Prepared for:

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Geotechnical Engineering Design Report
I-80 Bridge and Tunnel Rehabilitation – Scope B
Rock Stabilization Analysis
Elko County, Nevada

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October 5, 2012
Kleinfelder Project No. 124637

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Mr. Craig Smart
HDR Engineering , Inc.
7180 Pollock Drive, Suite 200
Las Vegas, Nevada 89119

Subject: **Geotechnical Engineering Report**
I-80 Bridge and Tunnel Rehabilitation - Scope B
Rock Stabilization Analysis
Elko County, Nevada

Dear Mr. Smart:

Kleinfelder, Inc. (Kleinfelder) is pleased to present our geotechnical engineering report for the portal rock stabilization analysis performed as part of the Interstate 80 Carlin Bridge and Tunnel Rehabilitation – Scope B in Elko County, Nevada. The accompanying report provides the results of our field exploration and engineering analysis. Our geotechnical explorations and services were performed in accordance with the scope of services between HDR Engineering, Inc. and Kleinfelder, Inc., dated August 16, 2011.

We appreciate the opportunity to provide these services to you. Should you require additional information or have any questions or concerns regarding this report, please feel free to contact us at (425) 636-7900 or (702) 736-2936.

Sincerely,

KLEINFELDER, INC.

Chad R Lukkarila

Chad Lukkarila, PE
Senior Engineer

Ann Backstrom

Ann Backstrom, PE
Principal Professional

Attachment: Geotechnical Engineering Report

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1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION

The project is located along Interstate 80 (I-80) at Carlin Canyon in Elko County and includes a total of eight bridges and two tunnels for evaluation, seismic retrofit and comprehensive rehabilitation. The two, side by side tunnels are referred to as the Carlin Tunnels and are approximately 1,800 feet in length. A site vicinity map is provided on Figure 1. The scope of services for the two tunnels includes the rock stabilization analysis addressed in this report and concrete pavement/drainage system evaluations addressed in a separate report.

1.2 SCOPE OF SERVICES

This report addresses the rock stabilization analyses of existing rock slopes above and behind the portal walls. Large blocks are perched above and behind the portal walls and have previously been strapped with cables. In addition to the stability of the blocks, an assessment of overall slope stability was requested. Our scope included reconnaissance and geologic mapping around the portals of the two tunnels, rock slope stability and rockfall hazard mitigation analyses, and preparation of this report addressing the results of geomechanical, stability and rockfall analyses and mitigation alternatives for stabilizing or reducing the impact of the rock slope hazards identified.

2.0 FIELD INVESTIGATION

2.1 REGIONAL GEOLOGY

The I-80 Carlin tunnels lie within the Basin and Range Province of northeastern Nevada. The Basin and Range is characterized by roughly north-south trending, parallel, fault-bounded and uplifted ranges separated by intervening, down-dropped basins. The site geology is dominated by the Carlin Canyon unconformity, a classic geologic feature formed where Mississippian sedimentary layers were uplifted to a vertical position, eroded, then overlain by later Pennsylvanian sedimentary layers, followed by another period of uplift (Cashman and others, 2008). Younger, horizontal Quaternary and Tertiary deposits cap the area and outcrop along I-80 east of the tunnels.

Bedding in the vicinity of the tunnel portals dips steeply to the east-northeast. The Humboldt River traverses the I-80 alignment east and west of the Carlin tunnels resulting in erosional topographic features and the deposition of Quaternary alluvial deposits. In general, the sedimentary rock units at the tunnel area are overlain by alluvial, colluvium and talus deposits.

2.2 FIELD RECONNAISSANCE

During April 2012, Kleinfelder conducted field mapping at existing rock cut slopes located above and behind the portal walls for the east and west bound tunnels. We collected information in accordance with *The Rock Slopes Reference Manual* (FHWA A-HI-99-007, 1998). Discontinuity information that we collected includes the following:

- Location of the discontinuity
- Type of discontinuity
- Discontinuity orientation (dip and dip direction)
- Discontinuity persistence
- Discontinuity aperture width

- Discontinuity filling (or lack of)
- Barton's joint roughness coefficient (JRC) value

Rock mass information that we collected includes the following:

- Locality type
- Slope length
- Slope height
- Rock mass color
- Rock mass grain size
- Field estimates of intact rock uniaxial compressive strength
- Rock mass fabric
- Rock mass block size
- Rock mass state of weathering
- Number of discontinuity sets

Additionally, we looked for signs of instability from the existing cut slopes and natural rock outcrops above the tunnels. We measured the existing catchment areas and fence heights at the base of the existing cut slopes and noted the rock blocks sizes that had fallen into the catchment areas. To assist with stability and rockfall hazard analyses, we developed multiple profiles of the existing cut slopes using hand-held laser range finder surveying equipment that measures the horizontal and vertical distance from a fixed point to a point on the cut slope. Figures 2 and 3 show photographs and profile locations of the west and east end portal cut slopes. Figures 4 through 6 show profiles of the west end cut slope. Figures 7 through 11 show profiles of the east end cut slope. In all cases except East End Line 5, the profiles extend up to the top of the ridge above the cut slopes.

2.2 GEOLOGIC AND SLOPE CONDITIONS

2.2.1 West End Portal Cut Slope

The west end portal cut slope is broken into two main geologic domains: older steeply dipping sedimentary units consisting of the Tonka Formation underlying the south half of the slope and younger, shallowly dipping Quaternary deposits composed of partially cemented talus and colluvium forming the north half of the slope. Additional description of the southern portion of the slope is described in the following paragraphs. No rock is exposed in the lower portion of the northern cut slope. A natural rock outcrop is present at the top of the ridge about 200 to 300 feet vertically above the existing cut slope. The talus blocks are less than four inches in size and sub-angular to sub-rounded. The matrix around the talus blocks is a fine grained sand and silt. The talus slope is at an inclination of 35 to 40 degrees. At the top of the talus slope (just below the natural slope), the talus/colluvium material is partially cemented and standing at an inclination of about 60 to 80 degrees.

The south end of the cut slope is composed of interbedded sandstone and shale units of the Tonka Formation. The bedding is high-angled and dips to the east-northeast at about 80 degrees. The sandstone is slightly weathered and moderately to highly fractured. The shale unit is highly weathered and moderately to highly fractured. Thin beds of sandstone are found within the shale unit. The overall inclination of the south end of the existing cut slope is about 45 degrees with small sections (5 to 10 feet in height) of the sandstone slope standing at an inclination of about 65-80 degrees.

The natural backslope above the cut is inclined at about 25 to 30 degrees (about 2H:1V; Horizontal:Vertical).

We discussed the rockfall with Mr. Aaron Hand, NDOT Area Supervisor, at the site. Mr. Hand stated that the amount of rockfall varies based on the amount of precipitation per year and that NDOT cleans off the bench once per year during the annual tunnel work. Additionally, Mr. Hand stated that typical block size ranges from 4 to 12 inches with more rockfall on the south end of the cut slope and near the contact of the talus and rock. We observed similar size blocks in the catchment during our field reconnaissance and apparent areas of impact from rockfall in the fence and concrete wall at base of slope. Mr. Hand indicated that impacted noted was from rockfall and not from machinery cleaning up rock debris in the catchment area. NDOT stated that they had not observed rockfall that has bounced over the fence. Along the north end, below the talus slope, NDOT built a berm against the slope, as stated by Mr. Hand and observed during our field reconnaissance, to help contain the talus material.

2.2.2 East End Portal Cut Slope

The east end portal cut slope is composed of limestone of varied weathering and fracturing occurring as part of the Strathearn Limestone. A near vertical fault zone strikes across the cut slope from southeast to northwest and is shown on Figure 3. The rock mass within the fault zone and to the south end of the cut slope is highly to moderately weathered and intensely to highly fractured. The rock mass to the north of the fault zone is moderately to slightly weathered and highly to moderately fractured. A construction bench is located about 50 to 75 feet above the catchment area (Figure 3). The bench is sub-horizontal to slightly declined towards the east tunnel entrance at an inclination of about 3H:1V to 4H:1V (Figure 10). The bench is partially covered with talus, grasses, soil, and rock blocks. The construction bench is accessible on foot up the natural slope to the north of the portal cut slope area. A former construction access road is visible on the portal cut slope face, but is not safely accessible because of erosion on the slope face (Figure 3).

