Geotechnical Investigation for
Lake Tahoe Environmental Improvement Program
Phase III
US 50
Douglas County, NV

NDOT Project Identification Number 60461
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Table Of Contents

FIGURES

TABLES

APPENDICES

1.0 INTRODUCTION

1.1 General

Wood Rodgers is responsible for the preparation of an EIP Quality Assurance design for the US50 roadway segment as defined in the Drainage Design Report. The Quality Assurance design documents include Quality Assurance level Plans, Specifications and Design Report which consists of two parts: the Drainage Design Report, and this Geotechnical Design Report. Wood Rodgers has prepared the Quality Assurance Drainage Design Report per the NDOT Drainage Design Manual requirements (December, 2006). The Quality Assurance Geotechnical Design Report is a stand alone document, but it is accompanied by the Drainage Design Report, which contains information specific to hydrologic, hydraulic, and erosion control design aspects of this project. Selected graphics inserted within the text of this geotechnical report are also presented in Appendix A. Calculations specific to the Geotechnical Design Report are presented under separate cover and include the Structural Design Calculations specific to the catchment wall design for Slope 27.

1.2 Scope

The scope of work for this portion of the project addresses surface stabilization and erosion mitigation recommendations associated with the selected slopes thereby reducing erosion and sediment transport, conflicts with vehicle traffic, and long-term operation and maintenance costs. The cut slopes under consideration are located along the south/east side of U.S. Highway 50 near the Glenbrook Community and Cave Rock tunnel.

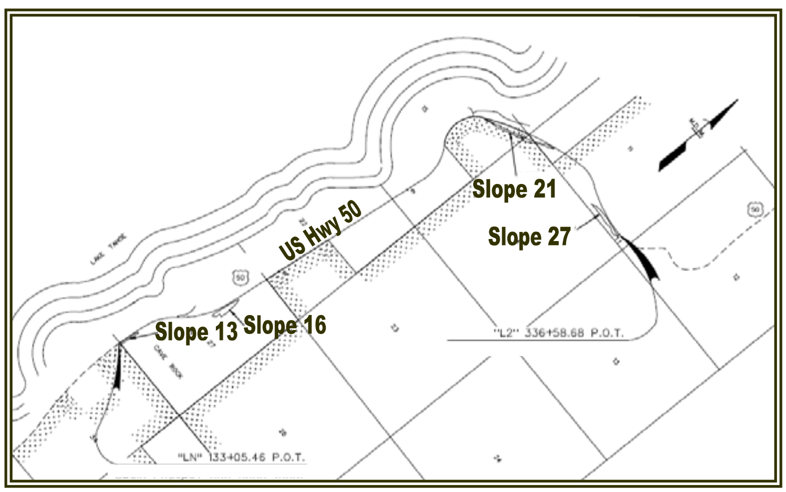

Figure 1 – Vicinity Map

As shown in Figure 1, the project area includes specific slopes between Milepost 9.51 (just north of the Cave Rock Tunnel) to Milepost 11.67 (just east of Glenbrook). Specifically addressed are Cut Slopes 13, 16, 21, and 27. The rock slope designation comes from the 1993 Rockfall Hazard Rating System (RHRS) thesis report prepared by Robert Flatland and Robert Watters, Ph.D./Thesis Advisor, a portion of which addresses these slopes. The cut slopes under consideration are located along the south/east side of U.S. Highway 50 near the Glenbrook Community and southward toward the Cave Rock tunnel.

1.3 Other Reports and Investigations

Previous work specific to Slopes 21 and 27 was performed by Wood Rodgers during December, 2006. The work was prepared under Mr. Mark Doehring, PE, and was entitled: *Geotechnical Investigation for Lake Tahoe Environmental Improvement Program, Phase III, U.S. Hwy 50, Washoe County, Nevada.*

2.0 PROJECT DESCRIPTION

Our work consists of developing slope protection and stabilization measures which will improve runoff water quality, reduce long-term maintenance costs, and address potential rockfall hazards. Ultimately the planned improvements must stabilize the existing slope surface, establish acceptable design grades, and meet NDOT's vegetation goals. Surface treatments and mitigation measures are discussed in Section 6.3 Approaches to Mitigation and specifically assigned and addressed for each slope in Section 9.0 RECOMMENDATIONS.

Briefly, the project scope entailed:

- Reducing the overall gradient of the selected slopes to allow for placement of surface improvements such as riprap and top-soiled riprap.
- Scaling of loose rock fragments on existing slopes.
- Placing manufactured facing elements on steeply sloping faces to facilitate revegetation.
- Installing rockfall drapes where the previously mentioned approaches cannot be utilized due to competency of the rock, associated costs to mitigate as addressed above, or right-of-way limitations.

Information provided to our staff during the course of the investigation included:

- Plan and profile views of the slopes addressed in this investigation.
- Geotechnical Investigation for Lake Tahoe Environmental Improvement Program, Phase III, US Hwy 50, SR 28, 207, and 431, Washoe and Douglas Counties, Nevada. Prepared by Wood Rodgers, Incorporated, December 2008.

Because of the nature of the scope of this investigation, presentation of design loads is not applicable. However, Section 12.3.5 of the NDOT Structures Manual presents a peak ground acceleration of 0.50 g for Douglas County. AASHTO 2002 Division IA, Section 7.4.3, allows the use of a horizontal coefficient equal to 50 percent of the peak ground acceleration for analyses, and therefore a horizontal coefficient of 0.25g was incorporated in our calculations.

3.0 GEOLOGIC CONDITIONS AND SEISMICITY

3.1 Local Geology

The Lake Tahoe area is located on the California-Nevada state line at the eastern edge of the northern Sierra Nevada Mountain Range. The Tahoe Basin is underlain at depth by the Sierra Nevada Batholith consisting of Mesozoic "granitic type" rocks, predominantly granodiorite (Hyne, 1972). Tertiary volcanic rocks, including basalts, andesites, latites, trachytes, and various tuffs and mudflows commonly overlay the granitics and are often exposed at the surface (Grose, 1985). In addition to the igneous rock units, deposits of alluvium, colluvium, and glacial sediments are locally abundant.

Lake Tahoe is dammed in the north near Tahoe City by andesitic mudflow breccias. During the Pleistocene the lake level rose and dropped dramatically due to recurrent damming by alpine glaciers (Hyne, 1972). This rise in lake level, as much as 450 feet, subsequently resulted in a rise in the ground water table, which led to extensive physical weathering of the granitic rocks present along the perimeter of the lake. Warhaftig (1965) proposed that the weathering of granitic rocks in the Sierra Nevada is often a function of microfracturing caused by the expansion of biotite that is continuously exposed to water. The extent of the microfracturing is directly responsible for changes in the engineering properties of the intact rock (Krank and Watters, 1983). Granitics existing above the groundwater table experience relatively minor physical weathering: while those that are or were below the water table for extended periods of time are extremely weathered and readily break down to grus or "decomposed granite" as typical of Slopes 13 and 16.

The 1985 U.S. Geological Survey Glenbrook 7 ½ minute quadrangle map (Grose, 1985, scale 1:24000) was reviewed to aid in qualifying host rock features in the study area and is shown in Figure 2. Essentially, three different rock type formations are generally noted within the slope areas under consideration.

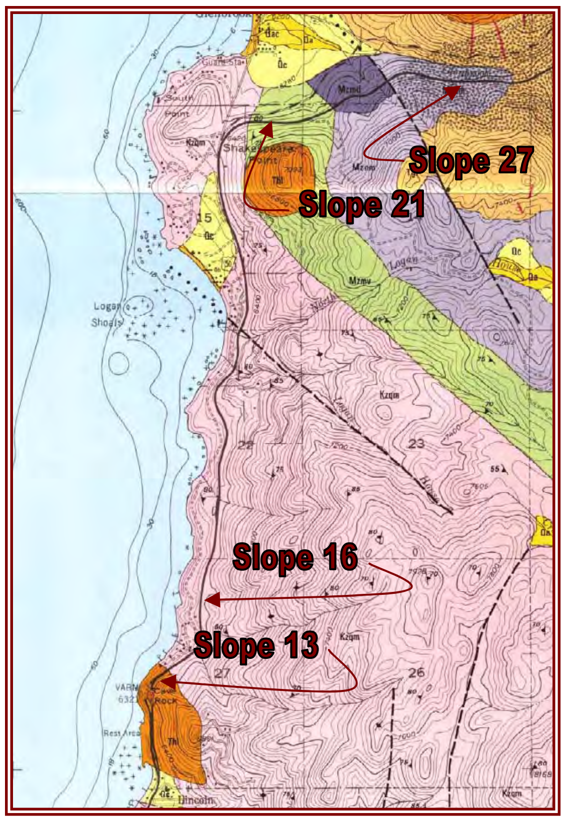

Figure 2 – Geologic Map of Project Area

Slope 13 is mapped in an area comprised of porphyritic hornblende-sanidine latite described in the referenced literature as "erosion resistant, intrusive masses."

Slope 16 is mapped within the granodiorite of Zephyr Cove. Granodiorites are usually stable and typically weather to very dense sands and gravels possessing varying amounts of non-plastic fines.

Slope 21 lies within a thin band of exposed metamorphosed tuff and flows. These units had a tendency to be more competent near the eastern limits of the slope. Some surface sloughing is evident near the western limits of the slope and the fine-grained nature of the native tuffs becomes apparent due to the minor rills in the slope face. The hummocky surface of the western limits of Slope 21 can be seen in Figure 3.

Figure 3 – Slope 21, West End

Slope 27 lies within altered and decomposed biotite monzogranite of North Logan House Creek which can give rise to the significant variation in weathering and competency within the unit. Although not readily apparent in Figure 4, the ground surface has raveled and slumped within the western limits of Slope 27 while the cornice shown in Figure 4 depicts a more competent bedrock mass.

Figure 4 – Slope 27, East End

3.2 Faulting and Seismicity

The lake itself is located within a large graben which is approximately 6,225 feet above sea level. The normal faulting that created the Tahoe Basin is related to Basin and Range structures, which continue eastward to the Wasatch Mountains of Utah. According to Hyne (1972), the region is still seismically active with the most earthquakes occurring north of the basin near the Truckee area. The mountains of the Sierra Nevada to the west were extensively glaciated during the Pleistocene; while the Carson Range to the east experienced only limited glaciation, probably due to its location in the rain shadow of the Sierra crest which rises to elevations of nearly 11, 000 feet.

Figure 5 shows the USGS Quaternary Fault Map for the area under consideration with project limits identified. As can be seen, although the project site is in a historically, relatively active seismic area, only one Quaternary Fault is located within the planned improvements. NDOT procedures and policies provide that for noncritical structures, the acceleration coefficient be obtained from Article 3.2 of Division IA of AASHTO, 2002. For our purposes an expected bedrock acceleration of 0.25g was used in our analyses.

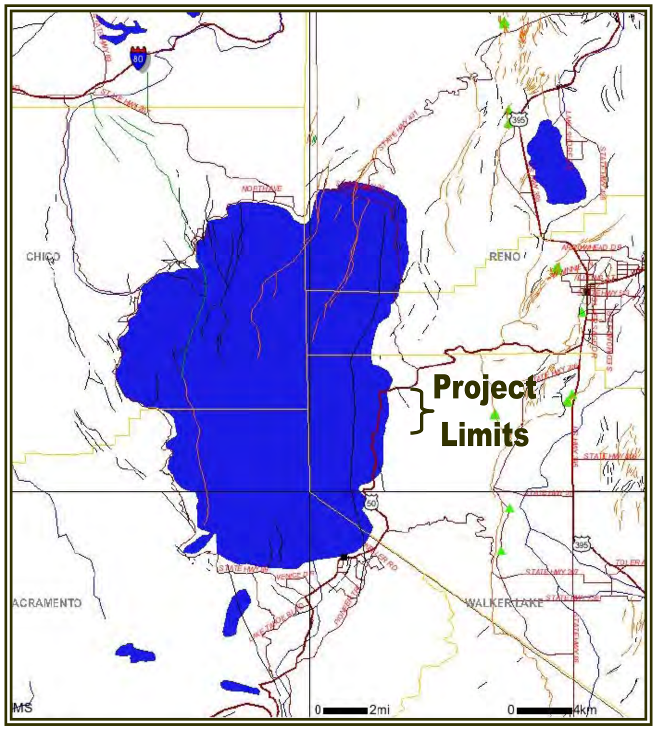

Figure 5 – USGS Quaternary Fault Map
(http://gldims.cr.usgs.gov/qfault/viewer.htm.)

4.0 FIELD INVESTIGATIONS

Subsurface conditions along Slopes 21 and 27 were investigated in October 2009 by advancing a series of twelve borings utilizing a Sonic Spyder #2 drill rig employing a 4 inch diameter casing. Because the planned improvements for Slopes 13 and 16 were not considered structural, no explorations were advanced in these slope locations. Weathered bedrock was sampled with the California Modified Sampler (CMS) driven by a 140-pound drive hammer with a 30-inch stroke. The number of blows to drive the sampler one-foot into undisturbed soil (Blow Count) is an indication of the density and shear strength of the material. All borings were advanced and logged from existing grade, and the maximum depth of exploration was 20 feet. Representative samples were returned to our Reno laboratory for testing. Logs of these explorations are presented in Appendix B, Subsurface Explorations.

Wood Rodgers' personnel examined and classified all rock in the field in general accordance with ASTM D 5878-08 Standard Guides for Using Rock-Mass Classification Systems for Engineering Purposes and characterized the excavated material in general accordance with ASTM D 2488 (Description and Identification of Soils). Soil classification methods were used due to the rocks' tendencies to drill and recover as a dense to very dense soil. A rock mass classification chart and a Unified Soil Classification System key have been included with the Logs of Borings in Appendix B of this report. The logs represent field interpretations of the subsurface conditions and laboratory test results. The lines designating the interface between various strata on the logs represent the approximate positions of the interface. The actual transition between the strata may be gradual.

CMS values were approximately correlated to Standard Penetration Test values utilizing preliminary correlations developed by the California Geological Survey (CGS) as presented in Figure 6. This correlation was used for information purposes only in an attempt to present the blow count information in a readily known tangible form (*Penetration Test Comparisons: Modified California versus Standard Penetration Test Samplers*, Bott, Jacqueline D. J., and Knudsen, Keith L., California Geological Survey, 185 Berry Street, Suite 210, San Francisco, CA 94107, jbott@consrv.ca.gov.) Please note that the CGS document refers to the test method as MCS, in lieu of the CMS standard adopted herein.

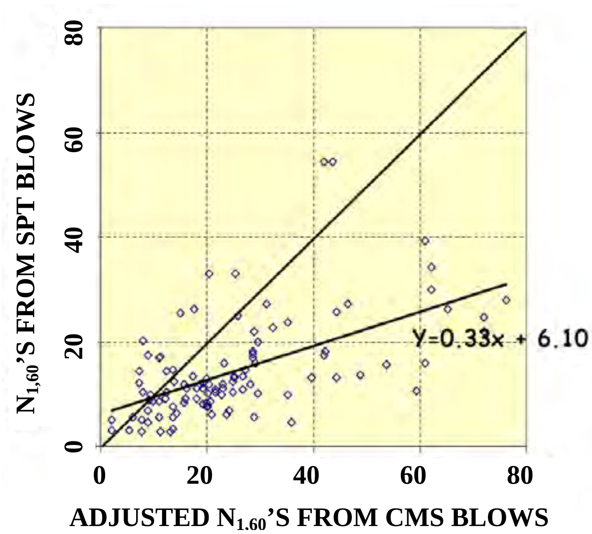

Figure 6 – Preliminary Correlation between CMS and SPT Blows

Rock mechanics and rock slope stability evaluations associated with central portion of Slope 27 were performed by McMillen, LLC. The means and methods employed, as well as their findings are presented in their report which has been included as Appendix E – Reports by Others.

In addition to advancing borings, ReMi geophysical survey techniques were employed to remotely assess the rock mass properties within the slopes. The ReMi study was performed by Marvin E. Davis and Associates. The ReMi methods provide an effective means to obtain subsurface information by obtaining vertical S-wave profiles to depths up to 300 feet. The method is based on the same theories as spectral analysis of surface waves (SASW) and multi-analysis of surface waves (MASW) with the benefit of utilizing lightweight, portable seismic equipment. One and two dimensional shear wave velocity profiles are developed which aid in the assessment of thickness and consistency of soils and weathered bedrock. ReMi data as used in our assessment of rock mass properties have been presented in Appendix D, In Situ Test Data. The Marvin E. Davis and Associates report is presented in Appendix E of this report as an Appendix to the McMillen Report.

5.0 LABORATORY ANALYSES

Laboratory test data is presented in Table 1 below:

Table 1 – Summary of Test Data

Sample Location				Classification					Strength				
				Gradation		Atterberg Limits			Unit Weight		**Un-confined	Direct Shear	
Slope	Boring	Station	Depth (Ft.)	%+#4	%-#200	LL	PI	USCS	γ_d (PCF)	%m	q_u (PSI)	Φ (°)	C (PSF)
SLOPE 21	B-1	290+00	0 - 5	0	70.4	34	5	ML	95.5	16.3	-	-	-
			5 – 10	19	38.7	-	NP	SM	108.8	6.7	-	35.7	0
	B-2	291+00	1 – 4	2	52.4	-	NP	ML	99.9	13.9	-	-	-
			12 – 14	0	59.3	-	NP	ML	104.5	7.8	-	38.0	0
	B-7	286+00	2 ½ - 4	2	50.0	-	NP	ML/SM	-	8.4	-	-	-
			6 - 8	0	38.1	-	NP	SM	103.9	6.6	-	-	-
	B-8	284+85	8 - 10	-	-	-	-	-	*** 2.48 BSG	-	6,600	-	-
	B-9	283+85	7 - 9	-	-	-	-	-		-	6,700	-	-
			12 ½ - 14 ½	-	-	-	-	-		-	4,300	-	-
			16 - 18	-	-	-	-	-		-	6,900	-	-
	B-12	276+50	2 - 4	5	50.5	-	NP	ML	-	14.3	-	-	-
			7 ½ - 9	6	42.2	-	NP	SM	-	6.7	-	-	-
SLOPE 27	B-3	330+30	2 – 4	41	17.3	29	12	SC	99.7	9.1	-	39.2	566
	B-4	331+70	Surface*	-	-	-	-	-	-	-	8,900	-	-
			1 – 3	48	22.1	19	4	SM/GM	-	4.9	-	-	-
			3 – 5	58	18.1	19	3	GM	-	3.5	-	-	-
			5 – 10	-	-	-	-	-	-	-	3,000	-	-
	B-5	333+70	Surface*	-	-	-	-	-	-	-	10,300	-	-
	B-6	335+40	2 – 4	28	24.7	32	12	SC	113.4	11.1	-	34.9	427
			7 – 9	43	20.8	29	13	SC	-	7.4	-	-	-

*Rockfall Samples **Unconfined Compression *** BSG – Bulk Specific Gravity

Test methods were performed in accordance with ASTM Standards. For Slope 21 the classification tests associated with Borings 2 and 7 are partially indicative of the competency of the rock as well as the material properties typically associated with this type of bedrock unit. The rock encountered in these areas was competent enough to degrade into 'rock flour' as the sonic barrel slowly progressed through the unit. In addition the metamorphosed tuffs and flows will typically express lower plasticities and greater percentages of -#200 inherent to their fine-grained rock matrix.

The classification test data associated with Slope 27 more closely approximates the anticipated excavation characteristics of the bedrock. The host rock of Slope 27 was coarser grained and more weathered and altered than Slope 21.

Laboratory test data is also presented in Appendix C.

6.0 DISCUSSION

6.1 General

The cut slopes under consideration are located along the south side of U.S. Highway 50 near the Glenbrook Community and to the south near Lakeridge. General observations for all slopes include:

- Frost penetration, spring runoff and intermittent seeps and springs contribute to erosion, rockfall, and general surface instability. This instability is exacerbated by the steepness at which the cut slopes were originally graded.
- Soil covers the surface or upper crests of most of the slopes. Large trees typically grow along the crest and in some instances runoff and surface instability have led to erosion and root exposure.

6.2 Site Conditions

The site was visited during July 2009 to assess surface conditions, potential erosion and incidental rockfall associated with the slopes currently under consideration. Specific site observations for each slope are discussed below.

6.2.1 Slope 13

Slope 13 is a relatively gentle slope comprised primarily of deeply to moderately weathered rock. Granitic in nature, the rock in this area typically weathers to non-plastic sands and gravels. The slope is approximately 15 feet high at its maximum and generally faces west. As evidenced by the cobbles and sediment in the toe ditch in the adjacent photograph, cobbles within the slope face and overburden become locally undermined and come to rest in the toe ditch which serves as both a catchment area and conveyance for runoff. Recommendations are presented to address isolated rockfall potential and surface erosion.

Figure 7 – Slope 13

6.2.2 Slope 16

Slope 16 is relatively stable and presents fewer issues from a geotechnical perspective. This area has been mapped as being comprised of granodiorite of Zephyr Cove. Bedrock outcrops are evident in the upper hillsides of Slope 16 and the bedrock has a tendency to weather to dense, non-plastic sand to sandy gravel consistency. Some scaling may be required along the ridgeline of the slope where the outcrops and residual knobs and boulders can present isolated rockfall issues. Vegetation is fairly well established along the slope face adding to its stability and aesthetics. Recommendations are presented to address surface erosion.

Figure 8 – Slope 16

6.2.3 Slope 21

Figure 9 – Slope 21

Slope 21 is a moderately steep slope (~ 1 ¼ :1 (H:V)), and comprised primarily of deeply to moderately weathered rock. However, more competent rock outcroppings are also evident across the slope. The cut slope is approximately 40 feet high at its maximum and generally faces northwest. The rock in this slope is a gray to green metamorphosed tuff that is moderately weathered with very few to no identifiable joint or fracture patterns. As evidenced by the hummocky surface indicated in the insert, isolated small slope failures have occurred in the western portion of the slope. These failures appear to be induced by springs, and are likely triggered as surface runoff and snowmelt increase in late spring and early summer. Recommendations are presented to address surface erosion and protection of the slope surface particularly where isolated surface failures have led to development of a cornice which accelerates weathering along the crest where head cutting is evident.

Figure 10 – CMS Sample & Bedrock Characteristics

Based on our exploration program and the ReMi data, the Tuff units comprising the slope toe have been weathered to a very dense soil/weak rock consistency to depths approaching 30 to 40 feet below the original ground surface. Undisturbed, the subsurface unit's appearance reflects the original and relatively uncompromised characteristics of the bedrock as shown in Figure 10; however, the unit has a tendency to break down readily when exposed to mechanical effort such as advancing with the sonic core barrel or application of minor blows with a rock hammer. Blow count, and shear wave velocity data indicate that the material is overall moderately strong and capable of supporting the planned improvements. If shear wave velocities on the order of 1,000 feet per second are used as the approximate marker between soil and rock, ReMi data indicates the depth to bedrock is on the order of 15 to 18 feet below grade within the upper reaches of the slope. Plate D-1 of Appendix D presents results of the ReMi surveys associated with Slope 21.

6.2.4 Slope 27

Slope 27 is a relatively steep rock/soil slope. Figure 11 shows the variability in material quality and physical properties along the slope. The slope approaches 50 feet in height and generally faces north to northwest. The host rock is deeply weathered, altered, heavily jointed and fractured biotite monzogranite presenting strong competent outcrops and intensely weathered and decomposed soil slopes. Previous work indicates that the more competent rock associated with Slope 27 exhibits planar and wedge failures in conjunction with some toppling failures. A spring is also present within the northern portion of the slope. Of the slopes under current consideration, Slope 27 presents the most significant and difficult profile to address. Recommendations are presented herein to address the continued weathering and erosion at the crest and further encroachment upon the right-of-way boundary. Rockfall potential associated with the steeper outcrops and proposed mitigation measures are addressed in the McMillen report presented in Appendix E.

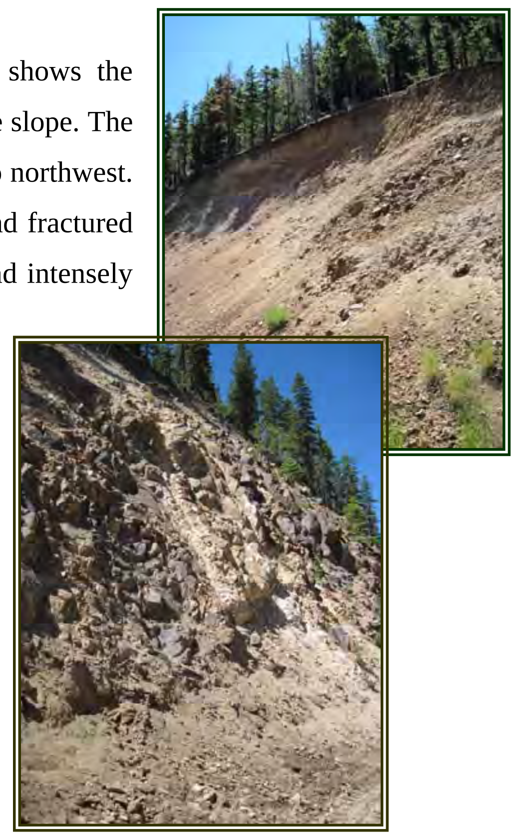

Figure 11 – Slope 27

As with Slope 21, the profile encountered in our explorations consisted of weathered and altered rock through the depths of our borings. ReMi data also indicates that the rock profile has been weathered and decomposed in the upper 20 feet of original ground at the top of the slope, and the upper 20 feet of the toe of the slope. If shear wave velocities on the order of 1,000 feet per second are used as the approximate marker between soil and rock, ReMi data indicates the depth to bedrock is on the order of 10 to 15 feet below grade within the upper reaches of the slope. Plate D-2 of Appendix D presents the results of the ReMi surveys associated with Slope 27. Blow count, and shear wave velocity data indicate that the material is overall moderately strong and capable of supporting the planned improvements.

6.3 Approaches to Mitigation

Various alternatives have been evaluated and selected for treatment of the selected slopes. Most slopes require more than one approach to address their issues comprehensively. Proper coordination along the transition between mitigation measure types is critical to successful implementation of the stabilization measures. This is especially critical on Slope 27 where the deeply weathered areas will be stabilized using mechanical means while the central portion of the slope, where rock is exposed, will be addressed by scaling and installation of wire mesh (cable anchored).

6.3.1 Scaling

Scaling is required for Slopes 13, 16, and the central portion of Slope 27. Scaling is the process by which unstable rock particles are dislodged from the surface of the slope and collected in a safe manner. Most of the planned scaling will occur at the top of the slopes where erosion occurs and undermines cobble and boulder sized particles exposed in the slope face. However, as in the case of Slope 27, scaling of heavily jointed rock mass is also required (Reference the McMillen report, Appendix E). Scaling is a subjective process and the contractor should be specifically trained and experienced in this type work. Trees that pose a potential hazard from toppling should be cut down or otherwise addressed. The root system of removed trees shall be left in place to inhibit slope erosion.

6.3.2 Grading, Riprap, and Top-soiled Riprap

Where right-of-way and topography allow, slopes shall be graded to a 1½:1 (H:V) slope inclination. Where existing slopes are slightly steeper than 1½:1 and the distance to 'catch' the existing slope becomes excessive, slopes shall be graded to 1¼:1 (H:V) as indicated on the plans. Once graded, the slopes shall be prepared for the placement of riprap or top-soiled riprap.

6.3.3 Toe Walls

Toe walls will be incorporated in the treatment of Slopes 13 and 16 to secure the riprap facing. Foundation excavations for walls shall be in cut and shall be cleared of all loose or disturbed material, or existing fills prior to placing concrete. Where toe walls consist of large boulders, the boulders shall be secured by embedding the base rock in Portland cement concrete or NDOT's Class B slurry. NDOT barrier rails can also serve as toe walls. Toe walls can eventually become backfilled with slope debris so maintenance can still become an issue.

6.3.4 Vegetated Reinforced Soil Slope (VRSS)

Vegetated Reinforced Soil Slope applications allow for the stabilization of slope faces in confined areas while providing a surface that accepts and encourages plant growth. A VRSS application is shown in Figure 12. VRSS applications can either be facing for a reinforced soil slope where global slope stability presents issues or can be anchored to stable slopes with soil or rock anchors. Reinforced soil slopes or anchored facings and processes are patented and as such strict adherence to the manufacturer's design, plans, and specifications is required, unless more restrictive requirements are stipulated in the contract documents. Ultimately the improvements must stabilize the existing slope surface, establish acceptable design grades, and meet NDOT's vegetation goals.

Figure 12 – Steeply Sloping Vegetated Face
(© Maccaferri-USA, reproduced with permission)

Slope 27 and Slope 21 show evidence of moisture seeping along the soil/bedrock interface. Incorporation of chimney/blanket drains will reduce the potential for buildup of hydrostatic pressures and are therefore required. In addition, designing the drains to intercept moisture and direct it toward the face of the slope would be beneficial toward establishing the face vegetation. As a minimum, the drain system should extend into the stronger, denser subsurface units as indicated by the ReMi data, i.e. approximately 10 feet for Slope 21, and approximately 15 feet for Slope 27.

6.3.5 Wire Mesh (Cable Anchored)

Placing a high-strength steel wire mesh on steep slopes stabilizes unconsolidated material and rocks and prevents stones and blocks of weathered rock from breaking out. The rock mass is first analyzed for global instabilities such as plane shear, wedge, step-path, or toppling. Any structural deficiencies are addressed by either removal of the potential failure zone or installing anchors to stabilize the rock mass. The rock face is then cleaned, or scaled, shaped, and finally covered with wire mesh.

In the passive mode, the slope is simply draped with the mesh. This allows weathering to continue but confines deposition of the falling rock and debris to immediately adjacent to the slope significantly reducing the rockfall energy and concentrating deposits. The passive system is typically reserved for more competent surfaces where surface erosion is more limited. Boundary ropes are often incorporated into the system for reinforcing and additional strength. Revegetation mats can be incorporated with the system to further reduce the potential for erosion and help return the slope to a more natural vegetated state. As is required for the Vegetated Reinforced Soil Slope applications, rockfall drape systems are proprietary and design and construction requirements shall be in strict conformance with the manufacturers' requirements and as further addressed in the special provisions of the project documents.

7.0 CALCULATIONS

Calculations associated with determination of the reported rock mass properties, global slope stability, bearing capacity, and rockfall analyses associated with riprap on steepened slopes are forwarded under separate cover.

8.0 ANALYSES

Specific recommendations for scaling means and methods are discussed in Section 9.0 and in the McMillen report in Appendix E. Earth pressures associated with slope debris collecting behind toe walls and allowable bearing capacity are presented in Section 8.2. Rock mass properties, design bearing capacity values, and global slope stability summaries associated with the VRSS alternatives are presented in Section 8.3.

8.1 Scaling

Figures 13 and 14 show areas recommended for scaling associated with Slopes 13 and 16. In addition scaling areas have been designated on the project plans. Figures 13 and 14 are presented to indicate the type of condition to be addressed by scaling and approximate scaling locations, but do present the entire slope face. Some rocks that have been circled, or are consistent with the conditions presented in Figures 13 and 14, may stay in place if during scaling attempts it is determined that the rock is a secure outcropping of bedrock. In addition, other rocks not indicated or circled may require scaling if the long term stability of the rock is questionable. Caution must be exercised during scaling to preclude otherwise stable rocks from becoming loosened, dislodged, or otherwise destabilized. Scaling required outside the limits depicted on the project plans or inconsistent with the conditions shown in Figures 13 and 14 may be determined during construction and shall be approved by the NDOT Engineer prior to scaling.

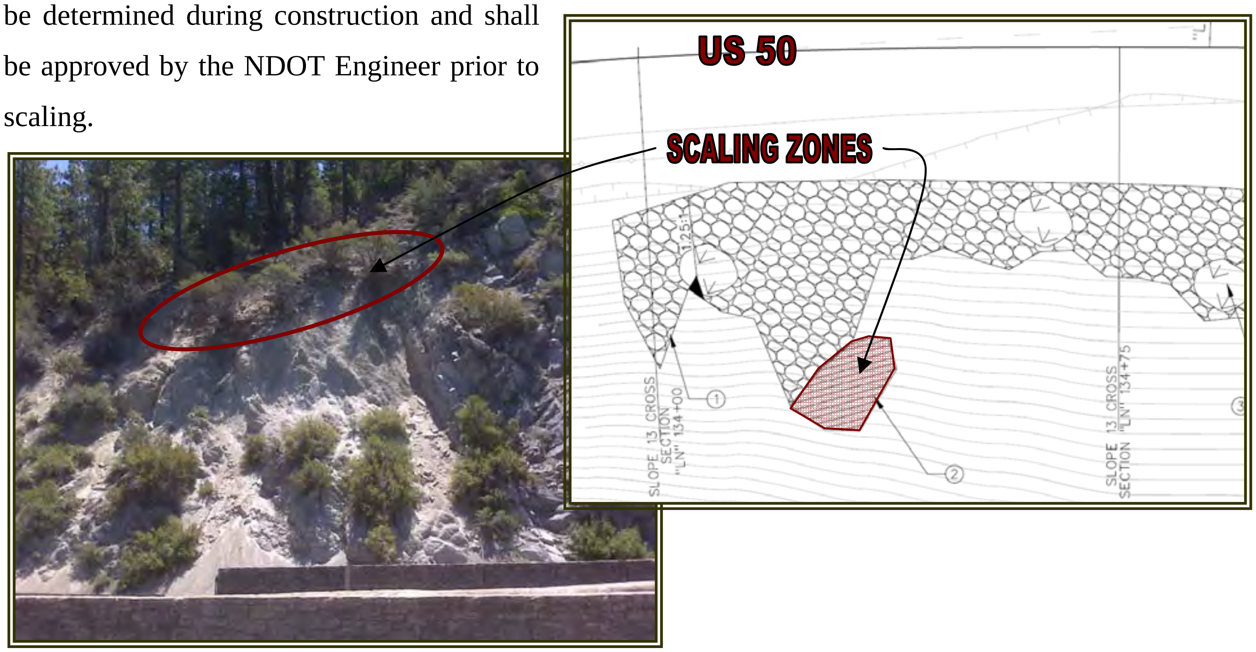

Figure 13 – Typical Scaling Zone Slope 13

The approximate volume of material expected to be generated during the scaling of Slope 13 is on the order of 10 to 15 cubic yards.

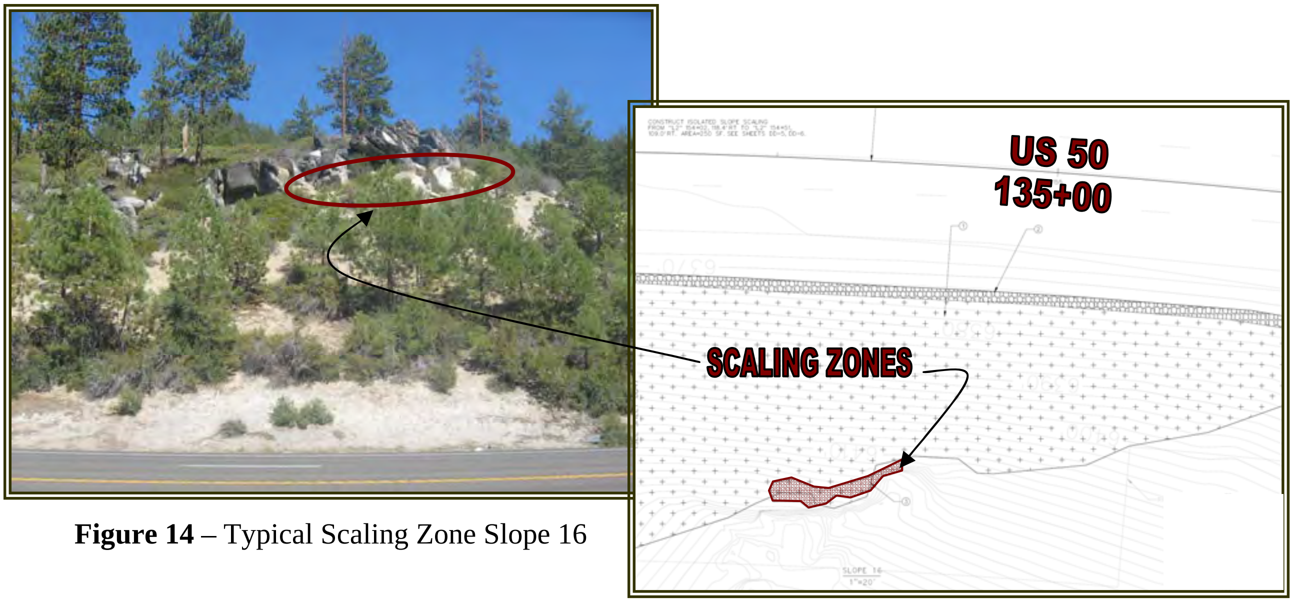

Figure 14 – Typical Scaling Zone Slope 16

The approximate volume of material expected to be generated during the scaling of Slope 16 is on the order of 20 to 25 cubic yards. Estimated scaling quantities associated with Slopes 13 and 16 are also presented in the project bid documents. It is the contractor's responsibility to protect existing improvements, personnel, and the travelling public during scaling operations.

Because the VRSS applications of Slope 21 and 27 extend to the crest of the existing cut slope, any required scaling or slope stabilization to protect existing improvements and construction personnel are considered integral to the construction of the specified improvements. Bank stability is the responsibility of the contractor, who is present at the site, able to observe changes in ground conditions, and has control over personnel and equipment.

Scaling associated with the wire mesh portion of Slope 27 is addressed in the McMillen report, included in Appendix E.

8.2 Steepened Riprap Slope Facing

Grading the slope surface to a 1.5:1 (H:V) inclination can create a significant disturbed area due to the distance required to 'catch' grades. We have evaluated mechanical stabilization of slopes steeper than 1.5:1 (H:V) using angular rip rap facing.

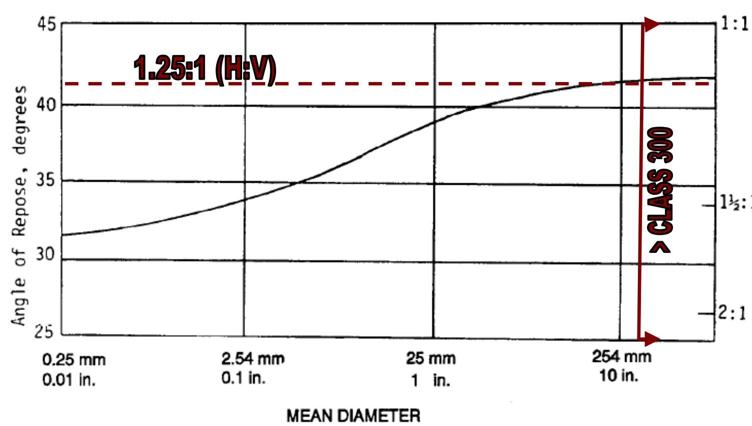

Figure 15 – Minimum Riprap Size vs. Slope Angle

Publications by the Corps of Engineers (EP 1110-1-16/ BMP-19 – Figure 15) and the State of Tennessee Division of Water Pollution Control allow for angular riprap to be place on slopes up to 1.25:1(H:V) when the median diameter (D_{50}) exceeds 10 inches, this would typically involve NDOT Class 300 riprap or larger. Placing riprap on such a significant slope will reduce the surface erosion which is a prime consideration of this project. However some unforeseeable maintenance issues associated with the steepened riprap slope may develop. Large boulders placed to form intermittent landscaped areas should be placed on a bench or key, approximately 1 ½ times the width of the boulder to comfortably seat the boulder. The key or bench should also slope gently into the hillside.

Unlike Tennessee, Nevada is in a seismically active zone. The Colorado Rockfall Simulation Program was utilized to roughly model the potential performance of the riprap on a steepened slope face in a strong earthquake. The slope was evaluated using the larger Class 550 riprap which has been specified for the slopes. Rockfall risk for an average slope of 1¼:1 (H:V) was considered and compared against a 1½:1 (H:V) slope. Rock size, drop heights, length of slope, surface roughness, tangential coefficients, and normal coefficients of restitution were varied and the volume of potential rockfall material was compared. A 1½:1 (H:V) slope will have approximately 4% less rockfall than a comparably surfaced 1¼:1 (H:V) slope. The potential for additional rockfall must be weighed against the benefit of reducing the amount of disturbed space when considering this approach.

8.3 Toe Walls

Lateral loads, such as wind or seismic, may be resisted by passive soil pressure and friction on the bottom of the footing. The recommended coefficient of base friction is 0.45 for a properly prepared subgrade. This value has been reduced by a factor of 1.5 on the ultimate soil strength. An allowable bearing capacity of 2,000 pounds per square foot may be used when evaluating subgrade support. This value may be increased by a factor of 1.33 for lateral loads such as seismic. At these design loads, calculated post-construction settlement associated with the toe walls is on the order of ½ inch or less.

Lateral earth pressures imposed on the structure are a function of soil type, moisture conditions, and adjacent slopes. Lateral resistance, i.e. passive pressure, is also a function of soil type and adjacent slope. Recommended lateral earth pressures are presented in Table 2 – Lateral Earth Pressures. These values assume a horizontal surface behind and in front of the toe walls.

Table 2 – Lateral Earth Pressures

Condition	Static (psf/ft)	Pseudo-Static (psf/ft)
Equivalent Active Fluid Pressure	35	57
Equivalent Passive Fluid Pressure	400	350

Because of the type and location of the planned structures, the potential for complete saturation and buildup of hydrostatic pressures is relatively limited. Although the values presented in Table 2 do not present values consistent with a saturated backfill, the active pressures have been increased to reflect higher backfill unit weights should some wetting of the backfill soils occur.

8.4 Vegetated Reinforced Soil Slope (VRSS)

Because the facing option needs to either be anchored to a stable slope, or a reinforced soil slope needs to be incorporated to buttress the embankment, the slope's rock mass properties had to be determined and the slope evaluated for global stability.

The rock slope was evaluated utilizing an iterative process between two rock mass evaluation methods until convergence of the Young's Modulus of the rock mass was attained. Shear wave velocity data obtained from the ReMi measurements provided a means to evaluate the elastic properties of the rock mass based on shear modulus and Poisson's ratio. Strength properties of the rock, i.e. cohesion and angle of internal friction based on unconfined compression strength test data and the data presented in the Rockfall Hazards Rating Program, as well as the Geological Strength Index (GSI) were evaluated using the computer program RocLab. The GSI was varied until convergence in Young's Modulus was attained. RocLab utilizes the rock strength parameters to evaluate the shear strength of closely jointed rock masses. When a hard rock mass contains a number of closely spaced joint sets, the behavior of the rock mass can vary substantially from what isolated test data would indicate. RocLab presents strength parameters for the rock mass. These values were used in evaluation of the global slope stability in the vicinity of the planned VRSS options for global stability.

The computer program XSTABL was also utilized to evaluate the global slope stability. XSTABL is useful for evaluating the weathered rock mass which behaves as a strong soil mass as opposed to a fractured rock material. Based on the rock mass parameters determined, the slopes under consideration meet the minimum factors-of-safety for global stability allowing the VRSS system to be approached as a facing rather than a buttressing fill. Factors of Safety determined from the XSTABL analyses are summarized in Table 3.

Table 3 – XSTABL Summary		
Location	Evaluation Condition	Factor of Safety
Slope 21 – Sta 276+22	Static	2.5
Slope 21 – Sta 276+22	Pseudo-Static	1.7
Slope 27 – Sta 330+81	Static	4.5
Slope 27 – 330+81	Pseudo-Static	3.4

The parameters determined from our rock mass evaluation are presented in Table 4 for use in consideration of Slopes 21 and 27. These values may also be used when considering anchor design for the VRSS system. However, because the rock mass varies between competent rock to an extensively altered and decomposed soil like material, anchor capacity will vary significantly across the slope face.

Table 4 – Summary of Rock Mass Properties*				
Slope	Description	Unit Weight γ_m – pcf	Internal Friction Angle – φ	Cohesion c- psf
21	Regolith – Weathered Tuff – Medium Dense - Coarse Sand with some Gravel Consistency	125	43	50
21	Weathered Tuff	150	34	1300
27	Regolith – Weathered Monzogranite – Medium Dense – Clay, Sand and Gravel Consistency	125	43	50
27	Weathered Monzogranite	135	44	3200
* Material is highly variable, and significant changes in physical properties across the slope face should be anticipated.				

An allowable bearing pressure of 5,000 pounds per square foot may be used when considering the VRSS design. This value may be increased by a factor of 1.33 when considering seismic loading. Ultimately the allowable bearing capacity and associated settlement are a function of final bearing depth, foundation support shape and structural characteristics and should be evaluated as part of the patentee's design package.

8.5 Wire Mesh (Cable Anchored)

Rockfall, and rock slope stability analyses and calculations are presented in Appendix E in the Rock Mechanics Cut Slope Stability Report, U.S. Highway 50, Slope 27, Douglas County, Nevada, by McMillen, LLC.

