SIGNIFICANT SAVINGS FROM PHASED INVESTIGATION AND DESIGN APPROACH TO LANDSLIDE REPAIR

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Abstract: In July 2001 a series of severe storms with extreme rain and winds resulted in heavy flooding in West Virginia. The flooding caused damage to several roads and bridges in six counties. A significant landslide developed along a 600-foot section of Cunard River Access Road on the New River Gorge National River creating an 85 feet high sheer face. Flood water reshaped a 27-foot high waterfall located at the base of the slide and formed a large scour pool. Removal of material at the slope face and water flowing through the slope contributed to the slide movement.

The subsurface field investigations to obtain soil and rock strength properties for the design progressed in several phases because of the difficulty accessing the unstable edge and toe of the slope. A proposed slide repair design was completed quickly and met environmental and aesthetic constraints and requirements for maintaining of public access during construction and maintaining the original alignment. The proposed slide repair design consisted of an anchored soldier pile wall at the toe of the slope and reinforced soil slope (RSS) for the embankment. The need for additional subsurface information at the toe of the slope for the design of the toe wall resulted in a third phase of field investigations. Observations during the progress of the design and the third phase of field investigations indicated the possibility for an alternative design that could provide significant construction cost and time savings.

The alternative design eliminated the toe wall and most of the RSS and focused on use of a rock ledge two-thirds of the way up the slide scarp. A fourth phase of field investigations was immediately performed in order to confirm persistence of the rock ledge. The ledge was found to be persistent and the alternative design was subsequently constructed with a cost savings of 50 % compared to the original proposed repair. This shows that a phased investigation and design approach can maximize the potential for identifying alternative concepts and, thereby, reduce costs.

INTRODUCTION

In July 2001 a landslide occurred on a section of Cunard River Access Road (Cunard Road) located in the New River Gorge National River (NRGNR) in Fayette County, West Virginia (Figure 1). The road closure affected the Park's visitors and the tourism industry because access to the boat ramp and the starting area for white water raft trips was not possible. A narrow one lane emergency access road was opened shortly after the flooding event along the cut side of the roadway embankment to provide temporary access to the boat ramp.

This paper is a case history of the repair of a dramatic landslide in an area of extreme environmental sensitivity and it presents the important finding that thorough, phased investigation, keeping repair options open, and advancing multiple designs simultaneously resulted in considerable savings without impact to the schedule. The landslide investigation and its repair were conducted by the Federal Highway Administration, Eastern Federal Lands Highway Division (EFLHD) and its contractors.