The cut slope face is intersected by two large gullies that may act as rockfall chutes. One gully is at the south end of the cut slope, is slightly curved and extends to near the top of the slope (East End Portal Slope Profile 2 – Figure 8). The second gully is located near the center of the cut slope and extends up to the construction bench (East End Portal Slope Profile 4 – Figure 10). The second gully is represented as the lower portion of East End Portal Slope Profile 4 downslope of the bench. The gully does not extend above the bench. Some rockfall has occurred from this gully near the center of the cut slope.

Three large blocks are perched above and behind the east end portal wall and are shown on Figure 12. The blocks are located on the inside edge (upslope side) of the construction bench located a height of about 75 to 80 feet above the catchment area. The three blocks (from right to left in Figure 12) have the following approximate dimensions:

1. 20'W x 19'H x 8'D (Width; Height; Depth)
2. 15'W x 28'H x 10'D
3. 10'W x 12'H x 5'D

The blocks appear to have been left in place during construction and have weathered in place and become potentially unstable. The blocks have previously been strapped with three approximately one-inch diameter cables stretched across the face of the blocks and the ends secured to the adjacent cut slope with one rock anchor on each end. Two of the cables are tight around the blocks and one has some slack. Information was not available about the installation method or length of the rock anchors supporting the cables. We did not observe tension cracks behind the blocks. However, the area behind the blocks is fine grained material that has eroded from the cut slope and cracks may have been covered.

The natural backslope above the cut slope is inclined at about 30 to 35 degrees (1.5H:1V). Natural outcrops 20 to 30 feet in height are located upslope of the cut slope. The outcrops are near vertical in inclination and moderately to slightly fractured. Based on our observations, these outcrops do not appear to be a source area for rockfall.

We did not observe impact marks from rockfall in the concrete wall at the base of slope. However, we did observe three large dents in the fence material. It is not known if these dents are from rockfall or from equipment used to clean the catchment bench and it could not be confirmed with NDOT personnel.

3.0 DESIGN AND RECOMMENDATIONS

3.1 GEOMECHANICAL ROCK MASS CLASSIFICATION

As part of our analysis, we completed geomechanical rock mass classifications. These classifications are a design tool and are estimated using the field data. Two of the more widely accepted classifications systems are the Rock Mass Rating System (RMR) by Bieniawski (1989) and the Geological Strength Index (GSI) from Hoek (1997).

The base RMR, also referred to as the geomechanics classification system, is based on the algebraic sum of five rock mass property ratings, namely:

- Rock quality designation (RQD)
- Strength of intact rock material
- Spacing of discontinuities
- Condition of discontinuities
- Groundwater conditions

Additional information on the field methods used to estimate RQD and rock strength are presented in the following sections 3.1.1 and 3.1.2. To estimate the RMR, we compared field data to published tables by Bieniawski (1989). Values for RMR can range from zero to 100. From the ratings, rock class and corresponding descriptions and engineering properties are assigned to the overall rock mass.

Bieniawski's (1989) RMR classification can be related to Hoek's (1997) GSI rating. The GSI rating can also be estimated directly from the information that we collected during our field mapping. The GSI values summarized below in Table 1 were estimated by correlation to the RMR.

3.1.1 Rock Quality Designation

Deere (1963) developed the RQD technique, which is simply estimated from percent of rock core recovery ≥ 10 cm (4 inches) in length compared to the total run. Additionally, Palmström (1982) developed a technique to estimate RQD while evaluating the rock face, where:

- RQD % = 115 – 3.3(J_v), where J_v is the volumetric joint count

In boreholes, RQD is a directionally dependent parameter and its value may change systematically depending upon borehole orientation. Therefore, using the volumetric joint count by Palmström is beneficial in reducing this directional dependence. To evaluate J_v in the field, one must select an open cleft in the rock which displays x, y and z dimensions. One then sums all the fractures along a 1-meter length in each of the 3-dimensions to obtain a volumetric joint count. The RQD is used in estimating RMR values.

3.1.2 Uniaxial Compressive Strength

To estimate intact rock strength, we employed a geological hammer to indent or break rock specimens. We compared our results to published tables by ISRM (1981) and Hoek and Bray (1981) on field estimates of rock strength. These values were converted to approximate uniaxial compressive strength of the rock and used to estimate RMR and Hoek-Brown (1997) empirical strength criteria. In design of the rock slopes, we focused on slope kinematics where structure of the rock mass controls stability. We used the Hoek-Brown nonlinear strength data, based on the RMR and GSI values, to evaluate rock mass slope stability where rock mass strength governs design.

3.1.3 Summary of Rock Mass Characteristics

Table 1 summarizes the geomechanical rock mass information collected during the mapping of the existing rock cut slopes. The material qualities are presented from North to South.

Table 1: Summary of Rock Mass Characteristics Collected During Field Reconnaissance

Location	Rock Type	Estimated Rock Strength (MPa)	Estimated Rock Strength (psi)	RQD	RMR	GSI
West End	Sandstone	25	3,625	10	44	39
West End	Sandstone	50	7,250	36	48	43
West End	Shale	5	725	0	40	35
West End	Sandstone	50	7,250	0	44	39
East End	Limestone	50	7,250	10	50	45
East End	Limestone	3	435	19	47	42
East End	Limestone	3	435	0	40	35
East End	Limestone	38	5,510	59	59	54
East End	Limestone	50	7,250	46	53	48

The existing cut slope above and behind the west end portal consists of moderately to slightly weathered sandstone and shale. Based on field tests, we estimated the sandstone rock mass strength to be weak to moderately strong and the shale rock mass strength to be very weak to weak. The sandstone RQD values estimated from the mapping data range from 0 to 36 combined with the shale RQD value of 0 indicates intensely to highly fractured rock. The estimates of RMR range 44 to 48 and GSI range from 39 to 43 for the sandstone and 40 (RMR) and 35 (GSI) for the shale. These ranges correspond to fair quality rock mass with some potential fair to poor quality areas based on qualitative ratings provided by Bieniawski, 1989.

The existing cut slope above and behind the east end portal consist of highly to slightly weathered limestone. On the south end and within the fault zone the limestone is highly weathered. To the north of the fault zone, the limestone is slightly to moderately weathered. Based on field tests, we estimated the rock mass strength to be very weak to moderately strong. The RQD values estimated from the mapping data range from 0 to 59 indicating intensely to moderately fractured rock. The estimates of RMR range from 40 to 59 and GSI range from 35 to 54 indicating a fair quality rock mass with some potential areas of fair to poor quality based on qualitative ratings provided by Bieniawski, 1989.

3.4 STEREONETS

Using the discontinuity data we collected at existing rock outcrops, we constructed pole plots on equal area stereonets using the computer programs Dips® Version 5.0. A pole represents an individual discontinuity. Stereonets provide a two-dimensional representation of the three-dimensional discontinuity data. The Dips® program contours the discontinuity data to identify trends in the data. Next, we outlined pole clusters or contoured populations of discontinuities. Great circles were then plotted for each major pole cluster. The pole clusters represent major discontinuity sets, which generally strike in a similar direction. Appendix A provides a "how-to" guide for reading stereonets and an example Markland Analysis.

The contoured pole plot stereonet, shown in the upper portion of Figure 13 for the west end, estimates four discontinuity sets (including bedding) that range in dip angle from 33 to 79 degrees. The great circles represent the average dip and dip direction for the discontinuity set. The contoured pole plot stereonets, shown in the upper portion of Figure 14 for the east end south of and in the fault zone, estimate five discontinuity set (including bedding) that range in dip angle from 29 to 80 degrees. The contoured pole plot stereonets, shown in Figure 13 for the east end north of the fault zone, estimate three discontinuity sets (including bedding) that range in dip angle from 43 to 72 degrees. The great circles represent the average dip and dip direction for the discontinuity set. As shown by the spread of the contoured populations on Figures 13 and 14, there are some minor variations in the discontinuity sets.