9.0 RECOMMENDATIONS

Unless modified by the Special Provisions, the means, methods, and materials required to perform the recommendations presented herein shall be in accordance with the NDOT Standard Specifications for Road and Bridge Construction (SSR&BC). Anchor design shall be in accordance with FHWA Publication No. FHWA-IF-99-015, Geotechnical Engineering Circular No. 4.

9.1 Slope 13

Scaling is required for the upper reaches of Slope 13. Scaling shall be to the extents indicated on the plans. Most of the planned scaling is to occur at the top of the slopes where active erosion occurs and undermines cobble and boulder sized particles exposed in the slope face. Scaling is a subjective process and the contractor should be specifically trained and experienced in this type work. In addition, the contractor is responsible for the means and methods to confine and contain dislodged rock within the limits of construction and without compromising safety. Slope scaling should begin at the top of the slope and proceed downward. Any loose or disturbed material that compromises safety shall be addressed. Chaining the slope face or dragging heavy objects is not allowed. The contractor must exercise due care in the execution of his duties to not to undermine otherwise stable soils and rock masses thereby creating conditions that would require additional scaling. During scaling activities, trees that pose a potential hazard from toppling should be cut down or otherwise addressed. The root system of removed trees shall be left in place to inhibit slope erosion. Reasonable precautions shall be implemented by the scaling contractor to limit damage to existing vegetation on and above cut slopes during scaling and tree cutting.

We anticipate the Contractor’s scaling equipment to include, but not be limited to: rappelling gear, rakes, pry bars, handheld hydraulic splitters, jackhammers, and construction equipment to raise manpower to the loose rock or outcrops or to use cables to pull loose boulders (rock larger than 12-inches in size) off the slopes. Except where grading is called for to address slope gradient, trackhoe or backhoe buckets shall not be used to scrape slopes.

Where existing vegetation is deemed adequately established no further mitigation efforts would be required. Where erosion is deteriorating the slope face and causing sediment build-up at the toe a combination of grading, vegetation, and riprap stabilization would be beneficial. Riprap size and placement shall be as indicated on the project plans. Based on our rockfall analysis, it is recommended that Class 550 riprap be used.

Prior to placing riprap, the slope surface shall be prepared as addressed in Section 610 of the SSR&BC. Riprap placement shall also be in accordance with Section 610 and the project's special provisions. Stone for riprap shall meet the requirements of Section 706.03.05 and should be large enough to reduce the potential for displacement. Minimum riprap size as a function of slope gradient is discussed in Section 8.0 of this report, Analyses. In addition, the base of the riprap zone shall be keyed into the slope toe. Minimum boulder size to key into the base of the slope should be Class 700 and on the order of 5 feet. The excavation created to key in the Class 700 toe boulder should be backfilled as required by Portland Cement Concrete. Boulders placed on the slope face to form intermittent landscaped areas should be placed on a bench or key adequately sized to comfortably seat the boulder. Riprap bedding is not recommended.

To facilitate vegetation, riprap can be cast with top soil to fill surface voids and hydroseeded with a mixture of native shrubs and grasses, a method referred to a top-soiled riprap. The topsoil is typically 'washed' into the riprap by application of water from a water truck. If for aesthetic purposes it is desired to create a slope with intermittent zones of vegetated and non-vegetated riprap, placing a weed inhibiting landscaping fabric beneath non-vegetated zones would be beneficial. The landscaping fabric must be placed so that it conforms closely to the subgrade and should be secured in accordance with the manufacturer's instructions. Woody vegetation planted in select areas will also help break up the potentially monotonous appearance of a riprap system.

All excavations should be performed in accordance with Section 203 and 206 of the SSR&BC and stabilized in accordance with local, state, and federal OSHA standards. Bank stability is the responsibility of the contractor. The contractor's onsite personnel are able to observe changes in ground conditions, and have control over personnel and equipment. Surcharge loads from adjacent embankments, equipment, etc. must be specifically evaluated for conditions created by the contractor. Site geotechnical units will vary between competent bedrock to OSHA Type C soils depending on the alteration and degradation of the bedrock.

9.2 Slope 16

Slope 16 is fairly stable and well vegetated. Therefore, improvements to Slope 16 will be limited to spot revegetation and a riprap zone to help stabilize the toe. Slope preparation for riprap shall be in accordance with the SSR&BC and as previously discussed for Slope 13.

9.3 Slope 21

It is our opinion the Vegetated Reinforced Soil Slope (VRSS) presents a unique solution to Slope 21. Slope strength parameters necessary for anchor design have been presented in Section 8.2. Slope debris and slough as encountered within the western limits of the slope must be removed prior to installing the VRSS (Section 203 SSR&BC). Reinforced soil slopes or anchored facings and processes are patented and as such strict adherence to the manufacturer's design, plans, and specifications is required, unless more restrictive requirements are stipulated in the contract documents. Alternatives must be presented for dealing with competent bedrock zones where the design limits of the VRSS system cannot be met without blasting, and, for addressing the condition where the facing unit does not extend flush with the backslope. Anchoring or shoring systems required to affix the facing to the slope or stabilize the facing are integral to the system and shall be as specified and required by the patentee/manufacturer.

Slopes to receive the VRSS treatment shall be excavated to the lines and extents indicated on the plans and shall be in accordance with Section 203 of the Standard Specifications for Road and Bridge Construction. Surcharge loads from adjacent embankments, equipment, etc. must be specifically evaluated for. Site geotechnical units will vary between competent bedrock to OSHA Type C soils depending on the alteration and degradation of the bedrock.

Any slope debris, remaining slough, or excessively loose or disturbed material generated during excavation shall be removed prior to preparing subgrade. The exposed subgrade should be moisture conditioned as necessary and proof-rolled to create a firm and relatively unyielding subgrade. The subgrade shall be deemed adequate if sufficient compaction can be readily attained in the initial lift during backfilling of the facing units.

9.4 Slope 27

Two approaches are recommended to address the significant variations in slope conditions evident along 27. A Vegetated Reinforced Soil Slope (VRSS) is recommended for the western and eastern ends of the slope, along with a rockfall netting/drape system for the central portion. Ultimately the planned improvements must stabilize the existing slope surface, establish acceptable design grades, and meet NDOT's vegetation goals.

Design considerations for the rockfall drape are presented in the McMillen report. Design parameters for the VRSS have been presented in Section 8.3 of this report. Slope debris and slough as encountered within the western limits of the slope must be removed prior to installing the VRSS (Section 203

SSR&BC). As part of the design submittal, the manufacturer/contractor must show how installation of their product shall be modified where the VRSS units do not extend to the backslope. The central portion of slope 27 and the upper reaches of the eastern limits of Slope 27 present a fairly competent bedrock face which will require special considerations and procedures for the VRSS fascia where the ability to excavate to the extent of the facing units is limited. Therefore, alternatives must be presented for dealing with competent bedrock zones where the design limits of the VRSS system cannot be met without blasting. Anchoring systems shall meet the requirements of the Special Provisions.

9.5 Prioritization of Slopes

Eminent slope failure and rockfall assessments have been provided in the previously referenced documents. Therefore, the scope of this prioritization discussion relates solely to our planned means of redress for the contracted tasks in our scope of services and more specifically, Slopes 21 and 27.

The steep terrain of Slopes 21 and 27 along with thick accumulation of either alluvium or regolith at the slope crown drive our approaches to these slopes. As the alluvium becomes wet, either due to infiltration or precipitation, the available cohesion becomes reduced, limited pore pressures may develop, and subsequently the 'glue' holding the headwall in place gives way. Freeze thaw cycles may also be contributory. This phenomenon is evidenced by the cornice structure, or headwall, which has developed at the top of the referenced slope areas. If the gradients of the lower reaches of slopes were less, the slough would accumulate near the bedrock/alluvium contact and the cornice would self-stabilize. Because the lower slope is so steep, the slough rolls downhill until the energy is either dissipated somewhere along the slope or the sediment is carried into the toe ditch and possibly onto the highway depending on the magnitude of the fall. This scenario will continue until the upper reaches of the slopes are stabilized. **The urgency to address Slope 27 is compounded when encroachment and additional environmental issues are triggered once the NDOT ROW is exceeded.** The prioritization of slope stabilization for this phase is presented in Table 5 below:

Table 5 – Prioritization of Slope Mitigation

	Slope	Station	Treatment
1	27	334+77 to 335+92	Steeply Sloping Vegetated Face w/ Anchors
2	27	329+14 to 331+68	Steeply Sloping Vegetated Face w/ Anchors
3	21	275+57 to 286+97	Steeply Sloping Vegetated Face w/ Anchors
4	13	133+94 to 137+02	Grade & Riprap

5	16	151+03 to 156+35	Revegetation
6	27	331+67 to 334+60	Rockfall Drape w/ Catchment Wall

10.0 REFERENCES

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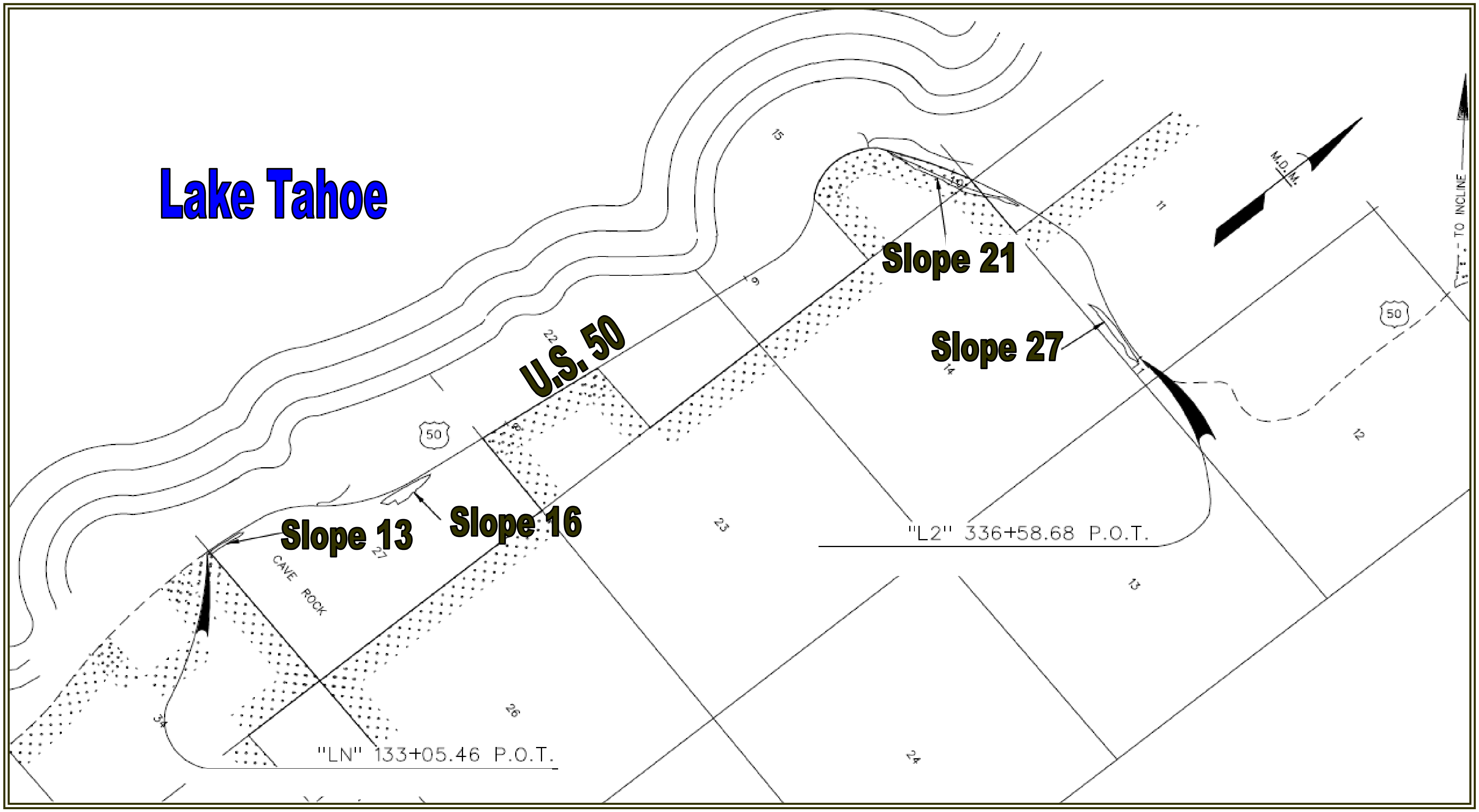
APPENDIX A
FIGURES



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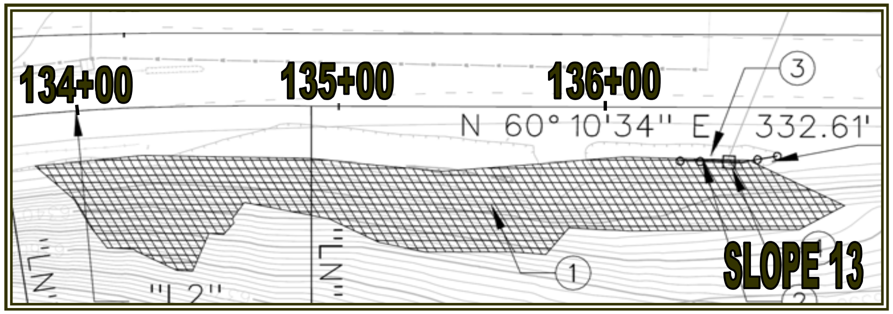
VICINITY MAP

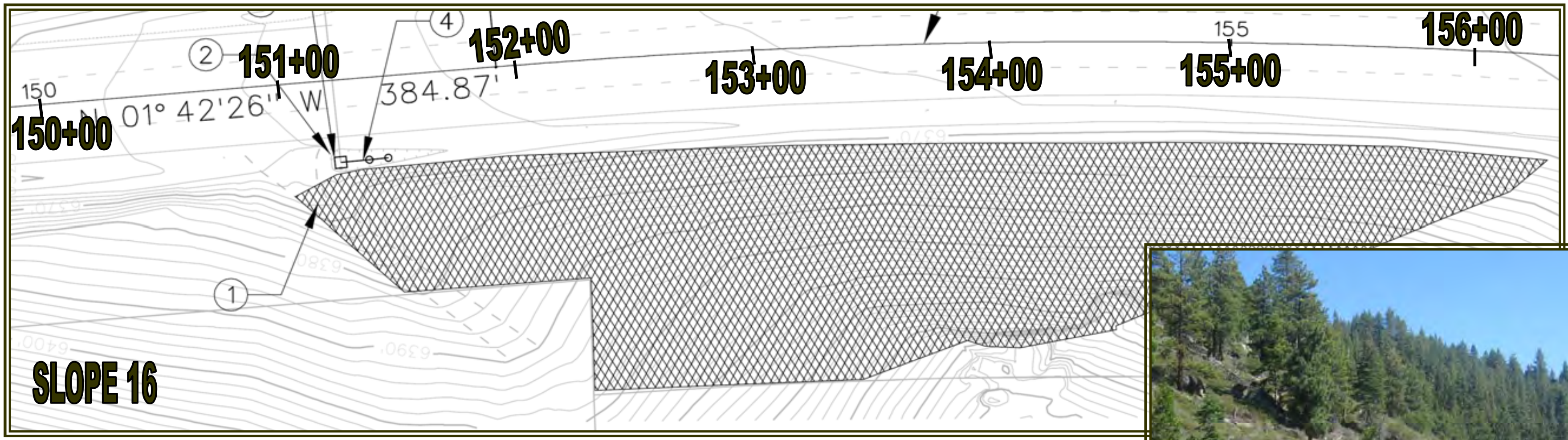
Geotechnical Investigation

LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US 50

Project No.: 8049.003
Date: 11/15/09

PLATE A-1





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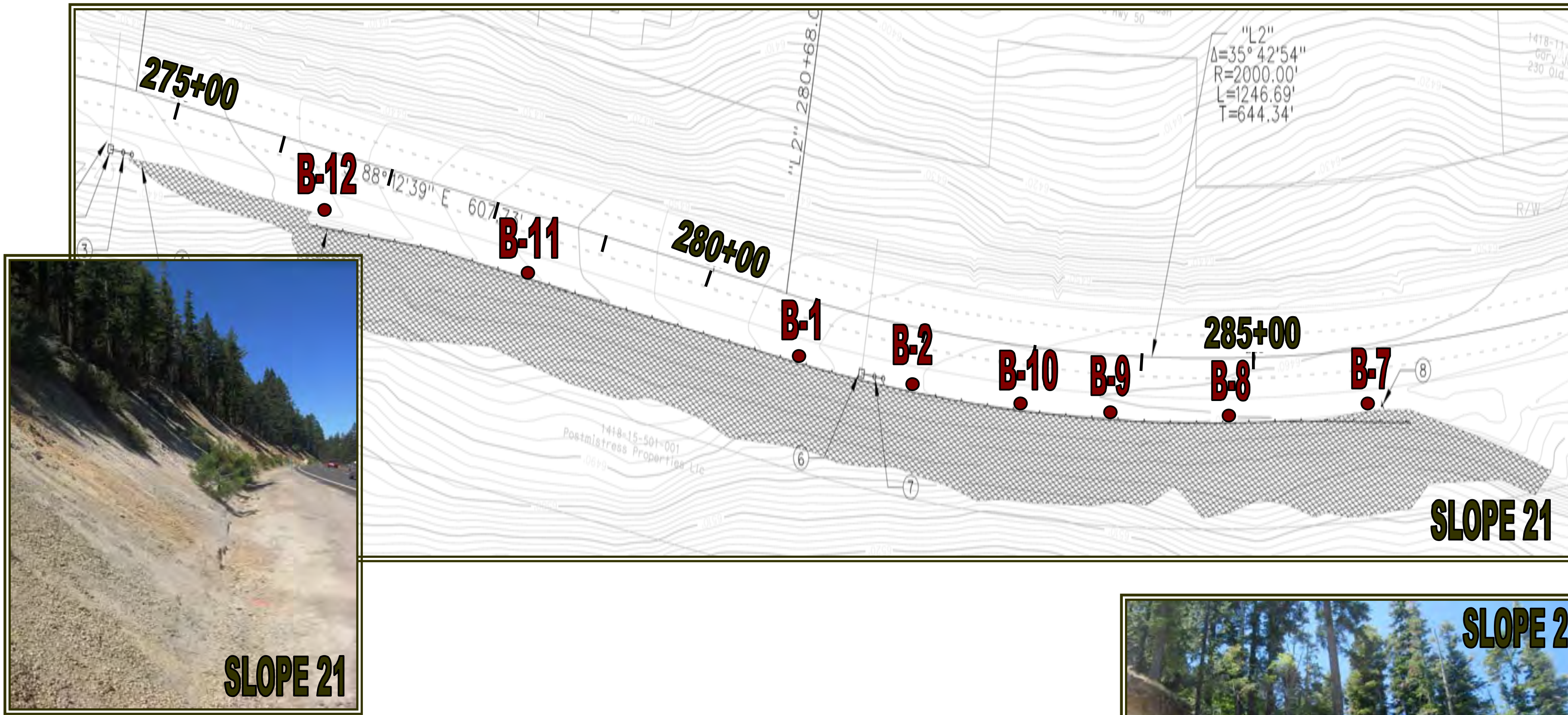
SLOPES 13 & 16
TOPOGRAPHIC MAPS

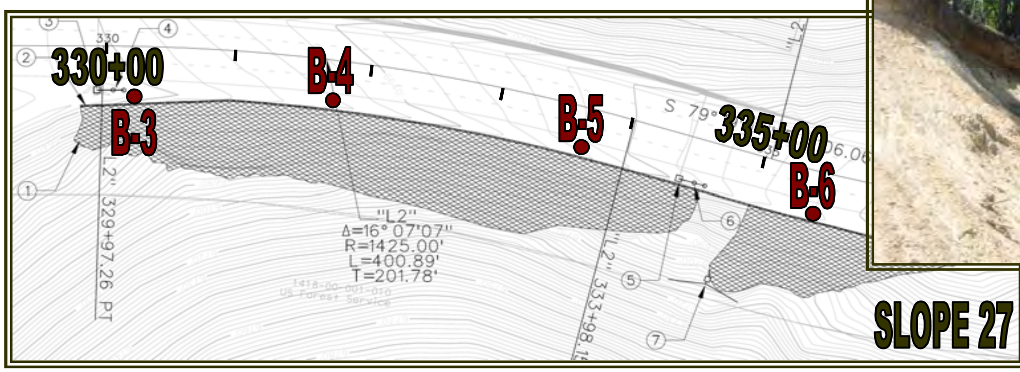
Geotechnical Investigation

LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US 50

Project No.: 8049.003
Date: 11/15/09

PLATE A-2





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SLOPES 21 & 27 TOPOGRAPHIC MAPS AND APPROXIMATE BORING LOCATIONS

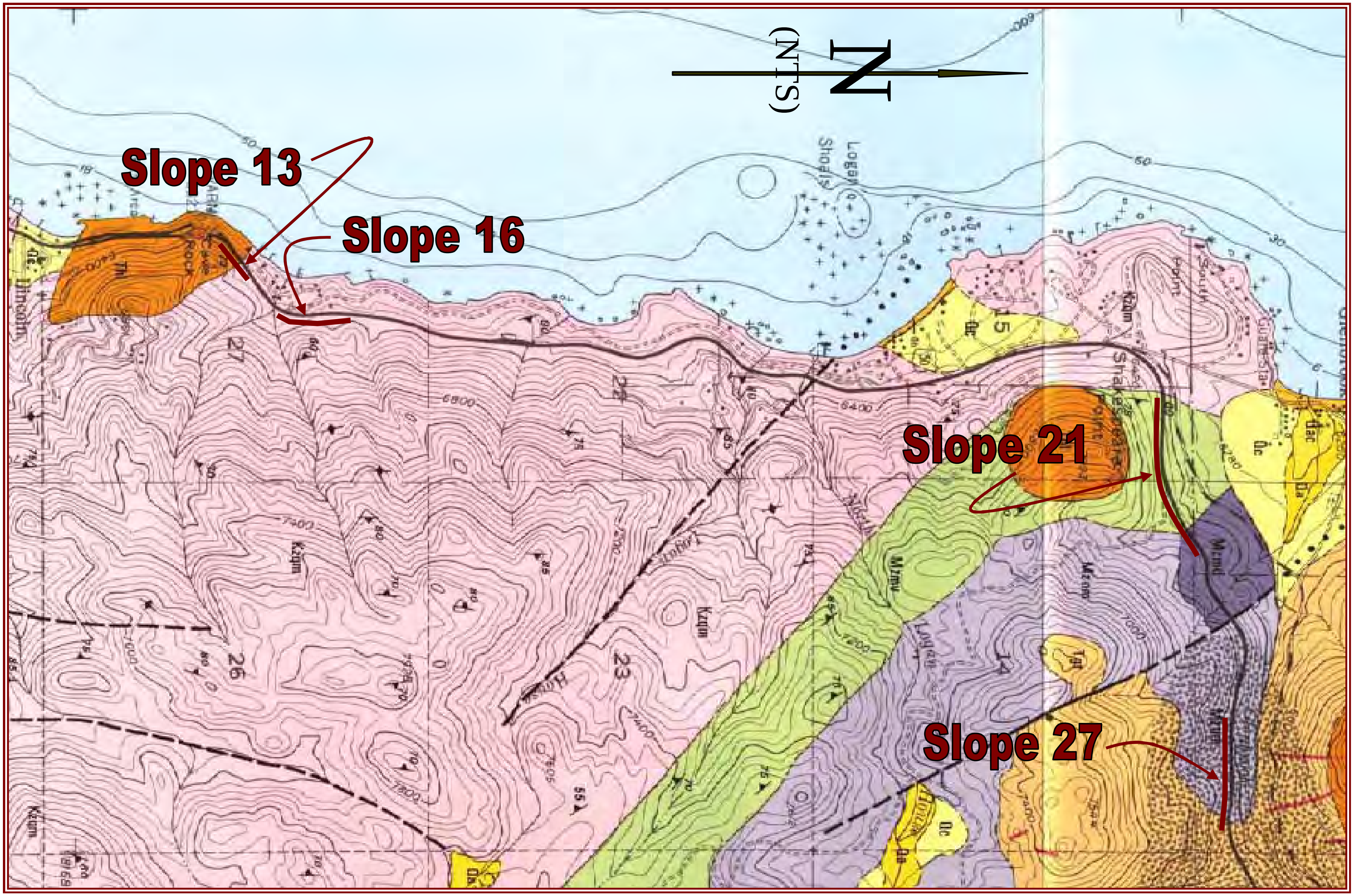
Geotechnical Investigation

LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US 50

Project No.: 8049.003

Date: 11/15/09

PLATE A-3



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GEOLOGIC MAP

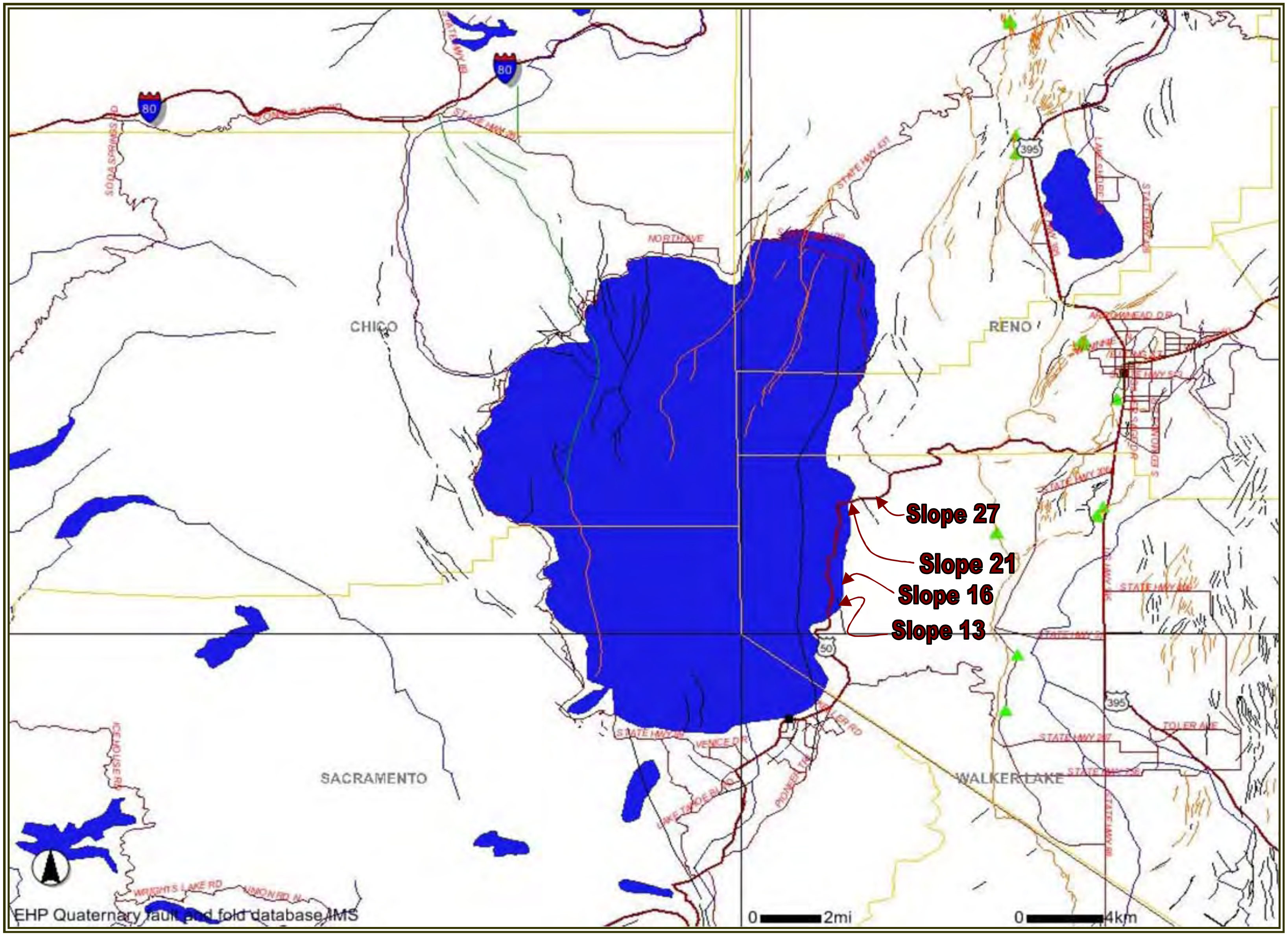
Geotechnical Investigation

LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US 50

Project No.: 8049.003

Date: 11/15/09

PLATE A-4



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USGS QUATERNARY FAULT MAP AND APPROXIMATE SLOPE LOCATIONS

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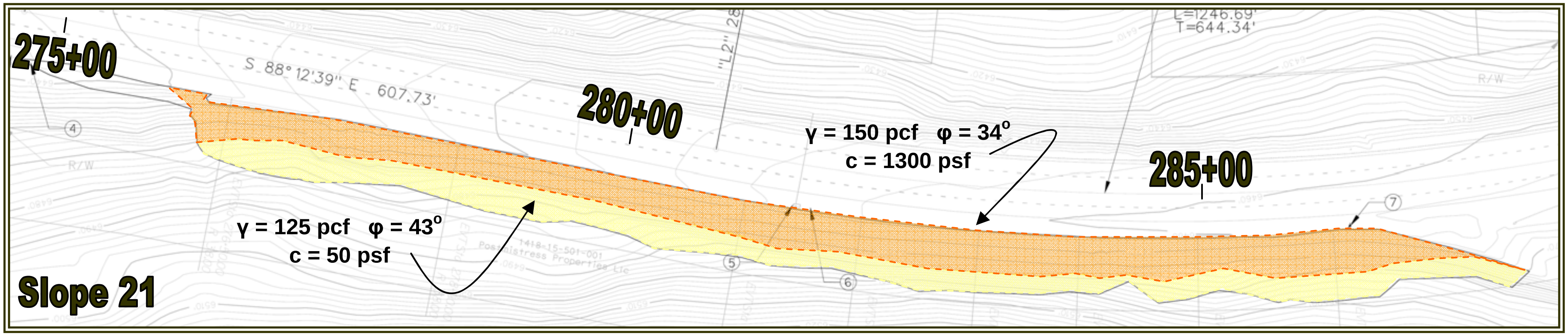
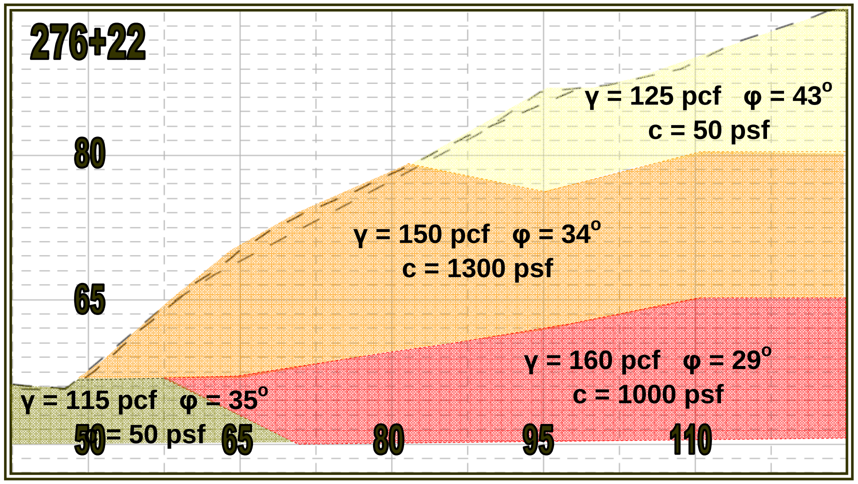
LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US 50

Project No.: 8049.003

Date: 11/15/09

PLATE A-5





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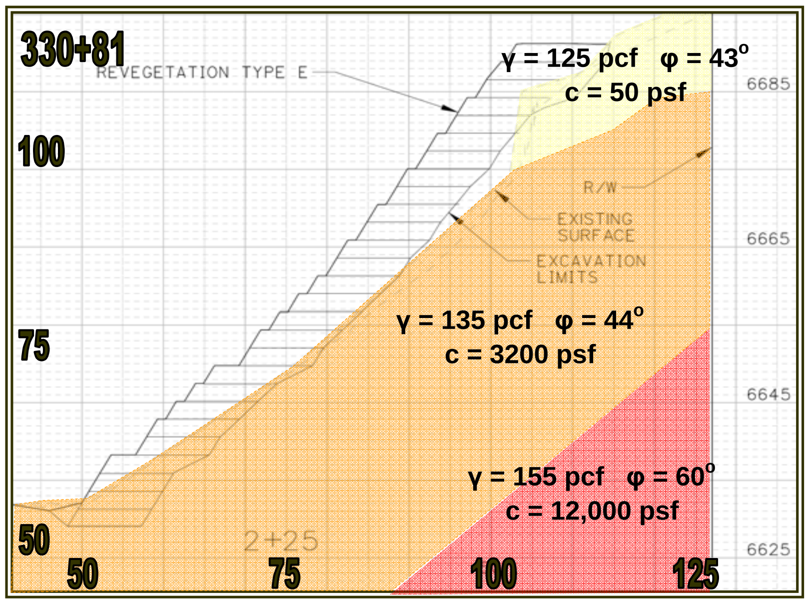
SLOPE 21
SECTION AND PLAN OF
ROCK MASS PROPERTIES

Geotechnical Investigation

LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US 50

Project No.: 8049.003
Date: 11/15/09

PLATE A-6



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SLOPE 27
SECTION AND PLAN OF
ROCK MASS PROPERTIES

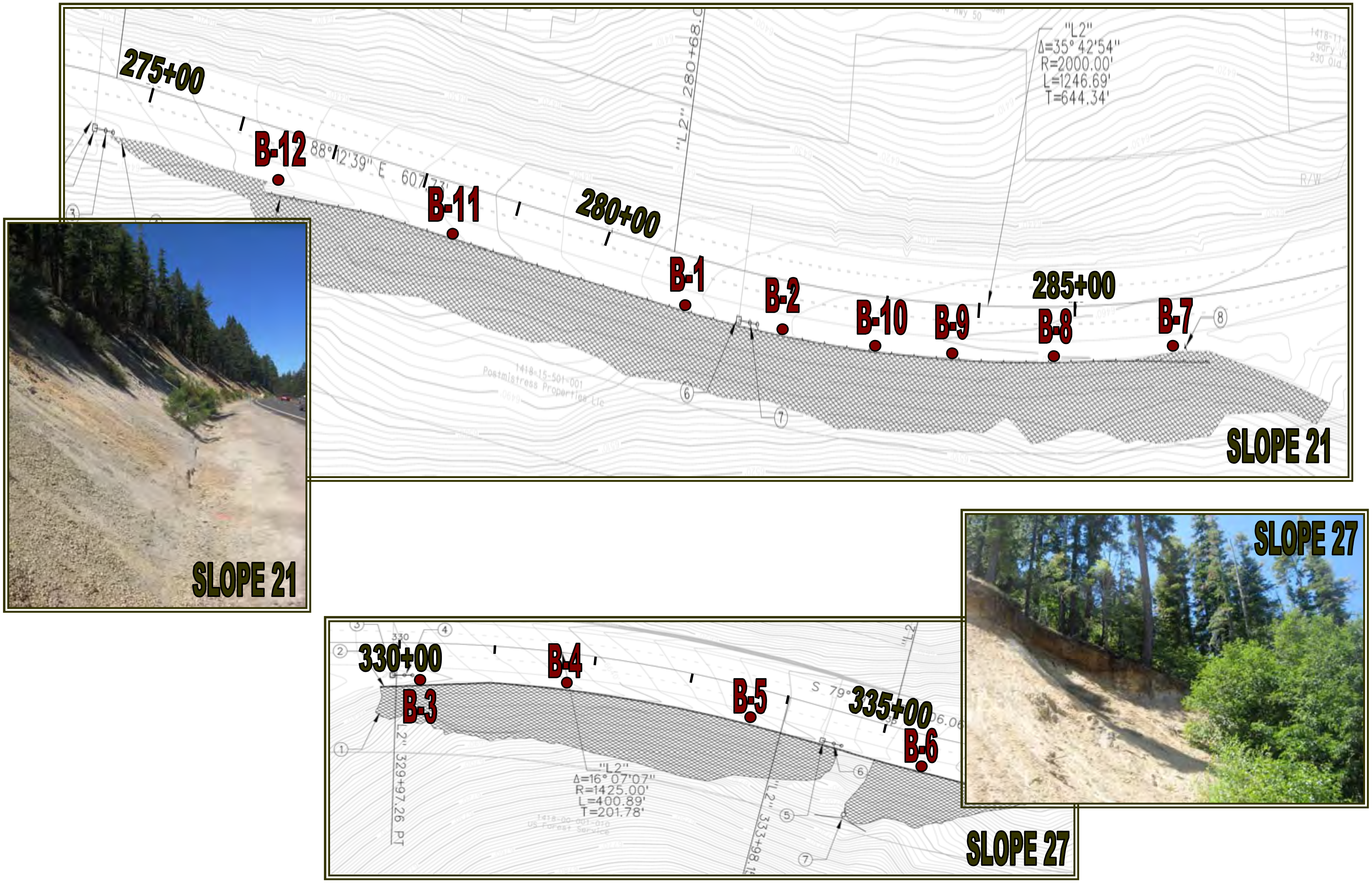
Geotechnical Investigation

LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US 50

Project No.: 8049.003
Date: 11/23/09

PLATE
A-7

APPENDIX B
SUBSURFACE EXPLORATIONS



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SITE PLAN AND APPROXIMATE EXPLORATION LOCATIONS

Geotechnical Investigation

LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US 50

Project No.: 8049.003

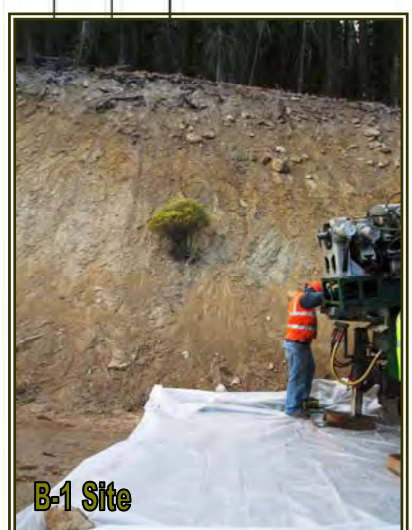
Date: 11/15/09

PLATE B-1

BORING LOG

SURF. EL. : 6455.5 FEET	WATER EL. : None	DEPTH : 20.25 FEET	LOCATION : Station 281+00 RT-Shoulder
EQUIPMENT : Sonic Spider #2			

DEPTH, Ft.	SOIL DESCRIPTION	% PASSING #200 SIEVE	BLOWS/Ft.	LIQUID LIMIT(%)	PLASTICITY INDEX (%)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LABORATORY TESTING
	WEATHERED BEDROCK; Metamorphosed Tuff and Flows (Mzmv) - massive to thickly bedded, moderately hard, friable, deeply weathered excavates to a very dense silty sand consistency (-200 increases to approximately 70% when abraded) bulk sample from 1.5 to 5'		30					
		70.4		34	5	16.3	95.5	MA
5			68					
	bulk sample from 6 to 10'							
	DSCD ø = 36°, c= 0 psf	38.7		NP	NP	6.7	108.8	DSCD= 36; MA
10			50/ 0.4'					
	bulk sample from 11.5 to 13.5'							
15			63					
	bulk sample from 17 to 18'							
20	Boring terminated at 20.25 feet. No free ground water encountered at time of drilling.		50/ 0.3'					



REMARKS :

FIELD ENG. : MJS	WATER DEPTH @ COMPL. : None	DATE : 10-06-09

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

LOG OF B-1
Lake Tahoe Environmental Improvement
Program US50 - Phase III

FIGURE
2

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

BORING LOG 8049.003_US50 SLOPES.GPJ WOOD RODGERS.GDT 11/16/09

BORING LOG

SURF. EL. : 6458.0 FEET	WATER EL. : None	DEPTH : 20.25 FEET	LOCATION : Station 282+00 RT-Shoulder
EQUIPMENT : Sonic Spider #2			

DEPTH, Ft.	SOIL DESCRIPTION	% PASSING #200 SIEVE	BLOWS/Ft.	LIQUID LIMIT(%)	PLASTICITY INDEX (%)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LABORATORY TESTING
	BROWN SILTY SAND (SM); dense, moist							
	ASPHALT; grindings							
	WEATHERED BEDROCK; Metamorphosed Tuff and Flows (Mzmv) - massive to thickly bedded, moderately hard, friable, deeply weathered excavates to a very dense silty sand consistency (-200 approaches 52% when abraded)		37					
	bulk sample from 1 to 4'	52.4		NP	NP	13.9	99.9	MA
5			58					
	bulk sample from 7 to 9'							
10	becoming dark gray at 10'		53					
	bulk sample from 12 to 14'							
	DSCD ø = 38°, c= 0 psf	27.4		NP	NP			DSCD= 38; MA
15						7.8	104.5	
			73					
	bulk sample from 17 to 19'							
20	Boring terminated at 20.25 feet. No free ground water encountered at time of drilling.		50/ 0.3'					

REMARKS :

FIELD ENG. : MJS	WATER DEPTH @ COMPL. : None	DATE : 10-06-09

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

LOG OF B-2
Lake Tahoe Environmental Improvement
Program US50 - Phase III

FIGURE
3

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

BORING LOG 8049.003_US50 SLOPES.GPJ WOOD RODGERS.GDT 11/16/09

BORING LOG

SURF. EL.: 6629.5 FEET	WATER EL.: None	DEPTH: 20 FEET	LOCATION: Station 330+30 RT-Shoulder

EQUIPMENT: Sonic Spider #2

DEPTH, Ft.	GRAPHIC LOG	SAMPLE	SOIL DESCRIPTION	% PASSING #200 SIEVE	BLOWS/Ft.	LIQUID LIMIT(%)	PLASTICITY INDEX (%)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LABORATORY TESTING
			ORANGE BROWN SILTY SAND (SM); medium dense, moist to wet							
			ASPHALT; grindings							
			ORANGE BROWN CLAYEY SAND (SC); medium dense, moist to wet							
			WEATHERED BEDROCK; Mapped as Biotite Monzogranite of North Logan House Creek (Mznm) -intensely to closely fractured, moderately soft to moderately hard, moderately strong to strong, moderately weathered, excavates to a very dense clayey sand with gravel consistency	17.3		29	12	9.1	99.7	DSCD= 39.2; MA
5			bulk sample from 2-4' DSCD ø = 39.2°, c= 56.6 psf							
					56					
			bulk sample from 7 to 9'							
10			jointed, fractured, and plastic at 10', very moist to wet							
					68/ 0.7'					
			becoming hard at 12.5'							
15										
			altered zone, soft at 16 to 16.5' estimated RQD > 50 from 16.5 to 18.5' bulk sample from 17 to 19'							
			altered zone, soft at 18.5 to 19.5'							
20			Boring terminated at 20 feet. No free ground water encountered at time of drilling.							

REMARKS:

FIELD ENG.: MJS	WATER DEPTH @ COMPL.: None	DATE: 10-07-09

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

LOG OF B-3
Lake Tahoe Environmental Improvement Program US50 - Phase III

FIGURE 4

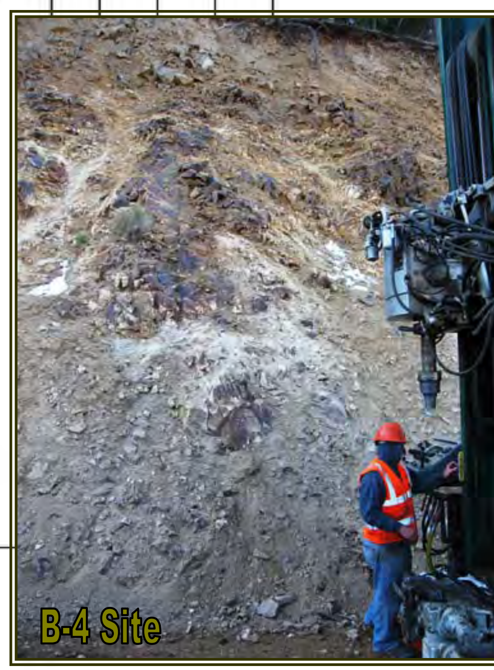
DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

BORING LOG

SURF. EL.: 6635.5 FEET	WATER EL.: None	DEPTH: 10 FEET	LOCATION: Station 331+70 RT-Shoulder
EQUIPMENT: Sonic Spider #2			

DEPTH, Ft.	SOIL DESCRIPTION	% PASSING #200 SIEVE	LIQUID LIMIT(%)	PLASTICITY INDEX (%)	MOISTURE CONTENT (%)	LABORATORY TESTING
0	ORANGE BROWN SILTY SAND (SM); medium dense, moist to wet					
1	WEATHERED BEDROCK; Mapped as Biotite Monzogranite of North Logan House Creek (Mznm) -intensely to closely fractured, moderately hard to hard, moderately strong to strong, moderately weathered, excavates to a very dense clayey sand consistency					
	bulk sample from 1 to 3'	22.1	19	4	4.9	MA
	bulk sample from 3 to 5'	18.1	19	3	3.5	MA
5						
	becoming very strong at 6'					
	bulk sample from 5 to 10'					
	unconfined - 3,000 PSI					
10	Boring terminated at 10 feet. No free ground water encountered at time of drilling.					
15						
20						

REMARKS:



FIELD ENG.: MJS	WATER DEPTH @ COMPL.: None	DATE: 10-07-09

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

LOG OF B-4
Lake Tahoe Environmental Improvement Program US50 - Phase III

FIGURE **5**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

BORING LOG

SURF. EL.: 6658.0 FEET	WATER EL.: None	DEPTH: 20 FEET	LOCATION: Station 335+40 RT-Shoulder
EQUIPMENT: Sonic Spider #2			

DEPTH, Ft.	SAMPLE	SOIL DESCRIPTION	% PASSING #200 SIEVE	BLOWS/Ft.	LIQUID LIMIT(%)	PLASTICITY INDEX (%)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LABORATORY TESTING
		ORANGE BROWN SILTY SAND (SM); medium dense, moist to wet							
		ASPHALT; grindings							
	X	WEATHERED BEDROCK; mapped as Altered Hornblende Diorite of Montreal Canyon (Mzmd) - closely to intensely fractures, moderately soft, plastic to friable, deep to moderately weathered excavated to a very dense clayey sand consistency		30					
		bulk sample from 2 to 4'							
		DSCD ø = 34.9°, c= 427 psf	24.7		32	12	11.1	113.4	DSCD= 34.9; MA
5									
	X			50/ 0.5'					
		bulk sample from 7 to 9'							
			20.8		29	13	7.4		MA
10	X	grades to hard near 10', joints become wet		50/ 0.5'					
		bulk sample from 12 to 14'							
15	O	no recovery		50/ 0.3'					
		bulk sample from 17 to 19'							
		becoming moderately soft between 18 and 19'							
20		Boring terminated at 20 feet. No free ground water encountered a time of drilling.							

