and boulders, driven piles and drilled shafts were not desirable. Driven piles would require a high percentage of pre-drilling to install the piles to the minimum tip elevation. Drilled shafts are rigid structural elements that would require fabrication of long reinforcing steel cages. Use of temporary or permanent casing would likely be required. Ground anchors require construction of structural concrete bearing blocks to distribute the load over the ground surface. Due to the low strength of the bedrock present at the site, numerous long anchors would likely be required. Rigorous testing is required to verify load carrying capacity. In addition, the most efficient location of ground anchors would be within the slope beneath the existing roadway level. Drilling angled holes for the insertion of the ground anchors may be difficult due to the presence of cobbles, boulders, and potentially shredded tire fill. Preliminary cost estimates indicated that the ground anchor option was cost prohibitive.

The preferred stabilization method was shear piles (micropiles) consisting of small diameter flexible elements installed across the slide plane to provide passive shear capacity, disrupt the slide plane, and reinforce the soil mass. Due to the depth of the slide plane below the ground surface and to reduce potential drilling difficulties, the shear piles were assumed to be installed vertically. By limiting the drill hole diameter to approximately 305 mm (12 in), it was assumed that the holes could be advanced utilizing rotary drilling methods that advance casing with the bit (i.e dual rotary drilling). The structural elements can be readily installed down slope of the existing roadway, primarily from the bench located at approximately elevation 7330 feet, in order to minimize impacts to traffic.

MICROPILE (SHEAR PILE) ANALYSIS

The stability model at Sta. 735+50 (design section) was used to evaluate the resisting force needed to improve the existing factor of safety from unity to the minimum required of 1.3. Using Slide 6.0, the required resisting force for a single row of piles to increase the factor of safety to 1.3 was 4,375 kN/m (300 kips/lf) of slope.

In order to reduce the resisting force per structural element, a total of four rows of piles were analyzed. Several iterations were conducted to optimize the spacing and to address up slope and down slope stability. The final configuration consisted of one row of piles near the crest of the lower toe slope and three rows on the bench below the existing roadway. Due to inefficiencies associated with spreading the rows out from the center of the slide mass, the required resisting force per row increased slightly to 1,108 kN/m (76 kips/lf).