3.4.1 Markland Analysis

To analyze the impact of the rock mass structure on stability, a Markland Analysis was used to estimate the kinematic potential for rock blocks to fail out of the existing or proposed slopes. The information required to perform an analysis are the design slope dip and dip direction, the orientation of the discontinuities within the rock mass, and the friction angle of the discontinuities. A kinematically potential wedge failure is identified when a point defining the line of intersection of two planes falls within the area included between the great circle defining the slope face and a circle defined by the angle of friction. A planar failure is a specialized form of a wedge failure that follows the same criteria above and also must fall within ± 20 degrees of the dip direction of the slope face. We plotted the orientations of the discontinuities on stereonets using the computer programs Dips® Version 5.0 by Rocscience and ROCKPACK III by C. F. Watts (2001). We plotted both poles and dip vectors. The poles tend to accentuate the orientation of steeply dipping discontinuities while the dip vectors lend themselves to performing Markland Analyses (Appendix A).

The Markland Analysis does not consider a cohesion intercept when modeling the strength of discontinuities. This method also assumes that the discontinuities are continuous and through going with no "bridging" within the discontinuity. The effect of "bridging" would allow a tensional component (or cohesion intercept) of discontinuity strength. The Markland Analysis assumes that the factor of safety of individual rock blocks may be estimated as follows. When the dip of a discontinuity or the plunge of the line of intersection is greater than the friction angle and the factor of safety is less than 1.0. When the dip of a discontinuity or the plunge of the line of intersection is less than the friction angle, the factor of safety is greater than 1.0. In either case, the dip or plunge has to be less than the dip of the slope face, or the structure will not daylight in or intersect the slope.

We assumed a rock discontinuity angle of friction of 30 degrees based on the geomechanical information that we collected in the field, experience with similar rock types and guidance from *The Rock Slopes Reference Manual* (FHWA, 1998).

The lower portion of Figure 13 displays the Markland Analysis completed for the existing west end cut slope. We estimated a dip direction of 305 degrees and a dip angle of 45 degrees (1H:1V) for the existing cut slope above the portal. Based on the data collected, the Markland Analysis indicates the potential for a planar-type failure from the cut slope at an inclination dipping out of slope at about 33 degrees. Additionally, there is a low potential for toppling-type failures from the cut slope. Based on the mapping, the RQD of the cut slope ranged from 0 to 36, indicating intensely to highly fractured rock mass. We did not observe large scale planar-type failures in the field. The sedimentary rock layers dip steeply to the east or into the slope. However, we did observe small planar fracture faces dipping out of the cut slope. It is our opinion that the potential planar and toppling type failure will result in small-scale rockfall and not large failures. We estimate that these will typically be less than one-foot in diameter and can be controlled by the cut slope catchment area (see Section 3.7).

The lower portion of Figure 14 displays the Markland Analyses completed for the existing east end cut slope north and south of the fault zone. We assumed a dip direction of 125 degrees for the cut slope above the portal. For the portion of the slope in the fault zone and south of the fault zone, we assumed a slope dip angle of 45 degrees based on the cross-sections. For the portion of the slope north of the fault zone, we assumed a slope dip angle of 55 degrees based on the cross-sections. For both sections of the cut slope, there are portions of the slope that are steeper, but the slope angles are based on overall slope inclination. Based on the data collected for the area south of the fault, the Markland Analysis indicates a low potential for toppling-type failures from the cut slope. Based on the data collected for the area north of the fault, the Markland Analysis indicates a low potential for wedge and toppling-type failures from the cut slope. We estimate that these will typically be less than two-feet in diameter and can be controlled by the cut slope catchment area (Section 3.7).

3.6 LIMIT EQUILIBRIUM SLOPE STABILITY ANALYSIS

We used limit equilibrium analyses via the computer program SLIDE V5® by Rocscience to estimate a global safety factor for the proposed slope geometries. We completed a limit equilibrium analysis at various sections for each end cut slope based on the slope geometry and geologic conditions.

To estimate the rock mass shear strength for a given material, we followed the recommendations outlined in Hoek and Brown (1997). This paper (and numerous others by Hoek but not cited here) describes the Hoek-Brown Strength Criterion. The Hoek-Brown Strength Criterion is an empirical rock strength criterion, which takes into consideration intact rock strength, as well as the influence of the discontinuities within the rock mass on the strength of the mass. Because rock mass shear strength cannot be measured in the laboratory, (i.e. large samples of rock are difficult and expensive to sample and test and will not contain a sufficient number of discontinuities to represent the rock mass), the rock mechanics community has adopted the Hoek-Brown Criterion for the purpose of slope stability calculations.

The input that is required to estimate the rock mass strength are the intact rock strength, the GSI, a Hoek-Brown m_i constant (empirical intact rock materials index based on rock type and 100's of triaxial tests), and the amount of disturbance (D) that the rock mass will be subjected to during construction. The first three of these input values were estimated during our field activities. Additionally, the m_i value was estimated from Hoek and Brown (1997). Based on discussions with Dr. Hoek, the disturbance factor, D, should only be applied to large scale cut slopes (open pit mines) and does not apply to road cut slopes, therefore, we have assumed a D factor of 0.

Based on United States Geological Survey Web-Based Seismic Application (USGS, 2012), the peak ground acceleration (PGA) for the project area is approximately 0.14g based on AASHTO (2009) 7 percent in 75 years probability of exceedance. For the west end, a seismic coefficient of 0.085 was used based on AASHTO for soil/rock Site Class C. For the east end, a seismic coefficient of 0.07 was used based on AASHTO for soil/rock Site Class B. Table 2 summarizes the properties used for each stability analysis and the resulting estimated factor of safety.

Table 2: Summary of Global Stability Analyses

Location	Slope Profile	Lithology	Rock Strength (MPa)	GSI	Static Safety Factor	Pseudo-Static Safety Factor
West End	1	Talus/ Colluvium	phi = 35^o, c = 500 psf		1.25	1.09
			phi = 37^o, c = 200 psf		1.00	0.88
			phi = 40^o, c = 100 psf		0.93	0.80
	2	Sandstone	25	40	3.58	3.11
	3	Shale/SS	5	38	2.19	1.89
East End	3	Limestone	25	45	1.69	1.50
		Faulted Limestone	3	38		
	4	Limestone	25	40	1.64	1.48
		Faulted Limestone	3	35		
	5	Limestone	38	48	4.64	4.23

For the West End Slope Profile 1, we completed multiple analyses in the talus/colluvium to assess sensitivity to phi angle and cohesion on the calculated factor of safety. The existing slope is composed of talus that was eroded from the partially cemented colluvium higher in the slope. The talus slope ranges in inclination from 35 to 40 degrees. As shown by the results in Table 2, the safety factors for the range of material properties evaluated are approaching or below 1.0 for static conditions. The failure surfaces predicted by the models are shallow failures (erosion, sloughing, raveling), which are consistent with field observations and the information regarding sediment accumulation and maintenance provided by the NDOT worker. We did not observe signs of potential deeper, large failures in the talus/colluvial materials. We expect shallow surficial sloughing, raveling and erosion to continue on this slope. NDOT Maintenance annually cleans up material from this section of the slope and has built a berm at the toe of the slope to contain the eroding material.

3.7 ROCK FALL HAZARD MITIGATION AND CATCHMENT DESIGN

As part of our design, we performed computer modeling rock fall simulations for each profile and the existing catchment area and fence height. We used the computer program Colorado Rockfall Simulation Program (CRSP) for our analysis at the critical station (maximum height) for each rock cut slope section. In our simulations, we modeled the average and maximum rock blocks expected to roll down the cut slope face. For each simulation, the model rolled 100 rock blocks. Based on field observations, we assumed rockfall originated from within the constructed area in all models, and not from the natural outcrop surroundings. We chose normal coefficients, tangential coefficients, and surface roughness values for the CRSP runs based on field observations and recommendations from the CRSP Manual for different slope descriptions related to slope lithology (Jones, et. al., 2000). Rock blocks were modeled as spherical with diameters ranging from 0.5 to 2.0 feet, based on our field observations and the Markland analyses. Additionally, the model estimated the maximum bounce height and kinetic energy that a rock block would exert on the fence. Table 3 summarizes the results of our rockfall analysis using the measured existing catchment widths and fence heights.