REMARKS:

FIELD ENG.: MJS	WATER DEPTH @ COMPL.: None	DATE: 10-07-09

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

LOG OF B-6
Lake Tahoe Environmental Improvement Program US50 - Phase III

FIGURE 7

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

BORING LOG 8049.003_US50 SLOPES.GPJ WOOD RODGERS.GDT 11/16/09

BORING LOG

SURF. EL. : 6462.0 FEET	WATER EL. : None	DEPTH : 10.5 FEET	LOCATION : Station 286+00 RT-Shoulder
EQUIPMENT : Sonic Spider #2			

DEPTH, Ft.	SOIL DESCRIPTION	% PASSING #200 SIEVE	BLOWS/Ft.	LIQUID LIMIT(%)	PLASTICITY INDEX (%)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	LABORATORY TESTING
	BROWN SILTY SAND (SM); medium dense, moist		27					
	WEATHERED BEDROCK; Mapped as Metamorphosed Tuff and Flows (Mzmv) - massive to thickly bedded, moderately soft to moderately hard, plastic to friable, deeply to moderately weathered excavates to a very dense clayey sand consistency							
	bulk sample from 2.5 to 4'	50.0		NP	NP	8.4		MA
5			23			6.6	103.9	
	bulk sample from 6 to 8'	38.1		NP	NP			MA
10			50/ 0.5'					
	Boring terminated at 10.5 feet. No free ground water encountered a time of drilling.							
15								
20								

REMARKS :

FIELD ENG. : MJS	WATER DEPTH @ COMPL. : None	DATE : 10-08-09

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

LOG OF B-7
Lake Tahoe Environmental Improvement Program US50 - Phase III

FIGURE **8**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

BORING LOG

SURF. EL.: 6461.0 FEET	WATER EL.: None	DEPTH: 10 FEET	LOCATION: Station 284+85 RT-Shoulder
EQUIPMENT: Sonic Spider #2			

DEPTH, Ft.	GRAPHIC LOG	SAMPLE	SOIL DESCRIPTION	% PASSING #200 SIEVE	BLOWS/Ft.	LIQUID LIMIT(%)	PLASTICITY INDEX (%)	LABORATORY TESTING
			BROWN SILTY SAND (SM); medium dense, moist		27			
			WEATHERED BEDROCK; Hornblende Diorite of Montreal Canyon (Mzmd) - closely to moderately fractured, moderately hard to hard, moderately weathered excavates to a very dense silty sand consistency bulk sample from 2 to 4'					
5					50/ 0.3'			
				38.1		NP	NP	MA
			bedrock becoming harder at 8', occasionally to moderately fractured bulk sample from 8 to 9' Unconfined Compression - 6,600 PSI					
10								
			Boring terminated at 10 feet. No free ground water encountered a time of drilling.					
15								
20								

REMARKS:

FIELD ENG.: MJS	WATER DEPTH @ COMPL.: None	DATE: 10-08-09

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

LOG OF B-8
Lake Tahoe Environmental Improvement Program US50 - Phase III

FIGURE 9

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

BORING LOG

SURF. EL. : 6460.0 FEET	WATER EL. : None	DEPTH : 18 FEET	LOCATION : Station 283+85 RT-Shoulder
EQUIPMENT : Sonic Spider #2			

DEPTH, Ft	GRAPHIC LOG	SAMPLE	SOIL DESCRIPTION	LABORATORY TESTING
			WEATHERED BEDROCK; Hornblende Diorite of Montreal Canyon (Mzmd) - closely to moderately fractured, occasionally fractured in some zones, moderately hard to hard, strong, moderately weathered excavates to a very dense silty sand with gravel consistency	
			bulk sample from 3 to 4.5'	
5				
			bulk sample from 7 to 9' Unconfined Compression - 6,700 PSI	
10				
			bulk sample from 12.5 to 14.5' Unconfined Compression - 4,300 PSI	
15				
			becoming hard, strong, and fresh at 16' bulk sample from 16 to 18' Unconfined Compression - 6,900 PSI	
			Boring terminated at 18 feet. No free ground water encountered a time of drilling.	
20				

REMARKS :

FIELD ENG. : MJS	WATER DEPTH @ COMPL. : None	DATE : 10-08-09

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

LOG OF B-9
Lake Tahoe Environmental Improvement Program US50 - Phase III

FIGURE **10**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

BORING LOG 8049.003_US50 SLOPES.GPJ WOOD RODGERS.GDT 11/16/09

BORING LOG

SURF. EL. : 6460.0 FEET	WATER EL. : None	DEPTH : 18 FEET	LOCATION : Station 283+00 RT-Shoulder
EQUIPMENT : Sonic Spider #2			

DEPTH, Ft.	GRAPHIC LOG	SAMPLE	SOIL DESCRIPTION	LABORATORY TESTING
			LIGHT BROWN SILTY SAND (SM); medium dense, moist	
			WEATHERED BEDROCK; Hornblende Diorite of Montreal Canyon (Mzmd) - closely to moderately fractured, occasionally fractured in some zones, moderately hard to hard, strong, moderately weathered excavates to a very dense silty sand with gravel consistency	
			bulk sample from 3 to 4.5'	
5				
10				
15			becoming hard, strong, and fresh at 16'	
			Boring terminated at 18 feet. No free ground water encountered a time of drilling.	
20				

REMARKS :

FIELD ENG. : MJS	WATER DEPTH @ COMPL. : None	DATE : 10-08-09

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

LOG OF B-10
Lake Tahoe Environmental Improvement
Program US50 - Phase III

FIGURE 11

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

BORING LOG

SURF. EL.: 6448.5 FEET	WATER EL.: None	DEPTH: 9 FEET	LOCATION: Station 278+50 RT-Shoulder
EQUIPMENT: Sonic Spider #2			

DEPTH, Ft	GRAPHIC LOG	SAMPLE	SOIL DESCRIPTION	BLOWS/Ft.	LABORATORY TESTING
			LIGHT BROWN SILTY SAND (SM); medium dense, moist	29	
			WEATHERED BEDROCK; Metamorphosed Tuff and Flows (Mzmv) - closely to intensely fractured, weak, friable, moderately weathered excavates to a very dense silty sand consistency bulk sample from 2.5 to 4.5'		
5			mottled at 5'	81	
			bulk sample from 7 to 8'		
			plastic zone at 9'		
10				20/ 0.1'	
			Boring terminated at 10.75 feet. No free ground water encountered a time of drilling.		
15					
20					

REMARKS:

FIELD ENG.: MJS	WATER DEPTH @ COMPL.: None	DATE: 10-08-09

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

LOG OF B-11
Lake Tahoe Environmental Improvement Program US50 - Phase III

FIGURE **12**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

BORING LOG

SURF. EL.: 6442.5 FEET	WATER EL.: None	DEPTH: 20 FEET	LOCATION: Station 276+50 RT-Shoulder
EQUIPMENT: Sonic Spider #2			

DEPTH, Ft.	GRAPHIC LOG	SAMPLE	SOIL DESCRIPTION	% PASSING #200 SIEVE	BLOWS/Ft.	LIQUID LIMIT(%)	PLASTICITY INDEX (%)	MOISTURE CONTENT (%)	LABORATORY TESTING
			LIGHT BROWN SILTY SAND (SM); medium dense, moist		28				
			WEATHERED BEDROCK; Metamorphosed Tuff and Flows (Mzmv) - closely to intensely fractured, moderately soft, friable, deeply weathered excavates to a very dense silty sand consistency, sample abraded during wash bulk sample from 2 to 4'	50.5		NP	NP	14.3	MA
5			mottled at 5'		46				
			bulk sample from 7.5 to 9', sample abraded during wash	42.2		NP	NP	6.7	MA
10					50/ 0.4'				
			bulk sample from 13.5 to 14.5'						
15			becoming harder at 15', closely to moderately fractured, moderately hard, and weak to moderately strong bulk sample from 16 to 18'						
20			Boring terminated at 20 feet. No free ground water encountered a time of drilling.						

B-12 5.5 - 6.0'

B-12 20 - 20.5'

REMARKS:

FIELD ENG.: MJS	WATER DEPTH @ COMPL.: None	DATE: 10-08-09

WOOD RODGERS
DEVELOPING INNOVATIVE DESIGN SOLUTIONS

LOG OF B-12
Lake Tahoe Environmental Improvement Program US50 - Phase III

FIGURE
13

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

UNIFIED SOIL CLASSIFICATION - ASTM D2487 - 00

MAJOR DIVISIONS					TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW		WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP		POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM		SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC		CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW		WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP		POORLY GRADED SANDS WITH OR WITHOUT GRAVELS, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM		SILTY SANDS WITH OR WITHOUT GRAVEL
			SC		CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML		INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL		INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL		ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH		INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH		ORGANIC CLAYS OR CLAYS OF MEDIUM TO HIGH PLASTICITY
	HIGHLY ORGANIC SOILS		PT		PEAT AND OTHER HIGHLY ORGANIC SOILS

M(80)	—	Moisture Content (%)
DD(105)	—	Dry Density (pcf)
Perm	—	Permeability
Consol	—	Consolidation
LL	—	Liquid Limit (%)
PI	—	Plastic Index (%)
G_s	—	Specific Gravity
MA	—	Particle Size Analysis
OC	—	Organic Content
R-Value	—	Resistance Value
CBR	—	California Bearing Ratio
■	—	"Undisturbed Sample"
⊠	—	Bulk or Classification Samples

	Shear Strength (psf)	Confining Pressure	
TxUU	3200	(2600)	— Unconsolidated Undrained Triaxial Shear
(FM) or (S)			(field moisture or saturated)
TxCU	3200	(2600)	— Consolidated Undrained Triaxial Shear
(P)			(with or without pore pressure measurement)
TxCD	3200	(2600)	— Consolidated Drained Triaxial Shear
SSCU	3200	(2600)	— Simple Shear Consolidated Undrained
(P)			(with or without pore pressure measurement)
SSCD	3200	(2600)	— Simple Shear Consolidated Drained
DSCD	2700	(2000)	— Consolidated Drained Direct Shear
DSCU	2000	(1000)	— Consolidated Undrained Direct Shear
UC	470		— Unconfined Compression
LVS	700		— Laboratory Vane Shear
DSUU			— Unconsolidated Undrained Direct Shear

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SOIL CLASSIFICATION CHART
AND TEST DATA KEY
LAKE TAHOE ENVIRONMENTAL
IMPROVEMENT
PROGRAM US50 - PHASE III

FIGURE
14

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
ALH	8049.003		10-12-09		

SOIL CLASSIFICATION 8049.003_US50 SLOPES.GPJ WOOD RODGERS.GDT 11/16/09

I CONSOLIDATION OF SEDIMENTARY ROCKS; usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

II BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick bedded
Blocky	2.0 to 4.0 ft.	thick-bedded
Slabby	0.2 to 2.0 ft.	thin-bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01 ft.	thinly laminated

III FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0 ft.
Occasionally fractured	2.0 to 4.0 ft.
Moderately fractured	0.2 to 2.0 ft.
Closely fractured	0.05 to 0.2 ft.
Intensely fractured	0.01 to 0.05 ft.
Crushed	less than 0.01 ft.

IV HARDNESS

1. Soft – Reserved for plastic material alone.
2. Low hardness – can be gauged deeply or carved easily with a knife blade.
3. Moderately hard – can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. Hard – can be scratched with difficulty; scratch produces little powder and is often faintly visible.
5. Very hard – cannot be scratched with knife blade; leaves a metallic streak.

V STRENGTH

1. Plastic or very low strength.
2. Friable – crumbles easily by rubbing with fingers.
3. Weak – An unfractured specimen of such material will crumble under light hammer blows.
4. Moderately strong – Specimen will withstand a few heavy hammer blows before breaking.
5. Strong – Specimen will withstand a few heavy ringing blows and will yield with difficulty only dust and small flying fragments.
6. Very strong – Specimen will resist ringing hammer blows and will yield with difficulty only dust and small flying fragments.

VI WEATHERING – The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation and freezing and thawing.

D. Deep – Moderate to complete mineral decomposition; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay silt.

M. Moderate – Slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

L. Little – No megascopic decomposition of minerals; little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.

F. Fresh – Unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.

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LAKE TAHOE ENVIRONMENTAL
IMPROVEMENT PROGRAM
US50 – PHASE III
DOUGLAS COUNTY, NEVADA

FIGURE 15

DRAWN BY ALH | JOB NUMBER 8049.003 | APPROVED BY | DATE 10-06-09

C:\ROCK_DESC.dwg 11/16/09 2:22pm sokelly

APPENDIX C
LABORATORY
TEST RESULTS

TABLE 1 – SUMMARY OF TEST DATA

Location				Classification					Strength				
				Gradation		Atterberg Limits			Unit Weight		Uncon Comp.	Direct Shear	
Slope	Boring	Station	Depth (Ft.)	%+#4	%-#200	LL	PI	USCS	γd (PCF)	%m	q_u (PSI)	Φ (o)	C (PSF)
SLOPE 21	B-1	290+00	0 - 5	0	70.4	34	5	ML	95.5	16.3	-	-	-
			5 – 10	19	38.7	-	NP	SM	108.8	6.7	-	35.7	0
	B-2	291+00	1 – 4	2	52.4	-	NP	ML	99.9	13.9	-	-	-
			12 – 14	0	59.3	-	NP	ML	104.5	7.8	-	38.0	0
	B-7	286+00	2 ½ - 4	2	50	-	NP	ML	-	8.4	-	-	-
			6 - 8	0	38.1	-	NP	ML	103.9	6.6	-	-	-
	B-8	284+85	8 - 10	-	-	-	-	-	2.48 BSG	-	6,600	-	-
	B-9	283+85	7 - 9	-	-	-	-	-		-	6,700	-	-
			12 ½ - 14 ½	-	-	-	-	-		-	4,300	-	-
			16 - 18	-	-	-	-	-		-	6,900	-	-
	B-12	276+50	2 - 4	5	50.5	-	NP	ML	-	14.3	-	-	-
			7 ½ - 9	6	42.2	-	NP	ML	-	6.7	-	-	-
SLOPE 27	B-3	330+30	2 - 4	41	17.3	29	12	SC	99.7	9.1	-	39.2	566
	B-4	331+70	Surface*	-	-	-	-	-	-	-	8,900	-	-
			1 - 3	48	22.1	19	4	SM	-	4.9	-	-	-
			3 - 5	58	18.1	19	3	GM	-	3.5	-	-	-
			5 - 10	-	-	-	-	-	-	-	3,000	-	-
	B-5	333+70	Surface*	-	-	-	-	-	-	-	10,300	-	-
	B-6	335+40	2 – 4	28	24.7	32	12	SC	113.4	11.1	-	34.9	427
			7 – 9	43	20.8	29	13	SC	-	7.4	-	-	-

*Rockfall Samples

WOOD RODGERS
5440 Reno Corporate Drive, Reno, NV 89511
9475 Double R Boulevard, Reno, NV 89521
Phone 775.823.4068 Fax 775.823.4066

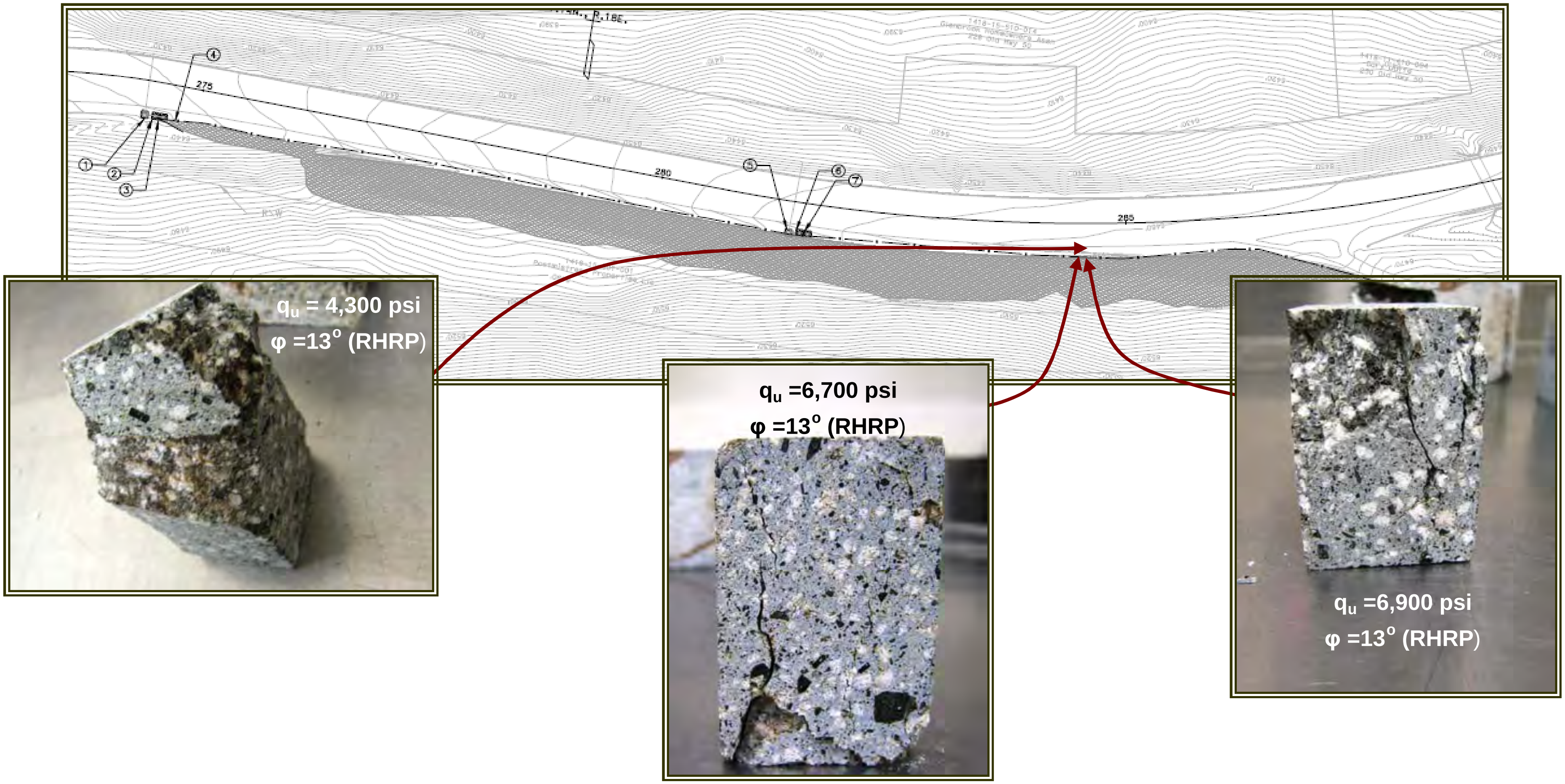
SUMMARY OF LABORATORY TEST DATA

Geotechnical Investigation

LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US50

Project No.: 8049.003
Date: 11/15/09

PLATE C-1



WOOD RODGERS
5440 Reno Corporate Drive, Reno, NV 89511
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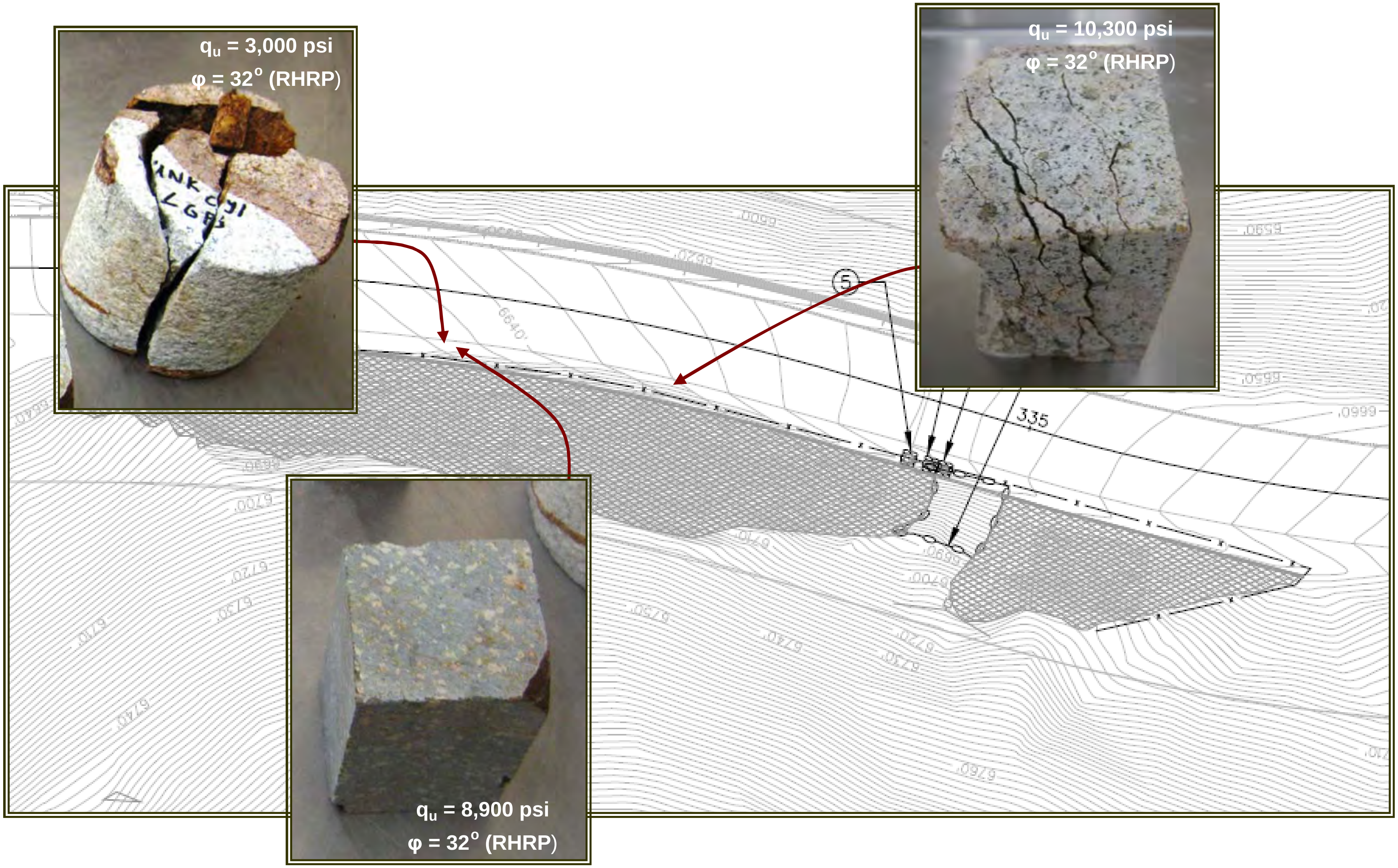
SLOPE 21
UNCONFINED COMPRESSION TEST

Geotechnical Investigation

LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US50

Project No.: 8049.003
Date: 10/28/09

PLATE C-2



WOOD RODGERS
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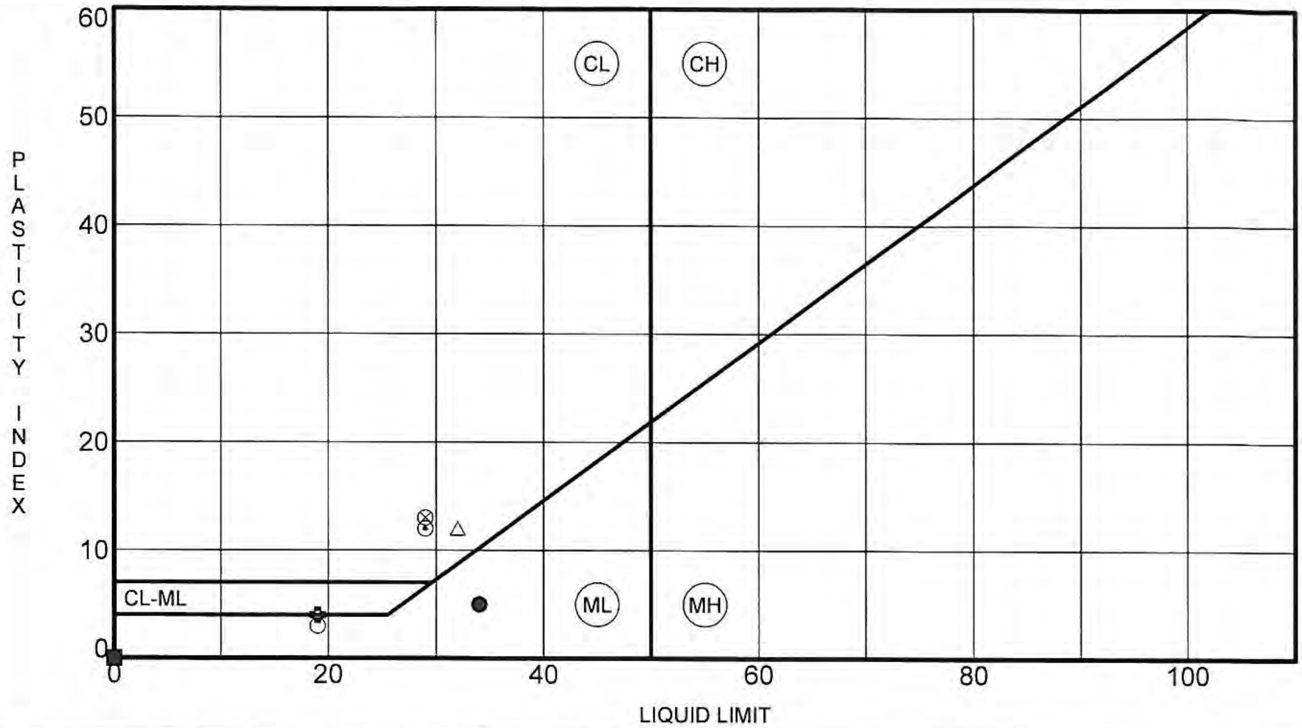
SLOPE 27
UNCONFINED COMPRESSION TEST

Geotechnical Investigation

LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US50

Project No.: 8049.003
Date: 10/28/09

PLATE C-3



Specimen Identification (ft)			LL	PL	PI	Fines	Classification
●	**B-01**	**1.5**	**34**	**29**	**5**	**30**	**Silty Sand(SM)-Mzmv**
⧗	**B-01**	**6.0**	**NP**	**NP**	**NP**	**39**	**Silty Sand w/Gravel(SM)-Mzmv**
▲	**B-02**	**1.0**	**NP**	**NP**	**NP**	**52**	**Sandy Silt(ML)-Mznm (altered)**
★	**B-02**	**12.0**	**NP**	**NP**	**NP**	**27**	**Silty Sand(SM)-Mznm (altered)**
⊙	**B-03**	**2.0**	**29**	**17**	**12**	**17**	**Gravelly Sand w/ Silt & Clay(SM/SC)-Mznm (altered)**
✿	**B-04**	**1.0**	**19**	**15**	**4**	**22**	**Gravelly Sand w/ Silt(SM/GM)-Mznm (altered)**
○	**B-04**	**3.0**	**19**	**16**	**3**	**18**	**Sandy Gravel w/Silt(GM)-Mzmv**
△	**B-06**	**2.0**	**32**	**20**	**12**	**25**	**Silty Sand w/ Gravel(SM)-Mzmv**
⊗	**B-06**	**7.0**	**29**	**16**	**13**	**21**	**Gravelly Sand w/ Silt(SM/GM)-Mzmv**
⊕	**B-07**	**2.5**	**NP**	**NP**	**NP**	**50**	**Sandy Silt(ML)-Mzmv**
□	**B-07**	**6.0**	**NP**	**NP**	**NP**	**38**	**Silty Sand(SM)-Mzmv**
◉	**B-12**	**2.0**	**NP**	**NP**	**NP**	**51**	**Sandy Silt(ML)-Mzmv**
◉	**B-12**	**7.5**	**NP**	**NP**	**NP**	**42**	**Silty Sand(SM)-Mzmv**

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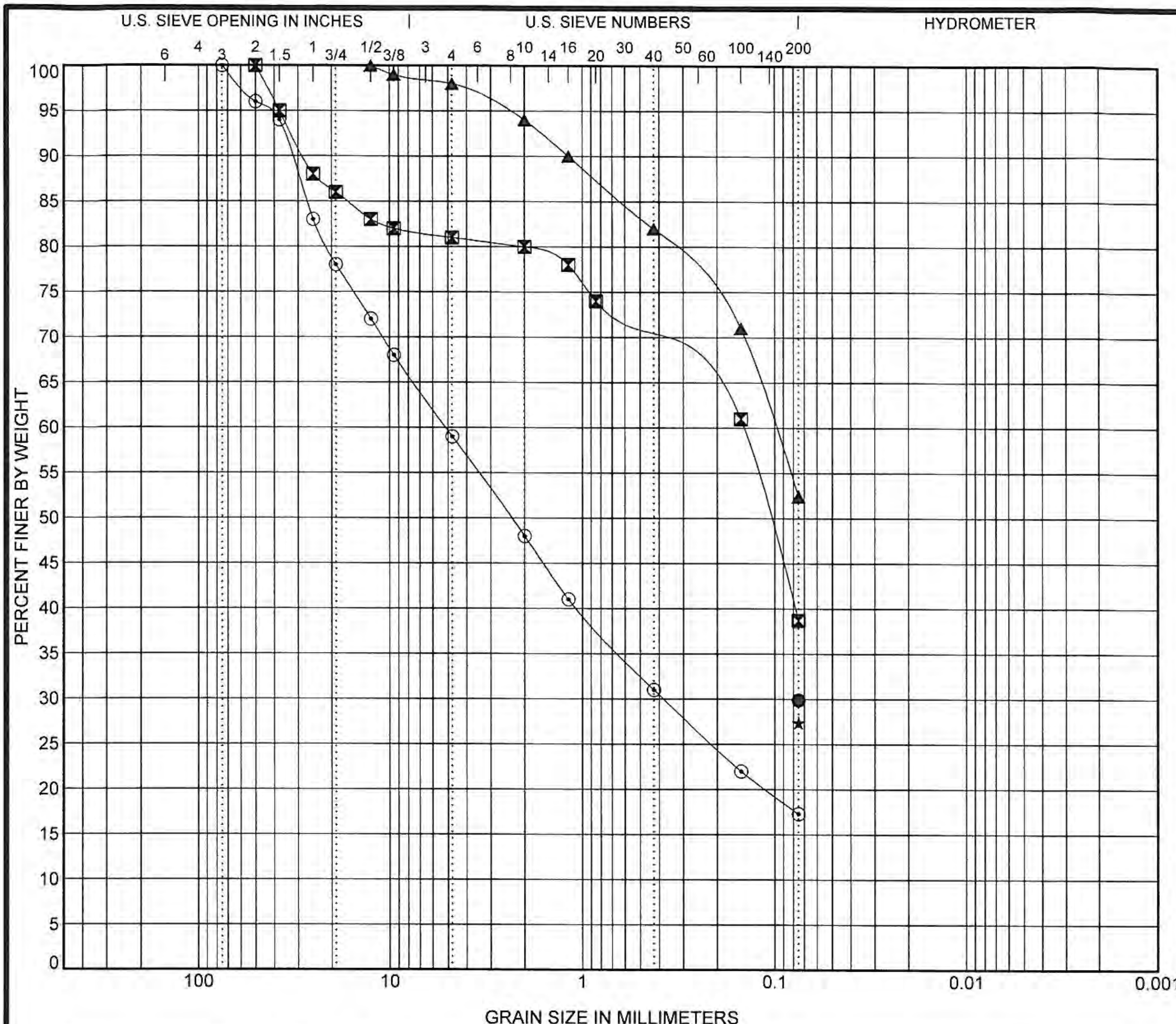
ATTERBERG LIMITS' RESULTS

Project: Lake Tahoe EIP US50-Phase III

Location: Douglas County

Number: 8049.003

US ATTERBERG LIMITS 8049.003 US50 EIP.GPJ WOOD RODGERS.GDT 11/19/09



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification (ft)		Classification	LL	PL	PI	Cc	Cu
●	B-01	1.5-5.0	**Silty Sand(SM)-Mzmv**	34	29	5		
⊠	B-01	6.0-10.0	**Silty Sand w/Gravel(SM)-Mzmv**	NP	NP	NP		
▲	B-02	1.0-4.0	**Sandy Silt(ML)-Mznm (altered)**	NP	NP	NP		
★	B-02	12.0-14.0	**Silty Sand(SM)-Mznm (altered)**	NP	NP	NP		
⊙	B-03	2.0-4.0	**Gravelly Sand w/ Silt & Clay(SM/SC)-Mznm (altered)**	29	17	12		

	Specimen Identification (ft)		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-01	1.5-5.0	0.075						29.9	
⊠	B-01	6.0-10.0	50	0.145			19.0	42.3	38.7	
▲	B-02	1.0-4.0	12.5	0.1			2.0	45.6	52.4	
★	B-02	12.0-14.0	0.075						27.4	
⊙	B-03	2.0-4.0	75	5.13	0.379		41.0	41.7	17.3	

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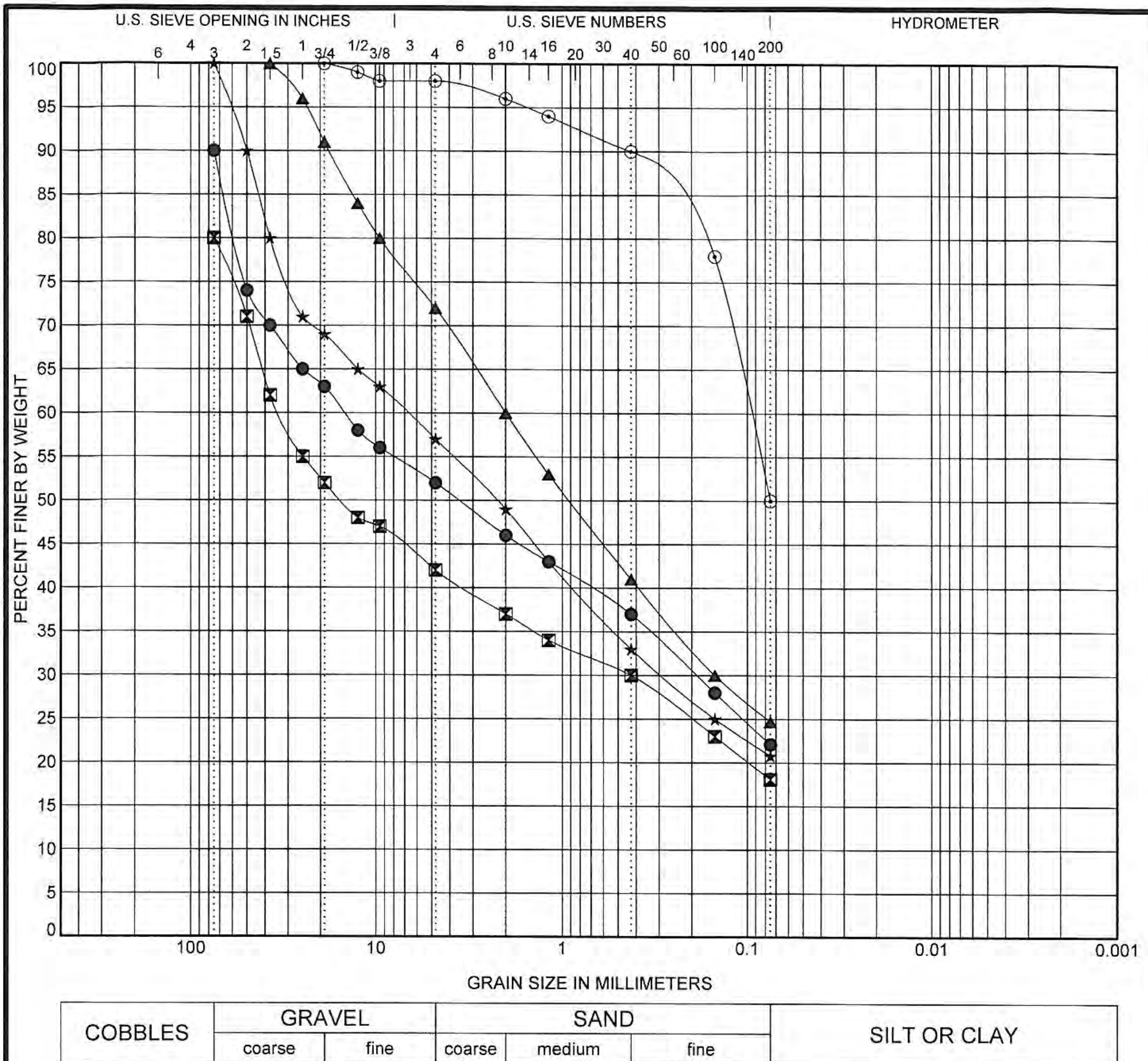
GRAIN SIZE DISTRIBUTION

Project: Lake Tahoe EIP US50-Phase III

Location: Douglas County

Number: 8049.003



	Specimen Identification (ft)		Classification	LL	PL	PI	Cc	Cu
●	B-04	1.0-3.0	**Gravelly Sand w/ Silt(SM/GM)-Mznm (altered)**	19	15	4		
⊠	B-04	3.0-5.0	**Sandy Gravel w/Silt(GM)-Mzmv**	19	16	3		
▲	B-06	2.0-4.0	**Silty Sand w/ Gravel(SM)-Mzmv**	32	20	12		
★	B-06	7.0-9.0	**Gravelly Sand w/ Silt(SM/GM)-Mzmv**	29	16	13		
⊙	B-07	2.5-4.0	**Sandy Silt(ML)-Mzmv**	NP	NP	NP		

	Specimen Identification (ft)		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-04	1.0-3.0	75	14.779	0.189		38.0	29.9	22.1	
⊠	B-04	3.0-5.0	75	33.398	0.425		38.0	23.9	18.1	
▲	B-06	2.0-4.0	37.5	2	0.15		28.0	47.3	24.7	
★	B-06	7.0-9.0	75	6.718	0.288		43.0	36.2	20.8	
⊙	B-07	2.5-4.0	19	0.096			2.0	48.0	50.0	

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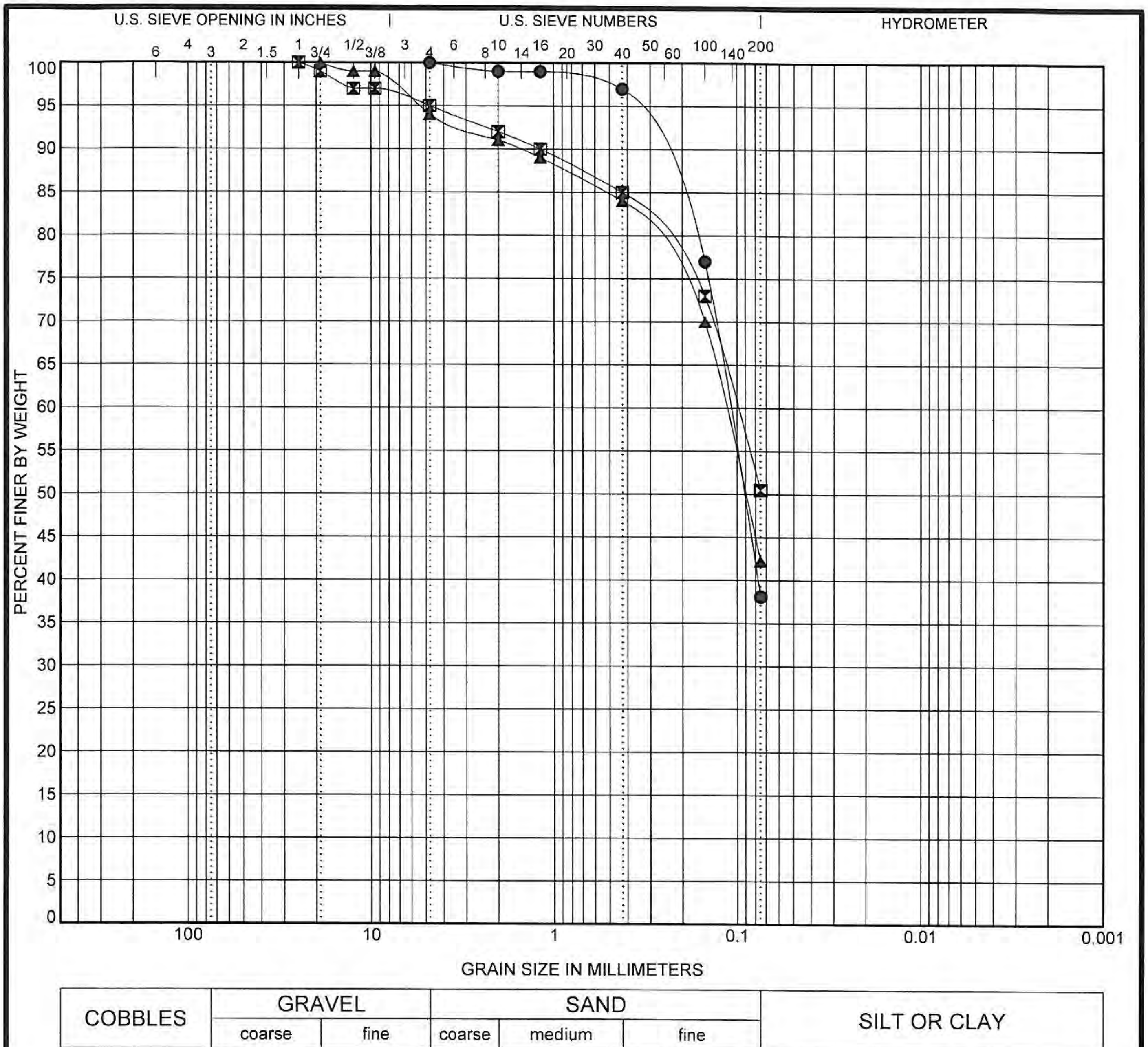
GRAIN SIZE DISTRIBUTION

Project: Lake Tahoe EIP US50-Phase III

Location: Douglas County

Number: 8049.003



Specimen Identification (ft)			Classification	LL	PL	PI	Cc	Cu
●	B-07	6.0-8.0	**Silty Sand(SM)-Mzmv**	**NP**	**NP**	**NP**		
⧗	B-12	2.0-4.0	**Sandy Silt(ML)-Mzmv**	**NP**	**NP**	**NP**		
▲	B-12	7.5-9.0	**Silty Sand(SM)-Mzmv**	**NP**	**NP**	**NP**		

Specimen Identification (ft)			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-07**	**6.0-8.0**	**4.75**	**0.111**			**0.0**	**61.9**	**38.1**	
⧗	**B-12**	**2.0-4.0**	**25**	**0.1**			**5.0**	**44.5**	**50.5**	
▲	**B-12**	**7.5-9.0**	**19**	**0.117**			**6.0**	**51.8**	**42.2**	