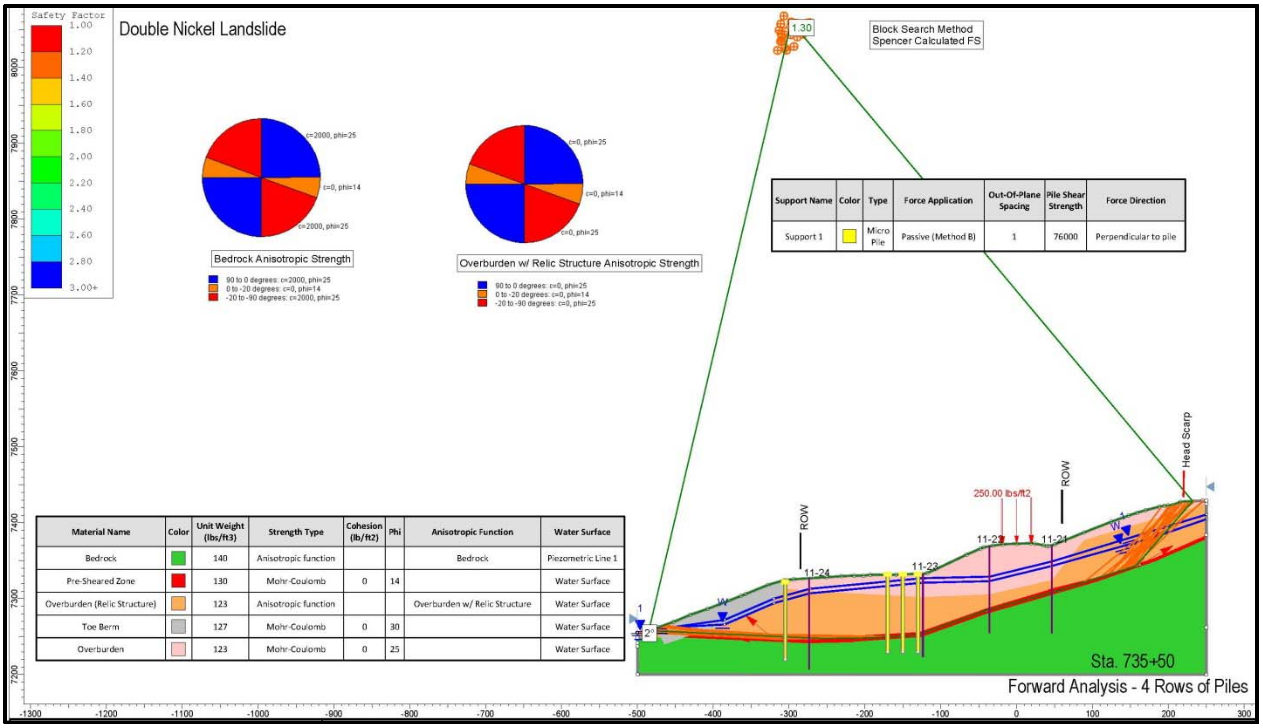

Figure 6. Slope Stability Model with Shear Piles (Micropiles).

The subsurface conditions along the section at Sta. 735+50 (design section) were used to develop the lateral loading parameters. LPile 6.0 software by Ensoft Inc. was used to model the piles under lateral load. Various methods to analyze piles for stabilizing a moving soil mass are presented in literature. Per FHWA's Drilled Shaft Manual, the lateral load applied to the structural element is the lesser of the maximum passive earth pressure of the moving soil or the resisting force necessary for overall stability of the slide mass. Using the soil parameters entered into LPile and the associated p-y curves, the ultimate passive earth pressure corresponding to approximately 63.5 mm (2.5 in) of displacement is 4,467 kN (1,005 kips) per pile. Based on a 1.8-m (6-ft) pile spacing for four rows, the resisting force needed per pile is 2,027 kN (456 kips). Thus, the piles were designed to resist a force of 2,027 kN (456 kips) corresponding to less displacement. Per Reese and Van Impe (2011), the 2,027-kN (456-kip) force can be applied as a triangular pressure distribution against the pile above the slide surface for a pile weak in bending. Due to the depth of the slide plane and overburden confinement, "flow" of the soil around the piles at the 1.8-m (6-ft) spacing was not anticipated. In addition, the material on the down slope side of the pile was assumed to remain in-place and provide resistance along the length of the pile (i.e. the piles were considered a soil reinforcing element). To account for variability of the soil mass and potential disturbance during installation of the shear piles, a reduction factor was applied to the lateral resistance of the soil mass on the down slope side of the pile.

Based on the loading conditions presented, LPile was then used to evaluate the structural response of the pile including shear force, bending moment, and deflection. The structural capacity at the threaded joints was calculated based on guidance provided in FHWA's Micropile Design and Construction Manual and was approximately one-half of the values calculated for the 302-mm (11.875-in) diameter casing with a wall thickness of 15 mm (0.582 in). Due to the depth of the slide plane, bending moment controls for a pile loaded with a triangular pressure distribution. Comparing the maximum calculated bending moment and shear force obtained from LPile and the structural capacity of the shear piles, the casing (F_y = 80 ksi) and joints are adequate in both bending and shear based on a triangular pressure distribution.

Where the slide plane is well defined at the center of the slide mass, a check was made considering the structural capacity of the shear pile if the 2,027 kN (456-kip) load was applied as a concentrated force at the slide plane. In this case, the shear capacity controls and the joints do not provide sufficient capacity. Therefore, restrictions on joint locations were included in the contract plans where the slide plane was well defined along the axis of the slide.