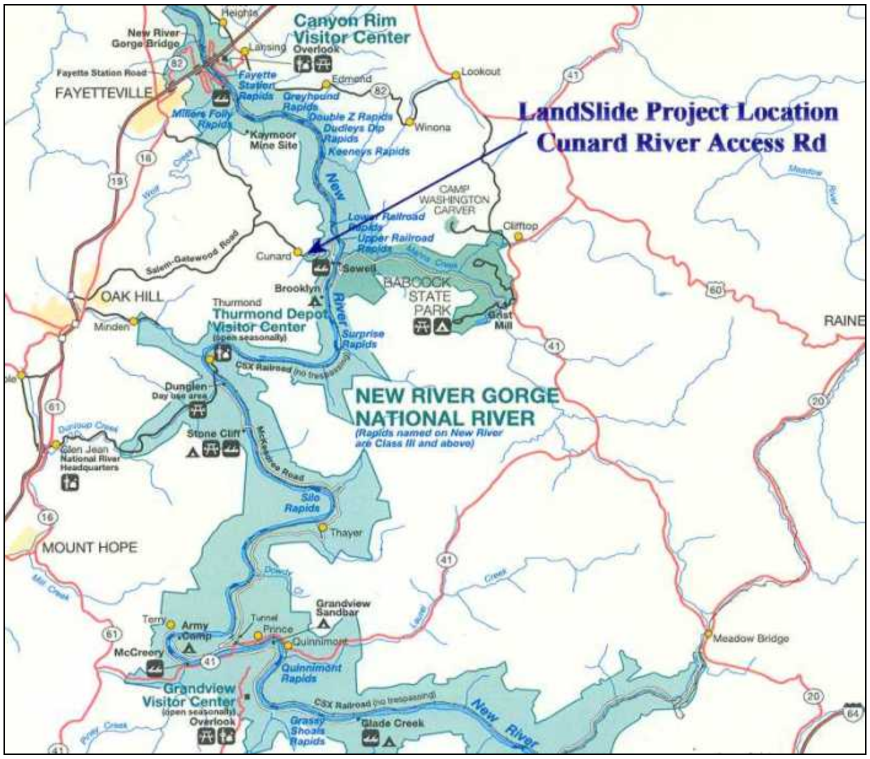

Figure 1. Project Location Map

LANDSLIDE DESCRIPTION

The landslide occurred along a 600 feet section of Cunard Road located near the beginning of the upper end of the 1.5 mile long road. Cunard Road is a narrow gravel road with grades of up to 15%, which runs nearly parallel to Coal Run, a tributary to the New River. A large volume of material consisting of soil, weathered rock fragments, coal slag from previous mining operations and trees had slid from the face of the slope into Coal run creating an 85-foot high scarp and slip surface. A long crescent shaped crack extending a few feet from the centerline had also developed along the roadway (Figure 2).

Figure 2. Initial Landslide crack developed through the roadway embankment

The initial crack width which was measured in the morning of the landslide event varied from 1/8th of an inch to a maximum of 4 inches with a maximum settlement of 3 inches. The crack width progressed within a short period of time to a maximum width of more that 1 foot and maximum settlement of 4 feet. Soils continued to slough from the face of the slope for several months.

A waterfall located near the upstream side of the slide area was reshaped by the action of the flood water. The newly developed 27-foot high waterfall face now consists of a 5-foot upper layer of horizontally bedded sandstone underlain by weathered shale and residual soil. A deep scour hole developed at the bottom of the waterfall within the weathered shale of the stream bed. The pool that was created by the scour hole had a maximum depth of 12 feet (Figure 3).

Figure 3. Flood reshaped waterfall located near upstream end of landslide

REGIONAL GEOLOGY

The landslide site is located within the Appalachian Plateau Physiographic Province and is underlain by rocks from the New River, Pocahontas, and Bluestone formations of the Pennsylvanian and Mississippian periods. The New River Formation consists predominantly of sandstone, with some shale, siltstone and coal. The Pocahontas Formation consists of coal-bearing sandstone with lesser amounts of siltstone and shale. The Bluestone Formations is mostly red, green and medium gray shale and sandstone (Cardwell *et al* 1968).

SUBSURFACE FIELD EXPLORATION

The subsurface field exploration program consisted of several phases of borings and rock coring. Seismic refraction surveys were also performed for locating top of bedrock and interpolating between the borings. The phases were dictated by the 1) instability of the soils and weathered rock fragments at the top of the slope, 2) the difficult access to the slope toe, 3) the need for additional data that was required for the landslide repair design analysis, and 4) the observation of new subsurface site conditions that would possibly assist in considering an alternate cost effective landslide repair design. The total cost of the geotechnical field subsurface investigations represented 0.65% of the Engineer's Estimate (EE) for the proposed design and was estimated to have a potential for providing 50 percent cost savings for the project. The phases of the subsurface field investigations included:

Phase I - Reconnaissance Borings

Eastern Federal Lands Highway Division (EFLHD) was drilling another roadway in the NRGNR when the landslide occurred. The team mobilized quickly to the landslide site and performed preliminary reconnaissance borings to probe for depth to bedrock and prepare for a detailed subsurface field investigation. The borings consisted of Standard Penetration Resistance Test Borings (SPT) to auger refusal at the top of the rock or boulders encountered at depths varying from 5 to 15.5 feet. The borings were drilled upslope of the cracked area of the roadway embankment because of instability of the slide area.

Phase II - Slide Repair Design Borings

Once the design criteria and the importance of rebuilding on the existing alignment with minimal impact to surrounding areas were established, supplemental borings were performed along the proposed roadway alignment to collect soil and rock condition and strength properties for the design analysis of the slide. These borings were drilled outside the temporary concrete safety barriers that were set around the edge of the slide. Borings were drilled and cored to depths of 20 to 29 feet. These borings were not located in ideal locations because of difficulty with access but the design alternative being considered at this time was a reinforced soil slope (RSS), and it was felt that these borings would provide sufficient information for design without incurring the cost and safety risk of drilling on the slide.