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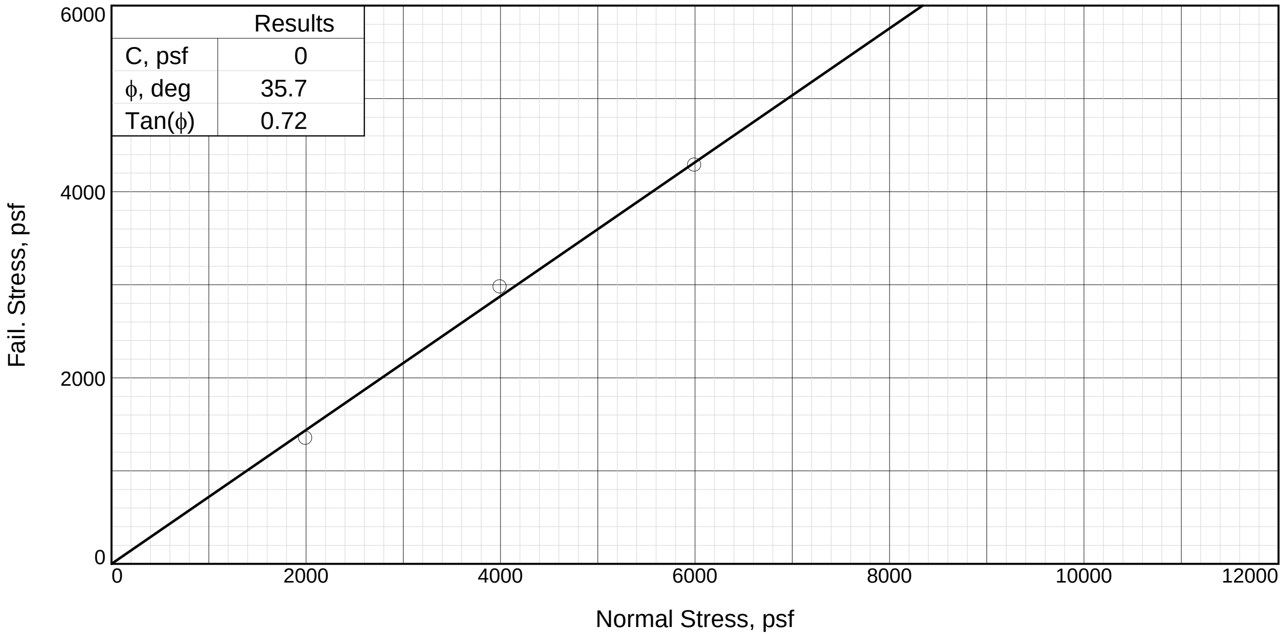
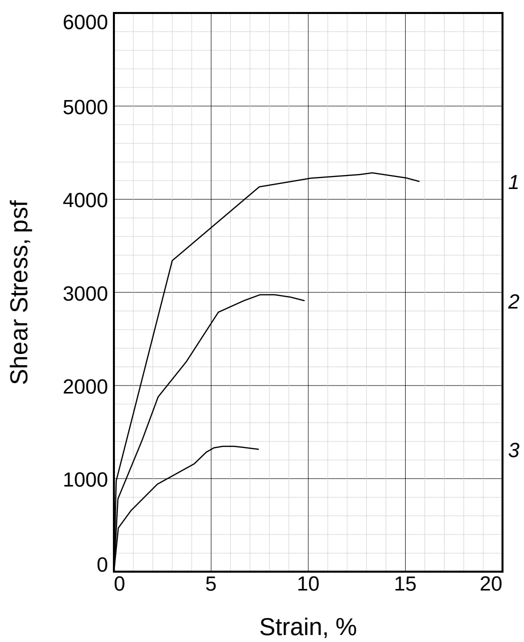
GRAIN SIZE DISTRIBUTION

Project: Lake Tahoe EIP US50-Phase III

Location: Douglas County

Number: 8049.003





Sample No.		1	2	3
Initial	Water Content, %	10.0	10.0	10.0
	Dry Density, pcf	109.0	108.8	108.8
	Saturation, %	49.4	49.2	49.2
	Void Ratio	0.5471	0.5492	0.5492
	Diameter, in.	2.420	2.420	2.420
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	14.6	16.5	18.2
	Dry Density, pcf	120.2	116.4	112.9
	Saturation, %	97.9	99.7	99.7
	Void Ratio	0.4018	0.4477	0.4925
	Diameter, in.	2.420	2.420	2.420
	Height, in.	0.906	0.935	0.963
Normal Stress, psf		6000.0	4000.0	2000.0
Fail. Stress, psf		4282.8	2974.2	1346.2
Strain, %		13.3	7.5	5.6
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.020	0.020	0.020

Sample Type: Remold
Description: Native

Assumed Specific Gravity= 2.7
Remarks: Laboratory Number 1546

Figure ________

Client: James Edward Engineering

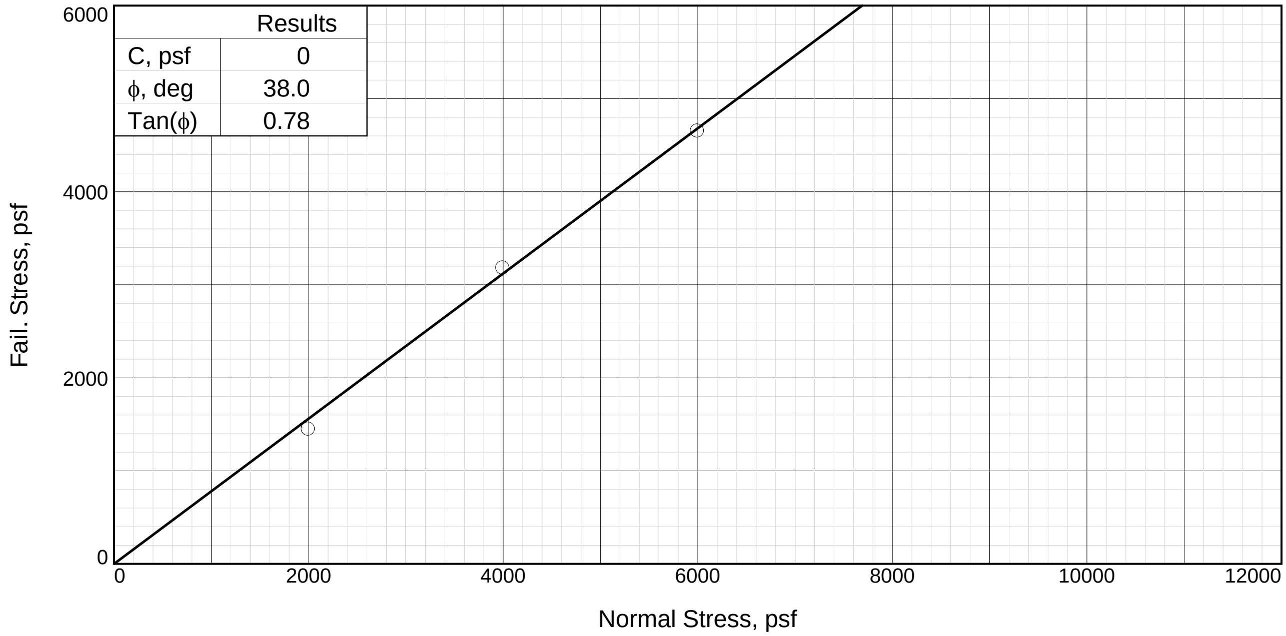
Project: James Edward Engineering - Testing As Ordered

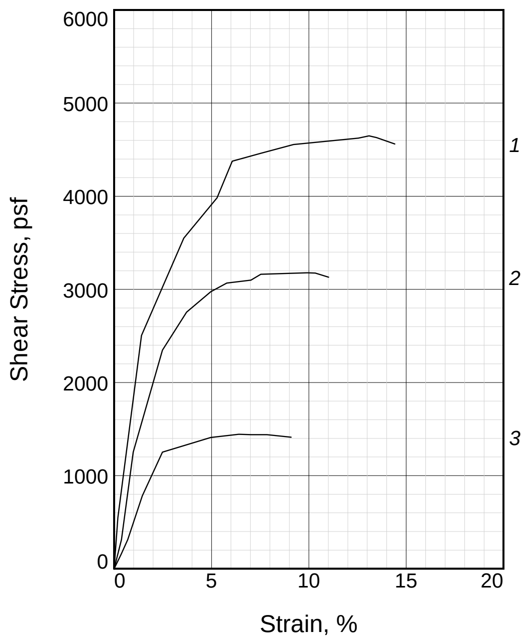
Source of Sample: LL 1546 **Depth:** 5.0' - 10.0'
Sample Number: B-1 (Sta 290+00)
Proj. No.: 0679-01-1 **Date Sampled:**

DIRECT SHEAR TEST REPORT

BLACK EAGLE CONSULTING, INC.

Tested By: G. Bomberger





Sample No.		1	2	3
Initial	Water Content, %	8.7	8.7	8.7
	Dry Density, pcf	104.5	104.4	104.5
	Saturation, %	38.3	38.2	38.4
	Void Ratio	0.6135	0.6147	0.6124
	Diameter, in.	2.420	2.420	2.420
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	17.2	19.1	18.9
	Dry Density, pcf	115.1	111.1	109.0
	Saturation, %	100.0	100.0	93.6
	Void Ratio	0.4640	0.5169	0.5459
	Diameter, in.	2.420	2.420	2.420
	Height, in.	0.907	0.939	0.959
Normal Stress, psf		6000.0	4000.0	2000.0
Fail. Stress, psf		4649.1	3177.7	1443.3
Strain, %		13.1	9.9	6.4
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.020	0.020	0.020

Sample Type: Remold
Description: Native

Assumed Specific Gravity= 2.7
Remarks: Laboratory Number 1546

Figure ________

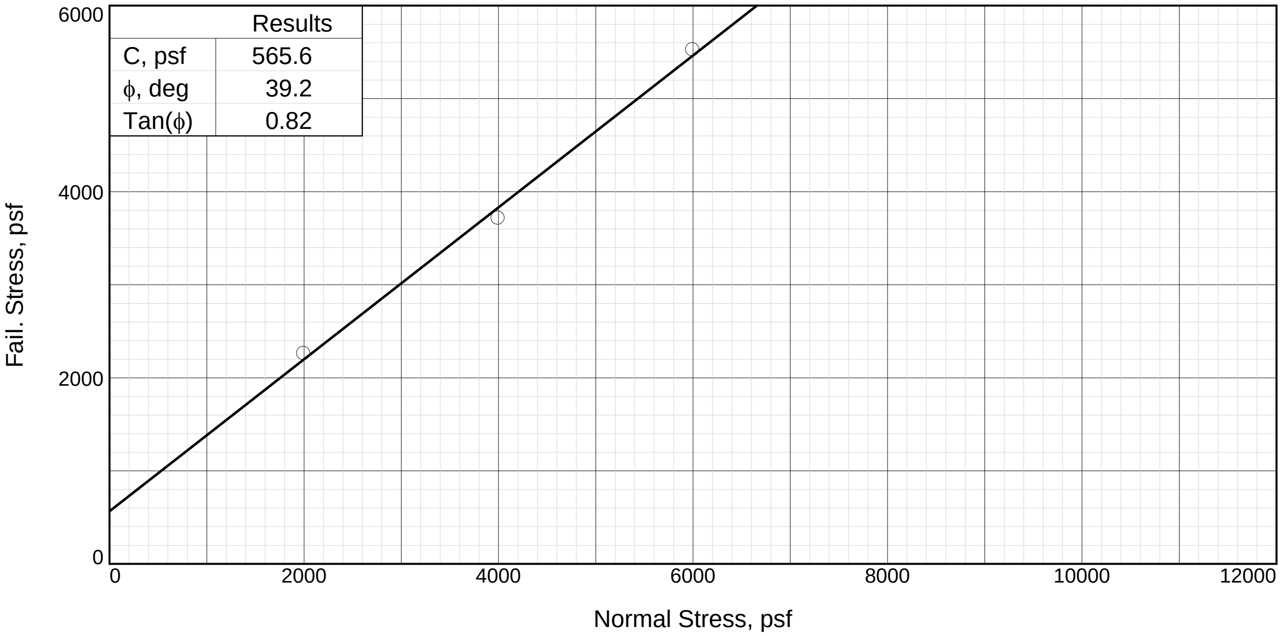
Client: James Edward Engineering

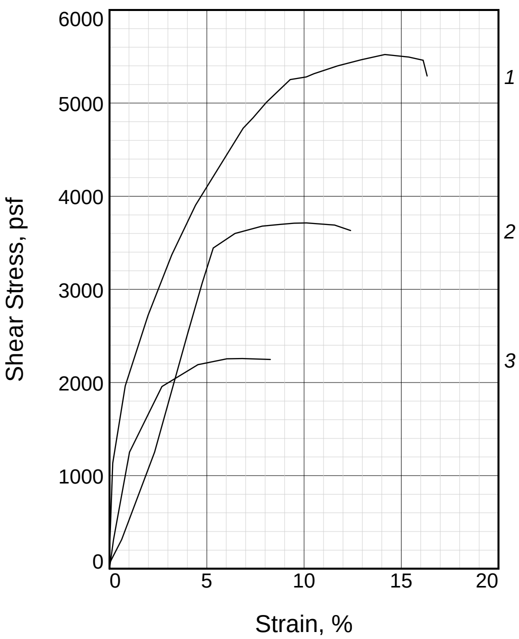
Project: James Edward Engineering

Source of Sample: LL 1546 **Depth:** 12.0' - 14.0'
Sample Number: B-2 (Sta 291+00)
Proj. No.: 0679-01-1 **Date Sampled:**

DIRECT SHEAR TEST REPORT
BLACK EAGLE CONSULTING, INC.
Reno, Nevada

Tested By: G. Bomberger





Sample No.		1	2	3
Initial	Water Content, %	12.6	12.6	12.6
	Dry Density, pcf	99.7	100.0	99.5
	Saturation, %	49.2	49.7	49.0
	Void Ratio	0.6911	0.6849	0.6937
	Diameter, in.	2.420	2.420	2.420
	Height, in.	0.999	1.000	1.000
At Test	Water Content, %	15.3	17.5	20.0
	Dry Density, pcf	119.2	114.3	109.4
	Saturation, %	100.0	99.9	99.8
	Void Ratio	0.4140	0.4743	0.5412
	Diameter, in.	2.420	2.420	2.420
	Height, in.	0.835	0.875	0.910
Normal Stress, psf		6000.0	4000.0	2000.0
Fail. Stress, psf		5522.6	3713.0	2257.2
Strain, %		14.2	10.1	6.8
Ult. Stress, psf				
Strain, %				
Strain rate, in./min.		0.020	0.020	0.020

Sample Type: Remold
Description: Native

Assumed Specific Gravity= 2.7
Remarks: Laboratory Number 1558

Figure ________

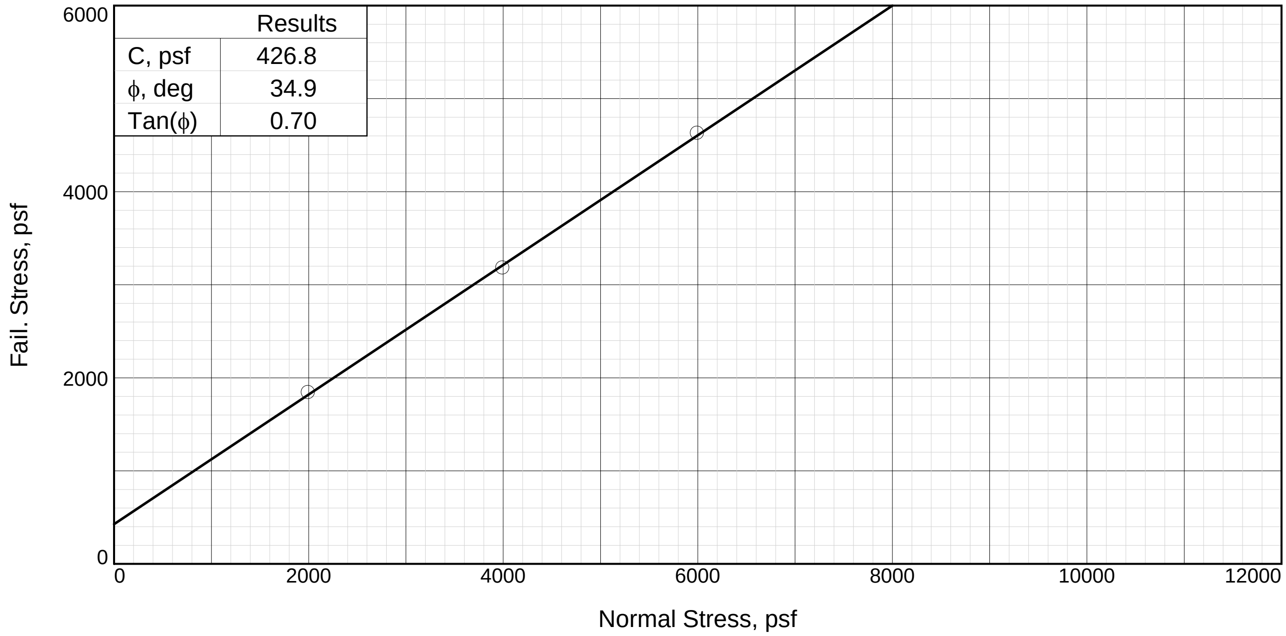
Client: James Edward Engineering

Project: James Edward Engineering - Testing As Ordered

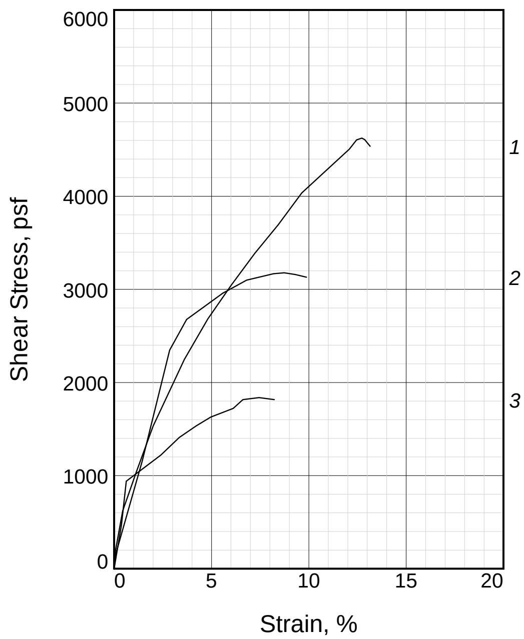
Source of Sample: B-3 **Depth:** 2.0' - 4.0'
Sample Number: Sta 330+30, Slope 27
Proj. No.: 0679-01-1 **Date Sampled:**

DIRECT SHEAR TEST REPORT
BLACK EAGLE CONSULTING, INC.
Reno, Nevada

Tested By: G. Bomberger



Results	
C, psf	426.8
ϕ, deg	34.9
Tan(ϕ)	0.70



	Sample No.	1	2	3
Initial	Water Content, %	8.3	8.3	8.3
	Dry Density, pcf	108.7	109.4	108.8
	Saturation, %	40.8	41.4	40.8
	Void Ratio	0.5499	0.5413	0.5488
	Diameter, in.	2.420	2.420	2.420
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	12.7	14.5	16.2
	Dry Density, pcf	125.1	120.8	117.0
	Saturation, %	98.9	99.4	99.2
	Void Ratio	0.3469	0.3948	0.4407
	Diameter, in.	2.420	2.420	2.420
	Height, in.	0.869	0.905	0.930
	Normal Stress, psf	6000.0	4000.0	2000.0
	Fail. Stress, psf	4624.0	3177.7	1837.7
	Strain, %	12.7	8.7	7.4
	Ult. Stress, psf			
	Strain, %			
	Strain rate, in./min.	0.020	0.020	0.020

Sample Type: Remold
Description: Native

Assumed Specific Gravity= 2.7
Remarks: laboratory Number 1558

Figure ________

Client: James Edward Engineering

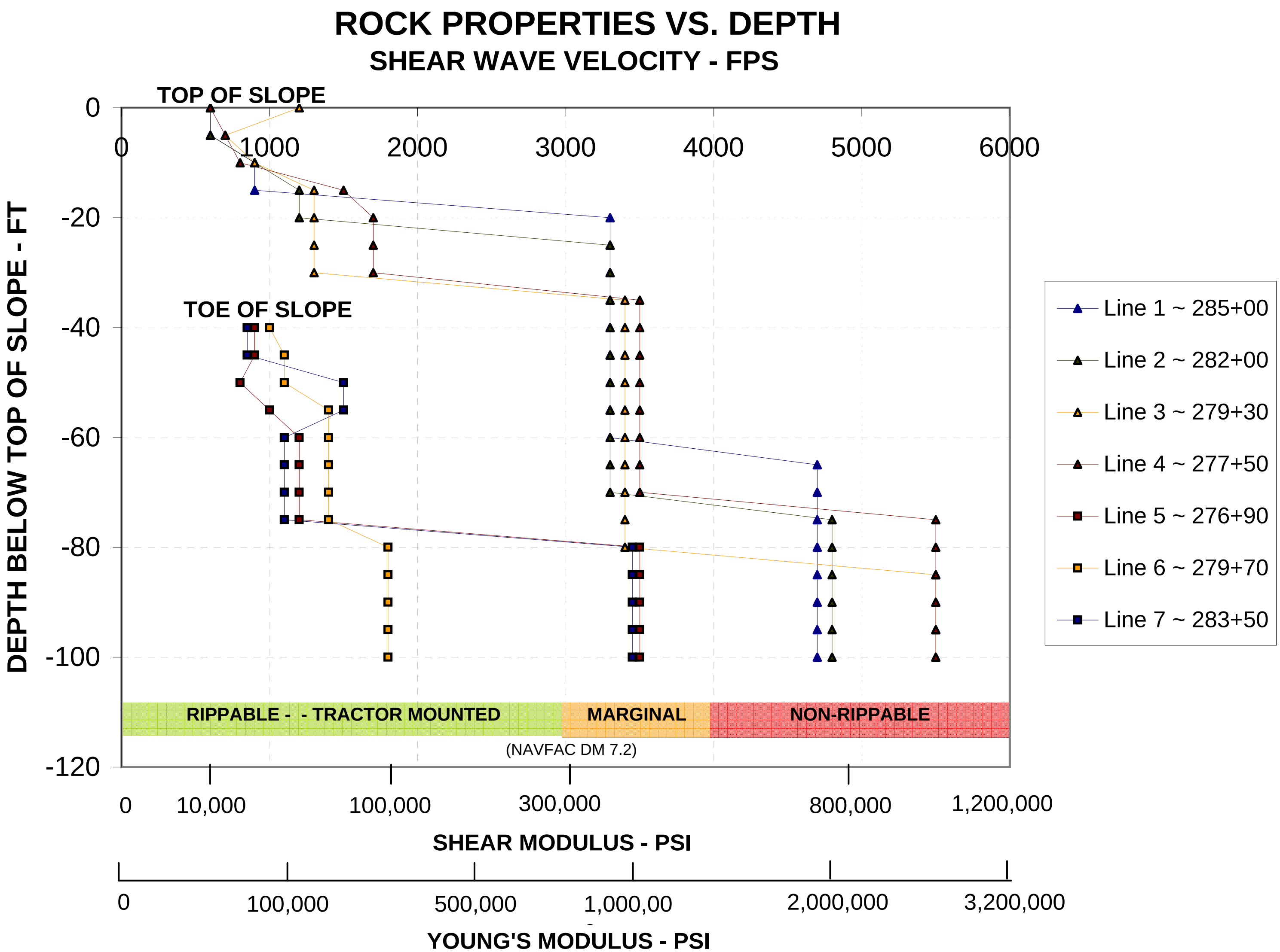
Project: James Edward Engineering - Testing As Ordered

Source of Sample: B-6 **Depth:** 2.0' - 4.0'
Sample Number: LL 1558
Proj. No.: 0679-01-1 **Date Sampled:**

DIRECT SHEAR TEST REPORT
BLACK EAGLE CONSULTING, INC.
Reno, Nevada

Tested By: G. Bomberger

APPENDIX D
IN SITU TEST DATA



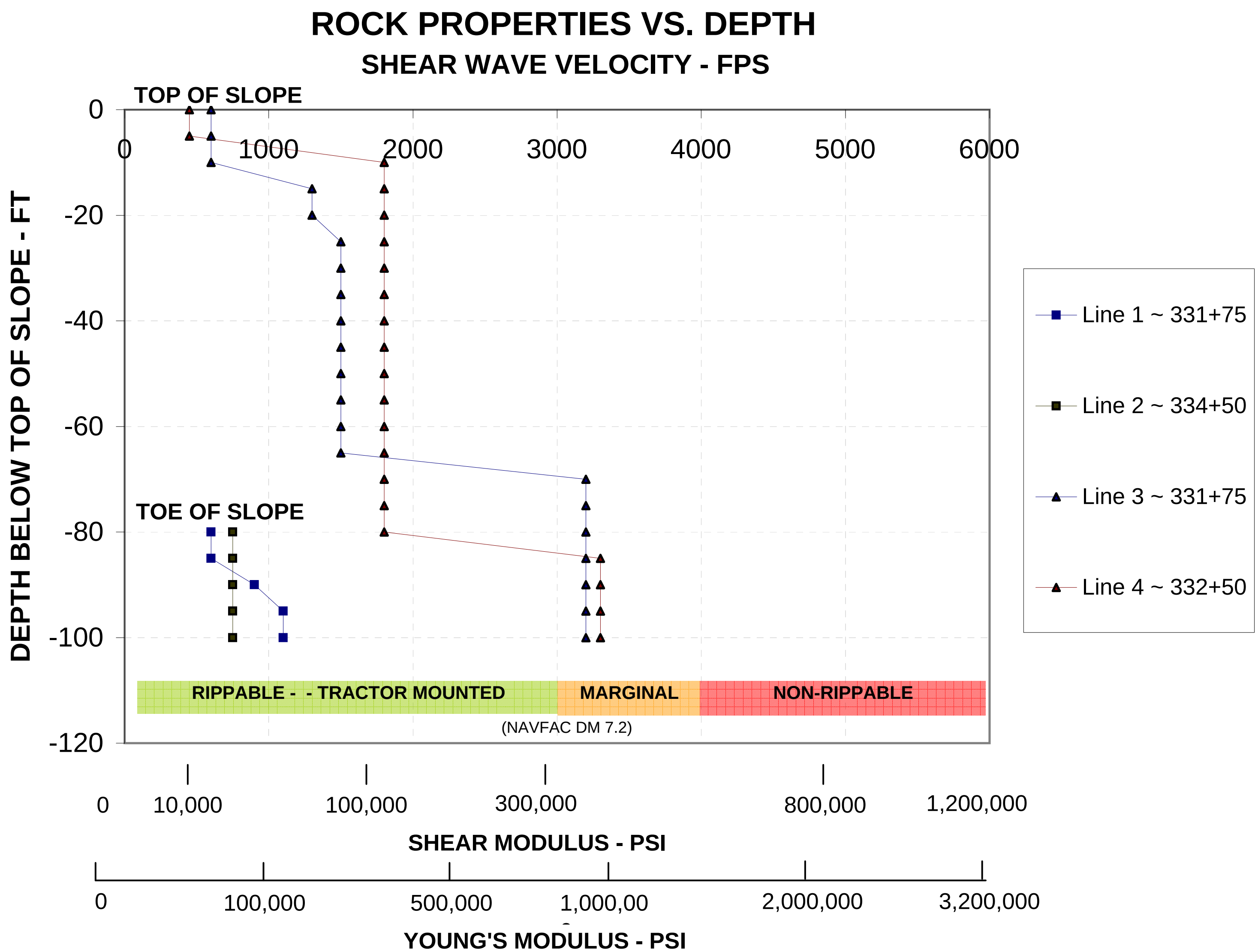
WOOD RODGERS
5440 Reno Corporate Drive, Reno, NV 89511
9475 Double R Boulevard, Reno, NV 89521
Phone 775.823.4068 Fax 775.823.4066

SLOPE 21
ROCK MASS PROPERTIES
BASED ON REMI DATA

Geotechnical Investigation
LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US50 - PHASE III

Project No.: 8049.003
Date: 10/28/09

PLATE D-1



WOOD RODGERS
5440 Reno Corporate Drive, Reno, NV 89511
9475 Double R Boulevard, Reno, NV 89521
Phone 775.823.4068 Fax 775.823.4066

SLOPE 27
ROCK MASS PROPERTIES
BASED ON REMI DATA

Geotechnical Investigation
LAKE TAHOE ENVIRONMENTAL IMPROVEMENT PROGRAM US50 - PHASE III

Project No.: 8049.003
Date: 10/28/09

PLATE D-2

APPENDIX E
REPORTS BY OTHERS

Wood Rodgers, Inc.

Rock Mechanics
Cut Slope Stability Report
U.S. Highway 50 – Slope 27
Douglas County, Nevada

Loren Jalbert, P.E.
Project Manager

McMILLEN, LLC

January 5, 2010

TABLE OF CONTENTS

APPENDICES

FIGURES

SECTION 1
INTRODUCTION

1.0 Project Description

Appendices modeling This rock mechanics cut slope stability report addresses existing rock Slope 27 located on the south side of U.S. Highway 50, on the west side of Spooner Summit near the community of Glenbrook, Nevada in Douglas County (Figure One – Project Location Map). Rock slope designation "Slope 27" was identified in the Rock Hazard Rating System report completed by Watters and Flatland in 1993 for the Nevada Department of Transportation (NDOT) for this section of U.S. Highway 50 (Figure Two – Site Location Map - Slope 27). In 1996, NDOT initiated the Lake Tahoe Environmental Improvement Master Planning program for all roads maintained in the Lake Tahoe Basin. The goal of the Master Plan is to comply with the environmental requirements of the Tahoe Regional Planning Agency (TRPA); and other regulatory agencies, to reduce and minimize erosion, improve water quality and drainage and improve the appearances of the cut slopes.

The purpose of the rock mechanics cut slope stability study is to define the current geometric stability of the rock slope(s), discontinuities relative to the existing slope angle, and determine the overall stability for the defined failure planes in Slope 27. Mitigation for geometric instability for these slopes needs to address two primary issues: safety (rockfalls from these slopes have caused automobile accidents); and, sediment control in the Tahoe basin. A separate issue to be addressed includes reduced long-term operation and maintenance of this slope.

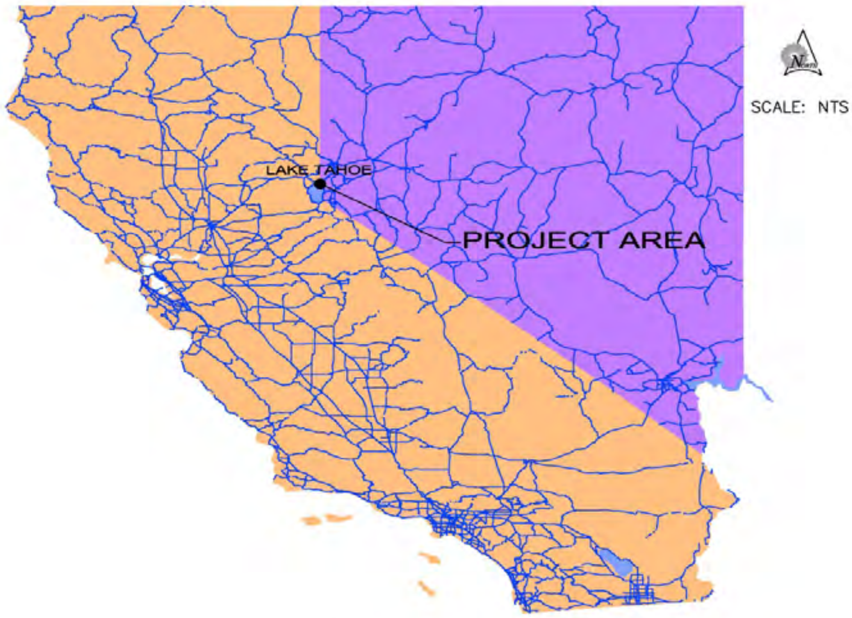
1.1 Objectives and Scope

The objectives of this rock mechanics cut slope stability analysis are to:

1. Perform a detailed rock mass analysis and slope evaluation for Slope 27 (Appendix A – Pictures).
2. Collect subsurface soil and rock information at the base of Slope 27 utilizing a sonic drilling rig. Analysis of soil and rock samples was completed by the Wood Rodgers, Inc. Materials Testing Laboratory. Results are presented in Wood Rodgers Geology report.
3. Conduct a refraction microtremor (ReMi) surface seismic survey at the crown or top of Slope 27 to determine subsurface soil conditions.
4. Define the current geometric stability of the bedrock, discontinuities relative to the existing slope angle, and determine the angle which the slope should be laid back to mitigate geometric instability.

In achieving the objectives, this scope of work included the following:

AREA MAP



LOCATION MAP

Figure One
Project Location Map
Glenbrook Hwy 50
Rock Mechanics Study

McMILLEN, LLC

Figure Two - Site Location Map
Slope 27

Map provided by Wood Rodgers

MCMILLEN, LLC

1. Review geologic reports and technical resources that were available for this area. The results of the review were used as background information to perform of the field program and incorporate into the finds as appropriate.
   - T. L. T. Grose, 1985., U.S. Geological Survey, Glenbrook 7 ½ Quadrangle Geological Map, Scale 1:24,000 (Submitted as Figure 3 under this report).
   - University of Nevada, Reno. April, 1993, "The Rockfall Hazard Rating System (RHRS) as Applied to the Cut Slopes of US 50 and SR 28 on the East Side of Lake Tahoe, Nevada" (Submitted as Appendix C under this report).
   - Harding Lawson Associates. April 1999, "Supplemental Rockfall Evaluation Report, U.S. Highway 50 – Slopes 29 and 30, Douglas County, Nevada, Report prepared for the State of Nevada, Department of Transportation, Project #42403.10.01.
   - Sunrise Engineering Inc. December, 2006, "Rock Mechanics Cut Slopes Stability Report", U.S. Highway 50 – Slopes 21 through 31, Douglas County, Nevada, Reported prepared Wood Rodgers, Reno, Nevada Office.

2. Performance of a detailed structural mapping program of the rock mass exposed in the cut slopes to document joint orientation and other properties of physical discontinuities. The joint orientation data was reviewed and synthesized before inputting into the computer program RockPack™ III that generates stereo nets. The RockPack™ III program provides statistical analysis for the relationship between the major joint patterns and the existing cut slope face angle.

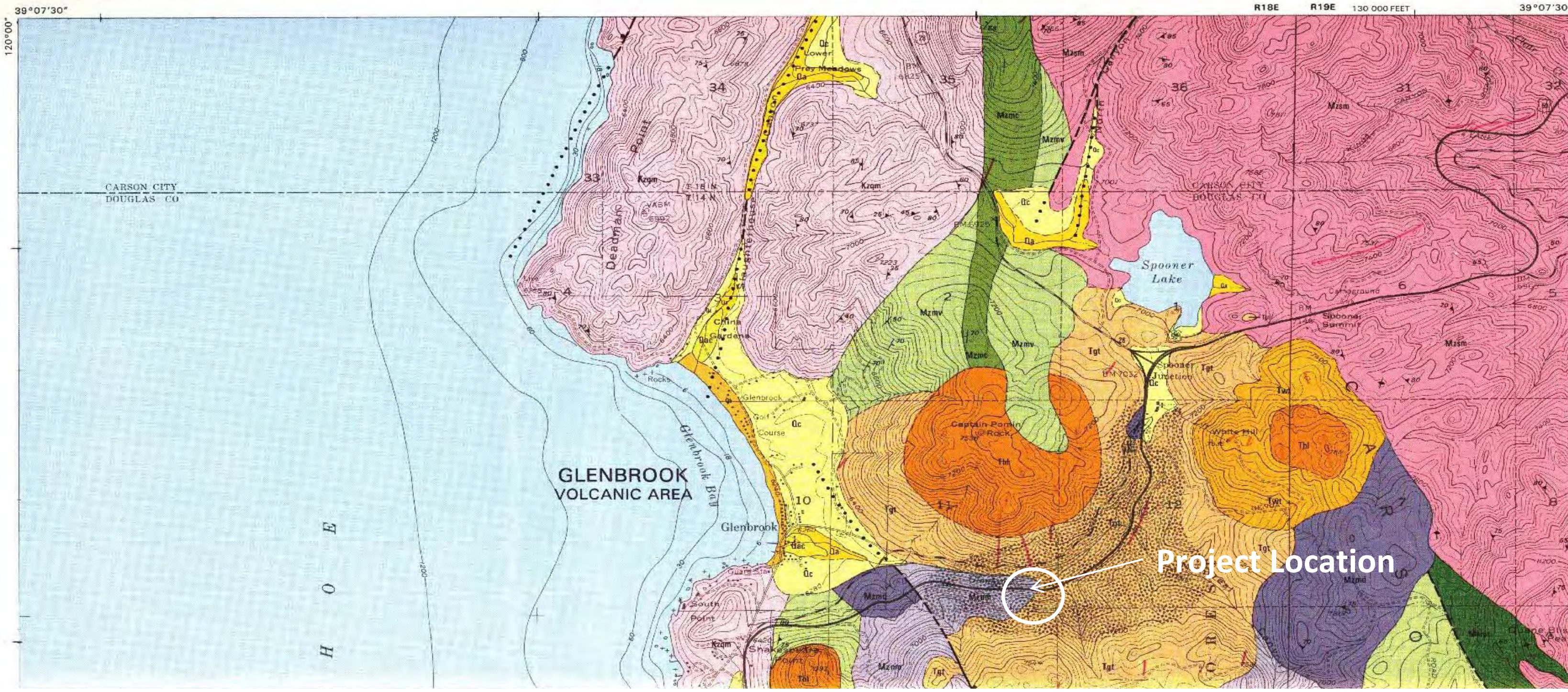
3. Preparation of this summary report.

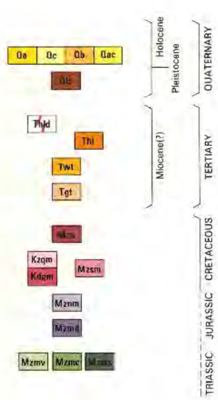
   - Identify the Rock Mass Rating for Slope 27 in determining suitable anchoring systems (rock bolts, dowels and soil nails) for rock slope draping.
   - Provide recommendations for mitigation and prevention of further erosion of Slopes 27.
   - Present photographs (see Appendix A); and, prepare surface area estimate for rock removal, i.e. scaling, for Slope 27.
   - Provide a rockfall/hazard discussion (bedrock slope description, percent soil cover, rockfall potential, loose rock estimate, degree of scaling required and associated estimate of cubic yards of rock to scale).

GLENBROOK QUAD

NEVADA BUREAU OF MINES AND GEOLOGY
LAKE TAHOE AREA MAP 2Bg

GLENBROOK QUADRANGLE
GEOLOGIC MAP





Qa **Sand and gravel alluvium** Arkosic transported debris from weathered granite. Occurs locally along low-gradient stretches of streams and in back-beach areas. Estimated thickness <10 m.

Qc **Sand and boulder colluvium** Arkosic, derived mostly from in-situ weathering of granitic rocks. Estimated thickness <25 m.

Qb **Beach sand** Arkosic, fine to very coarse grained. Estimated thickness <25 m.

Qac **Carbonaceous alluvium** Clayey to sandy, dark gray to black, variously organic. Occurs in marshy back-beach alluvial tracts.

Qls **Landslide debris** Granitic, unsorted, coarse. Two small masses associated with Late Quaternary(?) faults.

Thld **Porphyritic hornblende-sanidine latite dike** Same as Thl, but with phenocrysts 3–8 mm long, 30–60% by volume. Dikes and elliptical plugs range from 3–15 m wide and 10–1000 m long. They commonly occur within and marginal to intrusive masses. (Detailed mapping would reveal more than indicated herein.)

Thl **Porphyritic hornblende-sanidine latite** Medium gray, finely to coarsely porphyritic with ubiquitous euhedral sanidine and hornblende phenocrysts 1–8 mm long, 10–40% by volume, and few biotite phenocrysts in a cryptocrystalline to microcrystalline matrix. Comprises 3 circular and 1 elliptical erosionally resistant intrusive masses.

Twt **Vitric-crystal tuff of White Hill** Light gray, coarse grained with quartz, feldspar, and biotite crystals. Massive, unwelded. Occurs concentric to a latite intrusion.

Tgt **Hornblende trachyte** Medium to dark gray, finely to microporphyritic with phenocrysts (40% by volume) of sanidine, oligoclase-andesine, hornblende, and minor biotite set in a cryptocrystalline to microcrystalline matrix. Occurs as flows and compound vent filling near Glenbrook.

Mza **Alaskite** Light gray and reddish tan, finely crystalline, quartz rich, massive. May be altered granitic rocks.

Mzsm **Biotite-hornblende monzogranite of Spooner Summit** White to light gray, medium grained, idiomorphic, slightly porphyritic with hornblende laths locally 10 mm long. Massive-structureless to weakly foliated. Rare dioritic inclusions usually 1–20 cm long.

Kzqm **Hornblende-biotite quartz monzodiorite and granodiorite of Zephyr Cove** Light to medium gray, medium grained, hypidiomorphic. Massive-structureless to moderately foliated, most commonly weakly foliated on hornblende, biotite, and inclusions. Ubiquitous dioritic inclusions 2–50 cm long, 1% by volume. Generally equivalent to granodiorite of East Peak, Late Cretaceous, of Armin and others (1983).

Kdqm **Hornblende quartz monzonite and monzogranite** White to light gray, medium grained, hypidiomorphic, equigranular to porphyritic with euhedral-subhedral hornblende phenocrysts locally 12 mm long. Weakly to strongly foliated on hornblende, biotite, inclusions, and rare schlieren. Ubiquitous dioritic inclusions 2–100 cm long, 1% by volume. Equivalent to granodiorite of Daggett Pass, Cretaceous, 83–90 m.y. (K-Ar), of Armin and others (1983).

Mznm **Biotite monzogranite of North Logan House Creek** Tan to pink gray, fine to medium grained, allotriomorphic, seriate, slightly porphyritic locally with euhedral hornblende phenocrysts <8 mm long. Massive-structureless. Intrudes metavolcanic rocks and hornblende diorite.

Mzmd **Hornblende diorite of Montréal Canyon** Dark gray, fine to medium grained, hypidiomorphic, protoclastic, and locally saussuritized. Massive-structureless. Intruded and locally migmatized by Mznm and Mzsm.

Mzmv **Metamorphosed tuff and flows** Medium to dark gray, locally greenish, very fine grained to aphanitic. Massive to thick bedded and locally weakly foliated. Protolith of silicic crystal-vitric tuff and intermediate flows metamorphosed to biotite-sericite hornfels and semischist.

Mzmc **Metaconglomerate and metasandstone** Light-gray to dark-gray, medium-grained sandstone to fine conglomerate, rounded to angular, quartzose and volcanic. Massive to poorly bedded.

Mzms **Metasandstone** Dark gray to black, fine to very fine grained, angular grains. Massive to weakly foliated. Protolith of graywacke metamorphosed to biotite-quartz hornfels and schist.

Contact Long dashes where approximately located; short dashes where gradational and diffuse

Fault Dashed where inferred or approximately located; dotted where concealed. Ball on downthrown side

75 **Foliation** Inclined and vertical

50 **Bedding strike and dip** Inclined and vertical

70 **Shear zone strike and dip**

Area of alteration and oxidation Mainly argillization and propylitization

REFERENCE
Armin, R. A., and John, D. A. (1983) Geologic map of the Freel Peak 15-minute quadrangle, California and Nevada, *with* Quaternary geology by J. C. Dohrenwend: U.S. Geological Survey Miscellaneous Investigations Series Map I-1424, scale 1:62,500

T. L. T. Grose, 1985

Scale 1:24,000
CONTOUR INTERVAL 40 FEET
0 0.5 1 kilometer
0 0.5 1 mile
0 1000 2000 3000 4000 5000 feet

nbmg
Base map: U.S. Geological Survey Glenbrook 7½' quadrangle, 1969
First edition, first printing, 1985; 1000 copies
Printing: Williams and Heintz Map Corp., Washington, D.C.
Editing: Alice Sjoberg
Cartography: Larry Jacox
Color separation assistance: Michael Tracy
Typesetting: Rayetta Buckley
Pasteup: Mati Stephens

For sale by the Nevada Bureau of Mines and Geology, University of Nevada Reno, Reno, Nevada. 89557
Order Map 2Bg, $4.00

Figure Three - Geology Site Map

SECTION 2
FIELD PROGRAM

2.0 Field Services

The field program consisted of three tasks. All three tasks were completed concurrently during the week of October 6th, 2009. These tasks include:

1. Drilling: A sonic drilling program was completed by the Boart Longyear drilling company. Several boring(s) were drilled along the base of Slopes 21* and 27. Soil and rock samples were collected during the sonic drilling program. Soil and rock samples were analyzed at the Wood Rodger Materials testing laboratory in Reno, Nevada. Sonic boring locations in the field were determined by a Wood Rodger's representative. Sonic boring logs and material testing results are submitted under separate cover from this report. [*Slope 21 was subject to a separate drilling and ReMi seismic survey from Slope 27. Evaluation of Slope 21 was not part of this scope of services.]

2. Geophysical Survey: A refraction microtremor (ReMi) surface was completed by Marvin E. Davis & Associates, Inc. ReMi survey line(s) were completed above and below Slopes 21 and 27. The purpose of the ReMi survey was to provide a profile of the subsurface soil/rock conditions in 1-dimensional and 2-dimensional subsurface profiles to an approximate depth of 100 feet below the ground surface. A copy of the Marvin E. Davis & Associates Report is submitted under Appendix B.