FINAL DESIGN RECOMMENDATIONS

Based on the analysis presented above, the following graphics summarize the design recommendations for the shear piles.

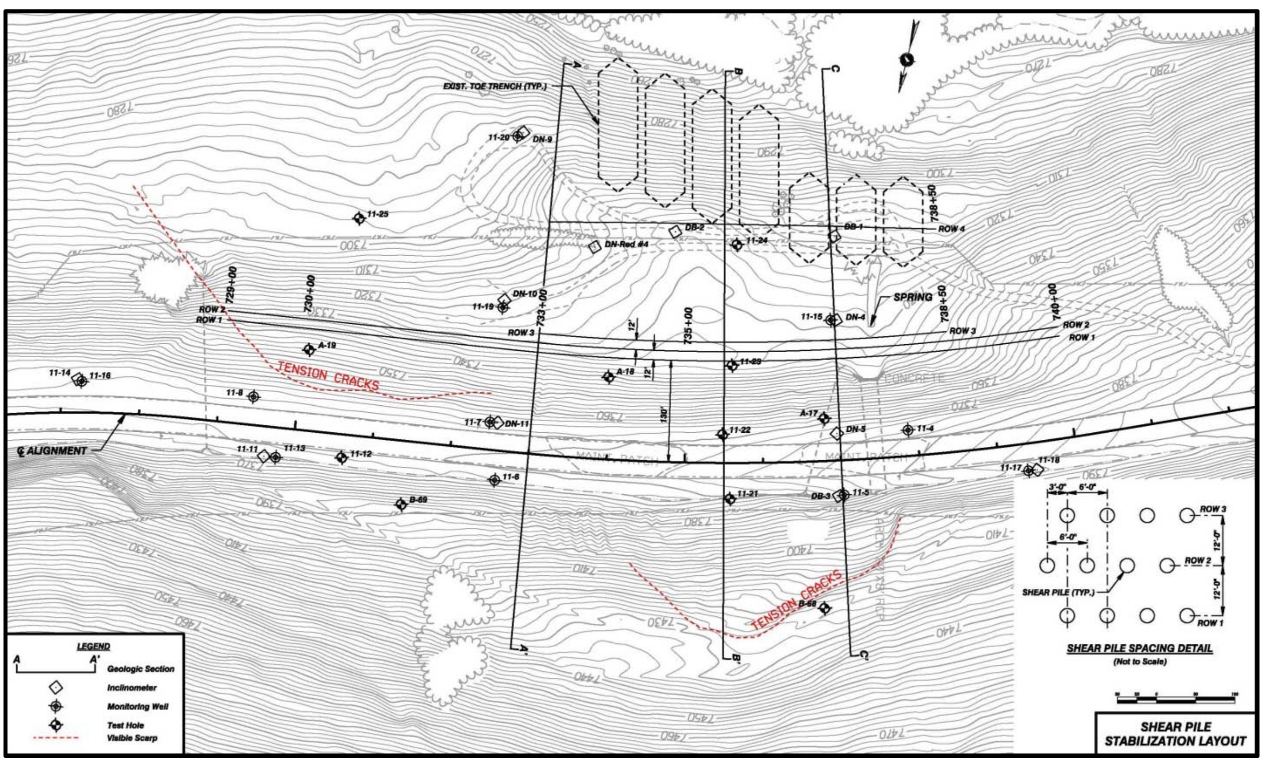

Figure 7. Shear Pile Stabilization Plan Layout.