Table 3: Summary of Rockfall Hazard Analyses

Location	Cross-Section	Catchment Width (ft)	Fence Height (ft)	Block Size (ft)	Percent Contained (%)	Max. Bounce Height (ft)	Max. Kinetic Energy (ft-lbs)
West End	1	18	6.5	0.5	>99	3.4	100
				1	>99	2.5	2,700
	2	20	6.5	0.5	>99	1.5	15
				1	>99	2.8	1,700
	3	12	6.5	0.5	>99	2.5	30
				1	>99	3.2	3,000
East End	1	14	7	1	>99	3.3	5,000
				2	>99	2.1	18,800
	2	14	7	1	>99	2.2	1,800
				2	>99	1.0	19,100
	3	16	7	1	>99	4.0	4,400
				2	>99	4.1	28,000
	4	15	12	1	>99	3.2	2,000
				2	>99	1.0	17,000
	5	10	7	1	>99	2.7	2,400
				2	>99	3.0	27,700

The CRSP analyses indicate that the existing catchment width and fence height listed in the table above provides adequate containment for the block sizes observed in the field and estimated from the rock properties collected.

4.0 CUT SLOPE STABILIZATION OPTIONS

4.1 WEST END STABILIZATION OPTIONS

The major concern for the west end cut slope is rockfall. Based on the analyses, the rockfall will be contained within the existing catchment area. The rockfall has the potential to generate kinetic energies up to about 3,000 ft-lbs (4 kJ). The resistance capacity of the existing fence material (chain-link) and posts should be evaluated considering the potential energies listed above. Manufacturers, such as Geobrugg or Maccaferri, develop and produce high tensile-strength rockfall fences and barriers to better contain rockfall with higher energies. Based on our discussions with NDOT staff at the site, the catchment bench is cleaned up once per year. However, they stated that the catchment area has been filled across the base to the fence during heavy precipitation years.

One option to reduce the energy and bounce of rockfall from the cut slope would be to drape the slope with a wire mesh slope rockfall protection system. The wire mesh slope system is supported with rock anchors at the top of the slope and draped down the face of the cut slope. Typically, the drape extends to 5 to 10 feet above the base of the slope. If rockfall occurs, it will slowly fall behind the drape system and fall out to the base of the slope.

For the north end of the slope, an additional option to control the talus material is to continue to construct the berm as NDOT currently has built. A berm is an effective low-cost alternative to reduce downslope movement of the talus material into the catchment area. NDOT should monitor the berm and reconstruct it on an annual basis to maintain containment.

4.2 EAST END STABILIZATION OPTIONS

4.2.1 General Slope Stabilization Options

The major concerns for the east end cut slope are rockfall and the stability of the large blocks on the slope. Based on the analyses, the rockfall will be contained within the existing catchment area. The rockfall has the potential to generate kinetic energies up to about 28,000 ft-lbs (38 kJ). The resistance capacity of the existing fence material (chain-link) and posts should be evaluated considering the potential energies listed above.

The following are additional options to reduce the potential for rockfall onto the roadway from the cut slope. Each option may be utilized independently or, for additional rockfall protection, used together with the other options presented for each area.

- South End Gully Area
    - Replace the existing fence with a designed rockfall fence. The fence should be designed at the same height and to contain rockfall with energies of up to 20,000 ft-lbs (28 kJ) (energy based on models in this specific area).
    - Install a fence at the toe of slope at road level to reduce the potential for rockfall to reach the road. Below the catchment bench, a 35-45 degree slope extends down to the edge of the roadway. A fence could be installed at the toe of this slope to further reduce the potential for rockfall to reach the roadway.
- South Section of Cut Slope to Mid Slope Gully
    - Replace the existing fence with a designed rockfall fence. The fence should be designed at the same height and to contain rockfall with energies of up to 28,000 ft-lbs (38 kJ) (energy based on models in this specific area).
    - Drape cut slope with a wire mesh slope rockfall protection system up to construction bench. The wire mesh slope system is supported with rock anchors at the top of the slope and draped down the face of the cut slope. Typically, the drape extends to 5 to 10 feet above the base of the slope. If rockfall occurs, it will slowly fall behind the drape system and fall out to the base of the slope.

- Mid Slope Gully
    - Replace the existing fence with a designed rockfall fence. The fence should be designed at the same height and to contain rockfall with energies of up to 17,000 ft-lbs (23 kJ) (energy based on models in this specific area).
    - Install a hybrid (modified) rockfall fence and drape system on the upper construction bench. The hybrid system includes a fence installed on the upper bench with a drape system hung below the fence to the base of the slope. If rockfall occurs from further up the slope, it will impact the fence to reduce the energy and then slowly fall behind the drape to the base of the slope.
    - The existing construction bench area is partially covered with talus, soil, and rock blocks. The bench is sloped down towards the east tunnel entrance at about 3H:1V to 4H:1V, which is typically sufficiently flat to reduce the potential for rolling. Currently there is no access to the bench. The bench did not appear to be full of debris at the time of our site visit, but should be monitored as part of maintenance operations. It is likely that debris removal will eventually be required.
- North End of Cut Slope
    - Replace the existing fence with a designed rockfall fence. The fence should be designed at the same height and to contain rockfall with energies of up to 27,000 ft-lbs (37 kJ) (energy based on models in this specific area).
    - Install a hybrid (modified) rockfall fence and drape system on the upper construction bench. The hybrid system includes a fence installed on the upper bench with a drape system hung below the fence to the base of the slope. If rockfall occurs from further up the slope, it will impact the fence to reduce the energy and then slowly fall behind the drape to the base of the slope. The hybrid system could be continued across the slope from mid-slope gully to the north end of the slope.

- Drape cut slope with a wire mesh slope rockfall protection system up to construction bench. The wire mesh slope system is supported with rock anchors at the top of the slope and draped down the face of the cut slope. Typically, the drape extends to 5 to 10 feet above the base of the slope. If rockfall occurs, it will slowly fall behind the drape system and fall out to the base of the slope.

4.2.2 East End Portal Large Blocks Stabilization

The stability of the existing large blocks cabled in place above the east end portal has not been quantitatively evaluated. The blocks are slightly inclined out of slope, down towards the road and east tunnel entrances. We did not observe tension cracks behind the blocks. As can be seen in Figure 12, the block on the right is undermined apparently due to erosion. We observed that two of the three cables were very tight indicating potential loading on the cables and anchors. Additionally, all three cables appear to be supported by one anchor on each end. The current block stability appears to depend on the integrity of the existing anchor / cable system. Kleinfelder has no information regarding the construction history of the support anchors or the types of anchor and cables used. Our opinion is that the blocks may be temporarily stable in the current condition, but should be remediated for long-term slope stability. The following stabilization options could reduce the potential for failure of the large blocks from the cut slope:

- Remove Blocks – The blocks could be remediated using boulder buster explosives to break up the blocks into small pieces. After the blocks have been reduced in size, options could consist of leaving them in place, excavation, or pushing down to the catchment bench and removal from there. Removal of the material could be difficult or create the potential for additional rockfall onto the catchment bench. The bench is 80 feet above the catchment area and slightly inclined down towards the roadway. There is not an access road to the bench, so equipment would have to be hiked up, craned up, or helicoptered into place. When the blocks are blasted, the debris will potentially go down the gully below the bench on to the catchment area. Blasting is feasible, but potentially difficult due to the access logistics.

- Rock Bolt in Place – The rock blocks could be anchored in place with tensioned rock bolts. Five to ten rock bolts could be installed through each block and tensioned to approximately 50,000 pounds (50 kips) to secure the blocks in place. Because of the current stabilization of the blocks, this option may not be a safe or feasible option to construct. Anchors will have to be installed between the cables into the rock blocks. The installation crew will potentially have to be on ropes or in a basket if a crane will reach the site. The vibration from drill could impact the existing anchors or cables. We recommend that a rock anchor specialty contractor be contacted to discuss rock anchor feasibility. Kleinfelder can provide names of contractors if requested.
- Support Blocks in Place with Spider© or Similar Product – Spider© mesh is developed by Geobrugg to support large blocks. A mesh is draped around blocks and tensioned to rock anchors installed around the outside edge of the blocks (not through the blocks). The Spider© mesh can form around the blocks to provide support and the tension is developed in the rock anchors. This is a feasible option because the large blocks do not have to be disturbed until the drape is hung in place and tensioned. The existing cable straps could be left in place. Additionally, the mesh can be colored (if needed) to match the rock coloration.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 GENERAL CONSIDERATIONS

Geotechnical design is usually based on direct knowledge of only limited amounts of information from widely spaced boreholes or surface outcrop mapping. The limitations to the ability to explore are based on practical considerations such as accessibility, cost, time, environmental conditions, property ownership, and reasonability. In general, geotechnical engineering recommendations are always considered preliminary until conditions are verified by observations during construction by the responsible geotechnical engineer (ASFE).