Phase III – Slope Toe Retaining Wall Borings

Based on observations of foundation conditions at the river level during Phase II, and consideration of river hydraulics and scour potential, it was concluded that a toe wall was needed to protect against scour. The risk of going to contract on this type of repair without site specific explorations is high so the need for subsurface soil and rock information at the slope toe became critical. EFLHD retained H. C. Nutting Company (HCN) to use a remote-controlled track-mounted CME 55 drill rig to avoid the risk of injury to field personnel. The remote controlled drill rig was lowered down to the stream level using a cable attached to a winch on a dozer and four borings were completed to depths of 31.0 to 37.5 feet (Figure 4). This phase of investigation, though seemingly slowing down the proposed design process, provided very important data for the project progress. Bedrock was found to be at a depth of 20 to 22 feet (not 8 feet, as was estimated based on surface observations made during Phase II), so a toe wall was indeed necessary and more extensive than would have been designed without these borings. Thus, the borings averted an impending construction contract modification and significant escalation in contract cost and schedule.

Figure 4. Slope toe retaining wall borings drilling operation

Phase IV - Alternate Design Borings and Seismic Refraction Survey Lines

Additional borings were performed by the EFLHD subsurface field investigation team for an alternate design. The alternate design was proposed after observation of a competent rock ledge near the top of the slide scarp during drilling for the proposed slope toe wall. The rock ledge appeared located beneath the inaccessible edge of the slide scarp. Continued sloughing of the slope soils had exposed a rock ledge that was not previously visible and was not expected based on the shape of the scarp and cracking at the ground surface.

The design alternative was triggered by the idea of using the rock ledge to support a wall nearer to the elevation of the road, saving much cost as compared to one supporting the road from the elevation of Coal Run.

The alternative would require a more detailed determination of the extent and depth of the rock ledge that underlay the proposed reconstruction of Cunard River Access Road. In addition, a determination of the condition of rock and presence of voids or soft zone in the rock would also be required.

In addition to borings that were drilled for the alternate design, seismic refraction surveys were performed with assistance from Central Federal Lands Highway Division (CFLHD). Seismic refraction lines were performed both parallel and perpendicular to the centerline of the proposed roadway alignment. A Geometrics Smartsies signal enhancement seismograph system with 12 vertical geophones, a sledgehammer and a striking plate was used for collection of the seismic data.

SUBSURFACE FINDINGS

Description of the subsurface soil and rock conditions encountered during the several phases of the subsurface field investigations are summarized in the following subsections. A generalized subsurface profile is presented in Figure 5.

Roadway Embankment Borings

Borings drilled along the proposed roadway embankment alignment during phases I, II and IV encountered dark gray and black weathered rock fragments and sand with coal and some clay to depths varying from 1.5 to 15 feet. Beneath the sand and coal layer, weathered sandstone fragments and boulders were encountered to depths varying from 7 to 20 feet. This layer was underlain by brown and gray, coarse grained, medium hard to hard, weathered to highly jointed sandstone to depths varying from 10 to 24 feet. Rock quality designations (RQD) measurements within the sandstone ranged from 0% to 64% indicating poor to fair rock structural quality. Jointed to relatively sound sandstone with RQD values between 74% and 100% was encountered to depths varying from 20 to 29 feet. The measured RQD values indicate fair to excellent rock structural quality. Gray and dark gray, highly jointed to jointed and relatively sound to sound shale with occasional sandy clay seams was encountered between depths of 29.5 and 44.5 feet. RQD's calculated within the shale varied between 63% and 100% indicating fair to excellent rock structural quality.

Slope Toe Retaining Wall Borings

Four (4) borings drilled and cored along the proposed alignment of the toe retaining wall encountered dark gray and black weathered rock fragments with some coal and sand, and little clay to depths varying from 16.0 to 25.0 feet. Dark gray, fine grained, medium hard to hard shale was encountered to the termination of the borings at depth between 31.0 and 37.5 feet. RQD values recorded within the shale generally ranged from 52% to 100% indicating that the rock condition varies from highly jointed to sound. The recorded RQD's indicate fair rock structural quality for the upper layer of the shale to excellent rock structural quality for the shale within the deeper depths. A brown and tan, weathered sandstone layer was encountered above the shale in two (2) of the borings.

A generalized subsurface profile of the borings drilled along the alignment of the roadway embankment is presented in Figure 6.