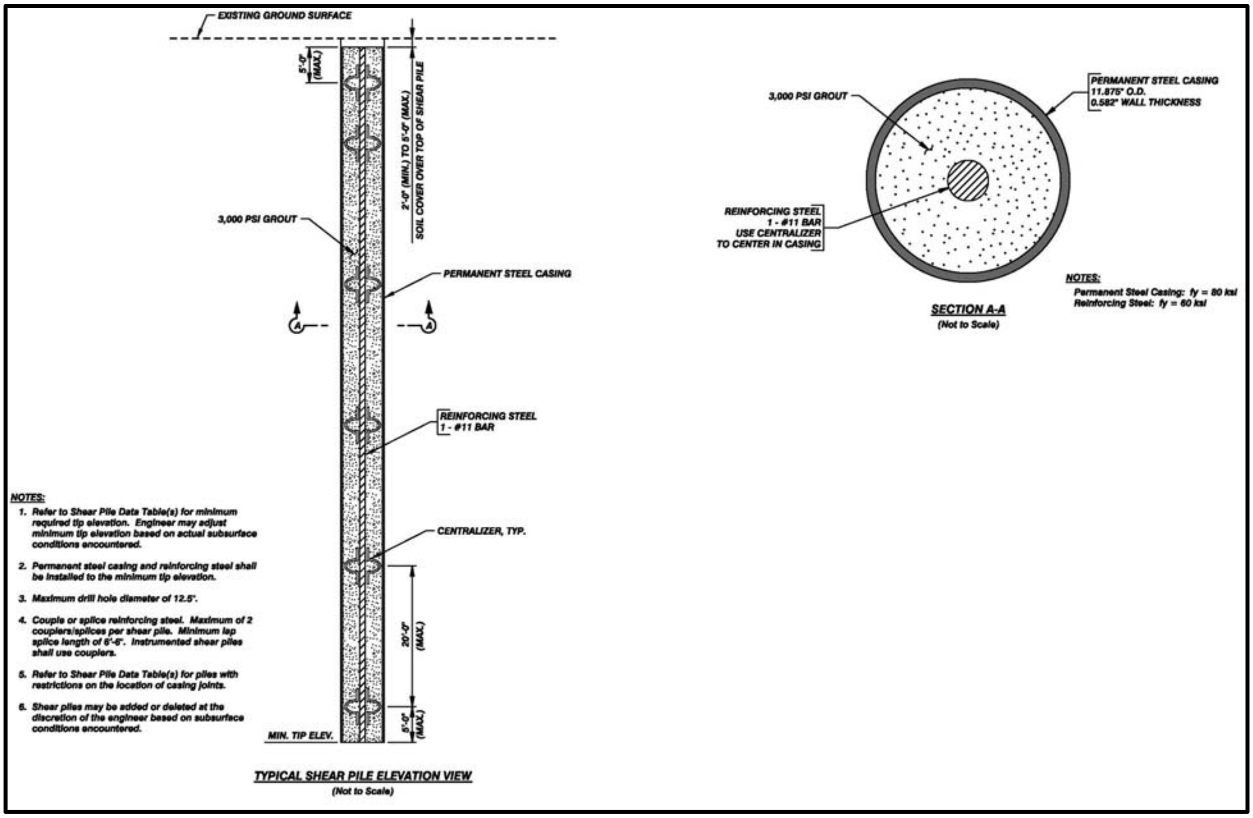

Figure 8. Typical Shear Pile Details.

As previously discussed, various methods of analyzing and designing shear piles are presented in the literature. Reasonable assumptions were made with respect to the soil-pile interaction in order to minimize the structural element required and provide a cost-effective repair. Based on the assumptions made during the analysis of the shear piles, additional instrumentation including inclinometers and strain gauges are required within the shear pile casing at select locations to monitor the response of the shear piles.

CONTRACT AWARD

Bids for the Double Nickel Slide were opened on January 12, 2012. A total of seven bids were submitted and ranged from \$5.884 million (US\$) to \$6.983 million with an engineer's estimate of \$6.177 million. The contract was awarded to Donald B. Murphy Contractors, Inc. of Federal Way, Washington.