We recommend that Kleinfelder provide cut slope stabilization observation by our rock engineers to validate our design recommendations. Our engineers will observe rock features such as jointing, faulting, joint irregularity, and orientation, and if conditions vary, will use this information for slope stability evaluation and potential modifications to design.

6.0 LIMITATIONS

The scope of services was limited to field mapping and observations of existing exposures in the west and east portal areas. It should be recognized that definition and evaluation of slope surface and subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on the field data collection and strength estimation methods described in the report; subsurface exploration and laboratory testing was not performed.

Recommendations contained in this report are based on field data collection and strength estimation methods described in the report and our present knowledge of the proposed construction. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder.

If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report. This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinions, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions.

The information included on graphic representations in this report has been compiled from a variety of sources and is subject to change without notice. Kleinfelder makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. These documents are not intended for use as a land survey product nor are they designed or intended as a construction design document. The use or misuse of the information contained on these graphic representations is at the sole risk of the party using or misusing the information.

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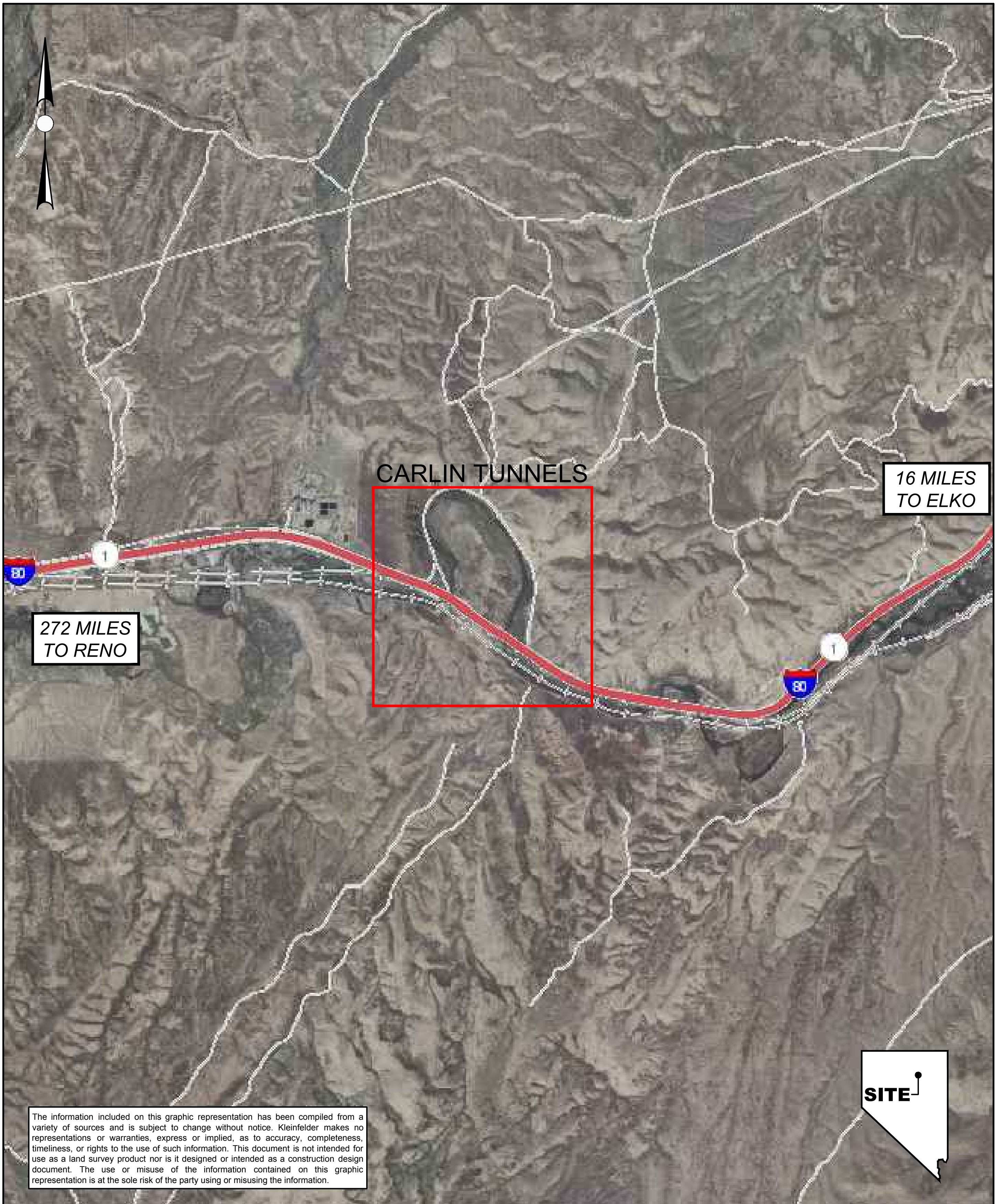
FIGURES



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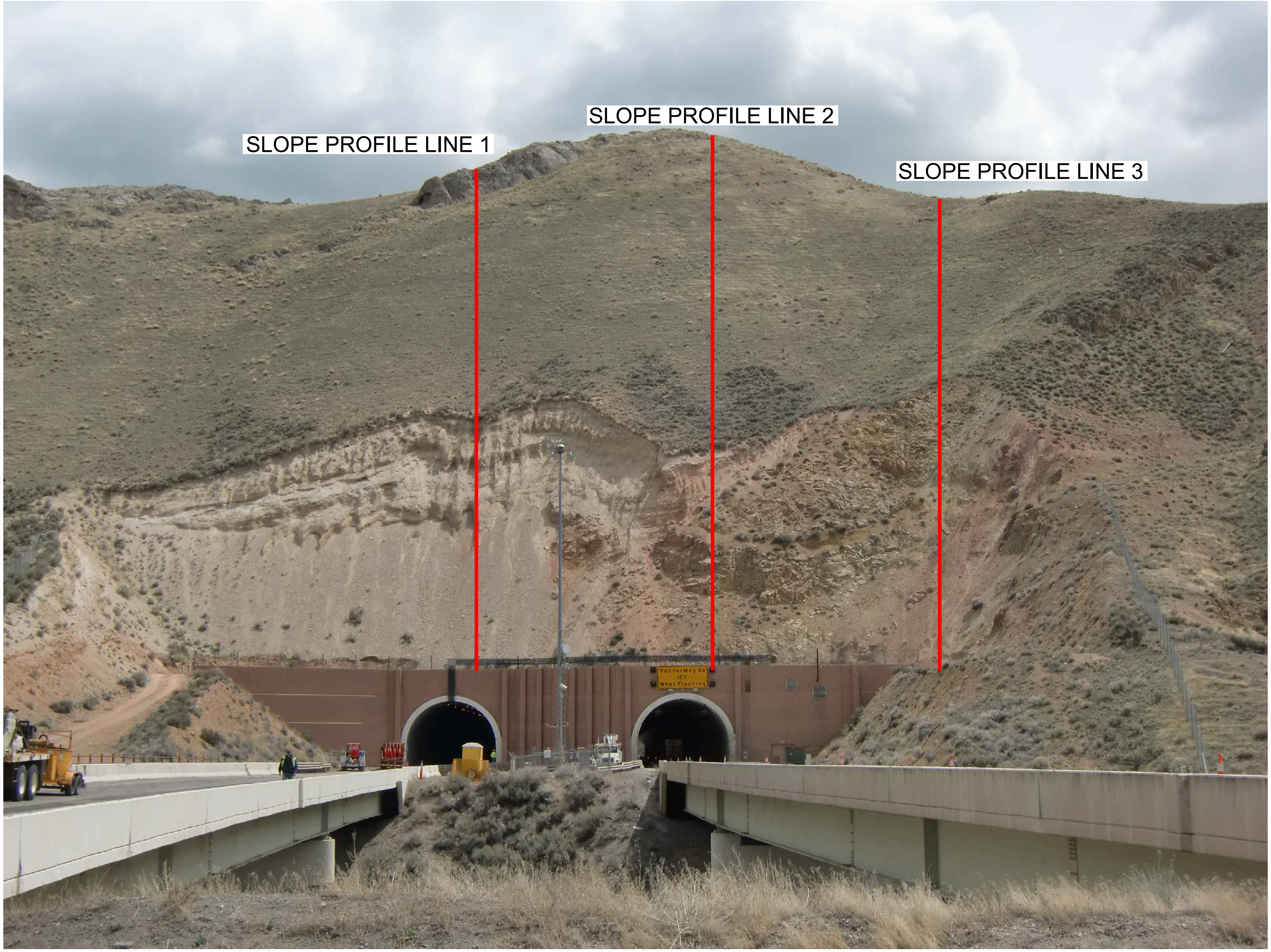
KLEINFELDER Bright People. Right Solutions.	PROJECT NO. 124637 DRAWN BY: JS CHECKED BY: CL DATE: May 2012 REVISED:	Site Vicinity I-80 Bridge and Tunnel Rehabilitation Scope B Rock Stabilization Analysis Elko County, Nevada	FIGURE 1 PAGE: 1 of 14