3. Rock Mechanics: The rock mechanics field work followed the Objectives and Scope of this report.

On the week of October 6 through the 8th, 2009, a detailed structural mapping program was conducted on the outcrop(s) of Slope 27. The purpose of the detailed structural mapping program was to evaluate the fracture pattern and overall stability of the outcrop faces to determine mitigation option(s) to eliminate the amount of rock debris falling onto Hwy 50 and increase overall global stability of Slope 27. The horizontal distance of rock outcrop examined for Slope 27 and the sampling interval of "strike and dips" is as follows:

Slope #	**Slope Interval**	**Sampling Interval (feet)**	**Joint Orientations (Strike and Dips)**
Slope 27	STA: 329+50 to 336+00	650	186

Two different rock type formations are identified along Slope 27. A full geologic description of each formation is defined in Section 2.1 of this report. Collectively, 650 linear feet of slope were examined in the field with a total of 186 joint orientations being obtained from these rock outcrops. The joint orientations were collected using a Brunton™ and a Suunto™ compass. Dip direction and dip magnitude (right hand rule) were recorded for RockPack™ III net computer program.

2.0.1 General Conditions of Slope 27

Slope 27 consists of two parallel slopes separated by a very undistinguished drainage basin having mature tree growth fed by multiple springs. The second cut slope of Slope 27 is much smaller in profile (length and height). For the purpose of this report, these two slopes are reported as single Slope 27 (see Appendix A). Although this investigation encompassed all of Slope 27, the evaluation of rock slope stability was focused upon the approximately 350 feet of slope immediately west of the dissecting drainage (Approximate Status “L2” 331+20 Rt to “L2” 334+70 Rt). Slope 27 is a relatively steep rock/soil slope with sporadic growth of pine trees. The slope is approximately 70 feet in height. This slope is generally facing north to northeast.

Moving from west to east (along a compass line), the first 214 linear feet of Slope 27 (Approximate Status “L2” 329+44 Rt to “L2” 331+59 Rt) consist of a soil/rock matrix with some relatively small, isolated rock outcrops. The soil predominately controls the failure mechanism in this slope. The most obvious slope failure mode for this section of Slope 27 is a slide or circular failure. The small isolated rock outcrops exhibit traditional wedge and sliding joint failure sets. These joint patterns are very small (less than 1 foot in length) and have no bearing on the overall stability of the slope. The angle of repose is 40 to 45 degrees.

From 215 feet to 497 feet (Approximate Status “L2” 331+60 Rt to “L2” 334+57 Rt), the slope consists of rock outcrop(s) having an angle of repose of 40 to 51 degrees. The rock outcrop(s) in this section consists of yellow to tan moderate to deeply weathered monzogranite that is extremely jointed, fractured and friable. On the eastern edge the rock outcrop (geology) changes medium to dark gray altered Hornblende Trachyte. Field evidence of circular, sliding, wedge, and toppling joint/fracture sets were observed. These structural conditions are controlled by the very close fracturing and jointing in the host rock.

From 498 to 650 feet (Approximate Status “L2” 334+58 Rt to “L2” 336+10 Rt), the slope consists of rock outcrop(s) having a angle of repose of 45 to 52 degrees. The altered Trachyte located on the eastern edge has wide to very wide fracturing, is intact to blocky and appears to be very competent in the field. The altered Trachyte is Slope 27 structurally exhibits both sliding and wedge failure type(s).

Most of the rock that has rolled into the ditch ranges from 3 to 12 inches in relative diameter. The rock debris is angular to sub angular. The ditch at the base of this slope is approximately 4 to 8 feet wide. There are no barrier rails to protect the roadway from rockfall.

2.1 Geologic Environment

The Lake Tahoe area, located on the California-Nevada State line at the eastern edge of the northern Sierra Nevada Mountain Range, is underlain by Mesozoic “granitic type” rocks, predominantly granodiorite (Hyne, 1972). Tertiary volcanic rocks, including basalts, andesites, latites, trachytes and various tuffs and mudflows commonly overlay the granitics and are often exposed at the surface (Grose, 1985). In addition to the igneous rock units, deposits of alluvium, colluvium, and glacial sediments are locally abundant.

Lake Tahoe is located in a large graben and is approximately 6,225 ft. above sea level. The normal faulting which created the Tahoe Basin is related to Basin and Range structures which continue eastward to the Wasatch Mountains of Utah, and according to Hyne (1972), the region is still seismically active with most earthquakes occurring to the north, near the Truckee area. The mountains of the Sierra Nevada to the west were extensively glaciated during the Pleistocene while the Carson Range to the east experienced only limited glaciation, probably due to its location in the rain shadow of the Sierra crest, which rises to elevations of nearly 11,000 ft.

Lake Tahoe is dammed in the north near Tahoe City by andesitic mudflow breccias and during the Pleistocene the lake level rose and dropped dramatically due to recurrent damming by alpine glaciers (Hyne, 1972). This rise in lake level, as much as 492 feet, subsequently resulted in a rise in the ground water table which led to extensive physical weathering of the granitic rocks which now exist on the perimeter of the lake. Warhiftig (1965) proposed that the weathering of granitic rocks in the Sierra Nevada is often a function of microfracturing caused by the expansion of biotite that is continuously exposed to water, and the extent of the microfracturing is directly responsible for changes in the engineering properties of the intact rock (Krank and Watters, 1983). Granitics existing above the groundwater table experience relatively minor physical weathering while those that are or were below the water table for extended periods of time are extremely weathered and readily breakdown to gruss or “decomposed granite” (D.G.). The granodioritic road cuts of US 50 express varying degrees of weathering probably resulting from ancient groundwater table locations.

In the western portion of Slope 27, the host rock consists of monzogranite with some altered Trachyte. The monzogranite is tan to gray in color. The monzogranite is fine to medium grained that includes metavolcanic rocks and hornblende diorite. Based on field observations, the monzogranite is moderately close to very close fractured, moderate to hard with some sections exhibiting little to no weathering and other sections of the rock have deep weathering.

In the central to eastern portion of Slope 27 the monzogranite host rock changes to an Altered Trachyte. The Altered Trachyte consists of moderately close to very close jointing and fractures, hard, moderately to highly weathered, altered yellow to tan. The Altered Trachyte is generally fine-grained, porphyritic, extrusive rock consisting of feldspars, biotite, hornblende and small amounts of quartz. It is commonly found in volcanic environments, usually occurring as flows and compound vent filling during the process of volcano building.

In the eastern portion of Slope 27, the host rock in this area is defined as having high alteration and oxidation, mainly argillization and propylitization. Based on field observations, the Trachyte is occasionally to intensely fractured (based on alteration) with both friable to weak zones followed by zones of strong to very strong. Weathering is also inconsistent with some zones exhibiting fresh to little weathering with other zones exhibiting deep weathering.

For this rock mechanics report, the 1985 U.S. Geological Survey Glenbrook 7 ½ quadrangle map (author T.L.T. Grose, scale 1:24000) was utilized to determine host rock features along U.S. 50 in the Glenbrook area (Figure Three – Geology Site Map).

General observations for Slope 27 include:

- The average or nominal size of rock falling from the outcrops, observed during this field investigation, indicates typical 3-inch to 8-inch three-sided wedge shape rocks, with isolated rocks near 18 inches in nominal diameter and a substantial amount of rock spoil near the ½-inch to 2-inch range consistent with deep weathering and decomposition.
- Frost and spring runoff are the controlling factors by which the rockfalls or rolls out occur along these slopes. As observed in the field during the 2009 investigation, rain events can also trigger rockfall events.
- Soil covers the surface or upper portions/crest of the slopes. Large trees are growing along the crest, however with soil erosion, roots are exposed and the tree eventually dies and falls down the slope (see Appendix A). The vegetated slope above Slope 27 exhibits a 45 degree inclination.
- Springs were noted in adjacent to the drainage basin located on the eastern side of Slope 27. The amount of water coming out of the spring(s) was observed to be less than one (1) cubic foot per second (estimated).
- A regional strike and dip cannot be given for this area due to the nature of deposition under volcanic building for the Glenbrook area. Most of the matrix or host rock has gone through metamorphic process that has included alteration and oxidation (mainly argillization and propylitization).
- The joints, fractures and micro faults in the rock mass control the stability of the slope. All three failure types – sliding, wedge and toppling were noted in all the slope areas. The condition of the joint(s) is fair to very poor (see Appendix A).
- Inclination of rock slope(s) in Slope 27 range from 40 to 52 degrees.

SECTION 3
ROCK MECHANICS ANALYSIS

3.0 Modeling

Slope 27 bedrock structure was analyzed with the aid of the RockPack™ III computer program. The analysis is based on lower hemisphere stereographic projections of sliding features. Planes can be represented as curved lines of intersection with the hemisphere or points representing the intersection of lines normal to the planes through the origin of the sphere intersecting the hemisphere. The curved lines are called great circles of the planes and the points are called poles of the planes. A schematic example showing planes and poles is shown on Figure 4 – Schematic Diagram of Lower Hemisphere - Equal Area Stereographic Projections.

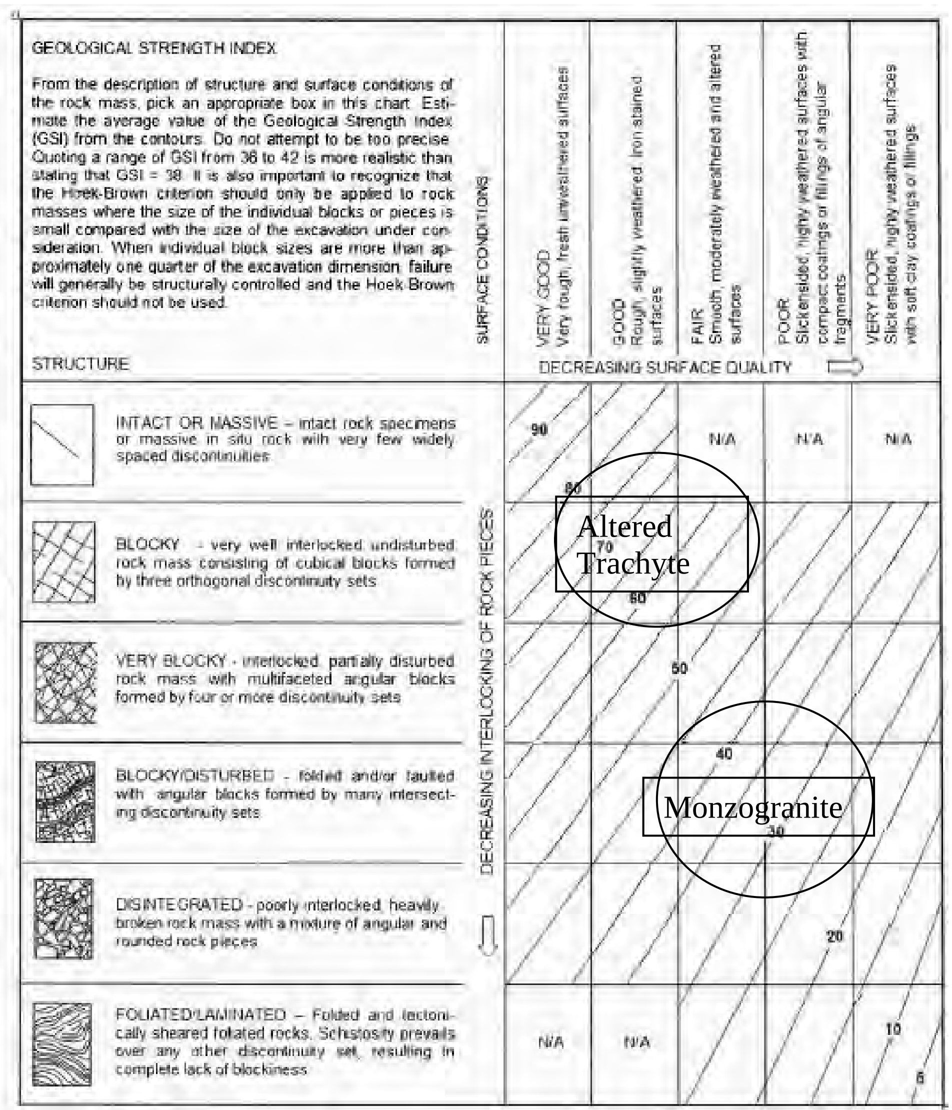
The monzogranite and the altered Trachyte do not exhibit bedding planes. The length of joints and fractures extend from several inches to several feet in length. As there is no predominate or major fracture sets, all repeating fracture sets were modeled. Several large fracture or fault sets were noted in Slope 27. In Appendix D – Major Fault Intersections, the computed graphical statistical stereo plots for the Slope 27 fault blocks were modeled. In Appendix D, the computed graphical statistical stereo plots for Slope 27 small joint intersections were modeled. Wedge, sliding and toppling failures were noted in both the fault and small joint intersection sets. The raw field data is submitted under Appendix E of this report.

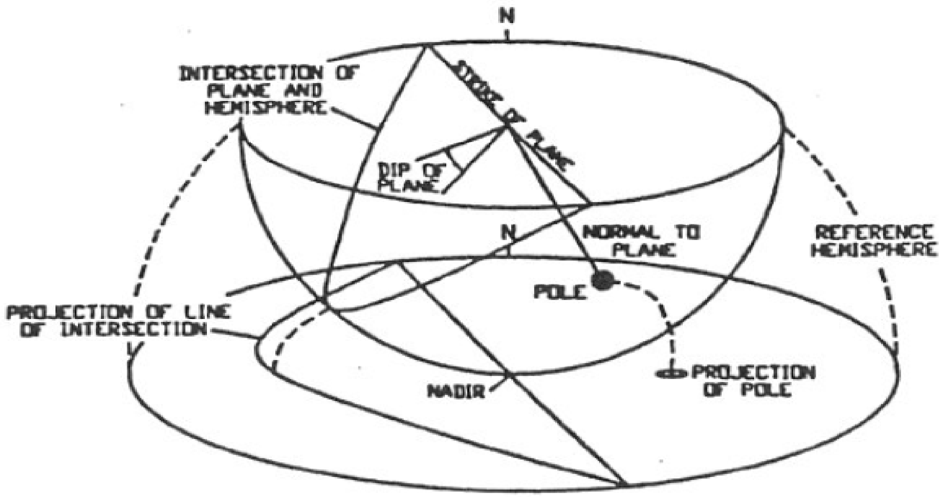
Specific rock mechanics information for the central portion of Slope 27:

Item:	**Description:**	**Value:**
Unit Weight (Wood Rodger Lab)	Rock in Slope 27 Soil above Slope 27	153 PCF 125 PCF
Internal Friction Angle (NDOT Rockfall Hazard Rating)	Monzogranite Trachyte Soil above Slope 27	32^0 36^0 43^0
Average Roughness of Joints (JRC)	Monzogranite Trachyte	12 to 16 8 to 12
Rockfall	Average diameter of rock that has rolled down the slope face	½ to 18" range Average 3 to 8"
Slope Profile	Angle of Slope above road Angle of Slope above crest	42^0 to 50^0 29^0 to 45^0
Compressive Strength Schmidt Hammer Rebound	Monzogranite	5670 PSI (Average) [1500; 1800; 3000;

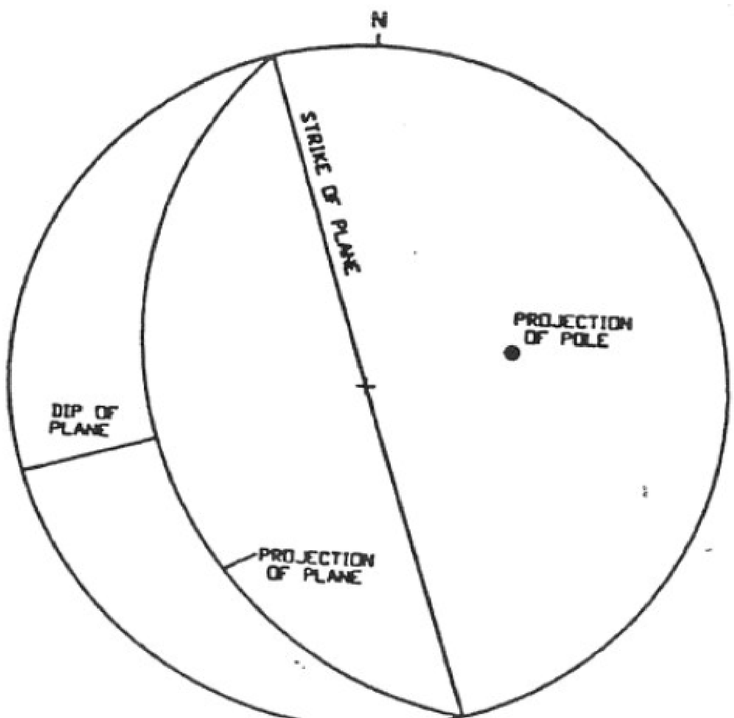
Test (12 tests taken)		3800; 5500; 7000; 8000; 8500; 8600; 9000]
Condition of Joints	Monzogranite Trachyte	Fair to very poor Good to poor
Inclination of major joints/faults (right hand rule)	Joints or Faults extending 20 feet in length	o 358/ 66 o 102/65 o 224/ 35 o 350/74 o 188/ 41 o 358/84 o 278/ 50 o 124/20 o 080/ 65 o 102/65

Geological Strength Index and joints in the rock mass are based on the following table:





PROJECTION ONTO HORIZONTAL PLANE BY ROTATION ABOUT NAIR OF REFERENCE HEMISHERE



LOWER HEMISPERE EQUAL AREA STEROGRAPHIC PROJECTION

Figure 4 - Schematic Diagram of Lower Hemisphere - Equal Area Stereographic Projections

Ref: Rock Slope Engineering, Third Edition Hoek & Bray

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The stereo analysis considers the orientations of joint planes and the orientation of the rock outcrop face. Four modes of rock slope failure are possible: circular (in heavily decomposed rock), slide, wedge and toppling failure (Figure 5 – Rock Mechanics Failure Types). Sliding is expected on joint surfaces and intersecting joint surfaces (wedges) which are inclined steeper than the angles of friction. The NDOT Rockfall Hazard Rating System report identifies the internal friction angle for host rock as the following: the Monzogranite at 32°; and the Trachyte between 36° to 37° (E. Hoek & J.W. Bray in "Rock Slope Engineering", 3rd edition 1981, approximate the internal friction angle of granite at 29°-35° for fine grained and 31°-35° for coarse grained) therefore an internal friction value of 32° was used in this stability analyses. The 32° friction angle (Appendix D) is represented on the stereo net diagram by a circle 32° from the center of the stereo net.

The potential for rock slope failure within the central portion of Slope 27 is based on the orientation of the joints. Joint planes and wedge intersections which fall between the center of the stereonet and the 32° internal friction circle are potentially unstable, depending on the dip orientation of the joint in relation to the cut slope. Sliding or plane failure will occur if the projections of joint planes or wedges fall on the 32° internal friction circle and outside/down dip of the cut slope; this condition is called "daylighted joint" or "daylighted wedge" because the joint planes or wedges are exposed in the existing cut slope. Joint planes which fall directly opposite to the slope or rock outcrop face may be subject to toppling failure even though the joints dip into the rock outcrop face. Circular failures are not represented in the stereo nets, but are rather a function of the raw point count (Figure 5) as collected from the field data.

Joint intersections within the central portion of Slope 27 do not follow a set pattern similar to a shale, sandstone or dolomite type rock. The host rock(s) at Slope 27 are volcanic in origin that has been altered or metamorphosed. Evidence of all four types of rock failure (wedge, sliding, toppling and circular) can be observed within the central portion of Slope 27. It can be assumed that the difference in failure types is a result of heavy weathering (decomposition) and selective break-down of the weaker microcrystalline matrix of the monzogranite and trachyte. Subsequently, in areas where these host rocks have experienced moderate to extreme weathering, rock debris can range from inches to several feet (<2 feet) in diameter. Calculating the factor of safety for the existing rock outcrop(s) indicates that at best, these slopes have a factor of safety of approximately one (1) under dry joint/fracture conditions. When water is added to the joint or fracture system, localized factors of safety fall under one (1) and rockfall event occurs. The highest potential for rock outcrop failure is in the spring and fall seasons, when both moisture and frost are combined. It should be clarified that failure mechanism noted for the central portion of Slope 27 are confined to near surface or surface manifestations resulting in isolated and localized rockfall events. No failure mechanism (large fracture sets extending several 10's of feet), commonly associated with a catastrophic or global event were noted in Slope 27.

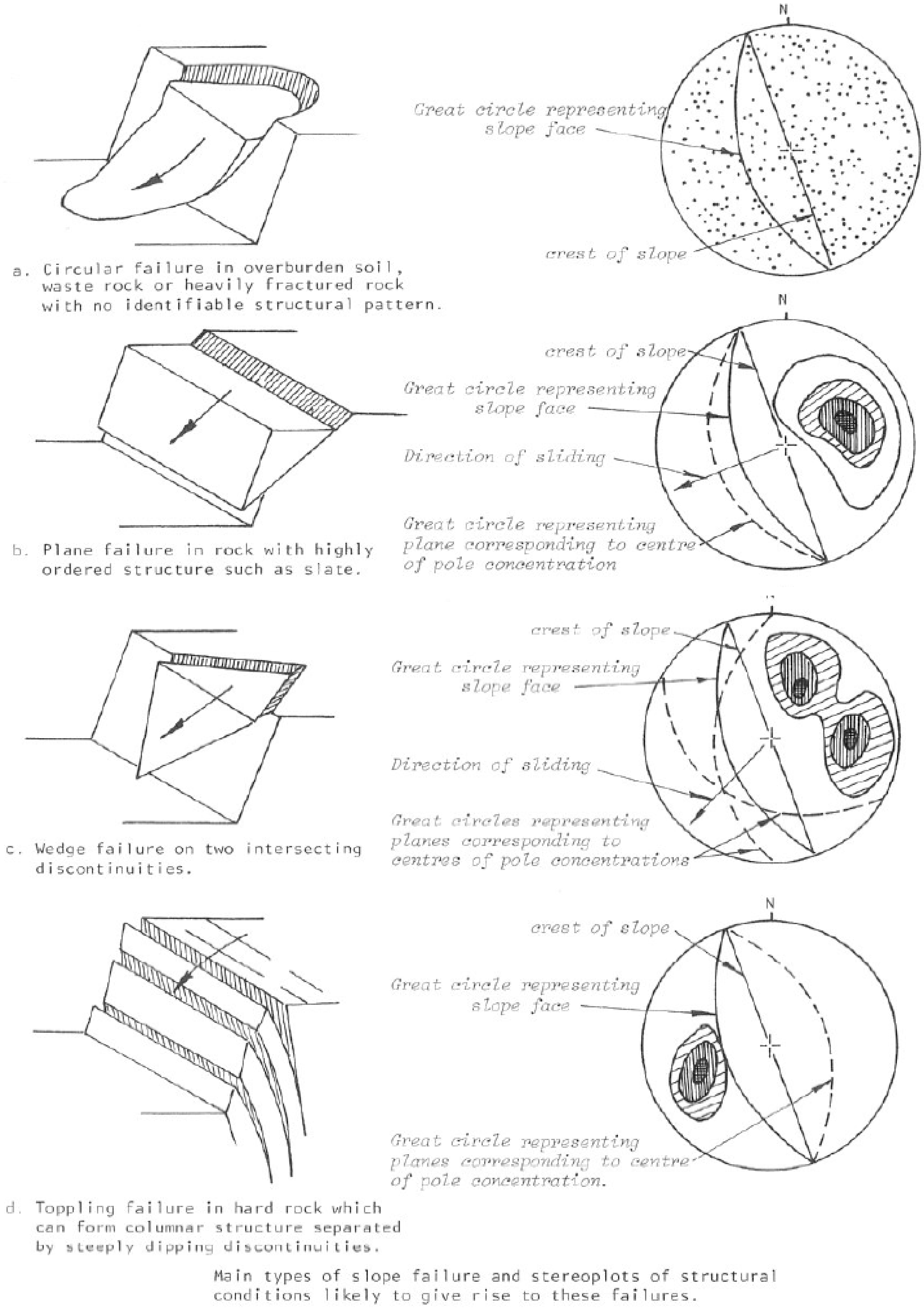

Figure 5: Rock Mechanics Failure Types in relation to the Stereo Plots

Ref: Rock Slope Engineering, Third Ed, Hoek & Bray

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SECTION 4
DISCUSSION OF FINDINGS

4.0 Findings

The currently proposed rockfall erosion mitigation method for the central portion of Slope 27 is the application of a wire mesh (cable anchored) on those sections of the slope with significant rock outcrops. Several wire mesh manufacturers and vendors exist in the industry that can provide acceptable products. A DRAFT Special Provision for wire mesh (cable anchored) is submitted under Appendix F. A discussion of other findings for central portion of Slope 27 is as follows.

During the field investigation, evidence of all four types of rock failures were observed in Slope 27 (see Appendix A). The four types of rock failure are a function of weathering and fracture patterns in the host rock. Slope 27 starts and ends with zones of friable, deeply weathered, almost decomposed rock that form 1:1 (H:V) debris/talus slopes of coarse small diameter rock fragments. As the host rock becomes more competent in the central portion of Slope 27, the potential for circular failures transition into slide, wedge and toppling type failures. Also noted within the competent host rock are thin zones of weaker, relatively in-competent rock that create debris chutes for smaller diameter rock typically between ½ to 6 inches in size. The natural undisturbed slope above Slope 27 ranges between 29 to 45 degrees, and is moderately vegetated with large conifer pines and frequent underbrush.

As identified in the Marvin E. Davis & Associates Report (Appendix B), the ReMi survey completed above the cut portion of Slope 27 (Plate 10) indicates that the first 9 to 10 feet (Plates 13 and 14) of the upper soil horizon consists of a soil and broken rock mix. Between 10 to 11 feet, the upper soil horizon (soil and rock) begins to grade to host rock (based on shear wave velocity). The host rock, consisting of Monzogranite and Trachyte extends to depths exceeding 70 to 80 feet. The upper 5 to 10 feet will not provide the necessary resistance for anchors and the anchors will need to be extended in the deeper, more competent rock encountered below the upper horizon. The shear wave velocity of approximately 1,000 feet per second (FPS) corresponds to competent rock, according to the IBC, corresponds to a blow count of approximately 50 blows per foot.

In central portion of Slope 27 the predominate rock failure mode (see Appendix D) is a wedge-type failure, which is schematically represented by the intersection of the great circles within the gray shaded Zone #1. However, to a much lesser degree, slide-type and toppling failure modes are also represented in the gray shaded zones. Observed rockfall from central portion of Slope 27 ranged from 1/2" to 18" in diameter, as noted in the talus slope and ditch line. Rocks exceeding 18 inches in diameter were very infrequent. The average size of the three-sided wedge rock debris located in the talus slope and ditch line was 3 to 8 inches size range.

Based on Figure 6, a conservative estimate of loose rock that can be removed in the area by scaling is approximately 650 to 1,000 cubic yards (see calculation on Figure 6). The

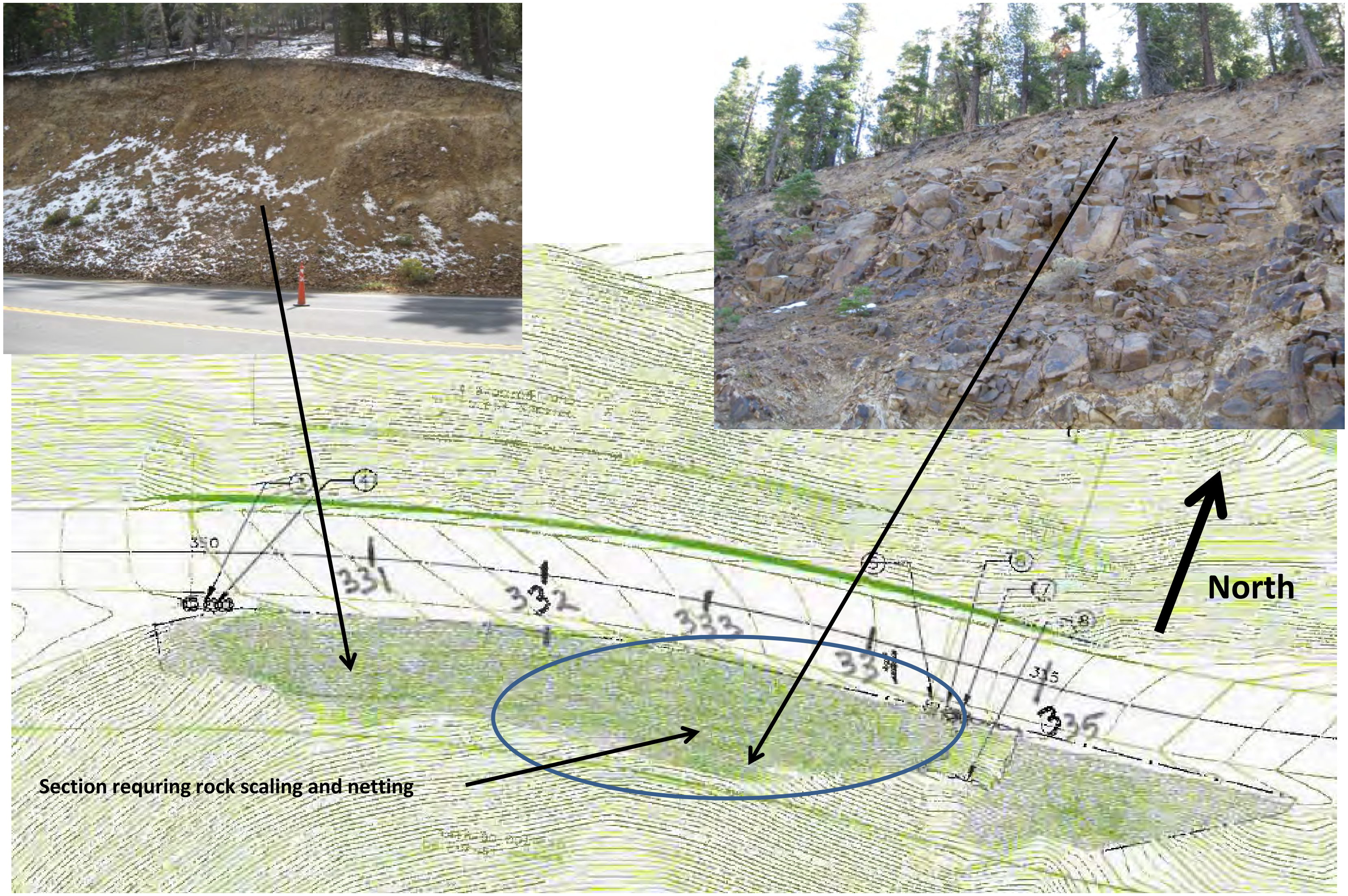

Figure 6 - Slope 27 Zone requiring rock scaling and netting

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Volume of rock removal = 400' (length of slope) x ~ 45' (vertical component) x 1' thick layer
Volume of rock removal = 650 to 1,000 CY

Map provide by Wood Rodgers

level and volume of rock scaling will be determined in the field by the engineer of record at the time that scaling operations occur. The volume of rock to be scaled is based on field observations. No direct measurements (prying, or repelling) of the slope were performed. Additional loose rock may be encountered on the edges of the slope where decomposition is stronger or less if the rock mass is more competent with depth or exhibits less weathering. Photographs of the cut slopes (Figure 6) indicate typical areas and rock zones to be scaled (pictures are not meant to indicate or represent actual field conditions of all rock requiring scaling).

Cable spacing at the top of the slope for wire mesh is based on snow load calculations. As outlined in the design guidelines for wire mesh/cable net slope protection as prepared by the Washington Department of Transportation, snow load plays a crucial role in determining cable. Calculations to determine cable spacing in the central portion of Slope 27 is submitted under Appendix G.

Based on the NDOT – RHRS report the predominant geo-hazard on the Glenbrook Slopes (Slope 27) is the quick collection and abundance of eroding small rock debris related to storm events and the winter season along the ditch line. The small rock comes from zones that are highly weathered, jointed and fractured. Soil erosion near the crest of the slopes may yield large rockfall failure onto the travel way of US 50. The potential for a large rock failure from the upper reaches of the central portion of Slope 27 is possible.

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APPENDIX A

PICTURES OF SLOPE 27

Slope 27 looking east along Hwy 50 – circled area exhibits slide failures

Debris along the toe of Slope 27

Upper portion of Slope 27 is a mix of soil and rock and undercuts the soil

Portions of Slope 27 have some mature vegetation starting to grow


Slope 27 exhibits wedge, sliding and topple failure fracture sequences


Example of toppling failure in Slope 27

Rock will be required to be scaled from portions of Slope 27

Altered Trachyte in the eastern side of Slope 27. The Trachyte is blocky to massive.

APPENDIX B

MARVIN E. DAVIS & ASSOCIATES INC – REMI REPORT

Marvin E. Davis & Associates, Inc.

Consulting Civil Engineers
P.O. Box 18449
Reno, Nevada 89511
PH (775) 853-9100
FAX (775) 853-9199

November 4, 2009
File No. 9059.001

Mr. Loren A. Jalbert
McMillen LLC.
910 Main Street, Suite 258
Boise, ID 83702

Subject: **ReMi Shear Wave Velocity Measurements**
Slopes 21 and 27, Highway 50 East of Highway 395
Douglas County, Nevada

Dear Mr. Jalbert,

This report presents the findings of the refraction microtremor (ReMi) shear wave velocity survey performed by Marvin E. Davis and Associates Inc. (MDA) for the subject project. The ReMi survey is part of an ongoing geotechnical investigation being performed by Wood Rodgers Inc., and McMillen LLC., for Slope 21 and Slope 27 on US 50 near Glenbrook, Nevada. The ReMi survey was performed on October 6 and 7, 2009 by our engineer Stella Montalvo and senior technician Justin Kamen. Loren Jalbert of McMillen LLC., and Mickey Smith of Wood Rodgers Inc., were also present at the site during our shear wave velocity measurements.

SITE DESCRIPTION

Slope 21 and Slope 27 are located on the south side of the Highway 50 near Glenbrook, Douglas County, Nevada, (Plate 1). Both slopes exhibit weathering and raveling at the slope faces. It is our understanding that McMillen LLC., will be providing slope retention (i.e. soil nail) design for both slopes.

PURPOSE

The ReMi survey results are used to provide a Site Class for the subsurface conditions as required by the 2006 International Building Code (IBC). The survey also provides 1-dimensional and 2-dimensional subsurface profiles to an approximate depth of 100 feet below the ground surface.

The 1-dimensional and 2-dimensional subsurface shear wave velocity profiles will aid McMillen engineers in assessing thickness and consistency of soils, the possible presence of boulders/cobbles, bedrock contact, and the rippability (by correlating with P waves) of the bedrock material before proceeding with the slope retention design.

METHODOLOGY

The ReMi method provides an effective and efficient means to obtain general information about large volumes of the subsurface in one and two dimensions per setup, where appropriate setup length is related to the depth of investigation. ReMi is described by Louie (2001), where it is applied to obtain vertical S–wave profiles to depths up to 300 feet for earthquake seismic site characterization. The methods' theoretical basis is the same as spectral analysis of surface waves (SASW) and multi-analysis of surface waves (MASW). However, field data is collected using modern standard small exploration seismic equipment. ReMi interpretation and analysis is performed using appropriate software that is available for desktop and notebook personal computers.

DATA ACQUISITION

We obtained ReMi data along 11 seismic lines, seven lines for Slope 21 and four lines for Slope 27, as shown on the Site Plans, Plate 2 and Plate 10. The seismic lines were positioned, as directed by McMillen and Wood Rodgers, to sample both the ridge and toe of the slopes. The lines were setback to a safe distance from the ridge edge of the slopes.

Surveys were performed in general accordance with the method described by Louie (2001) to develop vertical 1-dimensional and 2-dimensional S-wave velocity profiles. A multi-channel seismograph capable of storing up to 16,000 samples per channel at sample intervals with 1 to 2 milliseconds in SEG2 or SEGY format was used to collect ReMi data. We performed ReMi surveys using a DAQ link II - 24 bit Data – 12 channel signal enhancement seismograph.

Geophone cables with 12 geophone takeouts at typical 28 feet spacings were used. Vertical geophones with resonant frequencies of 4.5 Hz were used to obtain surface wave data for S-wave vertical profile analysis. Broad band ambient and controlled surface wave site noises were used as a surface wave energy source. Controlled surface wave energy sources included jogging alongside geophone arrays. The data transferred to field laptop and later transferred to Optim software facilities via e-mail for processing.

DATA ANALYSIS

Optim software representative Dr. Satish Pullammanappalill performed interpretations of each line using the most current SeisOpt ReMi software package. The 1-dimensional and 2-dimensional cross-sections of the lines were prepared by Dr. Pullammanappalill in coordination with our engineers.

The SeisOpt ReMi software consists of two modules. The first module is used to transform data files into a spectral energy shear wave frequency versus shear wave velocity (or slowness) presentation for each ReMi seismic setup. The interpreter then selects a dispersion curve consisting of the lower bound of the spectral energy shear wave velocity versus frequency trend and that dispersion curve is saved. Tracing the lower bound (slowest) of the shear wave velocity at each frequency selects the ambient energy propagating parallel to the geophone array, since energy propagating incident to the array will appear to have a faster propagating velocity. The second module allows the interpreter to model a dispersion curve with multiple layers and S-wave velocities to match the selected dispersion curve from the field data. The modeler interactively varies layer velocities and depths until the resulting dispersion curve best matches the previously selected dispersion points. An interpreted vertical S-wave profile is obtained through this process.

It must be understood that this type of interpretation may not result in a unique solution.

RESULTS AND DISCUSSION

The results of ReMi survey are represented by the 1-dimensional and 2-dimensional shear wave velocity profiles given on Plates 3 through 14. These sections depict variations in shear wave velocity profiles within 100 ft of the subsurface. The velocities are represented by color shading as indicated by the velocity scale shown below each cross-section.

Note that each measurement starts from the right side of the line (geophone 1, A) and proceeds to the left side of the line (geophone 12, A'). The 2-dimensional models also start at 95 feet and end at 205 feet away from the starting point. This is caused by the use of an interpolation process which is called grouping the traces. In general grouping the traces (depending on the data received) would consist of interpolation between the geophones (1 to 6, 2 to 7, 3 to 8 , 4 to 9, 5 to 10, 6 to 11 and 7 to 12) by loosing unreliable data from geophones 1 to 3 and 9 to 12. The longer the line (i.e. the larger the spacing between phones) the lower the resolution in the upper 15 to 20 feet, however, deeper than 20 feet below ground surface provides considerably greater resolution. The elevations provided to generate 2-dimensional model profiles can be off by as much as 5 feet to 7 feet due to the lower resolution.

The velocities obtained from ReMi surveys were assessed using Table 1613.5.2 given in International Building Code (IBC) 2006. This table is given below.

TABLE 1613.5.2
SITE CLASS DEFINITIONS

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, SEE SECTION 1613.5.5		
		Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H >$ 10 feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays (H >25 feet with plasticity index PI >75) 4. Very thick soft/medium stiff clays (H >120 feet)		

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m^2, 1 pound per square foot = 0.0479 kPa. N/A = Not applicable

Slope 21

The average shear wave velocity of the ridge of the slope ranged from 1,998 ft/s to 2,336 ft/s while average shear wave velocity of the toe of the slope ranged from 1,703 ft/s to 1,995 ft/s. These velocities indicated a Site Class of C, very dense soil and soft rock profile for the upper 100 feet with an undrained shear strength of Su>2,000 psf in accordance to the Table 1613.5.2 of IBC 2006.

Based on the IBC correlations, Lines 1 and 2 at the ridge of the slope indicated Class C and Class D soil profiles to 15 feet and 23 feet below ground surface (bgs), respectively. In both lines no Class A hard rock was correlated within the 100 feet bgs. Class B Rock was correlated in both lines below surface soils to the 100 feet bgs. Lines 3 and 4 indicated gradual increase in velocity with depth, from as low as 3,400 ft/s at 29 feet bgs to as high as 5,500 ft/s at 72 feet bgs. In both lines Class A Hard Rock was correlated at around 72 feet to 83 feet bgs.

The average shear wave velocities of the toe of slope exhibited slightly lower values compared to the ridge of the slope. This may be explained by the previous roadway cut and fill operations of the HWY 50 near these lines. Lines 5 and 7 exhibited similar characteristics. Both lines indicated Class C and D soils extending to approximately 39 feet bgs. Below these soils both lines indicated Class B Rock to depths of 74 and 83 feet bgs. Below Class B Rock both lines indicated Class A hard rock to the 100 feet bgs. The increase in velocity with depth can probably be attributed to a corresponding decrease in the degree of weathering in Lines 5 and 7. However one notable exception is Line 6. Line 6 indicated Class C soils extending to 83 feet bgs. Class B Rock was correlated below 83 feet. No Class A Hard Rock was correlated for Line 6. The deep Class C soil deposits may be indicative of moist to very wet conditions (drainage layer) which may have contributed to the increased weathering of the subsurface rock layers below Line 6.

Slope 27

The average shear wave velocity of the ridge of the slope ranged from 1,532 ft/s to 1,591 ft/s while average shear wave velocity of the toe of the slope ranged from 1,617 ft/s to 1,890 ft/s. These velocities similar to Slope 21 indicated a Site Class of C, very dense soil and soft rock profile for the upper 100 feet with an undrained shear strength of Su>2,000 psf in accordance to the Table 1613.5.2 of IBC 2006.

Based on the IBC correlations, Lines 3 and 4 at the ridge of the slope indicated Class D and Class E soil profiles for the upper 10 feet. Below these soft soils both lines indicated Class C soils to 69 feet and 81 feet bgs. Class B Rock was correlated in both lines below surface soils to the 100 feet bgs.

The average shear wave velocities of the toe of slope exhibited slightly higher values compared to the ridge of the slope. Lines 1 and 2 indicated that Class C and D soils extend to 30 to 39 feet bgs. Class B rock was correlated in both lines and extended to 77 to 85 feet bgs. In both lines Class A Hard Rock was correlated below Class B Rock to 100 feet bgs.

EXCAVATION CHARACTERISTICS (RIPPABILITY)

Seismic velocity tables relating seismic velocity (P-wave) and excavation characteristics have been developed from field tests by others. These tables list the seismic velocity of various types of bedrock materials and their relative ease of excavation using different types of rippers.

In general, geophysical methods are much more global in nature than borehole measurements. Under ideal rock conditions a typical correlation between P-wave (Vp) and S-wave (Vs) can be defined as Vp/Vs~1.7. However, under soil conditions the ratio can increase dramatically to as much as 10, normally because of a decrease in Vs due to soft soils.

This information should only be used as a general guide, however, as many other factors should also be considered. These factors include the rock jointing and fracture patterns, the experience of the equipment operator, and the equipment and excavation methods selected.

STANDARD CARE AND WARRANTY

The scope of services for this project consisted of using the ReMi method to define subsurface shear wave velocities and depths. The accuracy of our findings is subject to specific site conditions and limitations inherent to the ReMi technique. We performed our services in a manner consistent with the level of skill ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made.