CONSTRUCTION

Planning and Preparation

Drill Rigs

During the planning process several factors dictated the choice of equipment. Since the shear pile design required the 11 7/8" OD permanent casing to be advanced as the hole was drilled, it was decided to equip the drill rigs with two rotary heads. The upper rotary was set up to turn the inner drill rod, and the lower one was used to rotate the permanent casing.

Figure 9. Double Rotary Drill Head

Additionally, the specifications required that over 35% of the shear piles be installed with no casing joints between Elevations 7240 and 7260. This restriction, along with the large diameter (11 7/8") of the casing, necessitated the use of large rigs with high torque and long stroke capability. Two ABI 18/22 rigs were selected, and were outfitted with dual rotary drives.

Permanent Casing

The plans specified a minimum tip elevation for each shear pile, with the top of pile required to be between two feet and five feet below the existing ground elevation. This resulted in pile lengths ranging from a minimum of 78.2 feet to a maximum length of 101.6 feet. The average pile length was 91.3 feet.

One of the challenges in procuring the permanent casing was working out combinations of lengths of casing that would meet the minimum shear pile lengths specified in the contract drawings, while minimizing excess length, which would increase material costs and drilling time, without additional compensation from WYDOT.

In addition, use of the dual rotary drilling method required that the lengths of the permanent casing pieces be matched with the lengths of the inner drill rod sections. A significant effort went into optimizing this geometry and minimizing the number of different length pieces of casing.

Additional challenges in casing procurement resulted from the quantity of material (48,000 LF) required for the project. Typically, casing used in the U.S. micropile industry is API oilfield pipe that does not meet the rigid manufacturing specifications established by API for oilfield work. The price for this “mill secondary” casing is substantially less since it is not usable for its primary purpose.

Initial Drill Tooling Selection

Based on the information in the geotechnical report, the initial drill tooling selected for the project consisted of carbide-tipped “J” teeth welded to the lead end of the 11 7/8” OD permanent casing for the outer string, along with an aggressive claw-type bit with carbide-tipped “bullet” teeth coupled onto the 7 5/8” OD drill rod for the inner string. In addition, due to the known presence of the rock fill in the toe drains for some of the shear piles located in Row 4, an 8” downhole hammer and button bit set-up was also selected. It was anticipated that this percussive set-up would need to be used in selective locations to deal with the toe drain rock fill, as well as some of the hard sandstone seams identified during the geotechnical investigation.

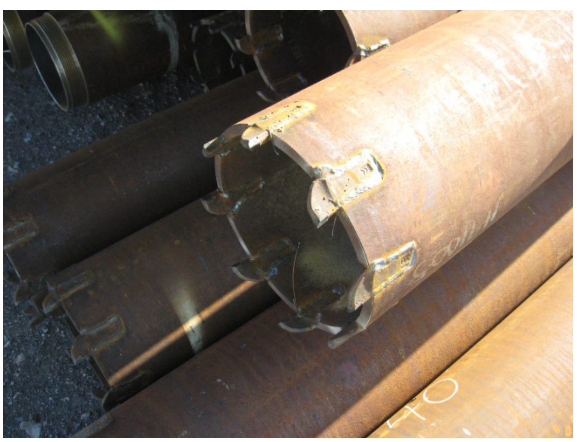

Figure 10. Initial Lead Casing Set-Up.

Figure 11. Claw Type Bit.

Support Equipment

The primary flushing medium selected for the drilling was air, with the intention to add water (and possibly clay inhibitor) as necessary to maintain adequate flush. The geotechnical investigation identified overburden consisting primarily of silts and clays which have a tendency to “collar off” the annulus between the inner rod and the casing, thereby hampering advancement of the drill hole. With this in mind it was anticipated that the amount of water added to the air flush would need to be adjusted by crews in the field to maintain productivity.