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West End Cut Slope

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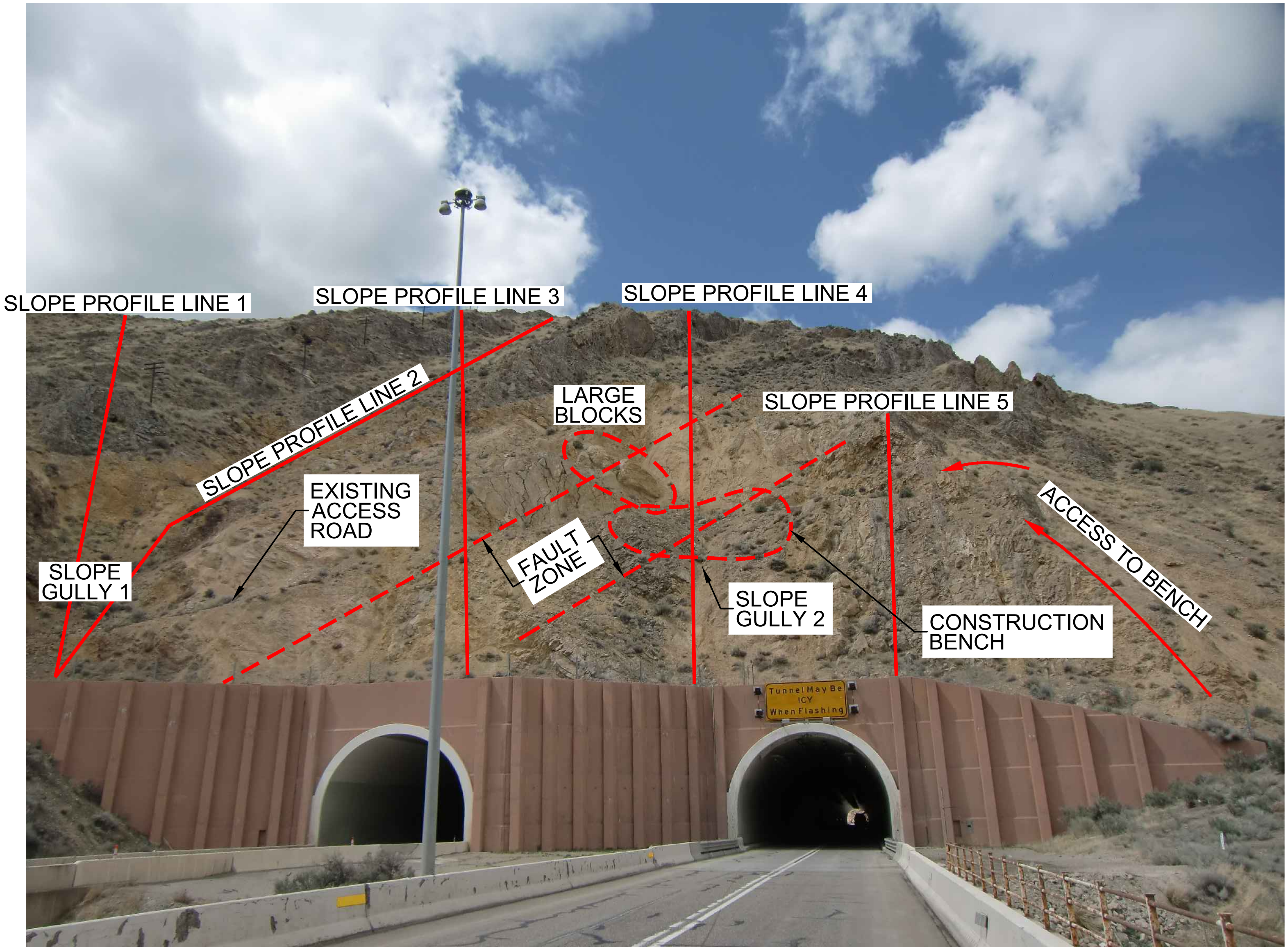
Rock Stabilization Analysis
Elko County, Nevada