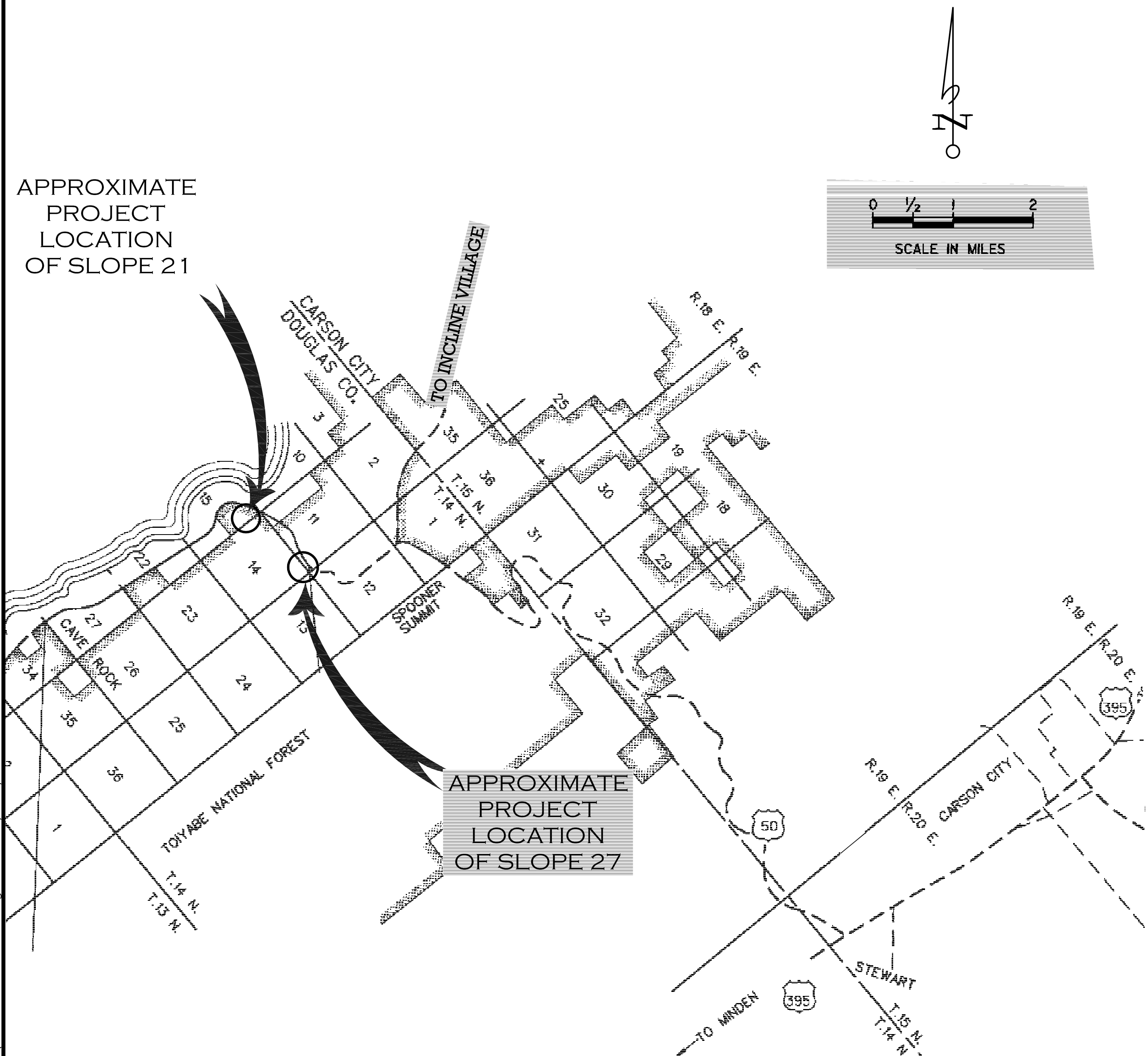
We appreciate the opportunity to provide our services to McMillen LLC., and look forward to working with you again on future projects. If you have any questions or need additional information, please do not hesitate to call.

Sincerely,
Marvin E. Davis & Associates, Inc.

PROFESSIONAL ENGINEER - STATE OF NEVADA
ERTAN OZMEN
Exp: 12/31/09
CIVIL
No. 16730
11/4/09

Ertan Ozmen, P.E.
Civil Engineer, CE 16730

Enclosures: Plate 1 Vicinity Map
Plate 2 Site Plan - Slope 21
Plates 3 through 9 - ReMi Lines 1 through 7 for Slope 21
Plate 10 Site Plan - Slope 27
Plates 11 through 14 - ReMi Lines 1 through 4 for Slope 27



REFERENCE: BASE MAP PROVIDED BY WOOD RODGERS, INC.

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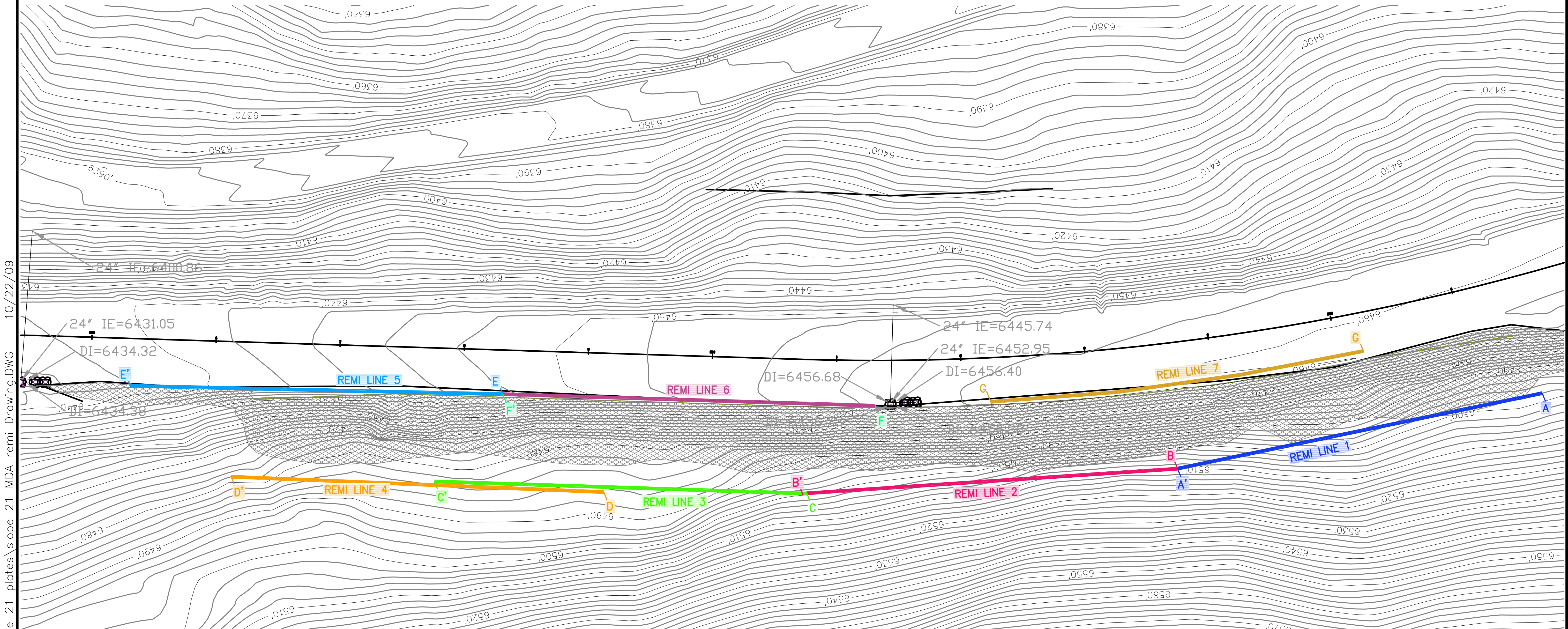
Marvin E. Davis & Associates, Inc. CONSULTING CIVIL ENGINEERS P.O. Box 18449 Reno, Nevada 89511 TEL (775)853-9100 FAX (775)853-9199	VICINITY MAP REMI SURVEY SLOPE TREATMENT PROJECT HIGHWAY 50 EAST OF HIGHWAY 395	PLATE
JOB # 9059.001 APPR: DATE: 10/20/09	DOUGLAS COUNTY NEVADA	1



REFERENCE: BASE MAP PROVIDED BY WOOD RODGERS, INC. ELECTRONICALLY ON OCTOBER 8, 2009.

LEGEND

- APPROXIMATE REMI LINE 1 LOCATION
- APPROXIMATE REMI LINE 2 LOCATION
- APPROXIMATE REMI LINE 3 LOCATION
- APPROXIMATE REMI LINE 4 LOCATION
- APPROXIMATE REMI LINE 5 LOCATION
- APPROXIMATE REMI LINE 6 LOCATION
- APPROXIMATE REMI LINE 7 LOCATION

NORTH
SCALE: 1:80

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SITE PLAN SLOPE 21

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

PLATE 2

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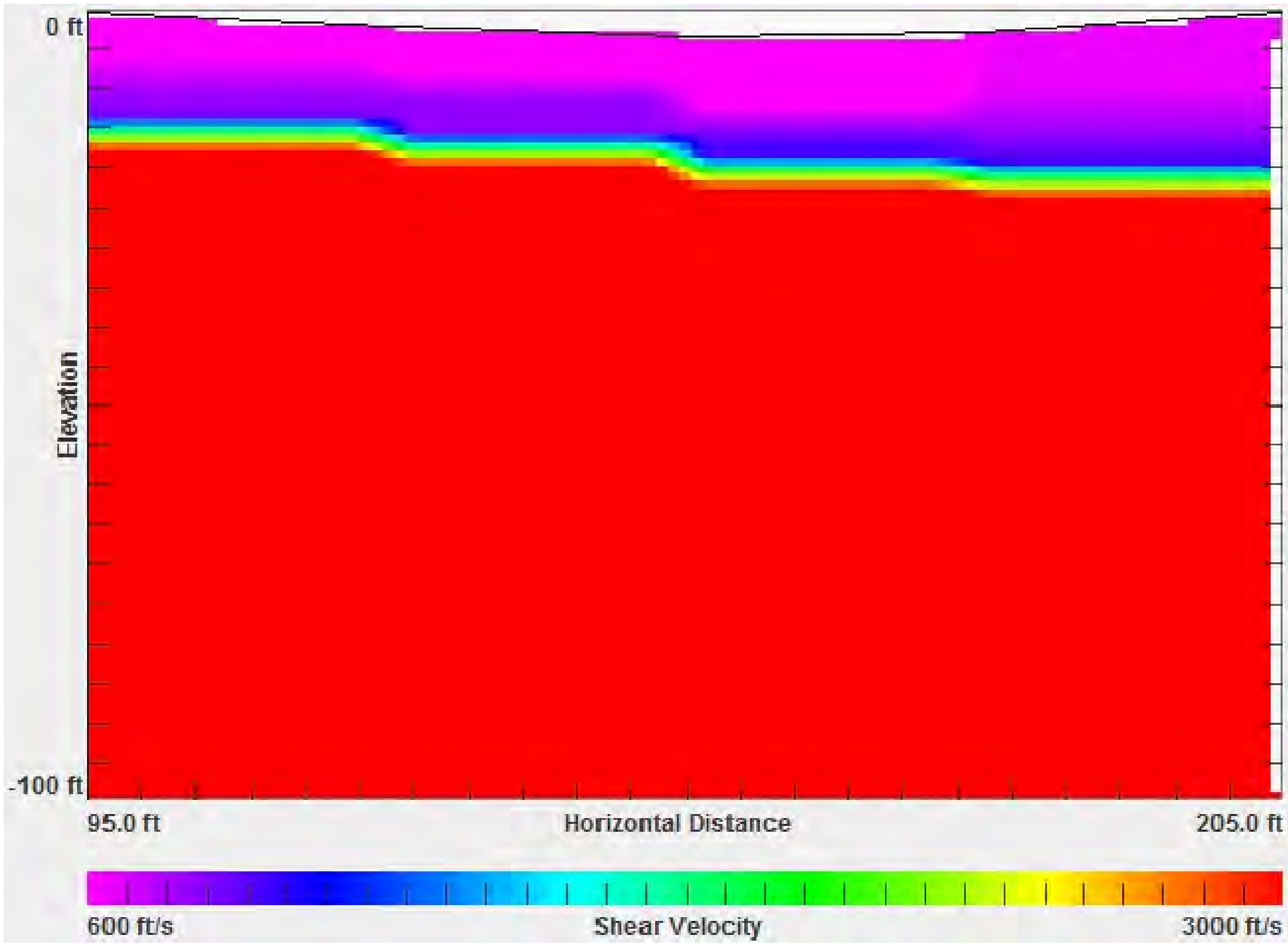
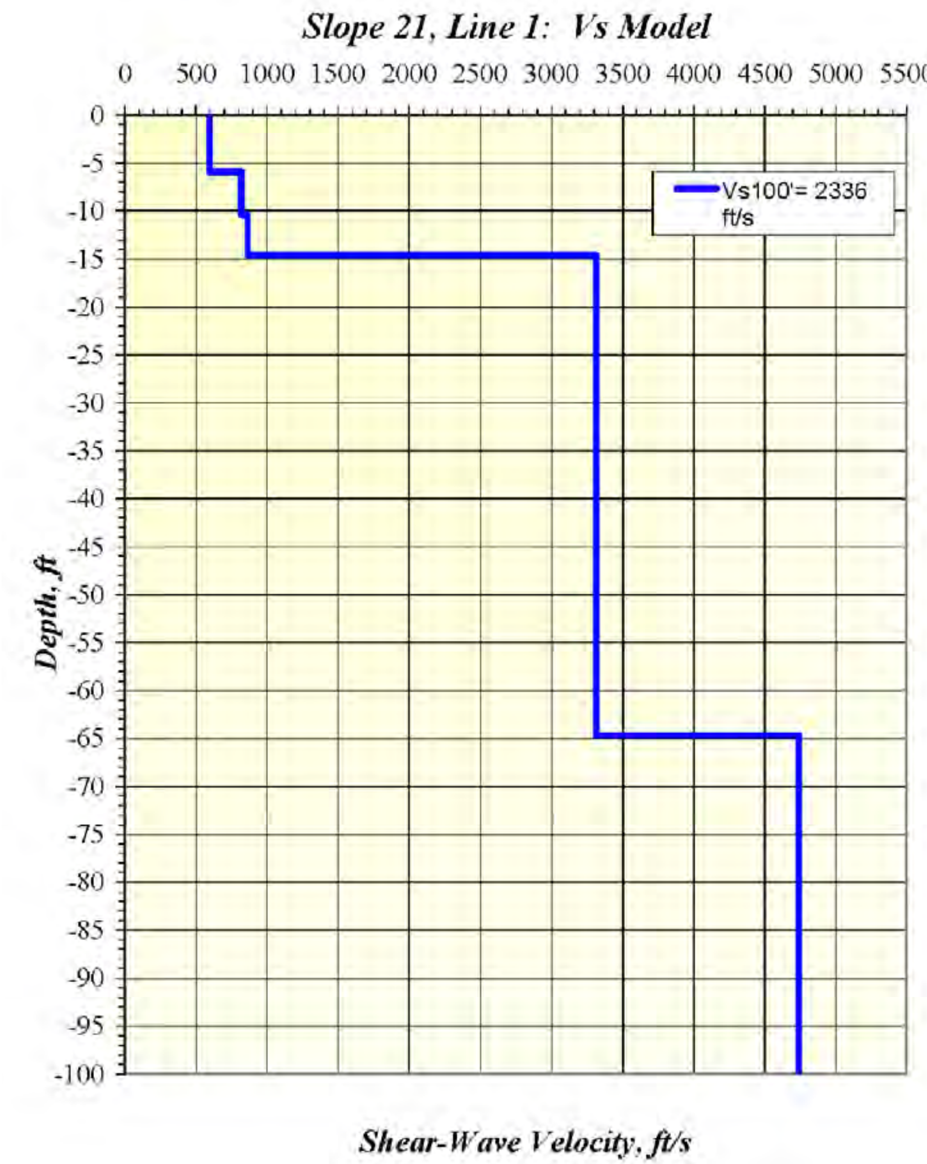


REMI LINE #1-2D
A-A'

SCALE IN FEET
VERTICAL: 1" = 20'-0"
HORIZONTAL: 1" = 13'-4"



REMI LINE #1-1D

20 10 0 20

SCALE IN FEET
1" = 20'-0"

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REMI LINE 1 SLOPE 21

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

PLATE
3

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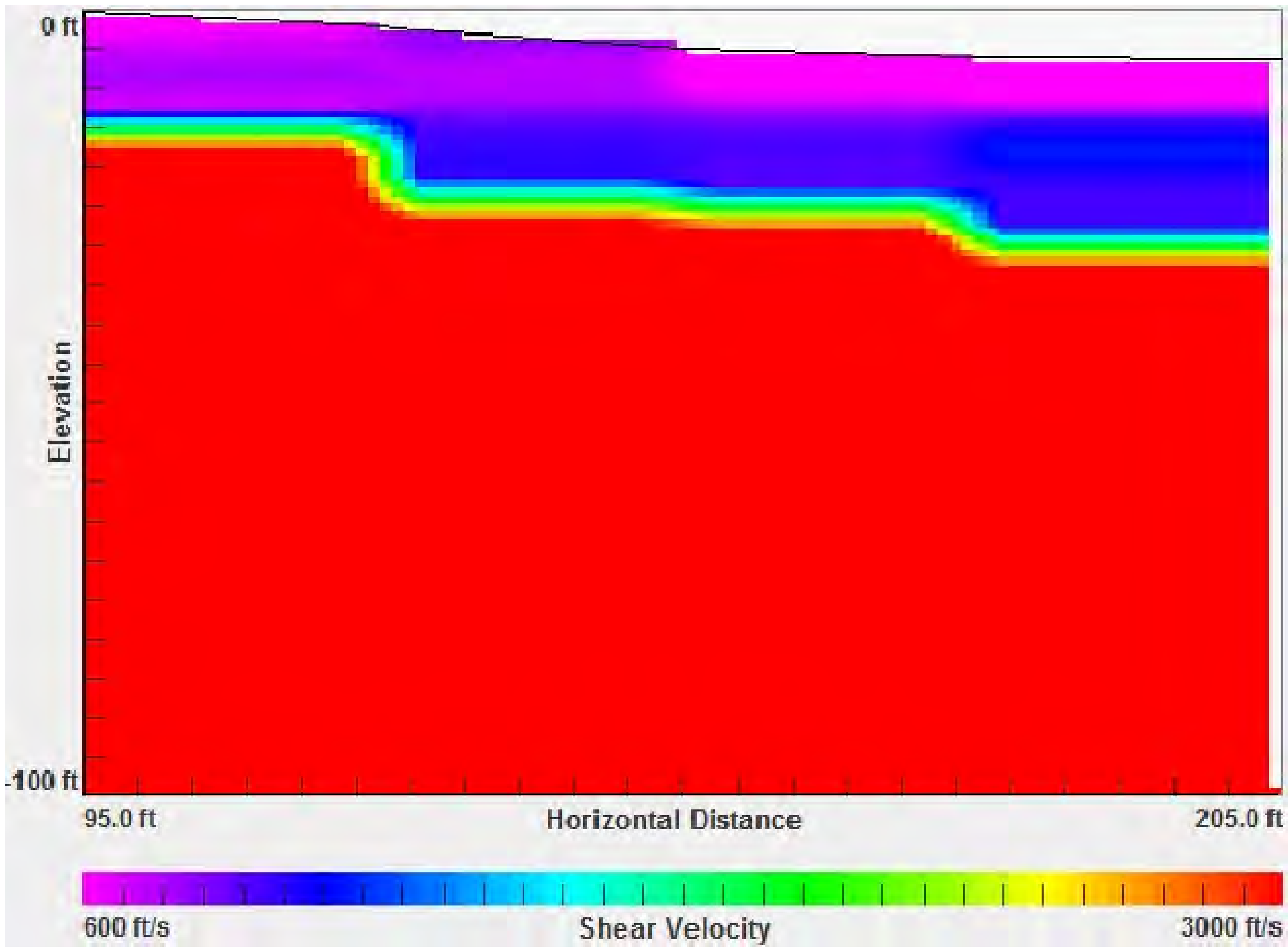

REMI LINE #2-2D

B-B'

SCALE IN FEET
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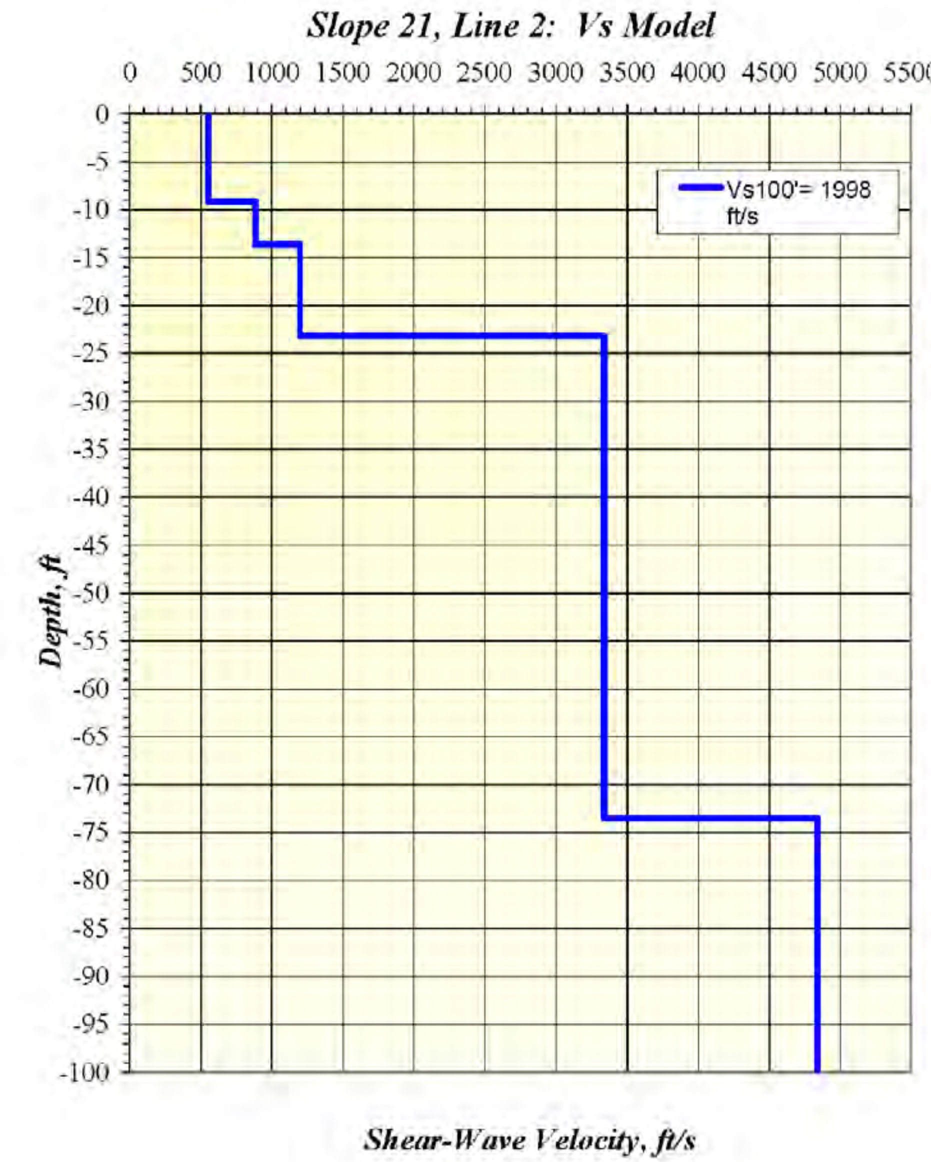
HORIZONTAL: 1" = 13'-4"



REMI LINE #2-1D

20 10 0 20

SCALE IN FEET
1" = 20'-0"

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REMI LINE 2 SLOPE 21

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

PLATE 4

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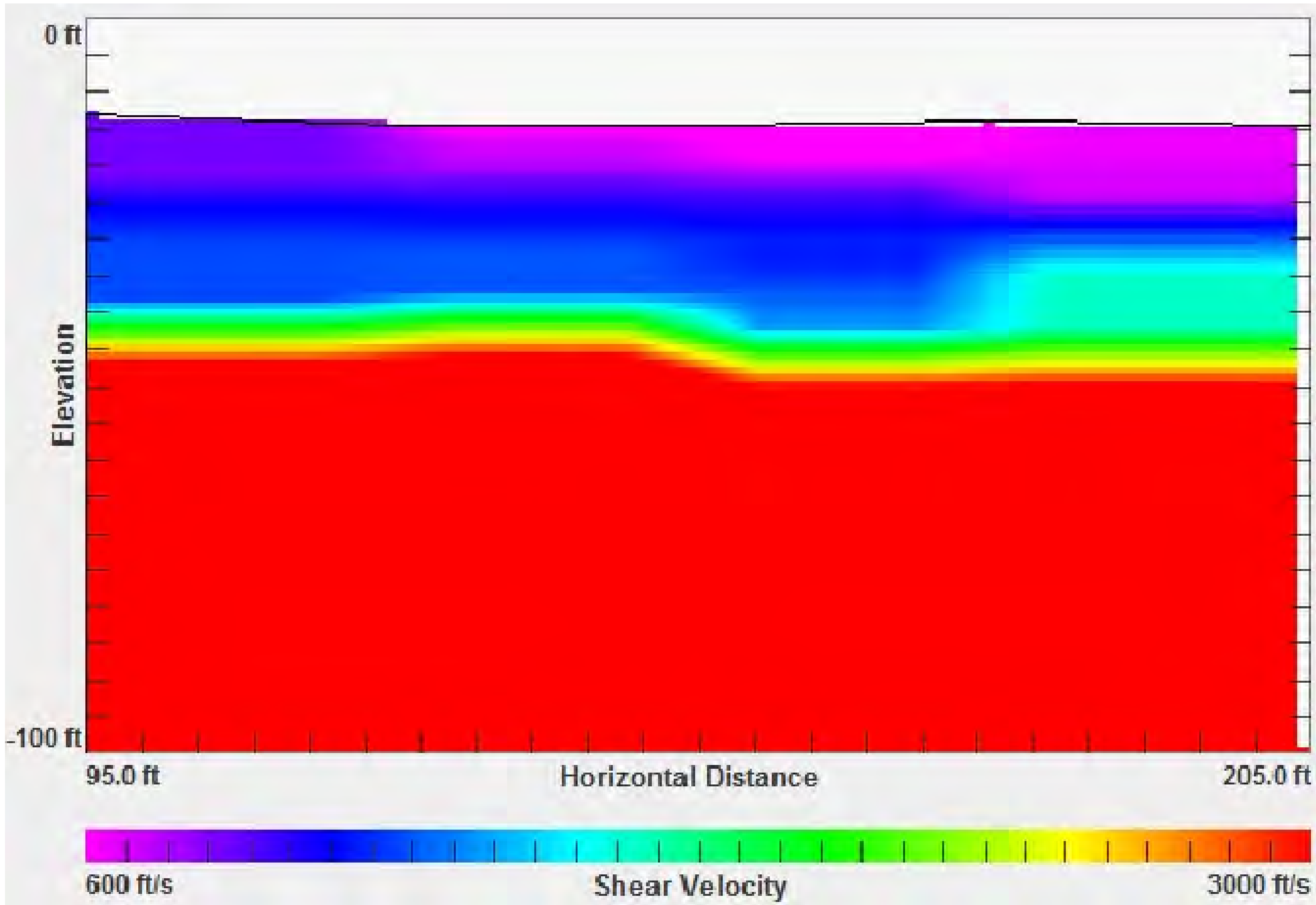

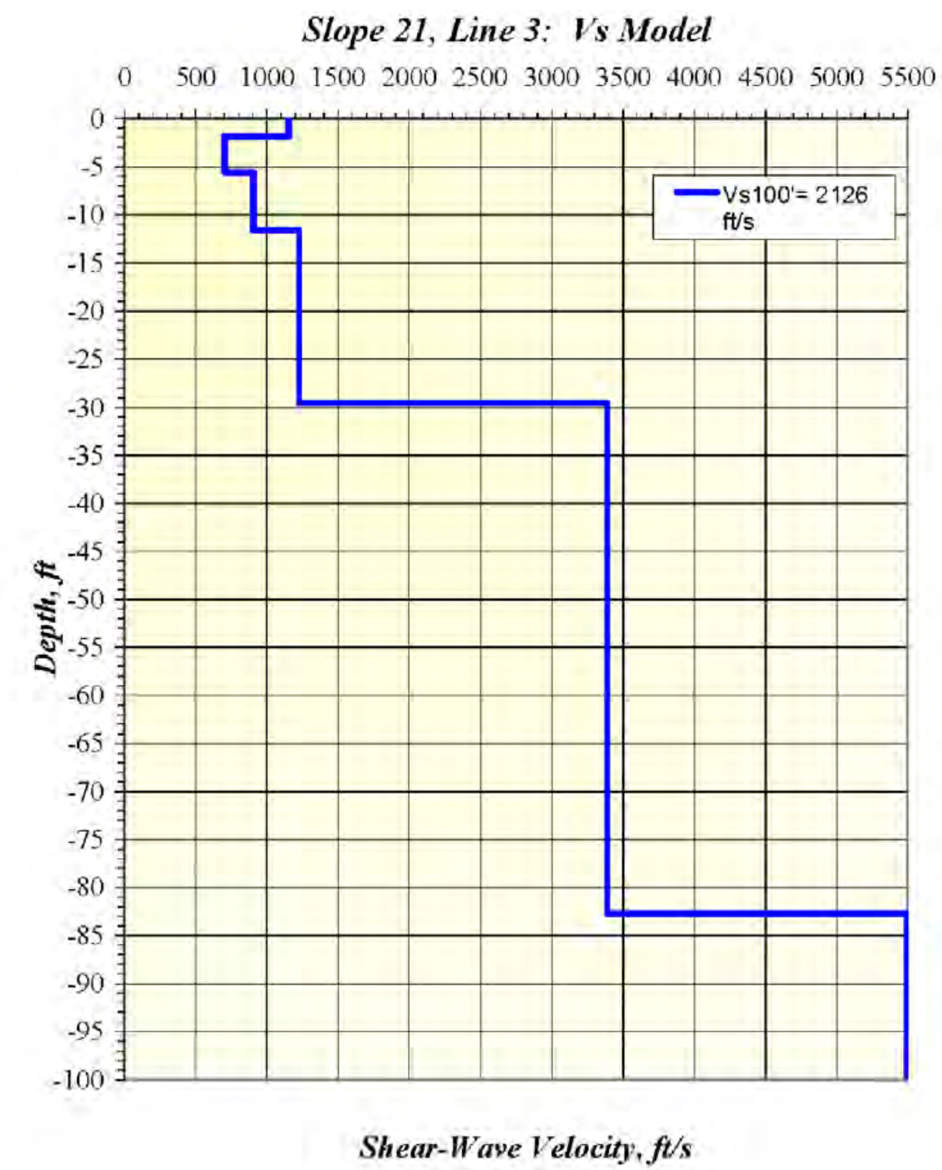

REMI LINE #3-2D

C-C'

SCALE IN FEET
VERTICAL: 1" = 20'-0"
HORIZONTAL: 1" = 13'-4"

REMI LINE #3-1D

20 10 0 20

SCALE IN FEET
1" = 20'-0"

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REMI LINE 3 SLOPE 21

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

PLATE 5

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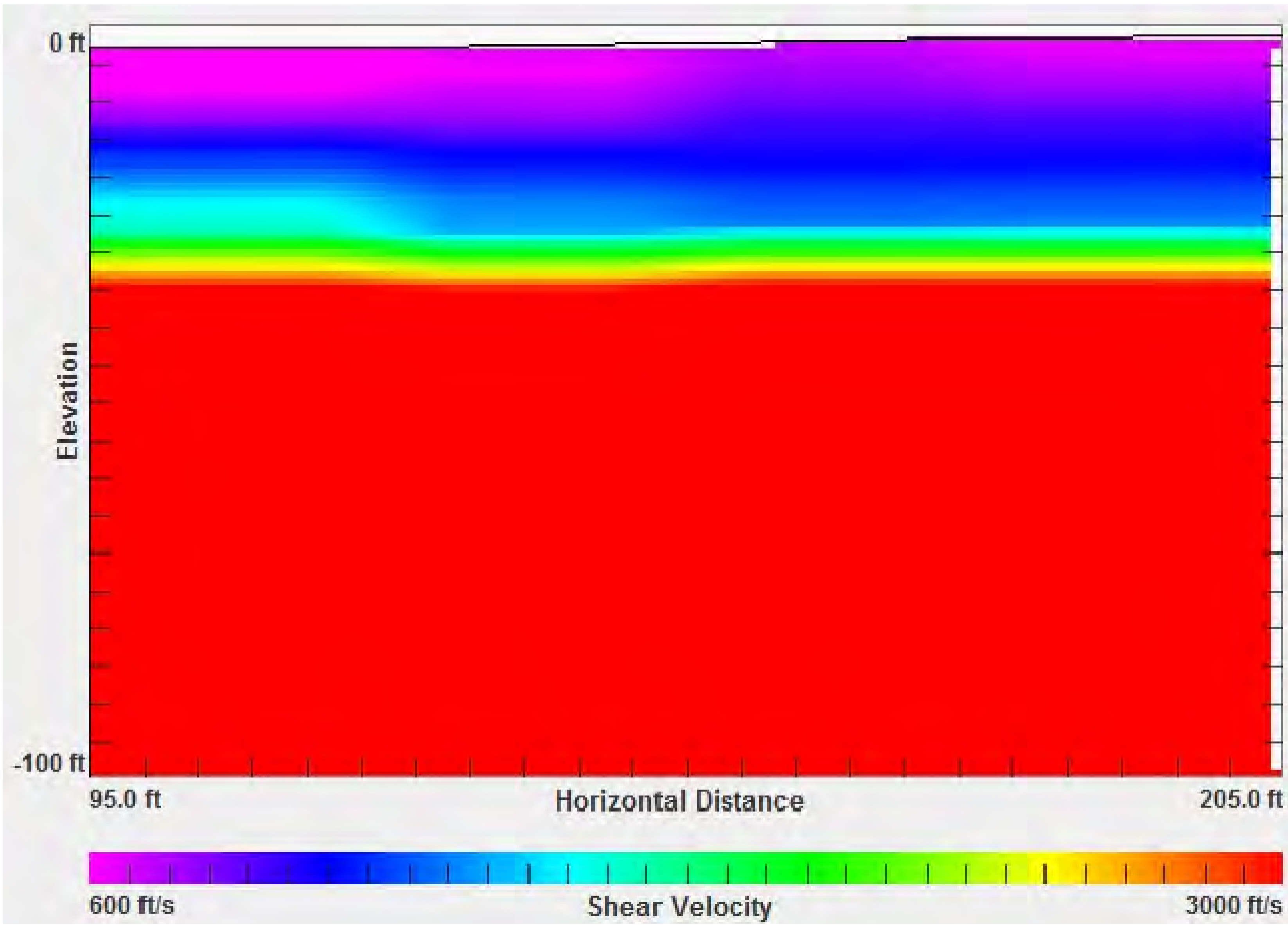
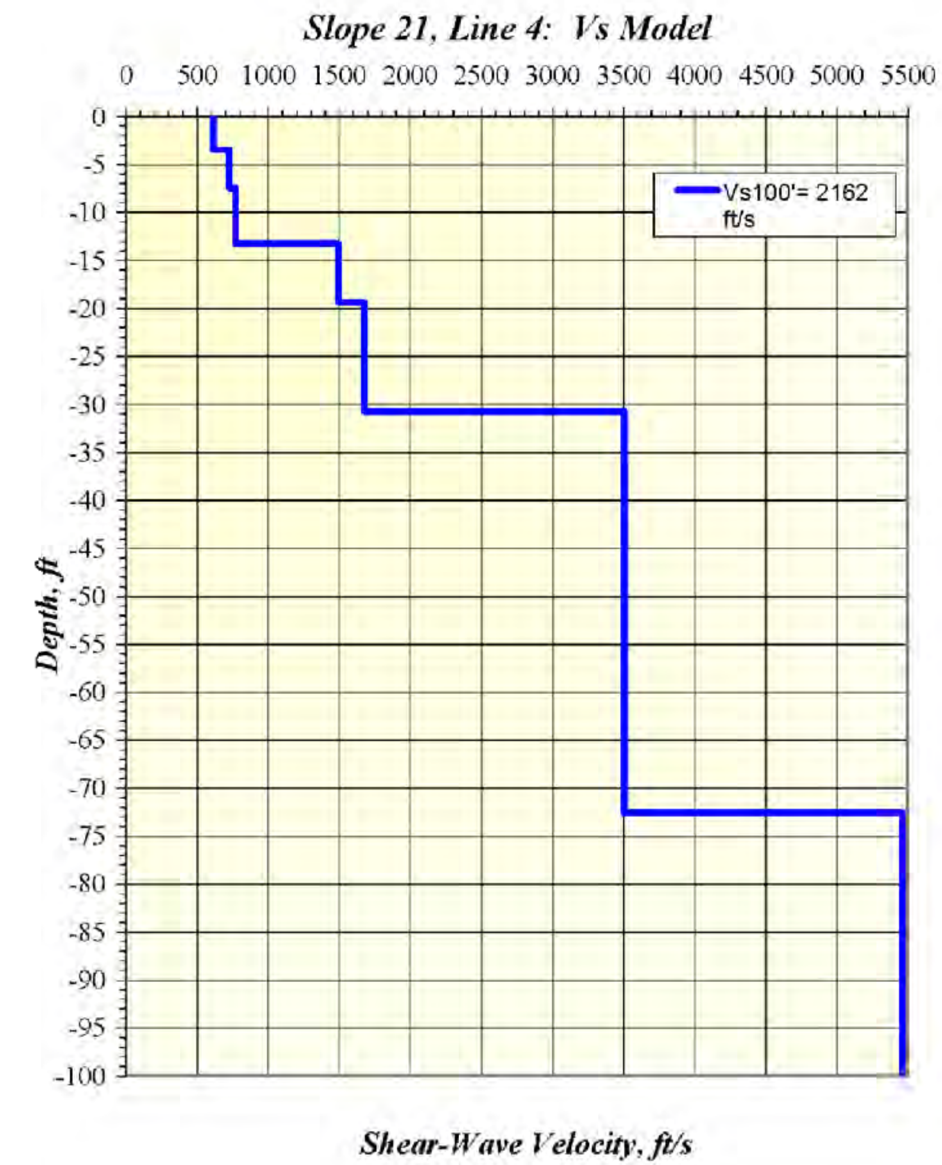


REMI LINE #4-2D

D-D'

SCALE IN FEET
VERTICAL: 1" = 20'-0"
HORIZONTAL: 1" = 13'-4"



REMI LINE #4-1D

20 10 0 20

SCALE IN FEET
1" = 20'-0"

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REMI LINE 4 SLOPE 21

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

PLATE 6

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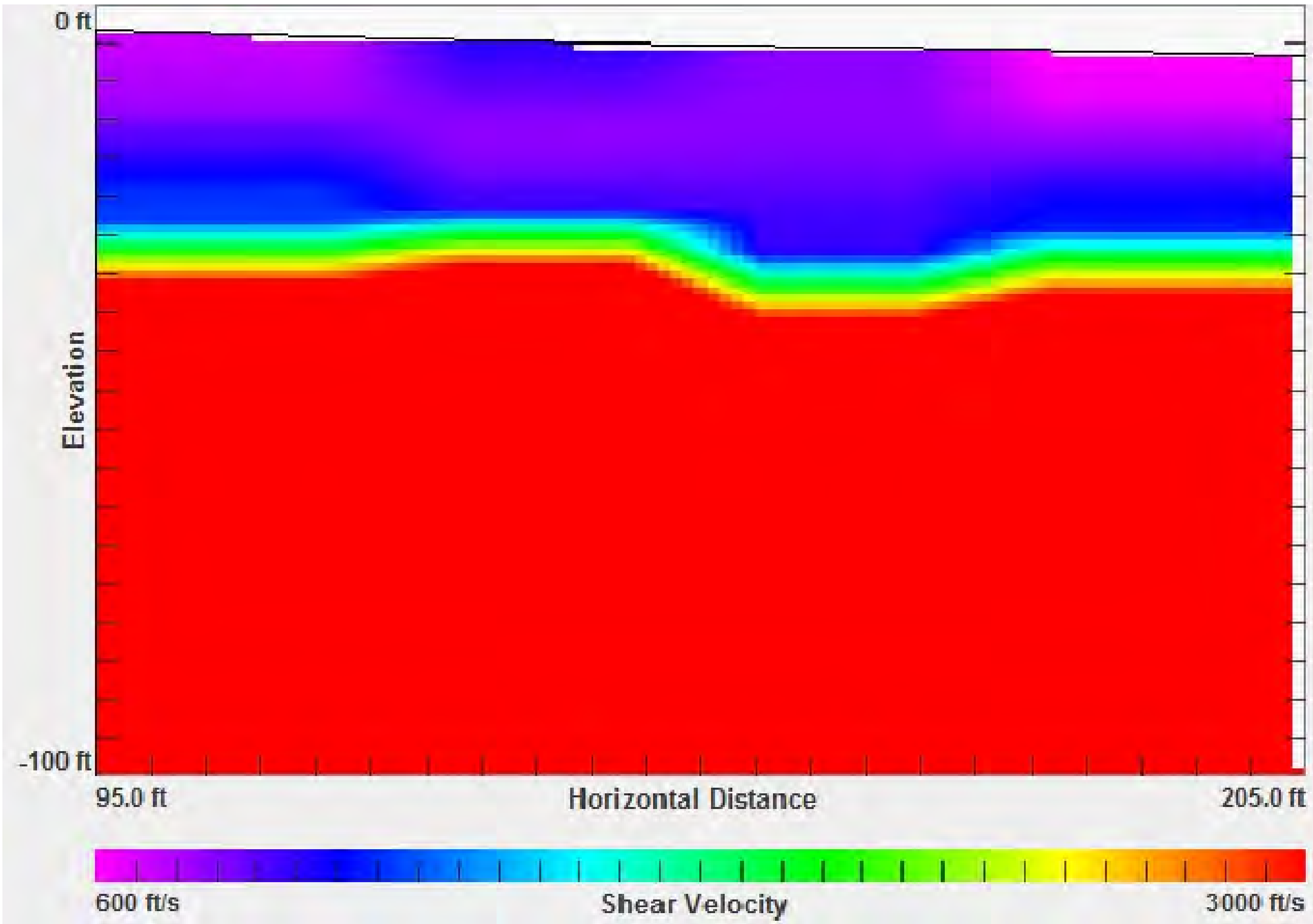
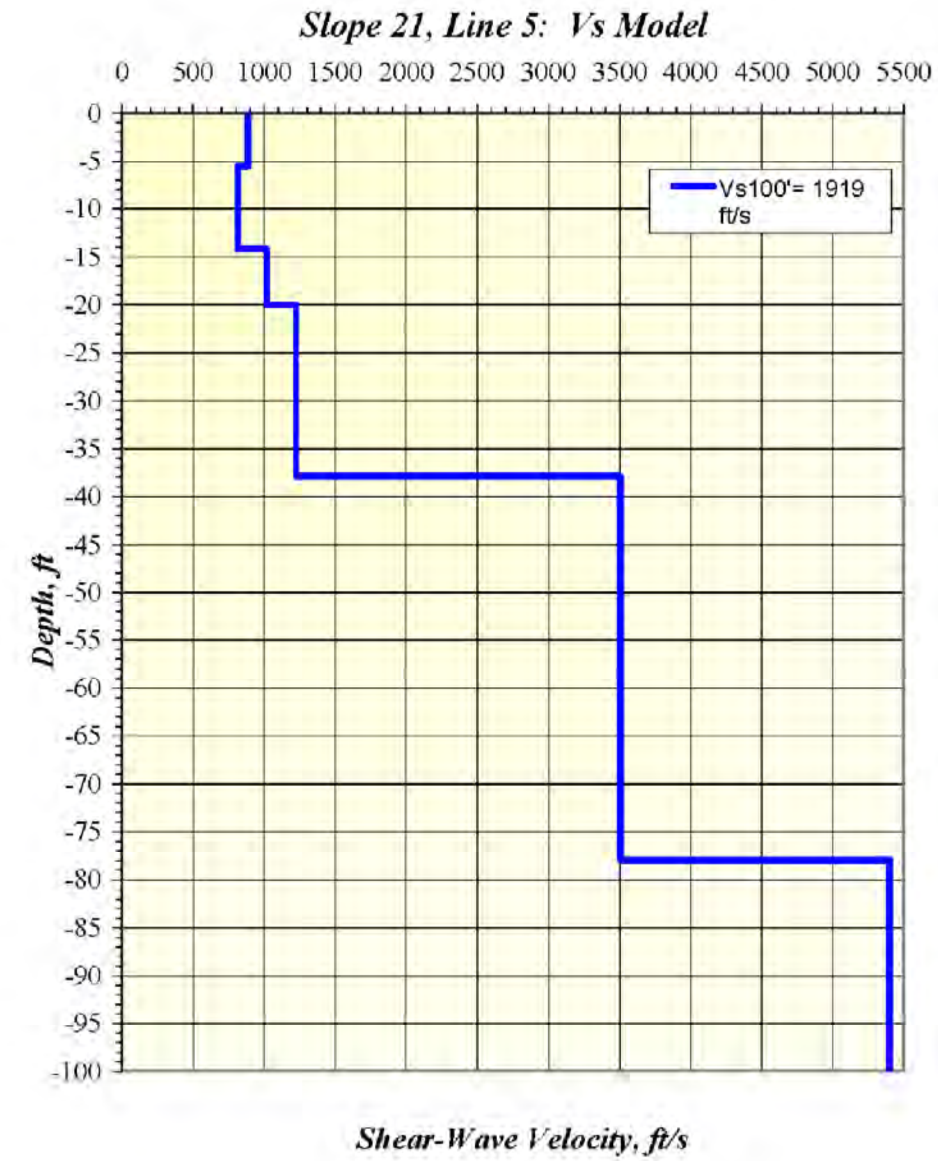


REMI LINE #5-2D
E-E'

SCALE IN FEET
VERTICAL: 1" = 20'-0"
HORIZONTAL: 1" = 13'-4"



REMI LINE #5-1D

20 10 0 20

SCALE IN FEET
1" = 20'-0"