An additional consideration in selecting the air compressor configuration was the elevation at the site. The project site was located at an elevation of 7,300 feet above sea level. At this elevation, the rated volume of an air compressor is significantly reduced. In order to maintain adequate uphole velocity of the drill cuttings between the inner drill rod and the permanent casing, the job was set up with three each 900 CFM / 350 PSI portable air compressors. All three units were attached to a single steel header pipe for distribution to the two drill rigs.

In addition, the length and weight of the drill rod and casing required special handling equipment. Casing was handled with a service crane and special rigging. The drill rods were handled with hydraulic excavators equipped with pipe clamps for loading the pipe into the casing during installation, and for removing the drill rod as it was "tripped out" after completion of drilling each hole.

Figure 12. Casing/Drill Rod Handling

Grouting

The contract required the 11 7/8" OD permanent casing be backfilled with cement grout. With the large quantity of piles for this project, an automated grout plant was selected to efficiently mix and pump the neat cement grout. A Scheltzke MPS 510 grout plant was chosen, and was set up with twin silos to allow use of bulk cement.

Site Preparation

Access and Benching

The contract drawings required the top of each shear pile to be installed to an elevation between two feet and five feet below existing ground. With the sloping topography at the jobsite, and with the size of the drill rigs required to install the permanent casing, a significant amount of grading work was required to create stable and relatively level access to each of the 526 shear pile locations.

During this grading work, numerous boulders were encountered near the surface. One of these boulders had a 2” diameter hole drilled through it, indicating it was “shot rock”, or rubble created from a blasting operation. This condition was not identified in the geotechnical report

Figure 13. Grading For Drill Benches.

Spoils Control

It was anticipated that there would be a significant volume of dirty water and wet spoils generated during 48,000 LF of shear pile installation. The site was set up with a settling pond at the low end of the site where waste water could be channeled and contained. In this same area, a location for spoils disposal was selected.

Shear Pile Installation

Shear pile installation started on the lower Row 4 piles as required by the sequence in the contract documents. The presence of the rock fill in the toe drains in this area was known, and it was anticipated that this condition would present challenges during installation of these piles. This was certainly the case.

There were challenges in the early stages of production at Row 4. In two instances, the casing could not be advanced to the required tip elevation. Initially, these challenges were attributed in part to the learning curve with the new equipment and drilling system, as well as the known ground conditions at Row 4. Various modifications were made to the drill tooling, including changes to the bit configuration and the type and quantity of cutting teeth on the permanent casing. In addition, the button bit and downhole hammer set-up was used on several shear piles during this period of the project. While production improved somewhat with these adjustments, it was still not adequate to meet the schedule requirements for the project.

Based on the experience gained during this initial production period, a new tooling system was selected. This system consisted of a 10” downhole hammer paired with an underreamer bit with a maximum diameter of 12 ½”. This system required the permanent casing to have a “blank” end with no cutting teeth. This system resulted in a much more reliable production rate than was achieved previously.

Figure 14. Down-The-Hole Hammer and Underreamer

However, this particular system was available in limited supply. With the remaining quantity of drilling left on the project, it was necessary to develop an additional tooling configuration that was available in adequate supply to allow the project to be completed on schedule. This additional system consisted of a 10” downhole hammer paired with a 10” button bit. The lead section of casing was modified with thicker carbide “J” teeth than originally selected. Various modifications were made to the quantity of the cutting teeth on each lead section of casing during the course of pile installation

Figure 15. More Aggressive Casing Leads

These two different systems were used for the remainder of the project. As a result of implementing these percussive systems, additional compressed air volume was required. A fourth compressor was added to the system so each drill rig would be supported by a pair of 900/350 air compressors.

During the shear pile installation with both of these systems, it was noteworthy that the downhole hammer would fire consistently through the clay and silt layers in addition to the rock formations indicating the continuous presence of harder materials. This was the case with either of the two systems (underreamer or button bit) used during the production phase of the work.