FIGURE

2



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East End Cut Slope

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Elko County, Nevada

FIGURE

3

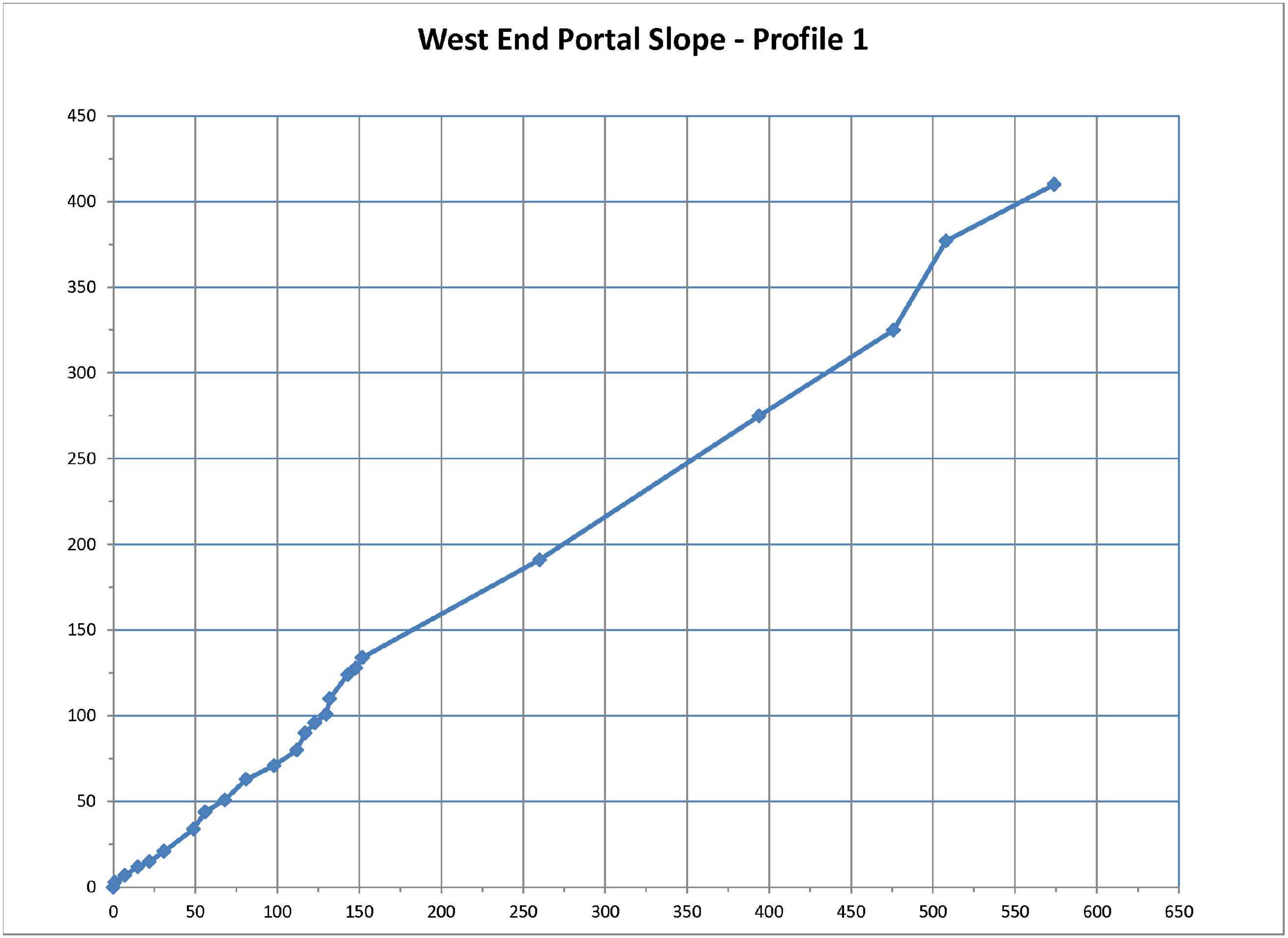

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FIGURE
4

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FIGURE
5

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Elko County, Nevada

FIGURE
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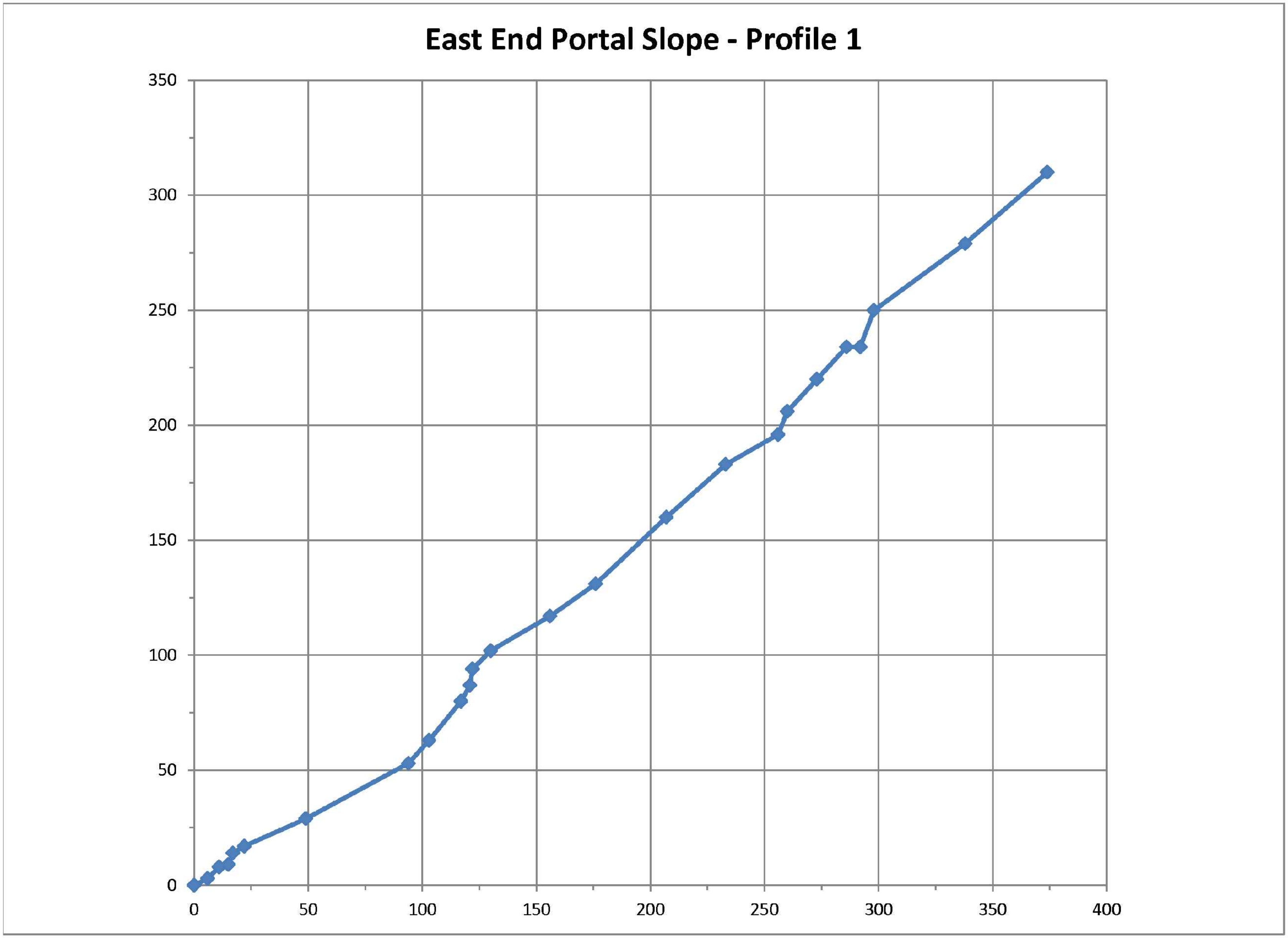