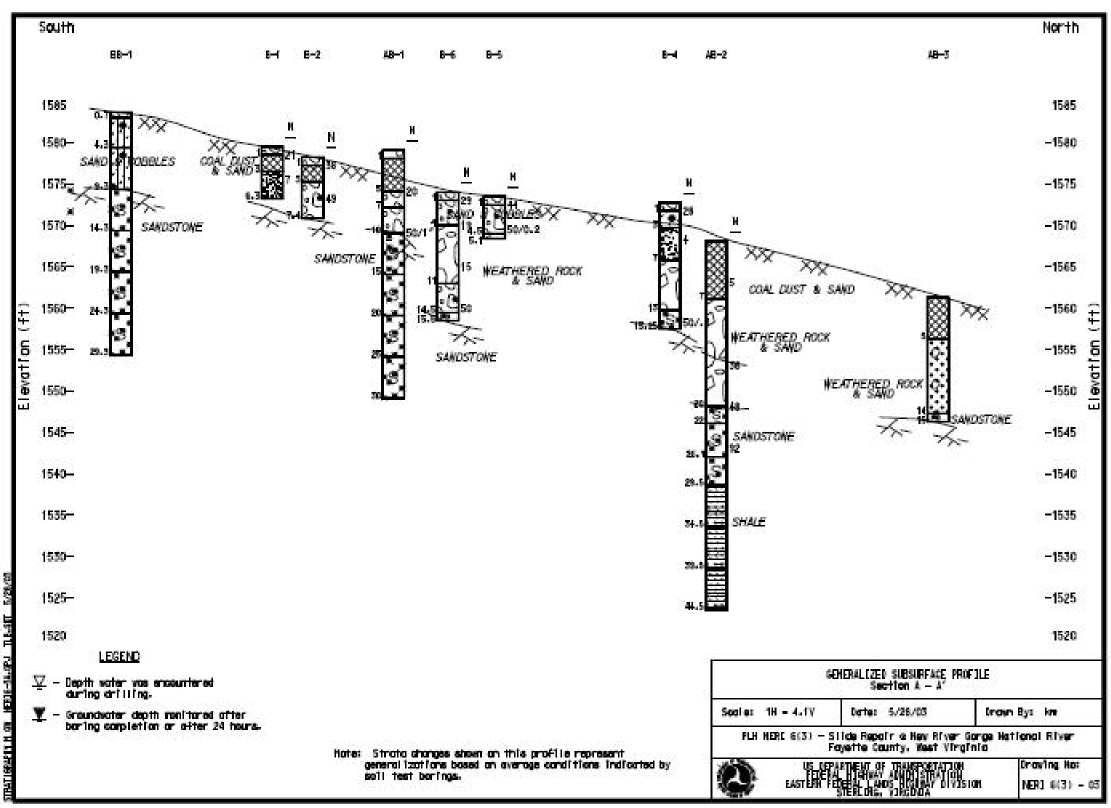

Figure 5. Generalized subsurface profile along roadway embankment

Seismic Refraction

Seismic refraction surveys were generally found to be in agreement with the results obtained from the borings performed along the seismic refraction line. Depth to rock varied between 10.0 and 15.0 feet along the outer line. Rock quality (velocity) was found to improve with depth from 9800 ft/s (3000 m/s) at depths between 10.0 and 15.0 feet to 16000 ft/s (5000 m/s) at depths between 18.0 and 20.0 feet. Sound rock was encountered at shallow depths in some of the borings. An example of a seismic profile along a line perpendicular to the roadway centerline at Station 29+60 is presented in Figure 6.

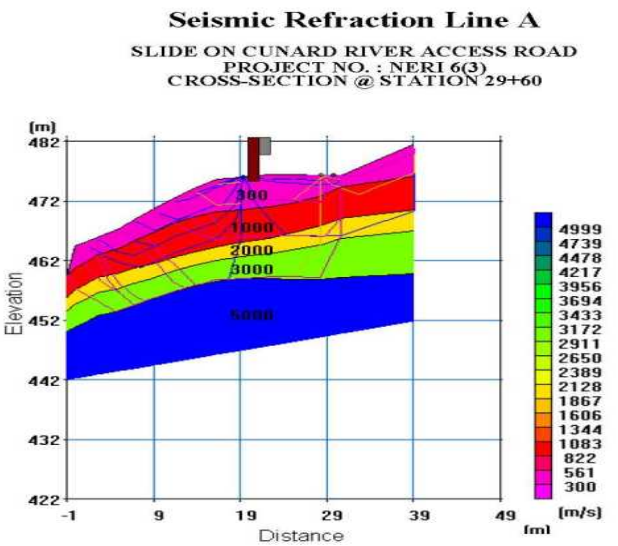

Figure 6. Seismic Refraction Line A @ Station 29+60

Groundwater

Groundwater was not encountered in any of the borings drilled on the roadway embankment. However, water was observed to continuously flow in the road cut side ditches and through fractures in the cut slope rock face along the road, located above the slide area. Groundwater was also encountered in the borings drilled along the slope toe at depths varying from 1 to 5 feet.