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REMI LINE 5 SLOPE 21

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

PLATE
7

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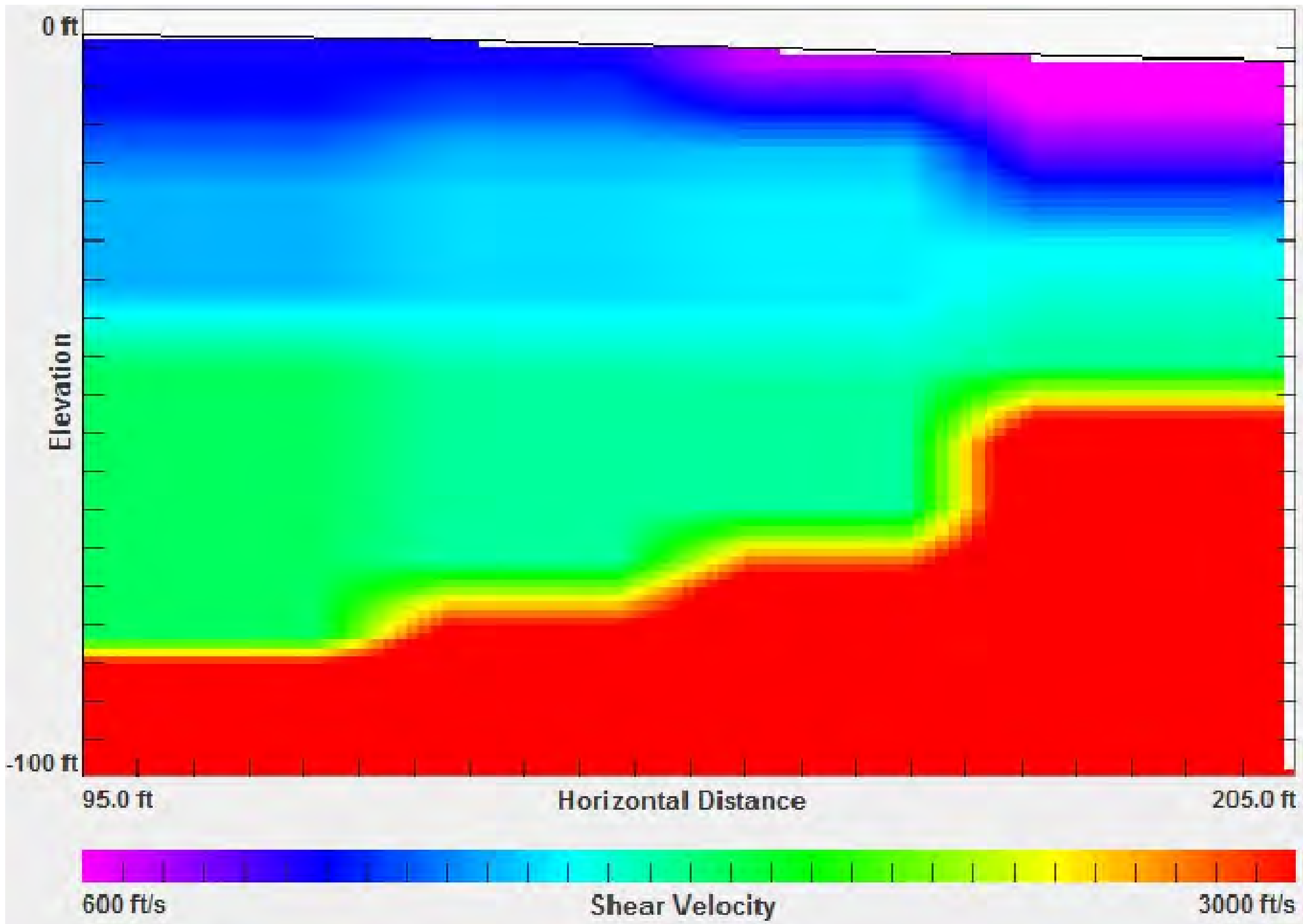
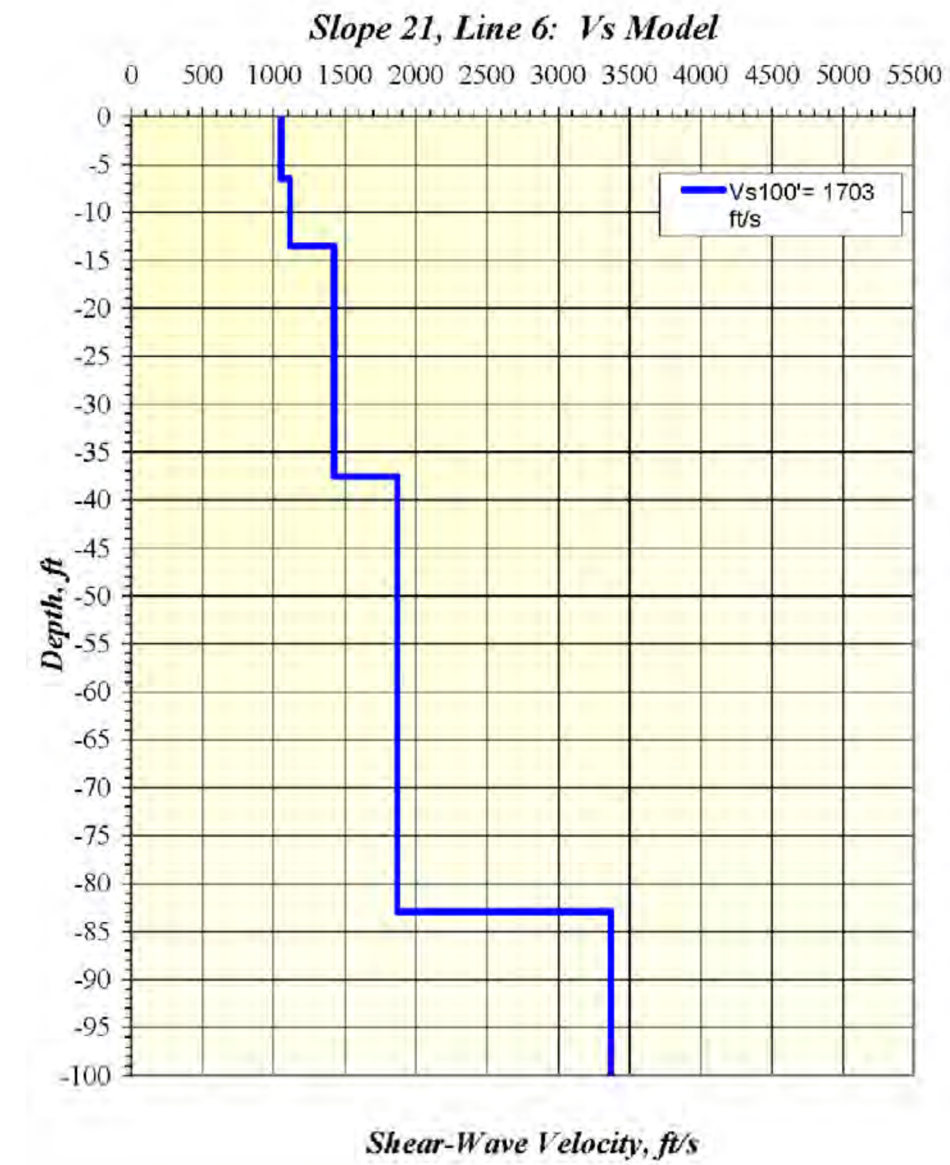


REMI LINE #6-2D

F-F'

SCALE IN FEET
VERTICAL: 1" = 20'-0"
HORIZONTAL: 1" = 13'-4"



REMI LINE #6-1D

20 10 0 20

SCALE IN FEET
1" = 20'-0"

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REMI LINE 6 SLOPE 21

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

PLATE 8

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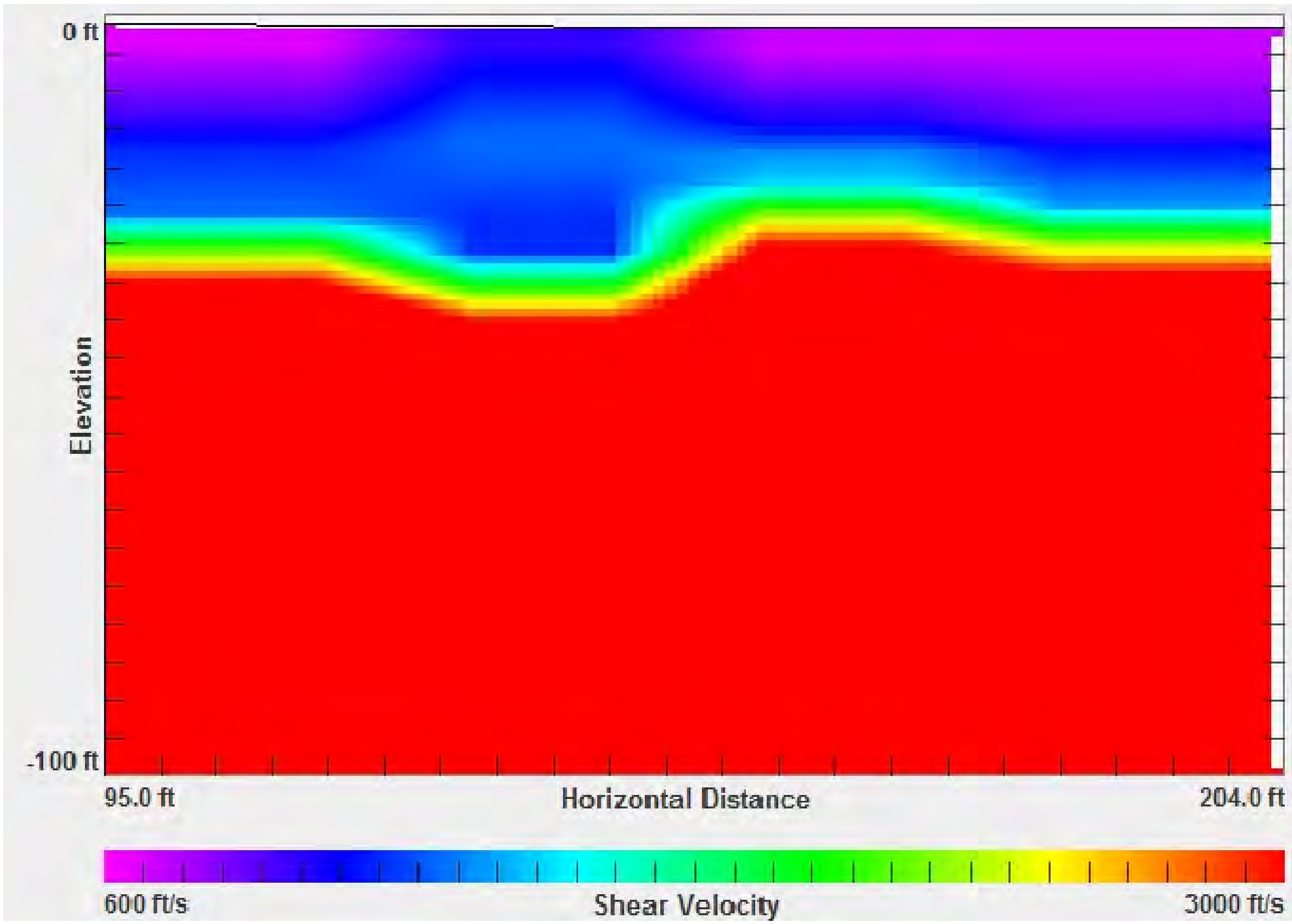
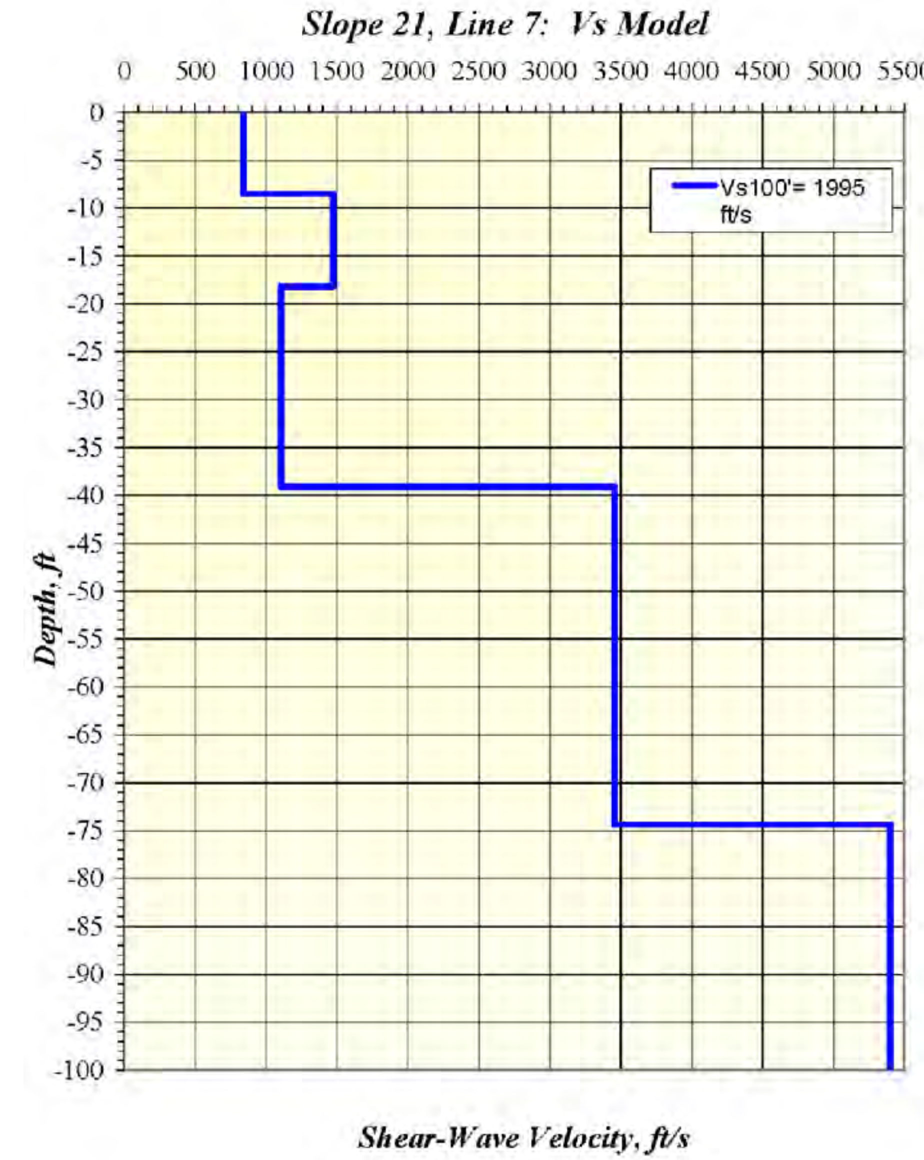


REMI LINE #7-2D
G-G'

SCALE IN FEET
VERTICAL: 1" = 20'-0"
HORIZONTAL: 1" = 13'-4"



REMI LINE #7-1D

20 10 0 20

SCALE IN FEET
1" = 20'-0"

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REMI LINE 7 SLOPE 21

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

PLATE
9

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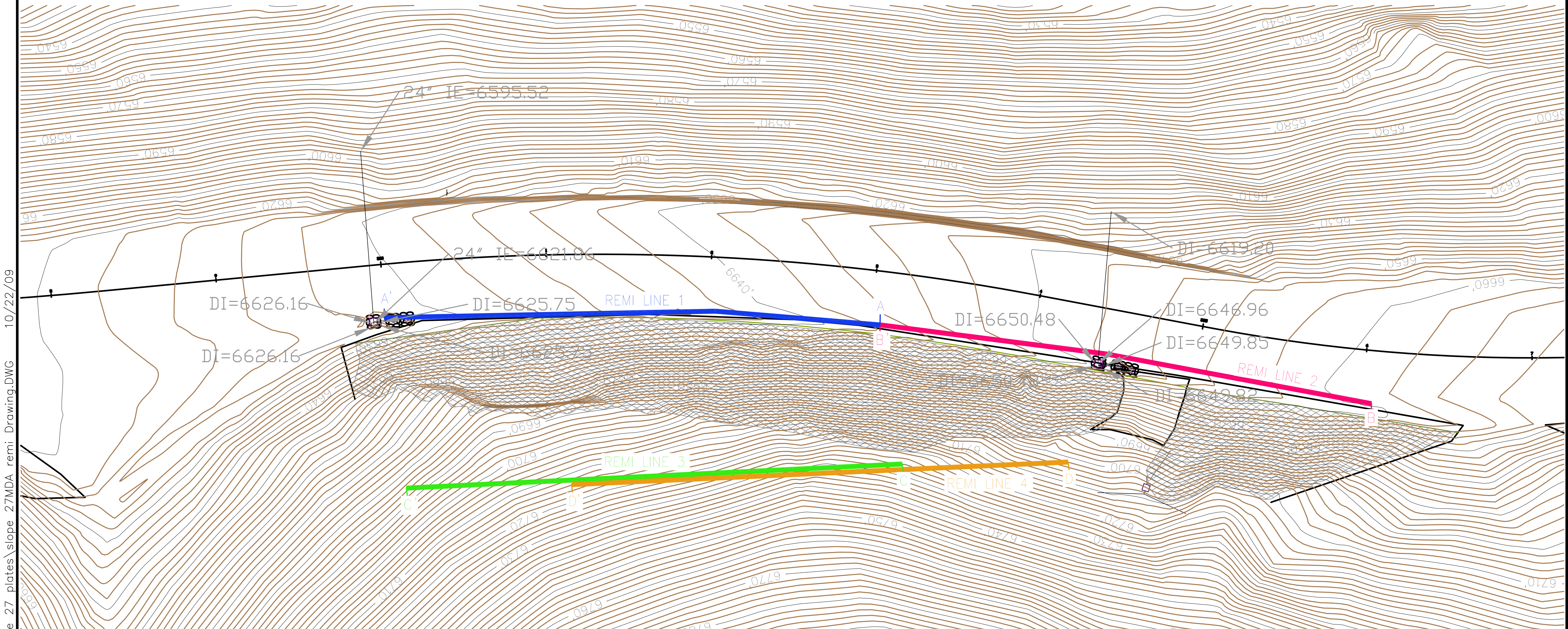

REFERENCE: BASE MAP PROVIDED BY WOOD RODGERS, INC. ELECTRONICALLY ON OCTOBER 8, 2009.

NORTH
SCALE: 1:60

LEGEND

- APPROXIMATE REMI LINE 1 LOCATION
- APPROXIMATE REMI LINE 2 LOCATION
- APPROXIMATE REMI LINE 3 LOCATION
- APPROXIMATE REMI LINE 4 LOCATION

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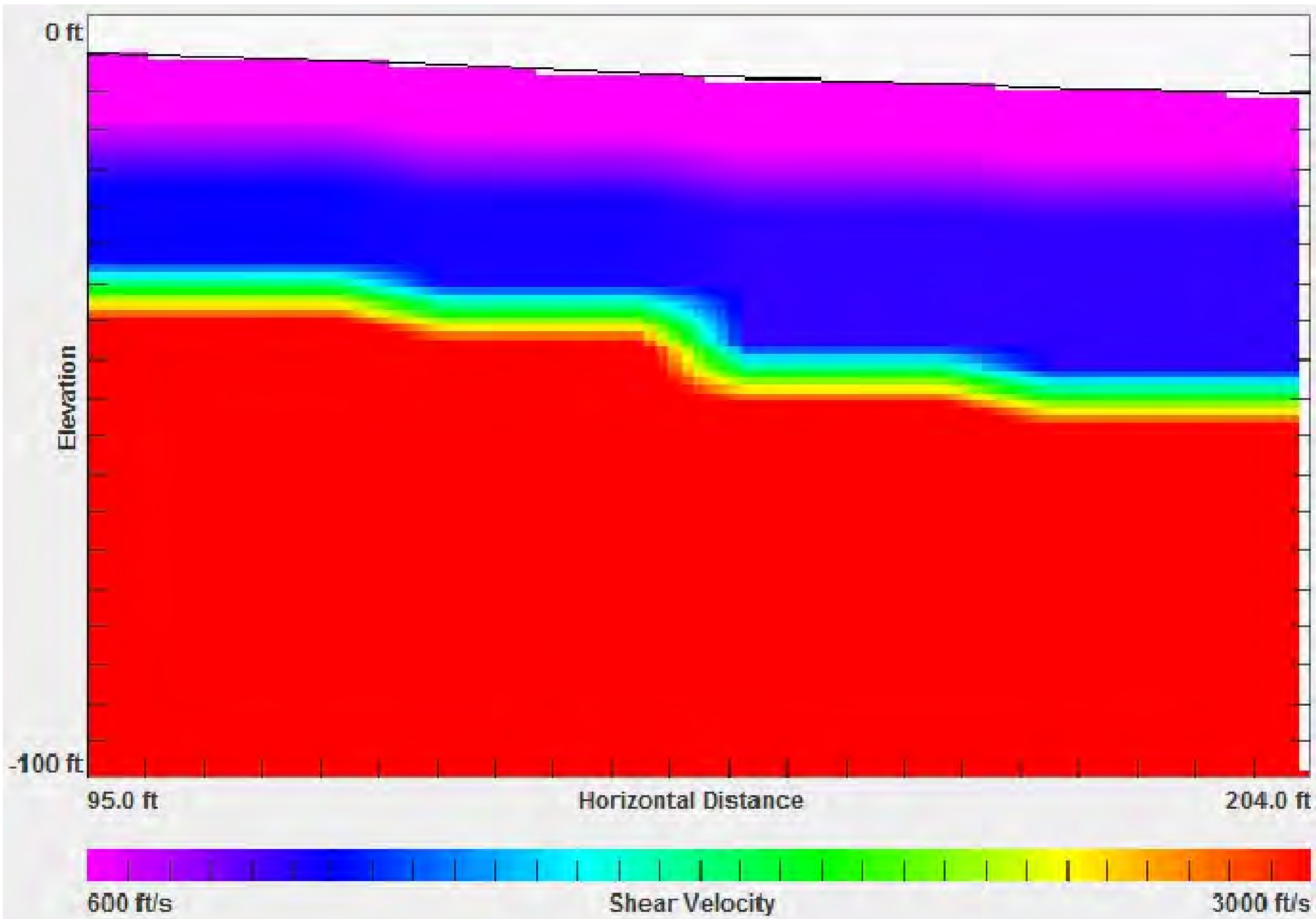
SITE PLAN SLOPE 27

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

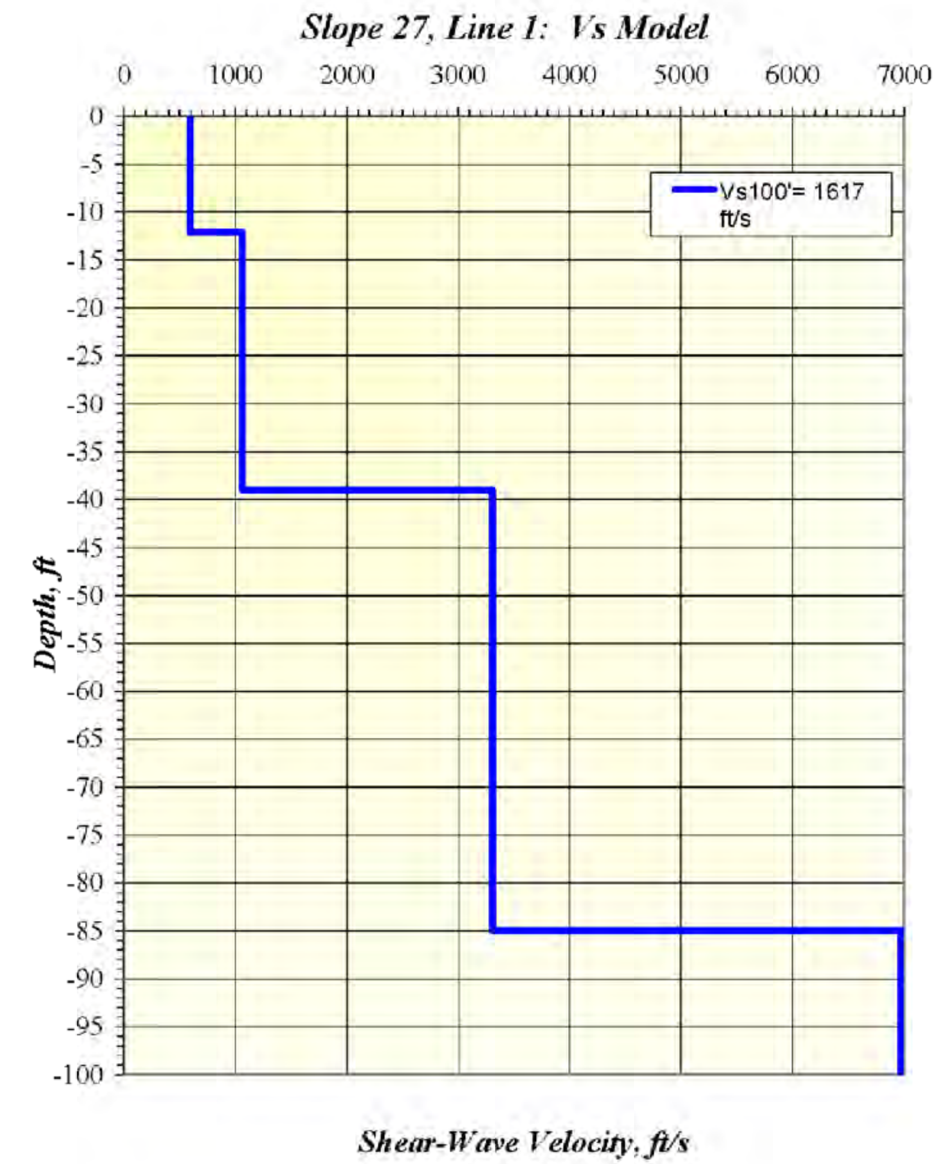
PLATE 10



REMI LINE #1-2D

A-A'

SCALE IN FEET
VERTICAL: 1" = 20'–0"
HORIZONTAL: 1" = 13'–4"



REMI LINE #1-1D

20 10 0 20

SCALE IN FEET
1" = 20'–0"

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REMI LINE 1 SLOPE 27

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

PLATE
11

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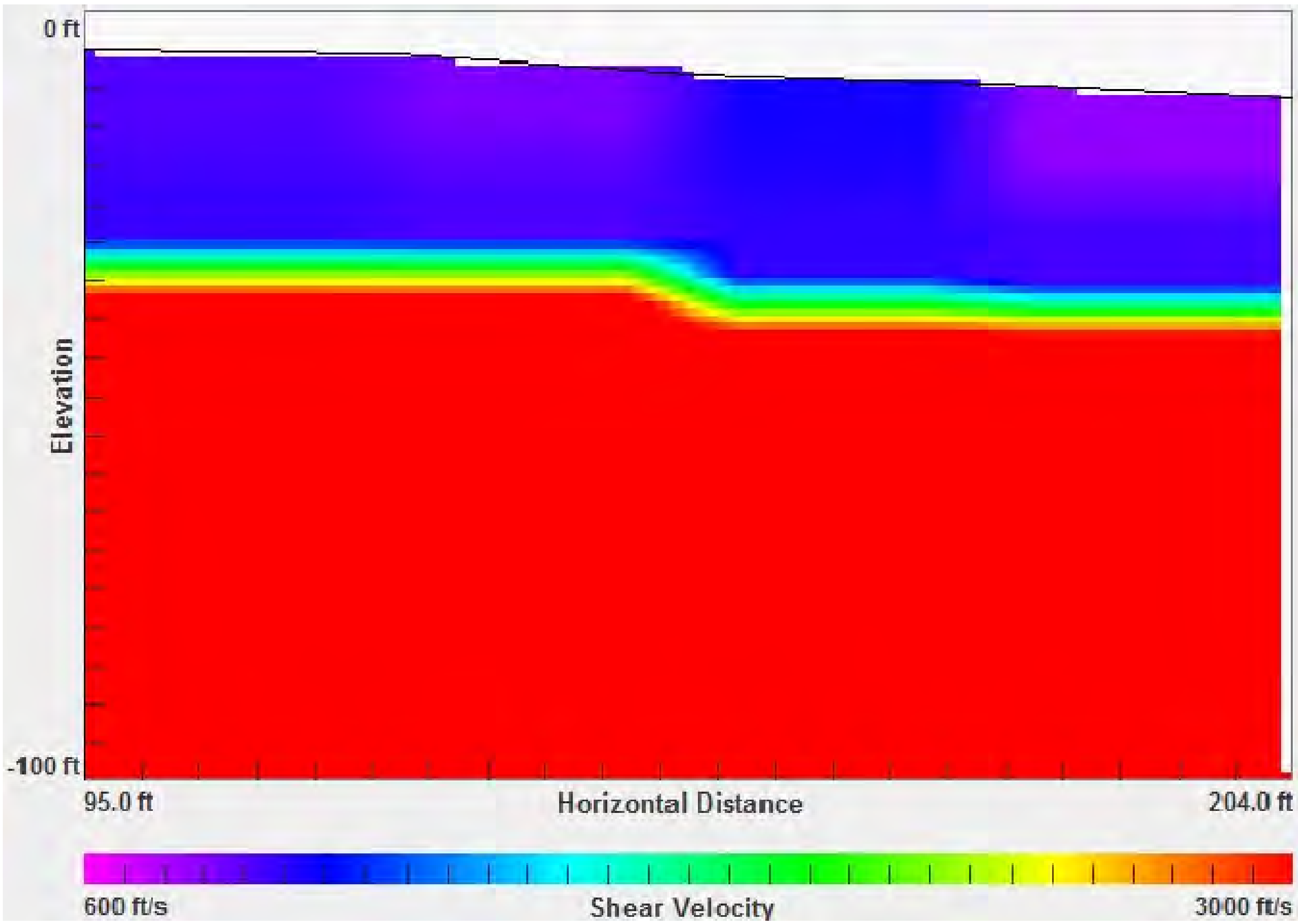
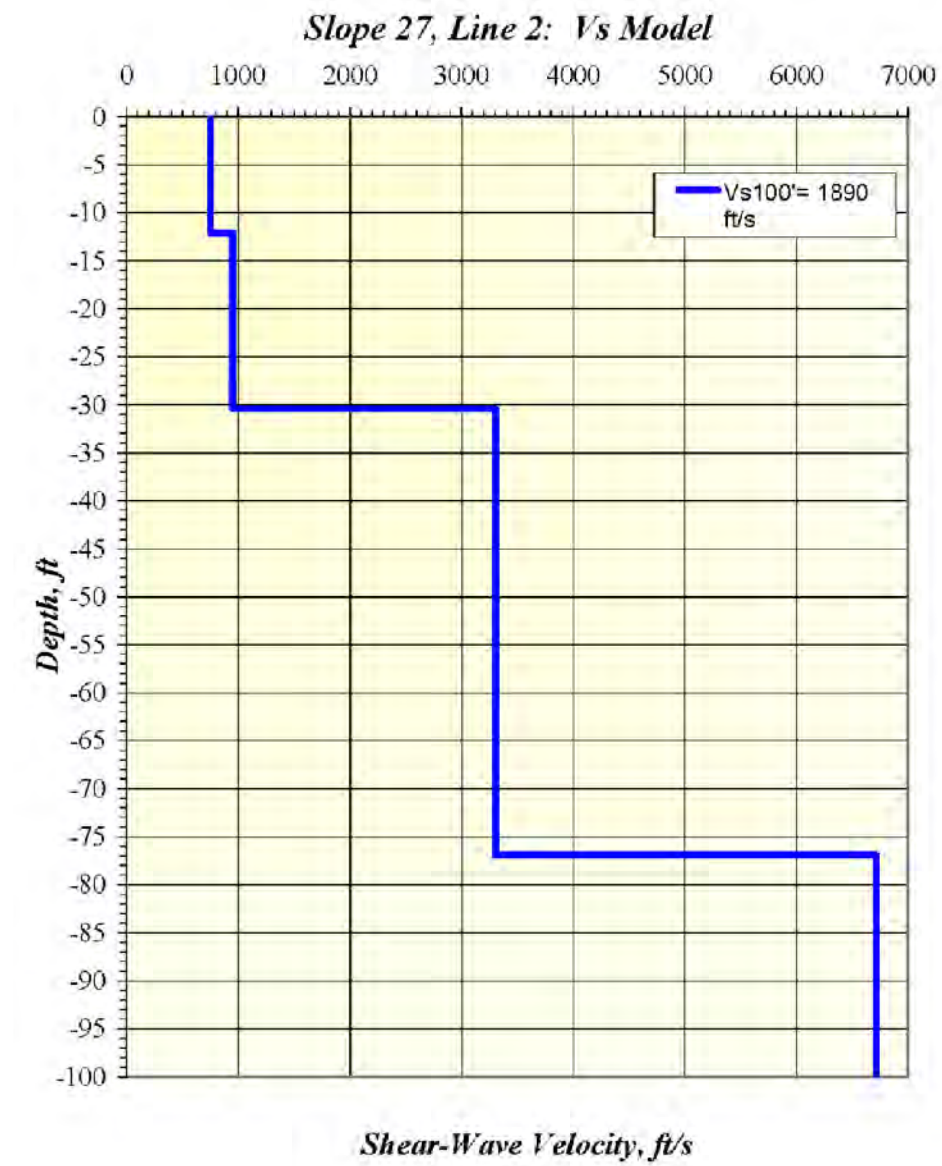


REMI LINE #2-2D

B-B'

SCALE IN FEET
VERTICAL: 1" = 20'–0"
HORIZONTAL: 1" = 13'–4"



REMI LINE #2-1D

SCALE IN FEET
1" = 20'–0"

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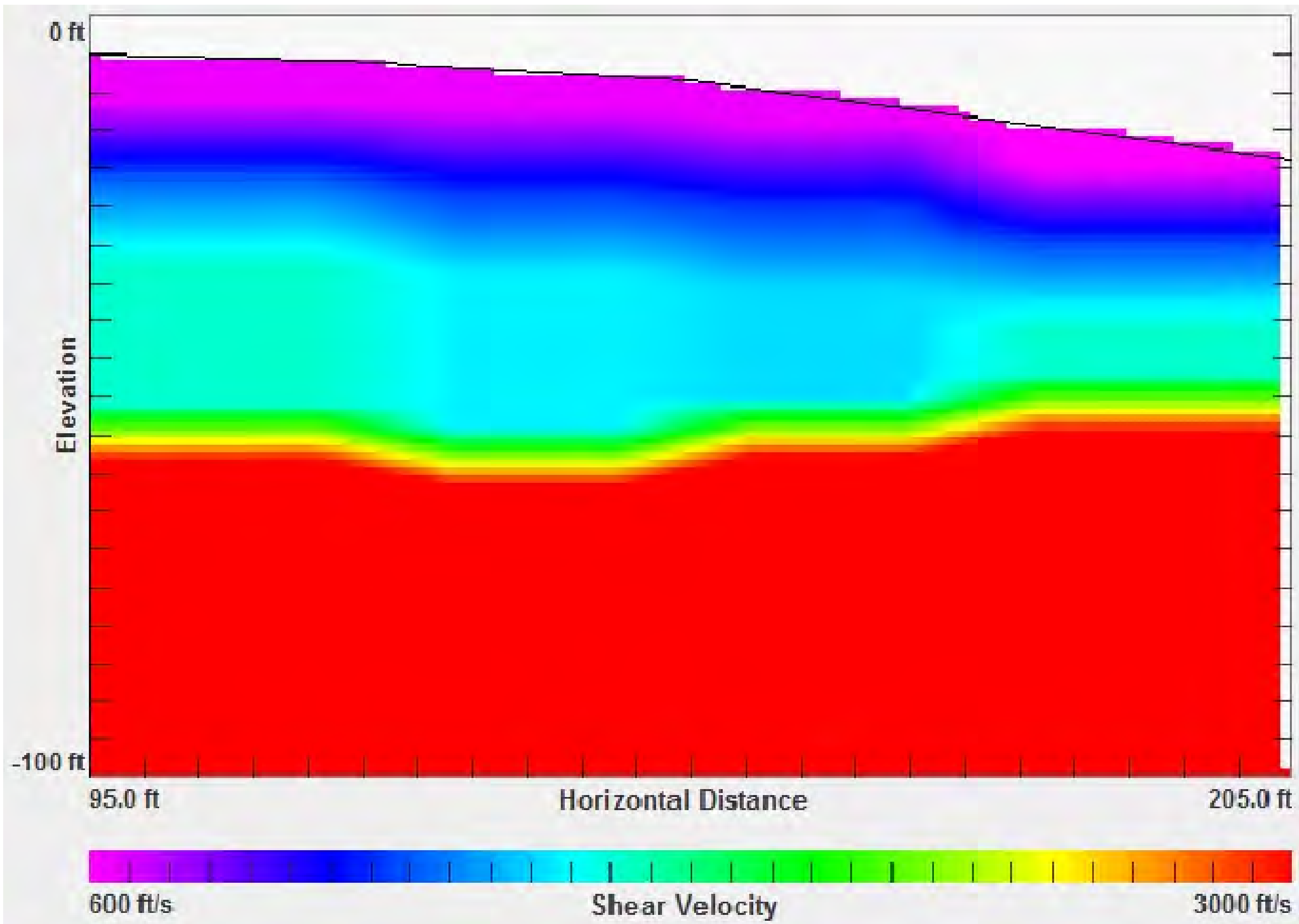
REMI LINE 2 SLOPE 27

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

PLATE 12

X:\PF\2009\9059.001 McMillen LLC\Drawings\slope 27 plates\Remi lines 1–4.dwg 10/22/09



REMI LINE #3-2D
C-C'

SCALE IN FEET
VERTICAL: 1" = 20'-0"
HORIZONTAL: 1" = 13'-4"

REMI LINE #3-1D

20 10 0 20

SCALE IN FEET
1" = 20'-0"

Marvin E. Davis & Associates, Inc.
CONSULTING CIVIL ENGINEERS
P.O. Box 18449
Reno, Nevada 89511
TEL (775)853-9100
FAX (775)853-9199

JOB # 9059.001 APPR:________ DATE: 10/20/09

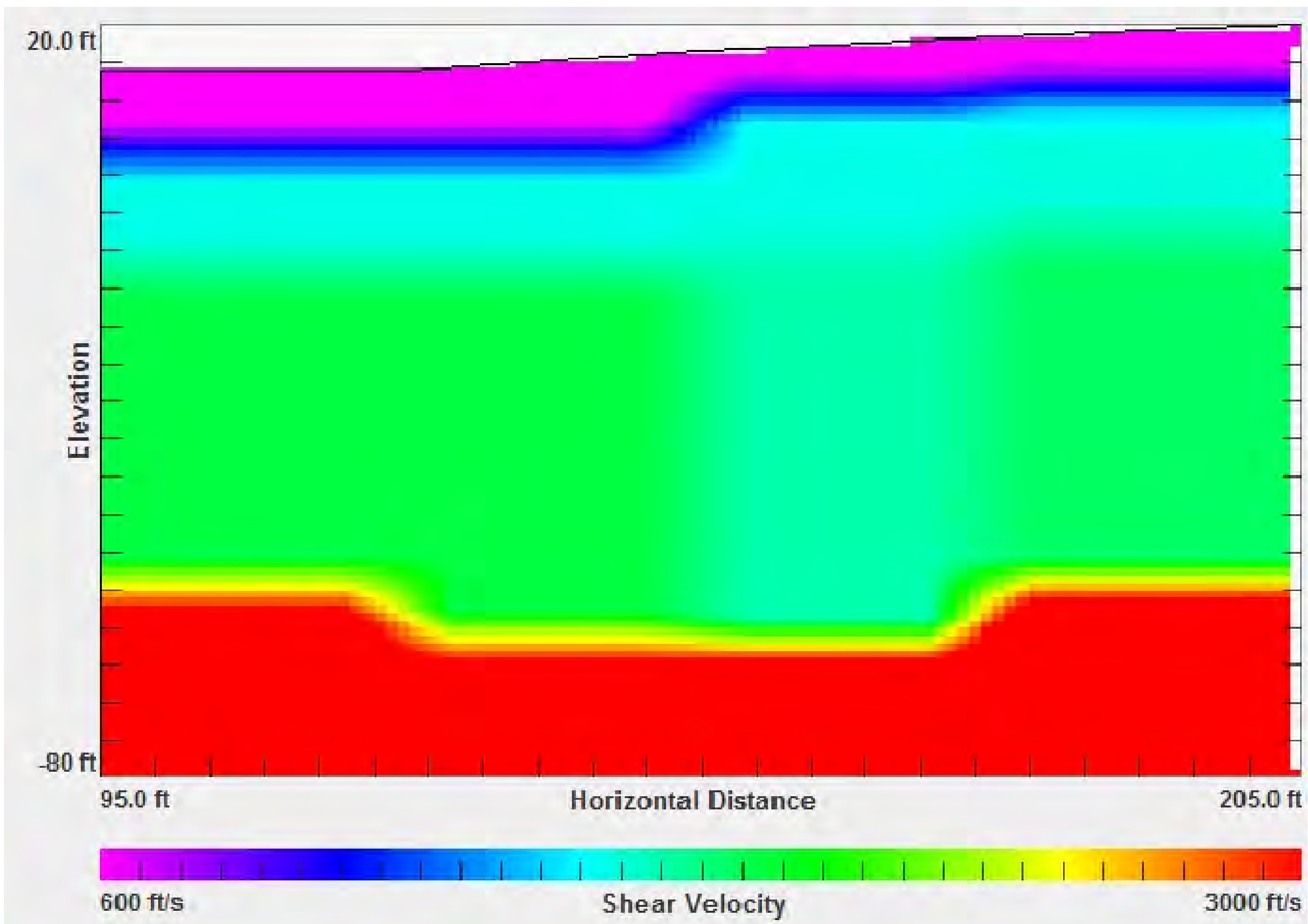
REMI LINE 3 SLOPE 27

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

PLATE
13

X:\PF\2009\9059.001 McMillen LLC\Drawings\slope 27 plates\Remi lines 1-4.dwg 10/22/09



REMI LINE #4-2D

D-D'

SCALE IN FEET
VERTICAL: 1" = 15'–0"
HORIZONTAL: 1" = 17'–3"

REMI LINE #4-1D

20 10 0 20

SCALE IN FEET
1" = 20'–0"

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JOB # 9059.001 APPR:__________ DATE: 10/20/09

REMI LINE 4 SLOPE 27

REMI SURVEY
SLOPE TREATMENT PROJECT
HIGHWAY 50 EAST OF HIGHWAY 395

DOUGLAS COUNTY NEVADA

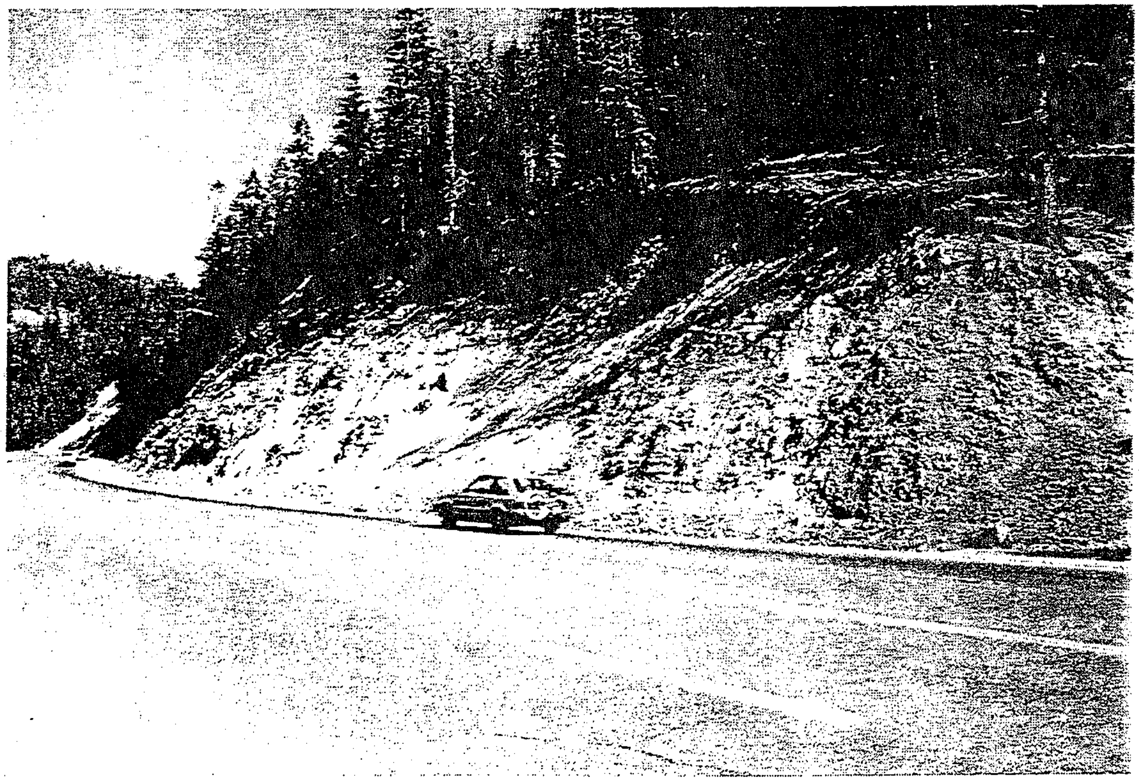
PLATE 14

X:\PF\2009\9059.001 McMillen LLC\Drawings\slope 27 plates\Remi lines 1–4.dwg 10/22/09

APPENDIX C

NDOT ROCKFALL HAZARD RATING FOR SLOPE 27

SLOPE 27



Slope 27 is a 70 ft. high, north facing slope composed of moderately to highly weathered monzogranite and altered trachyte. The ditch is well shaped and 14 ft. wide, and is consistently filled with rock debris. Historical evaluation indicates that large quantities of small sized rockfall debris frequently reach the roadway. In 1984 an automobile suffered minor damage in an accident resulting from rocks on the roadway.