Schedule and Production Summary

Installation of the 526 shear piles was completed in 15 weeks. The first three weeks and the final two weeks included one drill rig working a single shift. The other 10 weeks of production were performed with two drill rigs working a single shift, or one rig on a two shift basis. The shift structure varied as a result of rig maintenance and/or tooling availability.

Once 75% of the piles were installed, reconstruction of the roadway began. Re-grading of the drill benches and final site restoration activities followed immediately after shear pile installation was complete. Despite the challenges encountered early in the project, the job was finished within WYDOT's budget and construction schedule specified in the contract.

Figure 16. Shear Pile Installation

INSTRUMENTATION AND MONITORING

As required by the construction contract, instrumentation consisting of sister-bar strain meters and inclinometer casing was installed within six shear piles across the site. The sister-bar strain meters were installed as upslope and downslope pairs at specific elevations above, at, and below the suspected slide plane. Inclinometer casing was installed full-depth within the shear pile. The table below summarizes the instrumented shear piles.

Table 1: Instrumented Shear Piles Summary.

Shear Pile No.	Strain Meter Pair Elevations (ft)	Inclinometer Casing
SP1-30	7310, 7300	Full-Depth
SP1-109	7270, 7260, 7255, 7250	Full-Depth
SP1-150	7270, 7260	Full-Depth
SP2-130	7280, 7275, 7270	Full-Depth
SP3-10	7285, 7280, 7275	Full-Depth
SP4-40	7265, 7255, 7250, 7245	Full-Depth

Instrumented shear pile SP1-109 and SP4-40 are along the inferred axis of the slide plane. SP1-150 and 2-130 are on the right flank near the spring location while SP1-30 and SP3-10 are positioned on the left flank of the slide.

The strain meters are connected to data loggers and are set to take readings on a daily basis. Acquisition of strain meter readings began within a few days after installation with the baseline “zero” reading established approximately 28 to 30 days after installation to account for curing of the grout within the shear pile casing. The inclinometers are manually read by the Department. Initial baseline readings of the inclinometers were obtained in October 2012. Subsequent readings of the inclinometers were obtained in May 2013 and October 2013 along with downloading of the strain meter data loggers. Key observations to date include the following:

- The maximum calculated strain for the shear piles is approximately 5,000 με. The maximum recorded strain at any strain meter is less than 300 με (less than 6%).
- The maximum movement indicated by the inclinometer readings is approximately 2.5 mm (0.1 in.)
- Negative strain readings indicate compression while positive strain readings indicate tension. The strain meter data at SP1-109 illustrates the anticipated shear pile response with the upslope strain meter at elevation 7270 ft indicating tension and the corresponding downslope strain meter indicating compression. Strain meters at lower elevations indicate essentially no differential loading between the upslope and downslope pairs. The inclinometer data at SP1-109 indicates approximately 2.5 mm (0.1 in.) of movement in the A_o direction at elevation 7270, supporting the strain meter data.

CONCLUSIONS

The designed landslide stabilization met the aggressive schedule established by WYDOT and necessitated by the typical weather conditions at the site. The innovative use of vertically installed micropiles to provide passive shear resistance, disrupt the slide plane and reinforce the soil mass permitted rapid installation at a remote location in a short construction season. Initial production rates were slowed by the variable subsurface materials and delivery of materials and equipment to the site. Acquisition of different drill bits and modification to the cutting teeth on the casing improved production. Daily production rates increased in mid-June and installation of the 526 shear piles, over 14,600 m (48,000 ft), was complete by the end of August 2012. Instrumentation designed and installed within the production micropiles will provide invaluable insight into the response of the system. The combination of inclinometers and previously-installed piezometers will monitor the geotechnical parameters and pinpoint the depth of slide movement while the strain meters will monitor the structural response of the vertically-installed micropiles. The measured strain of the micropiles can be converted into load to better understand the mechanics of load transfer between the moving slide mass and the structural elements. To date, the instrumentation indicates stable conditions with no significant movement since the shear piles were installed.