LANDSLIDE REPAIR DESIGN ANALYSIS

Design Challenges

The design of the landslide repair had to overcome several challenges. The challenging factors included:

- *Establish original roadway alignment*: Establishing the original roadway alignment required widening the remaining portion of the slide scarp 29 feet from the scarp line towards Coal Run, in order to re-establish a missing lane, shoulder, and width for placement of guardrail. This would require extensive construction of roadway embankment.
- *Avoid rock excavation or tree removal from the cut side of the roadway:* This factor eliminated any realignment of the road away from the scarp line, and significantly complicated maintenance of public traffic during construction, another design challenge.

- *Maintain traffic control during construction:* The design was required to provide a one-lane, two-way access through the construction period for business and public access.
- *Access to the slide toe for construction.* The scarp face presented an unstable vertical drop, and downstream beyond the slide the vertical difference between the road and Coal Run was even greater, very steep and heavily vegetated.

Proposed Landslide Repair Design

Based on the available subsurface soil and rock information from Phases I and II of the subsurface field investigations and project design requirements, a landslide design consisting of excavating the remaining amount of loose soil and coal mix from the slope face, installing a retaining wall at the toe of the slope and constructing a geosynthetic reinforced soil slope (RSS) from top of the toe retaining wall to establish the original alignment of Cunard Road. This proposed design was adopted because it involved minimum or no disturbance of the ground and no clearing of trees and vegetation outside the limits of the landslide area.

The wall was required because of the constraint against realigning Coal Run, and the need to protect the reconstructed slope from future flood scouring. In addition, the wall was designed to resist a calculated 3 feet of scour depth.

RSS and Toe Retaining Wall Design Details

Preliminary analysis of the RSS and cantilevered soldier pile toe retaining wall was performed using limit equilibrium slope stability analysis. The geometry of the analyzed cross section consisted of a 1 (H): 1 (V) slope and a slope height of 77 feet from top of the toe retaining wall to the roadway level. Slope stability analysis was performed for both a block-translation failure and a circular failure. The global stability analysis of the critical section indicated that the lateral loads would be high and a critical failure plan would pass through the cantilevered retaining wall with a safety factor of less than 1. Based on the preliminary slope stability analysis results an anchored soldier pile retaining wall was selected.

Material strength properties used in the slope stability analyses were for the new fill soils, on-site sand and weathered rock fragments, jointed to sound sandstone and shale, and soldier piles supported toe retaining wall. A summary of the design analysis material strength properties is provided in Table 1.

Table 1. Slope Stability Material Strength Properties

Layer No.	Layer Description	Unit Weight (pcf)	Friction Angle (ϕ^o)	Cohesion (psf)
1	New fill (A-2-4)	120	34	0
2	Weathered rock fragments and sand	115	33	500
3	Weathered to jointed sandstone and shale	135	43	2000
4	Reinforced Concrete (Toe Retaining Wall)	150	45	3000

The design of the toe retaining wall design consisted of anchored soldier pile wall with concrete lagging. The preliminary design analysis of the toe retaining wall was performed using the methods from Principals of Foundation Design (Das 1998). A final design check was performed using the methods from FHWA (Mohoney, 1994) and (Sabatini *et el* 1999). The proposed design typical section detail is presented in Figure 7.