Mitigations: D or J, DM

DATA SUMMARY SHEET
ROCKFALL HAZARD RATING SYSTEM
LAKE TAHOE, NEVADA

SLOPE 27

PRELIMINARY RATING	A
HIGHWAY NAME	US 50
MILE POST	10.82-10.94
NDOT AIR PHOTO NO.	3-16
LOCAL GEOLOGY (Grose, 1985)	MONZOGRANITE AND TRACHYTE
DEGREE OF WEATHERING (From Krank and Watters, 1983)	III to IV

DETAILED RATING SUMMARY

SLOPE HEIGHT SCORE	22
DITCH EFFECTIVENESS SCORE	21
AVERAGE VEHICLE RISK SCORE	81
SITE DISTANCE SCORE	22
ROADWAY WIDTH SCORE	3
GEOLOGIC CHARACTER	
Structural condition score	17
Rock friction score	8
OR	OR
Erosional feature score	N/A
Differential erosion rate score	N/A
BLOCK SIZE SCORE	3
PRESENCE OF WATER SCORE	21
ROCKFALL HISTORY SCORE	66
OVERALL SCORE	**264**

SLOPE 27 ANALYSIS OF POTENTIAL MODES OF FAILURE

SLOPE ORIENTATION

DIP AZIMUTH: 13
DIP : 50

KINEMATIC ANALYSIS OF PLANAR FAILURE ALONG DISCONTINUITES

#	DIP AZIMUTH	DIP	FRICTION ANGLE	POTENTIAL FOR PLANAR FAILURE
1.	161	53	32	NO
2.	284	39	32	NO
3.	7	73	32	NO
4.	77	85	32	NO

AT SLOPE ANGLES STEEPER THAN 73.00, A POTENTIAL FAILURE PLANE WILL DAYLIGHT IN THE FACE OF A SLOPE WITH THE GIVEN ORIENTATION

KINEMATIC ANALYSIS OF DISCONTINUITY INTERSECTIONS

PLANES INVOLVED	PLUNGE	TREND	DAYLIGHTS IN DESIGN SLOPE	DAYLIGHTS IF THE SLOPE IS CUT STEEPER THAN
1 & 3	22.91	89.57	NO	90.00
1 & 4	52.99	160.33	NO	90.00
2 & 1	25.44	229.99	NO	90.00
3 & 2	38.77	291.22	NO	79.91
4 & 2	19.06	348.74	NO	90.00
4 & 3	72.97	3.59	NO	73.19

AT SLOPE ANGLES STEEPER THAN 73.19, A POTENTIAL WEDGE FAILURE WILL DAYLIGHT IN THE FACE OF A SLOPE WITH THE GIVEN ORIENTATION

ANALYSIS OF POTENTIAL TOPPLING FAILURE

BASIC CRITERIA INDICATES THAT POTENTIAL FOR TOPPLING IS LOW

DISCONTINUITY DATA

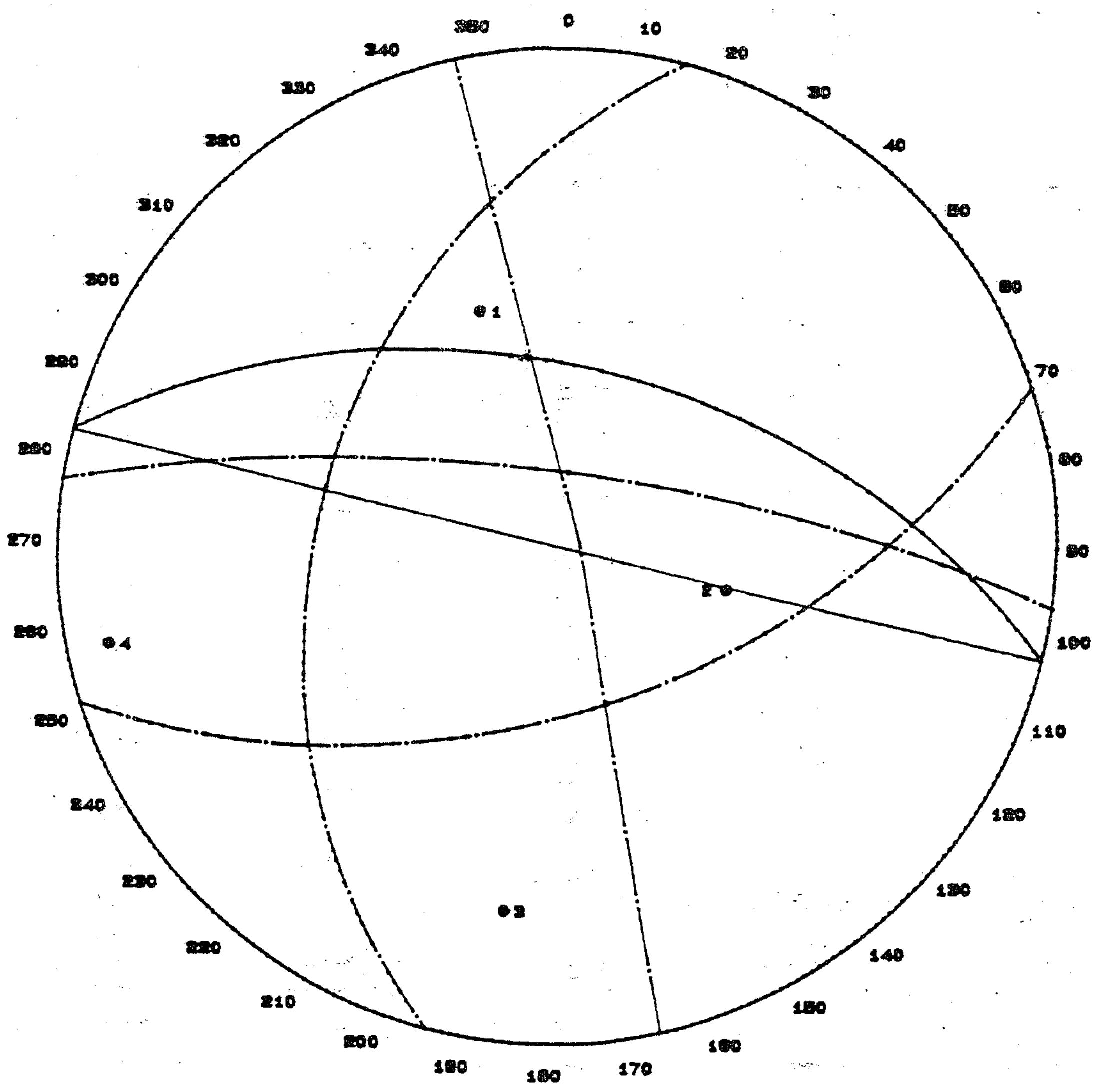
	AZIMUTH	DIP
1.	161.00	53.00
2.	284.00	36.00
3.	7.00	73.00
4.	77.00	85.00

LEGEND

SLOPE FACE ————————
DISCONTINUITY ——————·——

DATA

DIP OF SLOPE : 50
DIP AZIMUTH : 13

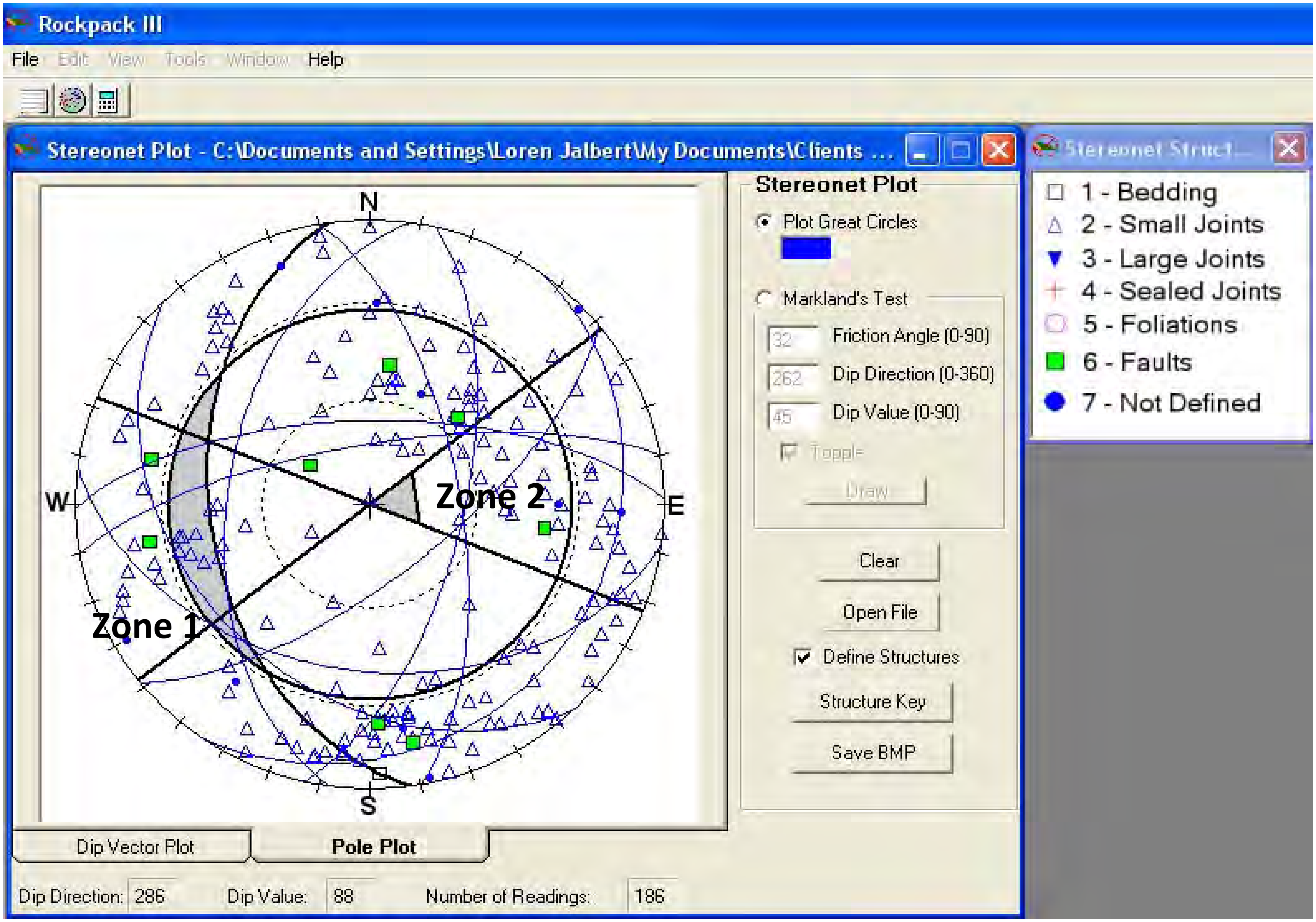


SLOPE 27
PROJECTION OF PLANES REPRESENTING MAJOR JOINT ORIENTATIONS

APPENDIX D

ROCKPACK^{TM} III STEREO NET RESULTS FAULTS

SLOPE 27

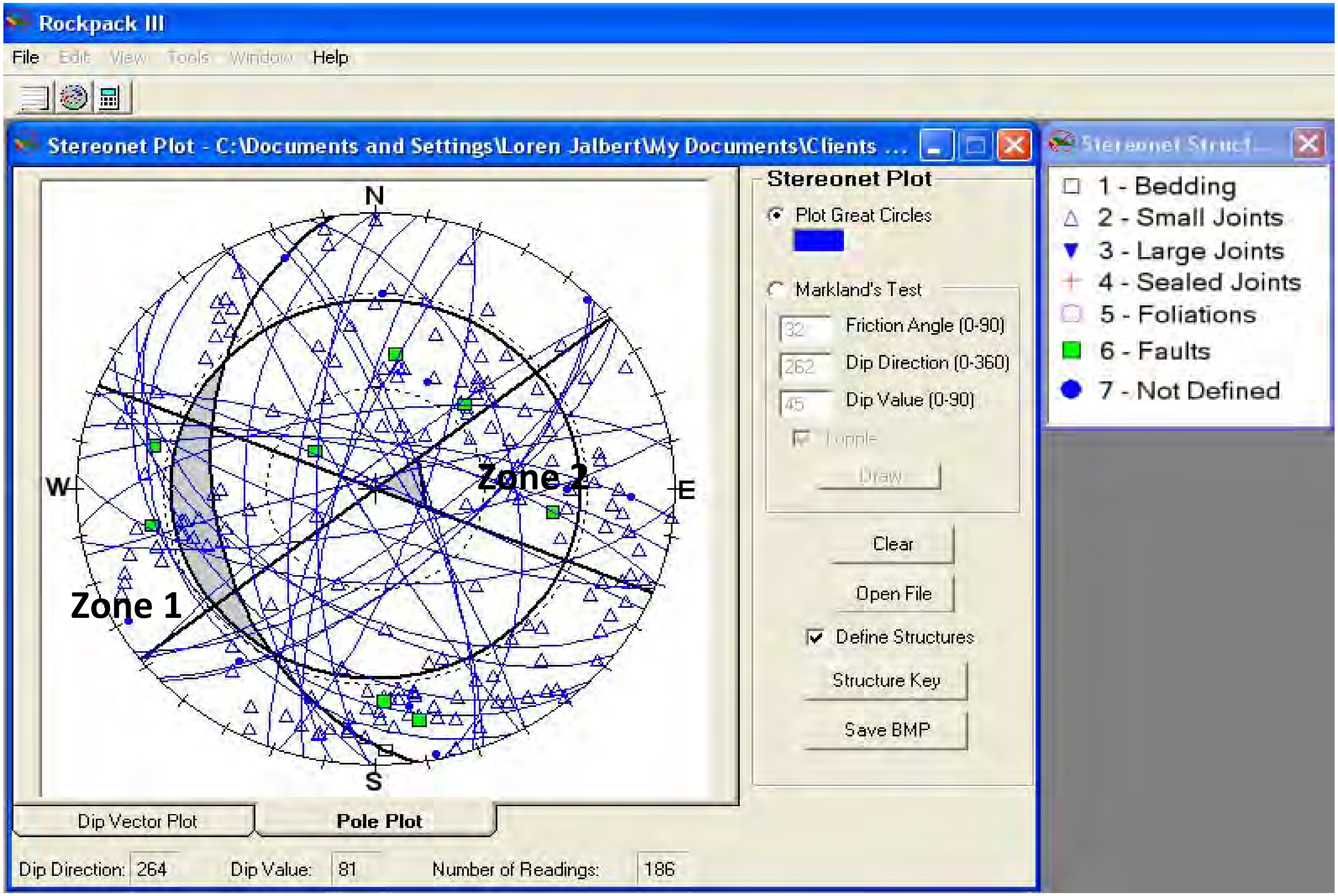


Slope 27: Major Fault Intersections (Green Squares)

Fault intersections in Zone 1 represent wedge and plane failures.

Fault Intersections in Zone 2 represent toppling failures.

SLOPE 27



Slope 27: Small Joint Intersections (Blue Triangles)

Small Joint intersections in Zone 1 represent wedge and plane failures.
Small Joint Intersections in Zone 2 represent toppling failures.

APPENDIX E

SLOPE 27 STRIKE AND DIP FIELD NOTES

Project: Glenbrook	Date 10/6/09 10/7/09	By: LA	Page: 1 of 2
Scanline Orientation: 268		Rock Type:	
Rockface Orientation: 274/40			
Northing 39°05.101	Westing 119°55.286	Lat:	Long:

Slope#: 27	Cell	Elev. 6630	Termination	Strike/Dip	Height	Width	Comments: Anomolies/water	
	Dist. (ft)	Length (ft)		Orientation	Curvature	Roughness		
				/				
1	Face	0		274 / 40	1	2	At Drop Inlet	Drill Rig at
2		16		180 / 87	1	3	30 Feet above scan line	
3		16		290 / 80	1	3	Small outcrop / soil around	the DropINLET
4		16	4X	040 / 45	1	2	4 - parallel ~3" Apart	
5		16		230 / 42				
6		25		140 / 64	1		30 Feet above scan line	
7		47		240 / 70				
8		47		320 / 60				
9		47		020 / 76			(Small sampling)	
10		46		220 / 21				
11		48		320 / 60				
12		48		020 / 030				
13		48		064 / 18				
14	6658	155		260 / 50			30 Feet above scan line	
15		155		164 / 40				
16		155		222 / 42				
17		155		210 / 18				
18		155		184 / 02				
19		155		326 / 80				
20		160		292 / 78				
21		160		090 / 43				
22		165		144 / 70			40 Feet above scan line	
23		165		080 / 50				
24		165		350 / 65				
25		165		230 / 70				
26		165		080 / 35				
27		165		264 / 35				
28		165		330 / 78				
29		165		226 / 35			Top of slope ~15-20 Feet	
30		165		298 / 70			(& broken rock / soil)	
31		165		240 / 48				
32		178		314 / 60				
33		178		200 / 50				
34		178		194 / 60				
35		178		274 / 68				
36		178		280 / 75				
37		178		170 / 85				
38		178		250 / 65				
39		185		160 / 46			RockFace 266 / 48	Drill Rig at
40		196		280 / 25				180 Feet
41				268 / 62				
42				242 / 84				
43				210 / 54				
44				320 / 84				
45				074 / 65				
46				088 / 45				
47				222 / 55				
48				142 / 75				
49				318 / 72				
50				248 / 40				
				/				

Dist: distance along the scan line
Length: Length measured above the scan line
Termination: A - At another Discontinuity I - Into Rock O - Obscured
Dip Direction: Right hand rule measured clockwise from true north
Curvature: Waviness on numerical scale 1 - straight to 5 very irregular (blocky)
Roughness: Friction on numerical scale 1 - very smooth to 5 very rough
Comments: water, infilling, vegatation, shear zone, fault rubble, etc.

Project: Glenbrook | Date: 10/7/09 | By: LAD | Page: 2 of 2

Scanline Orientation: | Rock Type:

Rockface Orientation: 286/45 | East side of slope 27

Northing: | Westing: | Lat: | Long:

Slope#: 27	Cell Dist. (ft)	Elev. 6652 Length (ft)	Termination	Strike/Dip Orientation	Height Curvature	Width Roughness	Comments: Anomolies/water
				/			
1	196			070/50			
2				036/70			
3				220/44			
4	204	6651		300/80			
5				216/40			
6				358/64			
7				320/88			
8				280/75			
9				236/55			
10				344/65			
11				080/54			
12				284/80			
13				116/70			
14				270/70			
15				070/78			
16	238			354/62			Major Joint set
17				222/35			
18				262/65			
19				080/55			
20				340/55			
21				184/36			
22				086/82			
23				332/76			
24				150/80			
25				260/40			
26				110/76			
27				240/30			
28				028/78			
29				342/76			Drill Rig @ 287 (Adjacent to competent
30	Add 2006			/			Rock Near East end of slope
31	Rock Data			/			- collected good rock samples
32	For this section			/			at this Drill location For Testing
33				/			
34	377			060/85			Rock at spring changes
35				080/65			in Texture / Type
36				182/60			Slope 27A
37				352/68			
38				272/74			
39				036/66			
40				348/88			
41				204/35			Drill Rig at 450 (center of 27A)
42				270/54			
43				160/78			
44				226/88			
45				/			
46				358/66			Major Fractures
47				224/35			Extending 20+ Feet in length
48				188/41			
49				278/50			
50				080/65			
				350/74			

358/84
124/20
102/65

Dist: distance along the scan line
Length: Length measured above the scan line
Termination: A - At another Discontinuity I - Into Rock O - Obscured
Dip Direction: Right hand rule measured clockwise from true north
Curvature: Waviness on numerical scale 1 - straight to 5 very irregular (blocky)
Roughness: Friction on numerical scale 1 - very smooth to 5 very rough
Comments: water, infilling, vegatation, shear zone, fault rubble, etc.

Schmidt Hammer
3800 PSI
5500 PSI
8500 PSI
8600 PSI
1500 PSI
1800 PSI

3000 → 7000 psi
10,000

Project:			Date:		By:			Page: 1 of 3	
	Cell	Elev.	Strike	Dip	Height	Width	Rock Type	Northing	Easting
	BEGIN	6652	262	51				39°05.0'	119°55.2'
	Dist. (ft)	Length (ft)	Termination	Orientation	Curvature	Roughness	Comments:	Photo	
1	0-100'			/			90%+ Soil Slope some small		
2				/	RK outcrops just @ slope surface, NEAR vert.				
3				/	AT TOP, Numerous TREES Need to be Remove				
4				/	RK is intensley - closely FRAC, Deeply weathered/Altered				
5				/	RK appears to be soil matrix supported AND AS				
6				/	soil erodes RK falls to ditch				
7				/	RK fall 2" → 30" - source seems to be vert				
8				/	section at top of slope				
9				/					
10	100-155			/	SAME AS ABOVE - @ 155 RK out				
11				/	CROP BEGINS @ MID slope / soil talus Below				
12				/					
13	214		I	190 / 70	2	3			
14	210		I	130 / 63	2	4			
15	223		I	294 / 72	2	4			
16	224		I	136 / 60					
17			I	316 / 40			GOES under scan & // to slope		
18	228			186 / 19			GOES over scan line @ 263		
19				180 / 57					
20				224 / 45			GOES under scan Line @	266	
21	268			172 / 50					
22				270 / 55			semi // to scan Line @	283	
23	279		I	330 / 67	2	3			
24	276			184 / 62					
25	211			290 / 85					
26				224 / 40			GOES over scan line @ 290		
27	265		I	322 / 82					
28	283		I	350 / 63	2	3			
29				/					
30	289-297			/			DEBRIS SHOOT		
31	308			200 / 78					
32	275			334 / 66					
33				/					
34	300'	6560	Scan Line	286 / 54				39°05.0'	119°55.2'
35	300			006 / 72					
36	285			192 / 37			Numerous Joints like this @ 6"-12" spacing		
37	298			338 / 82					
38				296 / 82			semi // to scan Line @	303	
39				149 / 74					
40	354			70 / 65					
41	313			350 / 72					
42	295			190 / 39					
43				240 / 37			GOES over scan Line @	322	
44				224 / 35			" " "	"	
45	323.5			344 / 87					
46				260 / 65			// to scan Line @	329	
47	372			68 / 79					
48	334.5			354 / 65	2	3			
49				256 / 32			GOES under scan Line @	338	
50				276 / 72			// scan Line @	342	

Dist: distance along the scan line
Length: length measured above the scan line
Termination: A - at another discontinuity, I - into rock O - Obscured
Dip direction: right hand rule measured clockwise from true north
Curvature: waviness on numerical scale 1 - straight to 5 very irriguaiar(blocky)
Roughness: Friction on numerical scale 1 - very smooth to 5 very rough
Comments: water, infilling, vegatation, shear zone, fault, rubble, etc.

Slope 5
Pg 2

Project:			Date:		By:			Page: 2 of 3	
	Cell	Elev.	Strike	Dip	Height	Width	Rock Type	Northing	Easting
	Dist. (ft)	Length (ft)	Termination	Orientation	Curvature	Roughness	Comments:	Photo	
1	310			310 / 86					
2	345.5		I	356 / 65	2	1	3 // @ 1' spacing		
3	350		A	359 / 69	1	1			
4	337		I	206 / 37	1	2			
5	355		A	002 / 62	1	2			
6	362		A	16 / 78	1	2			
7	363		I	12 / 79	1	2			
8	366		I	12 / 80	1	2			
9	369		I	10 / 77	1	2			
10			I	272 / 38			GOES UNDER SCANLINE @		372
11	371			352 / 71					
12				190 / 34			GOES UNDER SL @		379
13				74 / 53			GOES BACK into SLOPE		
14				280 / 67			// to SL @ 385		
15	387.5			26 / 86	3	3			
16	372			216 / 50			Numerous Joints like this @ 6"–24" spacing		
17				74 / 79					
18				70 / 45			GOES UNDER SL @ 388		
19	379			190 / 37					
20	387.5			350 / 64					
21	399			10 / 55					
22				76 / 56					
23				276 / 54			// to scanline @	400	
24	396			340 / 69					
25	400			002 / 68					
26	400	ELEV	6664	/				39°5.1'	119°55.2'
27				94 / 66			GOES // into SLOPE	@ 400	
28	403			006 / 76					
29	392			312 / 64					
30	417			210 / 64	2	2			
31	406			310 / 86	1	1			
32	395			309 / 80	1	1			
33				276 / 89	1	1	// to scan line @	411	
34	400			180 / 32	1	1			
35	417			359 / 72	2	3			
36	419			006 / 77					
37	426			16 / 67					
38				228 / 34			BELOW scan line @	439	
39	436			002 / 79					
40	463			80 / 70					
41	415			106 / 77					
42				250 / 52			BELOW scanline @	439	
43	412			130 / 37			GOES INTO HILL		
44	450.5		I	40 / 63	2	3			
45	449		I	356 / 75					
46				202 / 20			ABOVE scan line	@ 445	
47	440			170 / 79	2	3			
48				266 / 50			// to scanline @	451	
49				74 / 43			// to scan line going into slope @ 466		
50	444			154 / 30					

Dist: distance along the scan line
Length: length measured above the scan line
Termination: A - at another discontinuity, I - into rock O - Obscured
Dip direction: right hand rule measured clockwise from true north
Curvature: waviness on numerical scale 1 - straight to 5 very irrigualar(blocky)
Roughness: Friction on numerical scale 1 - very smooth to 5 very rough
Comments: water, infilling, vegatation, shear zone, fault, rubble, etc.

Slope 5
pg 3

Project:			Date:		By:			Page: 3 of 3	
	Cell	Elev.	Strike	Dip	Height	Width	Rock Type	Northing	Easting
	Dist. (ft)	Length (ft)	Termination	Orientation	Curvature	Roughness	Comments:	Photo	
1	461.5			346/74					
2	474			346/70					
3	497			356/42					
4				/					
5	500-600			/	Soil Slope, veg from 545-600,				
6				/	Active spring/seep @ top of slope @				556
7				/					
8				/					
9				/					
10				/					
11				/					
12				/					
13				/					
14				/					
15				/					
16				/					
17				/					
18				/					
19				/					
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36				/					
37				/					
38				/					
39				/					
40				/					
41				/					
42				/					
43				/					
44				/					
45				/					
46				/					
47				/					
48				/					
49				/					
50				/					

Dist: distance along the scan line

Length: length measured above the scan line

Termination: A - at another discontinuity, I - into rock O - Obscured

Dip direction: right hand rule measured clockwise from true north

Curvature: waviness on numerical scale 1 - straight to 5 very irriguslar(blocky)

Roughness: Friction on numerical scale 1 - very smooth to 5 very rough

Comments: water, infilling, vegatation, shear zone, fault, rubble, etc.

SLOPE 6 | Page 1 of 1

Slope 27(A) cell mapping data sheet

Project:			Date:		By:			Page: 1 of 1	
	Cell	Elev.	Strike	Dip	Height	Width	Rock Type	Northing	West ~~Easting~~
BEGIN		6667	288	45				39° 05.0'	119° 55.1'
END		6669						39° 05.0	119° 55.1'
	Dist. (ft)	Length (ft)	Termination	Orientation	Curvature	Roughness	Comments:	Photo	
1	0 - 100			/					
2				/					
3	25 - 90'			/			HAVE two Active SPRINGS APPROX 6' DOWN FROM		
4				/			CREST - ALSO HAVE DEBRIS Flow/FAILURE IN		
5				/			this Section - SLOPE is wet		
6				/					
7				/			SLOPE IS SOIL with no RK on SLOPE		
8				/			SURFACE. ONE small RK outcrop in center		
9				/			of slope But not enough to get READINGS		
10				/			VERY Few Rocks IN Ditch (<12"-)		
11				/					
12	98'			/			END SOIL SLOPE. BEGIN RK SLOPE		
13				/			TRAithite - INtensley FRactured, Deeply weathered		
14				/			to FRACURED FOR any READINGS		
15				/					
16	100 - 150	END		/			GOES BACK to SOIL @ 130' to end		
17				/					
18				/					
19				/					
20				/					
21				/					
22				/					
23				/					
24				/					
25				/					
26				/					
27				/					
28				/					
29				/					
30				/					
31				/					
32				/					
33				/					
34				/					
35				/					
36				/					
37				/					
38				/					
39				/					
40				/					
41				/					
42				/					
43				/					
44				/					
45				/					
46				/					
47				/					
48				/					
49				/					
50				/					

Dist: distance along the scan line
Length: length measured above the scan line
Termination: A - at another discontinuity, I - into rock O - Obscured
Dip direction: right hand rule measured clockwise from true north
Curvature: waviness on numerical scale 1 - straight to 5 very irriguалar(blocky)
Roughness: Friction on numerical scale 1 - very smooth to 5 very rough
Comments: water, infilling, vegatation, shear zone, fault, rubble, etc.

APPENDIX F

DRAFT SPECIAL PROVISION – ROCKFALL SLOPE DRAPE (RSD)

APPENDIX G

SNOW LOAD CALCULATIONS

Snow Loads @ Slope 27 based on Formula in Final Design Guidelines
See Exhibit A

$F_a = \rho g H L \sin\theta - \rho g H L \cos\theta \tan\phi$

P = 30 PSF – Douglas County design for snow loads approximately 6000 Feet Elevation Exhibit B

H – 1 Feet thick – Estimated
L – 124 Feet @ 42° Slope   143 Feet @ 50° Slope (Exhibit C)
θ – 42° Slope / 50° Slope.
φ – 50 (Ave) Interface Friction based final design Guidelines by Washington State Transportation Commission
36-59 per Exhibit A

$Fa_{36} = (30)(1)(124)(\sin 42) - (30)(1)(124)(\cos 42)(\tan 36)$

$Fa_{36} =$ 480 lbf/Ft   F.S. = 2 = 961
@ 20,000 lbF (anchor Load?) = 21 Feet Anchor Spacing
@ 30,000 lbF (anchor Loads) = 31 Feet

$= (30)(1)(143)(\sin 50) - (30)(1)(143)(\cos 50)(\tan 36)$

= 1283 lbF/Ft   F.S = 2 = 2565
@ 20,000 lbF = 8 Feet
@ 30,000 lbF = 12 Feet

$Fa_{59} = (30)(1)(124)(\sin 42) - (30)(1)(124)(\cos 42)(\tan 59)$

= –2111 lbF/Ft   FS = 2 = 4223.
@ 20,000 lbF (anchor Load) = 5 Feet Spacing
@ 30,000 lbF = 7 Feet Spacing

$= (30)(1)(143)(\sin 50) - (30)(1)(143)(\cos 50)(\tan 59)$

= –1303 lbF/Ft   F.S = 2 = 2606
@ 20,000 lbF = 8 Ft
30,000 lbF = 12 Feet.

Exhibit A

Final Design Guidelines

DESIGN GUIDELINES FOR

WIRE MESH/CABLE NET SLOPE PROTECTION

Balasingam Muhunthan
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Federal Highway Administration

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Exhibit A

2.3 INTERFACE FRICTION

Where the mesh is in contact with the slope, interface friction provides a resistance component to the stability of the system. The interface friction is controlled by macro and micro roughness of the surface. Macro roughness is defined by large-scale irregularities of the slope, and micro roughness is defined as the texture of the surface. Where the slope is planar and the surface is smooth, minimal interface friction may occur, and the mobilized force on the system is carried largely by the anchors. Where slopes are highly irregular and the surfaces are rough or have abrupt protrusions, very high interface friction may occur. In these cases, very little to no mobilized force may be imparted to the anchors.

Unfortunately, interface friction is a difficult parameter to quantify in practice. Furthermore, to include this contribution with the necessary resistance force for a system, a designer must estimate the amount of mesh contact. This task is also difficult, since mesh contact is influenced by slope configuration, fabric flexibility, and installation methods. Because of weathering, interface friction can also be a transient condition. For these reasons, the guidelines do not include the resistance contribution of interface friction to determine anchor requirements for mesh weight, debris load, and impact load. Instead, the guidelines apply a factor of safety to a range of system configurations for a vertical slope (no interface friction) to determine the anchor requirements for these loading conditions.

The one exception is that where snow load is anticipated, interface friction should be assessed. In the absence of either back-calculated or field measurements, the interface friction angle can be estimated for the observed slope irregularity and surface roughness by using the guidelines below.

i. *Rough*: The slope surface is very irregular and undulating and/or has many prominent protrusions on the surface (Figure 4). For such cases, the interface friction angle is assumed to be above 60°.

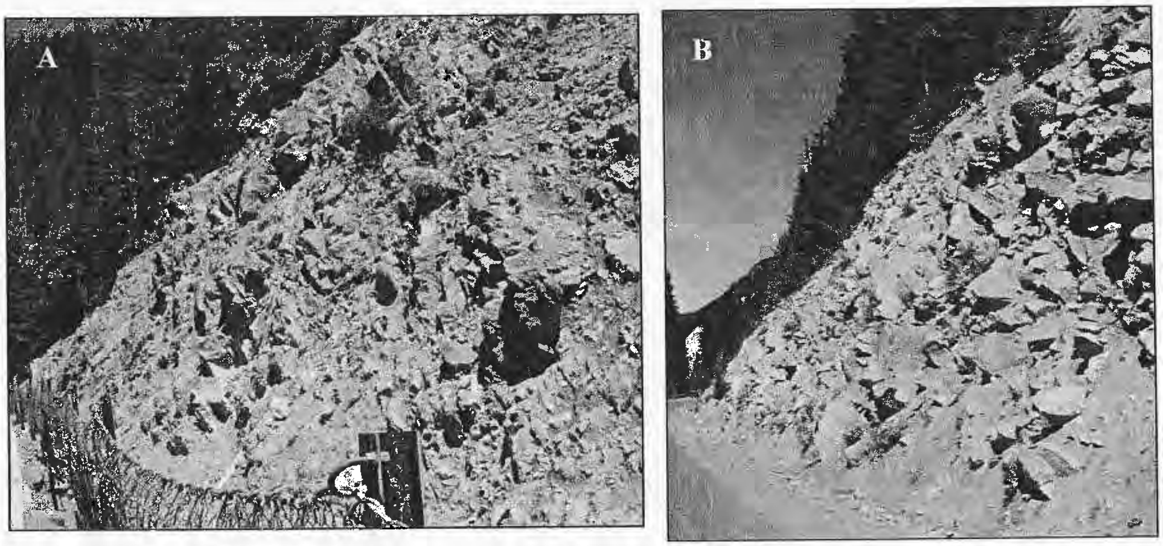

Figure 4. Rough slopes exhibit a high degree of surface roughness with planar, uniform profiles.

ii. *Undulating:* The slope is undulating, and the surface contains some minor protrusions (Figure 5). The interface friction angle is assumed to be between 36°-59°.

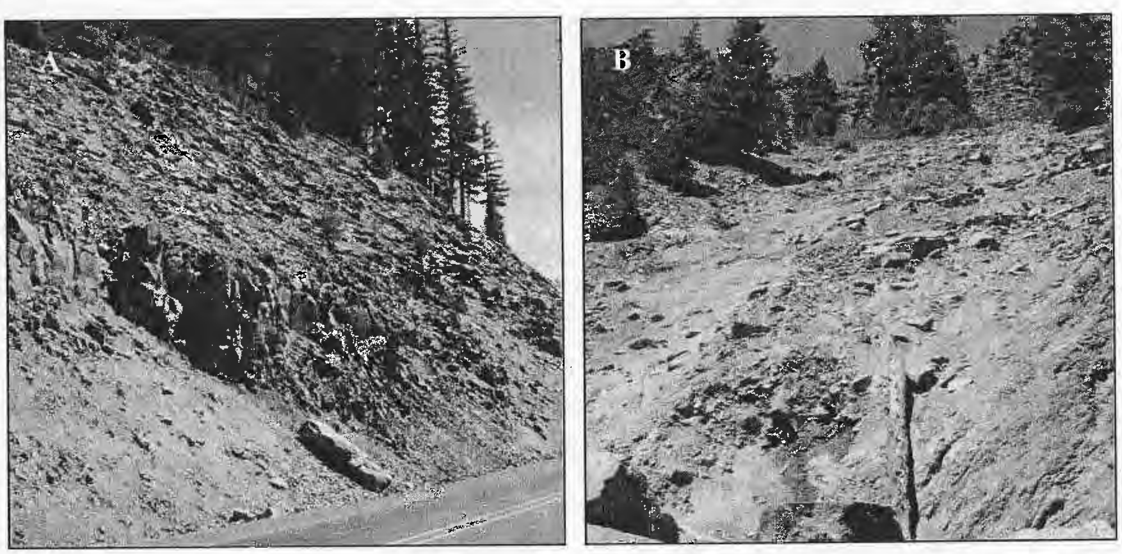

Figure 5. Undulating slopes exhibit profiles with (A) somewhat uniform particle distribution with limited overall roughness, and (B) numerous localized protrusions.

Exhibit A

over hexagonal wire mesh for backing. Hexagonal mesh, however, has greater strength than chain link, and thus probably provides somewhat better puncture resistance for small-sized rocks.

3.2 ANCHOR CAPACITY AND SPACING

While interface friction alone can provide, in some cases, sufficient resistance to hold a mesh system on a moderately inclined slope, anchors should provide the primary support for mesh systems. Unlike interface friction, the resistance contribution from anchors is easily quantifiable and unchanging over the life of the system. For these reasons, it is recommended that the design of system support for debris and impact loads relies solely on the anchors. Snow loads, however, require the consideration of interface friction to develop a cost-effective anchor design. These two approaches to anchor design are treated separately in the sections that follow.

Current practice in North America generally utilizes anchor elements that exceed a 20,000-lbf (90-kN) minimum yield strength in both tension and shear. Common tendons include a 1-inch (25-mm), continuously threaded deformed steel bar and ¾-inch (19-mm) wire rope. Consequently, a minimum capacity of 20,000 lbf (90 kN) has been assumed in the design chart presented below. The charts presented in figures A-19, A-20, and A-21 in Appendix A can be used for anchors of different capacity for common fabric types with an appropriate safety factor. Additional charts for common fabric types are provided in Appendix A (figures A-1 through A-18) that account for interface friction for slopes oriented at 45° and 60° with planar, undulating, and rough slope surfaces. An appropriate factor of safety should also be applied to these anchor loads and spacings.

Exhibit A

slope orientations. A similar factor of safety should then be applied to determine anchor capacity and spacing.

3.2.2 Snow Loads

As documented in the technical report (Muhunthan et al., 2005), snow loads have been responsible for numerous system failures. All known system failures have occurred as a result of anchor yielding, either through exceeding the strength or passive resistance of the ground or the yield strength of the tendon. No ancillary damage to the mesh, support ropes, or connections has been observed at any of these snow-related failures. The anchor capacities and spacings used at these sites were in general accordance with those presented in Table 2, supporting the conclusion that these spacings may be too wide for systems exposed to snow loads. However, if the anchors were assumed to carry the entire snowpack weight and interface friction was neglected, unrealistically large anchor loads would be calculated. The instrumented Tumwater Canyon and the U.S. 20 Rainy Pass sites summarized in the technical report clearly demonstrate the important resistance contribution provided by interface friction. The anchor force due to snowpack per unit width of mesh, F_a, can be calculated with the following equation:

$$F_a = \rho g H L \sin\theta - \rho g H L \cos\theta \tan\phi$$

where ρ is the overall density of the snowpack, g is the gravity constant (for metric units), H is the thickness of the snowpack oriented normally to the slope, L is the slope length of the installation, θ is the slope angle, and ϕ is the interface friction angle. The design challenge lies in characterizing the interface friction of the entire installation. The case histories presented in the technical report and the photos in section 1.3 can aid in this characterization.

Exhibit A

A safety factor of 2 to 3 should be applied to account for larger than anticipated snowpack and overestimation of interface friction.

It is evident from the equation that for slopes that have an interface friction equal to or greater than the slope angle, a snowpack should cause no load increase on the anchors. Conversely, when interface friction is less than the slope angle, a portion of the snow load is transferred to the anchors, and load increases rapidly as the angles diverge. Two examples are provided to illustrate the effect of interface friction (ρ = 25 lbf/ft^3; H = 2 ft; L = 150 ft; θ = 45°; ϕ = 30°, 40°) for an assumed anchor capacity of 20,000 lbf (90kN):

$F_{a30°}$ = 2240 lbf/ft; a FS=2 results in a roughly 5 ft anchor spacing

$F_{a40°}$ = 850 lbf/ft; a FS=2 results in a roughly 12 ft anchor spacing

Exhibit B

Douglas County Community Development
1594 Esmeralda Avenue
P.O. Box 218
Minden, Nevada 89423

Building Division: 775-782-6224
Engineering Division: 775-782-6235
Planning Division: 775-782-6217
Fax: 775-782-9007

COUNTY BUILDING CODES & DESIGN CRITERIA

Adopted Building Codes:
2006 International Building Code (IBC)
2006 International Residential Code (IRC)
2006 Uniform Plumbing Code (UPC)
2006 Uniform Mechanical Code (UMC)
2005 National Electrical Code (NEC)
2006 International Energy Conservation Code (IECC)
2006 International Fire Code (IFC)
Amendments to these codes are available on the County website: www.douglascountynv.gov

6,000 feet and under elevations:
Seismic Zone: IBC=D / IRC=D2 & E for some site specific areas
***Wind Speed:** 105 MPH (3-Second Gust)* **Exposure:** C
****Snow Load:** 30 psf**
Frost Depth: 18" (inches) minimum
Ice Shield Required: above 5,300 feet elevation
Soil Bearing: 1,500 psf maximum or site specific

Above 6,000 feet elevation:
Seismic Zone: IBC=D / IRC=D2 & E for some site specific areas
***Wind Speed:** 105 MPH (3-Second Gust)* **Exposure:** C
(Exposure B may be used in areas between ¼ mile of the lakeshore & below the elevation of 7,200 feet.)
Snow Load: 150 psf
Frost Depth: 24" (inches) minimum
Ice Shield Required: above 5,300 feet elevation
Soil Bearing: 1,500 psf maximum or site specific

Single Family Dwelling Design:
Minimum Roof Pitch: 4:12 (inches) – Maximum Building Height: 35'-0" (feet)
Minimum Soffet Eaves (overhang): 18" (inches) – 3 or More Gable ends and building offsets recessed/alcove or similar features – Minimum of 2-car off-street covered parking (carport or garage)

* ***Wind Speed (3-Second Gust):*** *Prior to the 2006 IBC & IRC accepted engineering practice was to base the wind design on the "Fastest Wind Speed", which was defined as the highest recorded wind velocity averaged over the time it takes a mile of air to pass a given point. However, since short-term velocities due to gusts may be higher, the 2006 IBC provides wind design maps based on a 3-Second Gust. According to the 2006 IBC Table 1609.3.1, the County minimum 105 MPH (3-Second Gust) wind speed can be converted to an 85 MPH (Fastest Wind Speed); however this must be clearly identified within the plans and the structural design calculations.*

** ***30 PSF Snow Load:*** *County policy requires that the 30 PSF ground snow load must be used as the design roof snow load. The 30 PSF snow load is the absolute minimum roof design load, design reductions will not be allowed to reduce the minimum roof design load.*

Exhibit C

McMILLEN, LLC

Project: Slope 27
Description: WOOD ROGERS - RSD
Prepared By: HAD
Page ___ of ___
Date: 12/4/09
Checked: ___
Job No. ___

Calculations base on Exhibit A - RSD

A - A' - length of slope ≠ 325 Feet
B - B' - Horizontal length of slope ≠ 111 Feet (92 + 19 Feet) ∟ Add 15' above slope crest / WSDOT Report June/2009 (Attached)
slope ∠ 42-50°

Slope Trigonometry

$C = \frac{b}{\cos a} = \frac{111 \text{ Feet}}{\cos 42/50°}$ = 149 Feet @ 42°
= 173 Feet @ 50°

@ $\frac{92 \text{ Feet}}{\cos 42/50}$ = 123.8 @ 42° = 143 @ 50°

Slope 27 Area - Slope Angle 42°

Area = 325 x 149 Feet = 48,425 SF
9,685 SF (20% contigency)
= 58,110 SF

Conversion SF to SM
1 SM = 10.7639 SF
= 5398.60 SM

WSDOT RSD Report / June 2009 = $ 139.51 SM
= $ 753,158.80

Slope 27 Area - Slope Angle 50°

Area = 325 x 173 Feet = 56,225 SF
11,245 SF (20% contigency)

Conversion SF to SM = 67,470 SF
= 10.7639 SF = 6268.17 SM

WSDOT RSD Report / June 2009 = $ 139.51 SM
= $ 874,472.97

∴ 6300 SM @ $ 140 / SM = $ 882,000