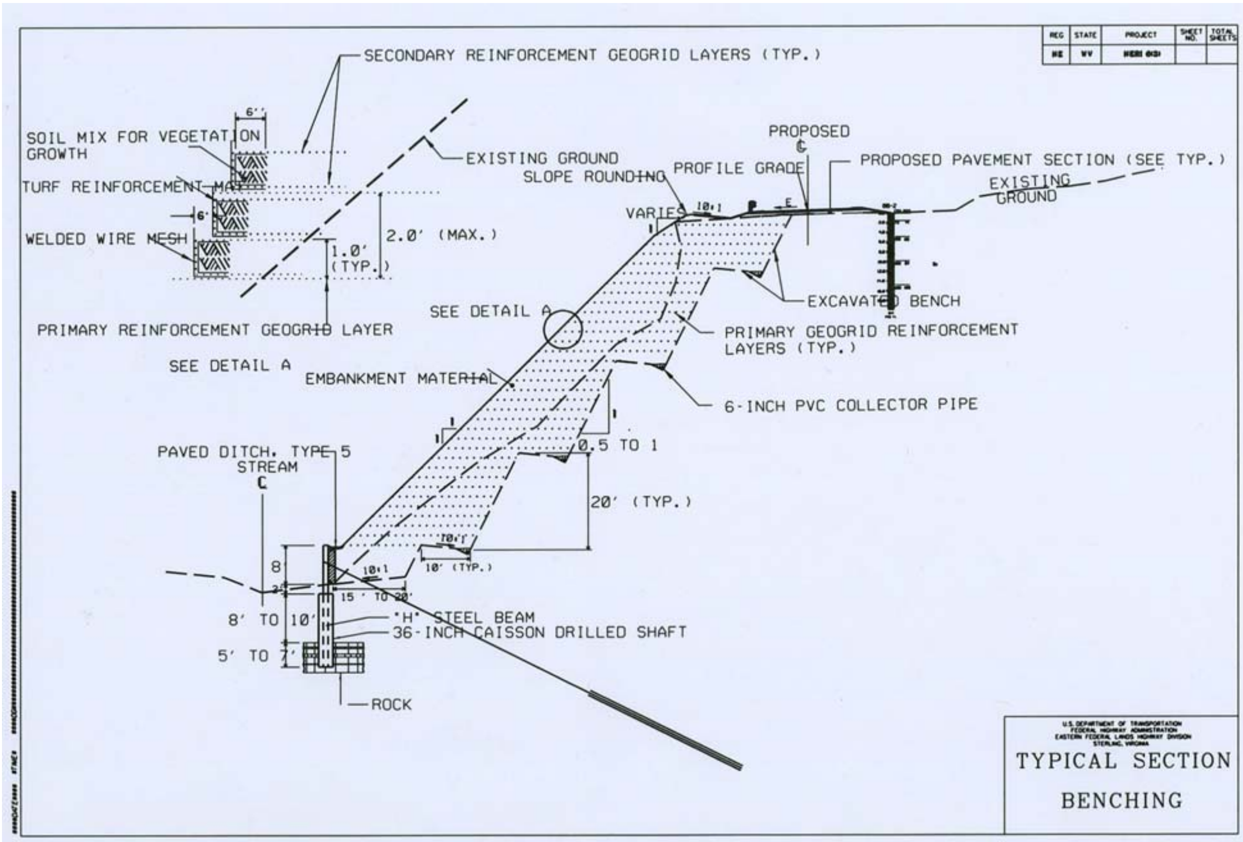

Figure 7. Proposed design typical section detail

Alternative Landslide Repair Design

As indicated previously, observations and careful study of the slope's soil and rock condition during the progress of the third phase of field investigations and the proposed design, indicated the possibility for an alternative design concept that could provide significant construction cost and time savings.

The alternate design would have to meet several challenges. These challenges included 1) the need for confirmation of subsurface observations and assumptions; 2) limited available time for additional field investigations and redesign; 3) the possibility that the second proposed design might not be constructible if the additional subsurface field investigation had unfavorable findings and 4) the fact that the first design was near completion with construction scheduled to follow shortly thereafter. However, based on the possibility of achieving 50% savings of the project's EE of $6.0 million, the decision was made to pursue the alternate design while maintaining the proposed design as a backup option.

The cost savings from the alternate design would be realized from eliminating the toe anchored soldier pile wall and most of the RSS. The alternate design instead would focus on use of an observed rock ledge some two-thirds of the way up the face of the slide.

Following completion of the additional subsurface field investigations and confirmation of favorable soil and rock conditions, the alternate design was completed. The alternate design consisted of a 12 to 18 feet high gabion basket wall with geosynthetic reinforcement at 3 feet vertical spacing. The wall was designed to bear on competent weathered rock or sound rock of the observed rock ledge. Stability analyses were performed assuming reinforcement geogrid panels were embedded 20 feet and tied to the gabion baskets. Reinforcement geogrid with 4000-lb/ft long-term design strength and select granular fill were assumed for the design analysis. A minimum safety factor of 1.4 was obtained based on the assumed and calculated material properties. Select granular fill was recommended because it meets the design friction angle, provides ease of construction with minimum compaction effort and is a good drainage material. Riprap was recommended for protection of the toe of the slope from scour. Class 6 riprap (up to 3530 lbs) with a 10 foot base width, 3 foot minimum embedment depth and 7.0 foot minimum height was placed at the toe of the slope in order to meet hydraulic requirements. A typical section of the details of the alternate design is presented in Figure 8.

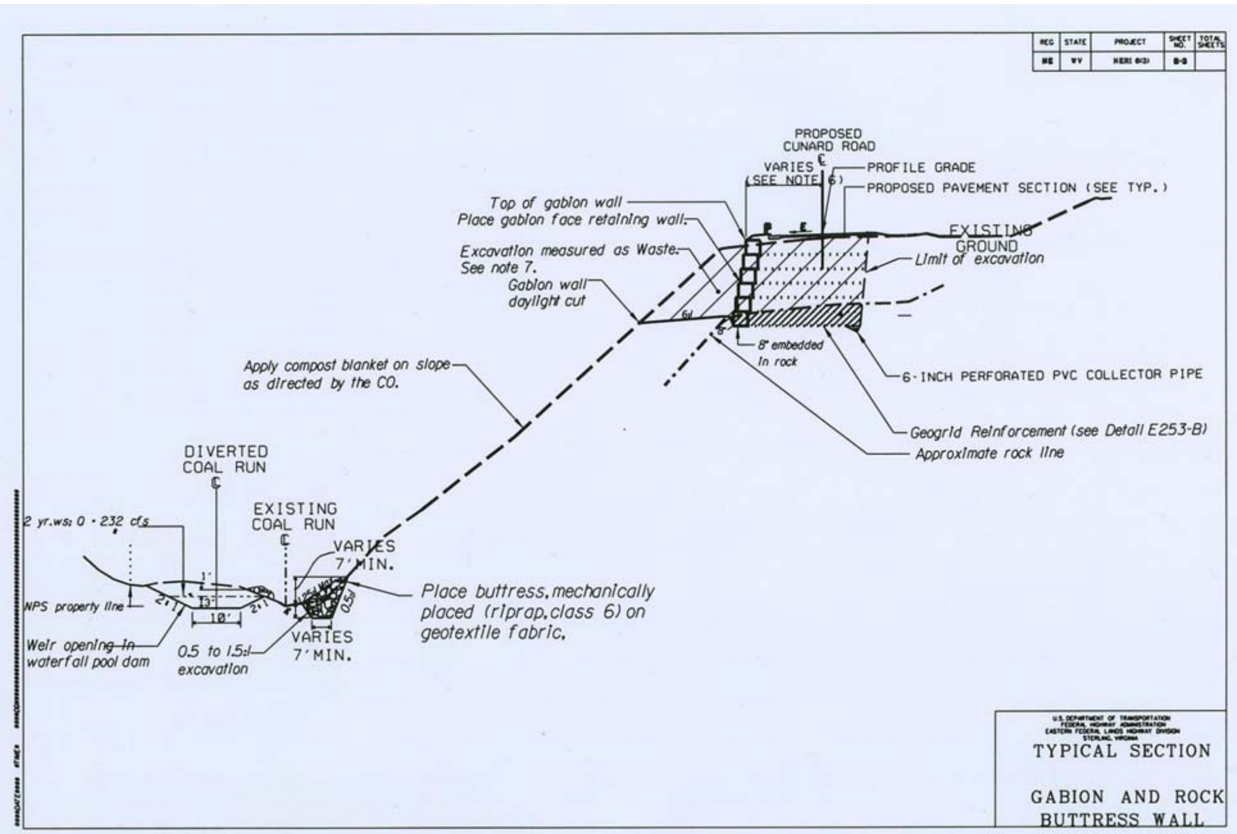

Figure 8. Alternate design typical section detail

Summary

Construction of the alternate design was completed as planned, meeting design assumptions and exceeding estimated cost savings (Figure 9). The phased investigation and design approach adopted during design of the landslide repair in this project proved to be a successful design method. Phased type of designs that are based on considering all possible risk factors, alternative plans and performing additional supporting field investigations can result in construction cost and time savings and hence an economical design.

Figure 9. Construction progress

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