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Elko County, Nevada

FIGURE
7

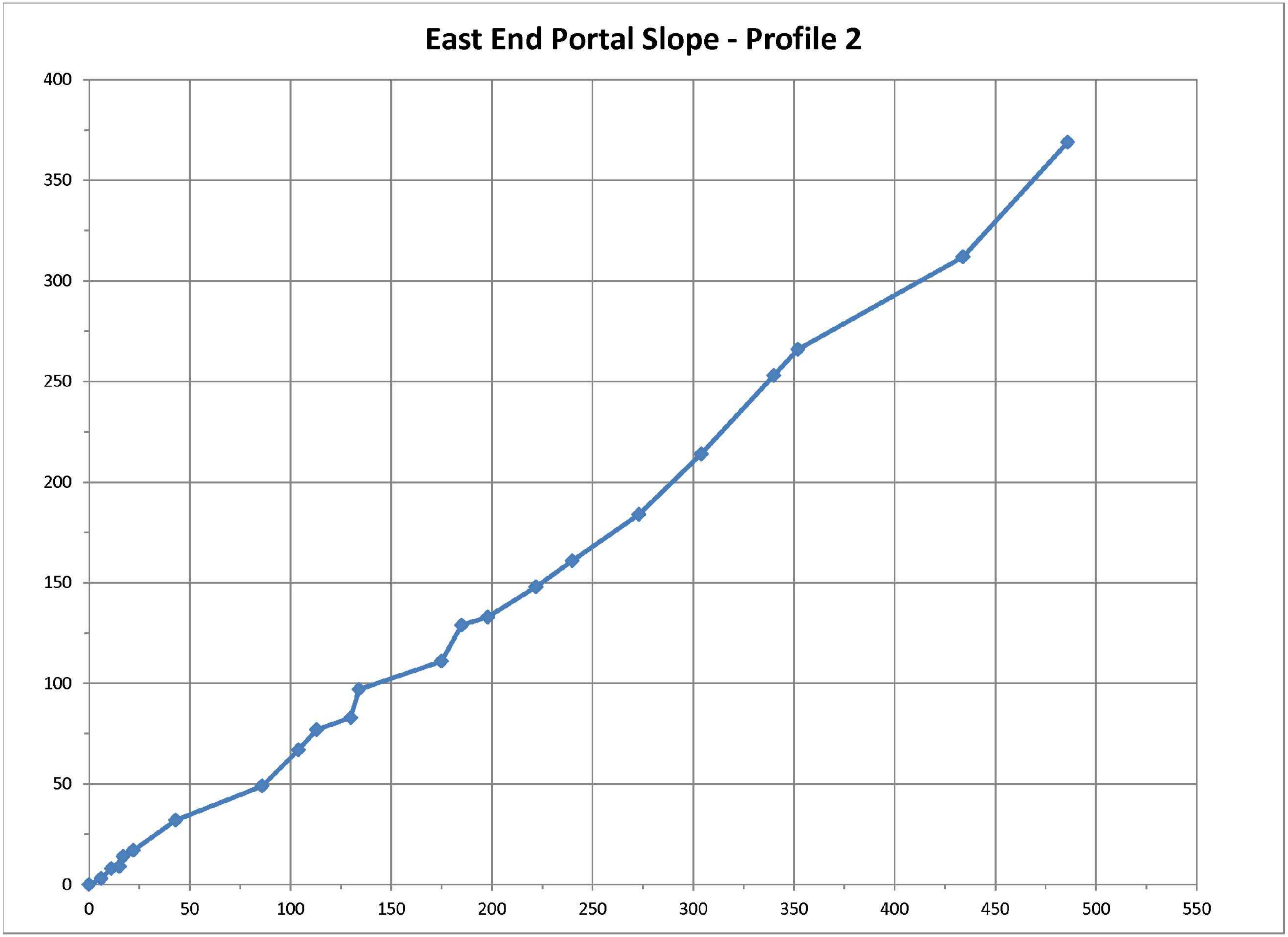

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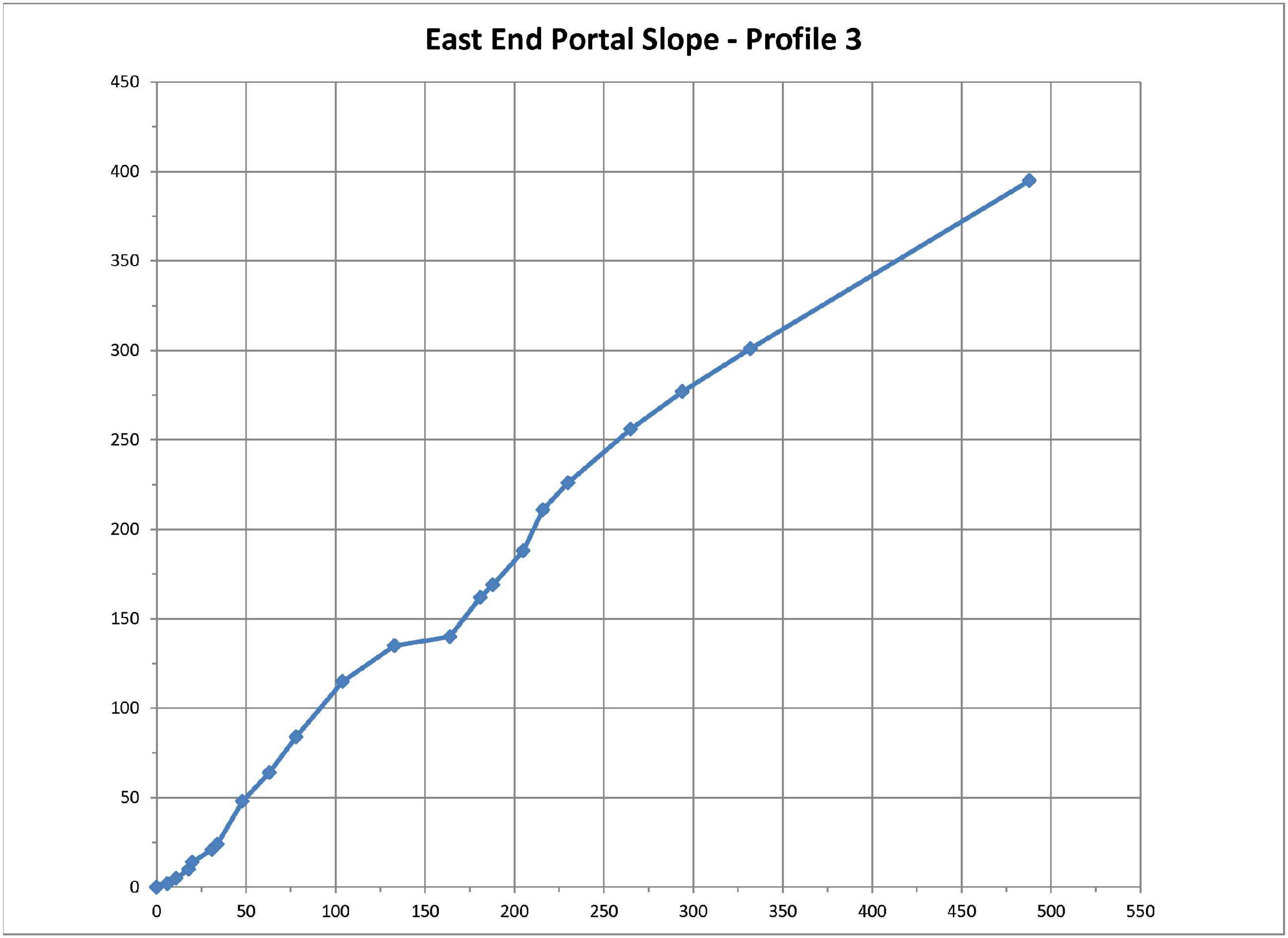
FIGURE
8



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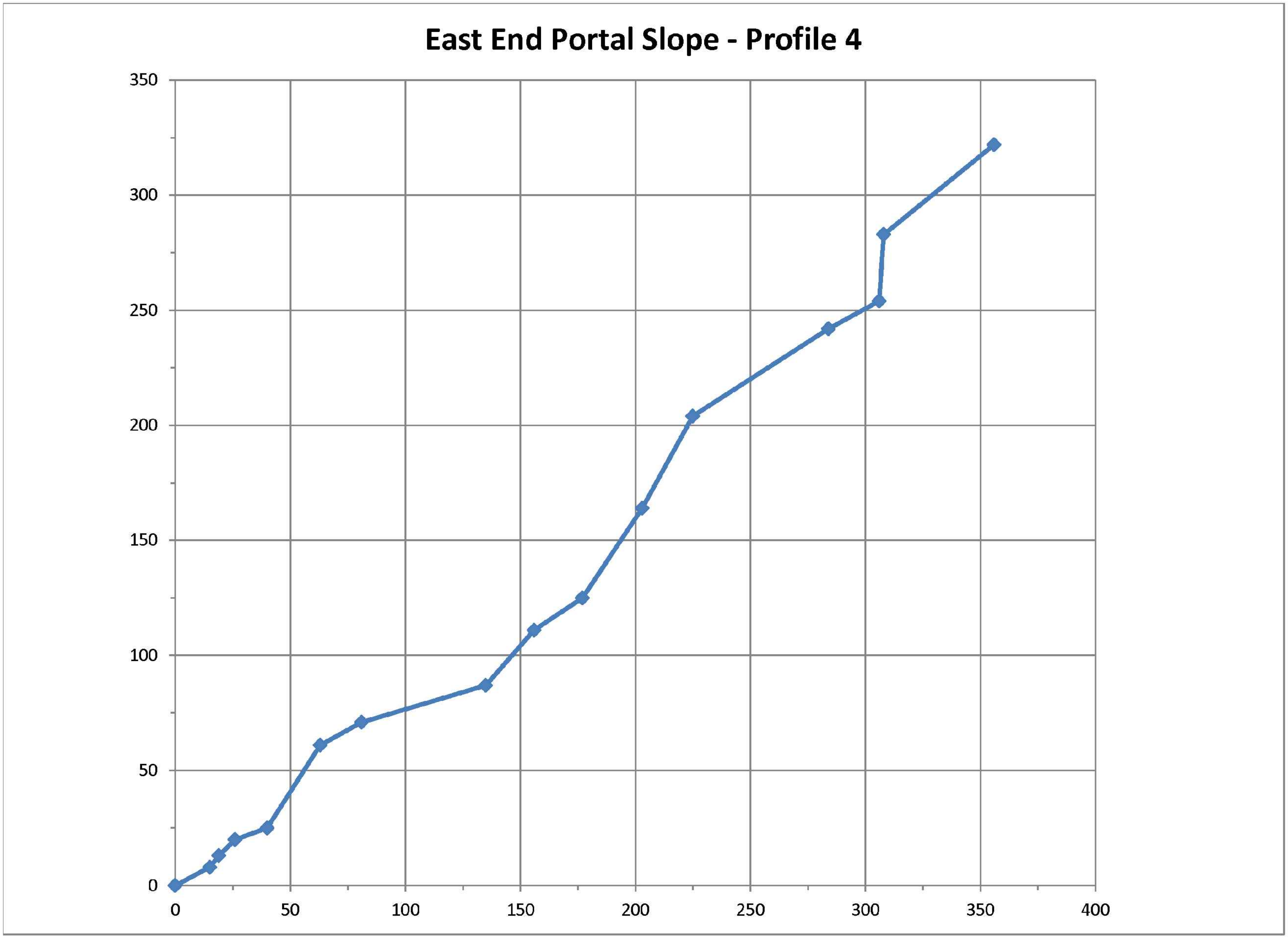
Rock Stabilization Analysis
Elko County, Nevada

FIGURE
9



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Rock Stabilization Analysis
Elko County, Nevada

FIGURE

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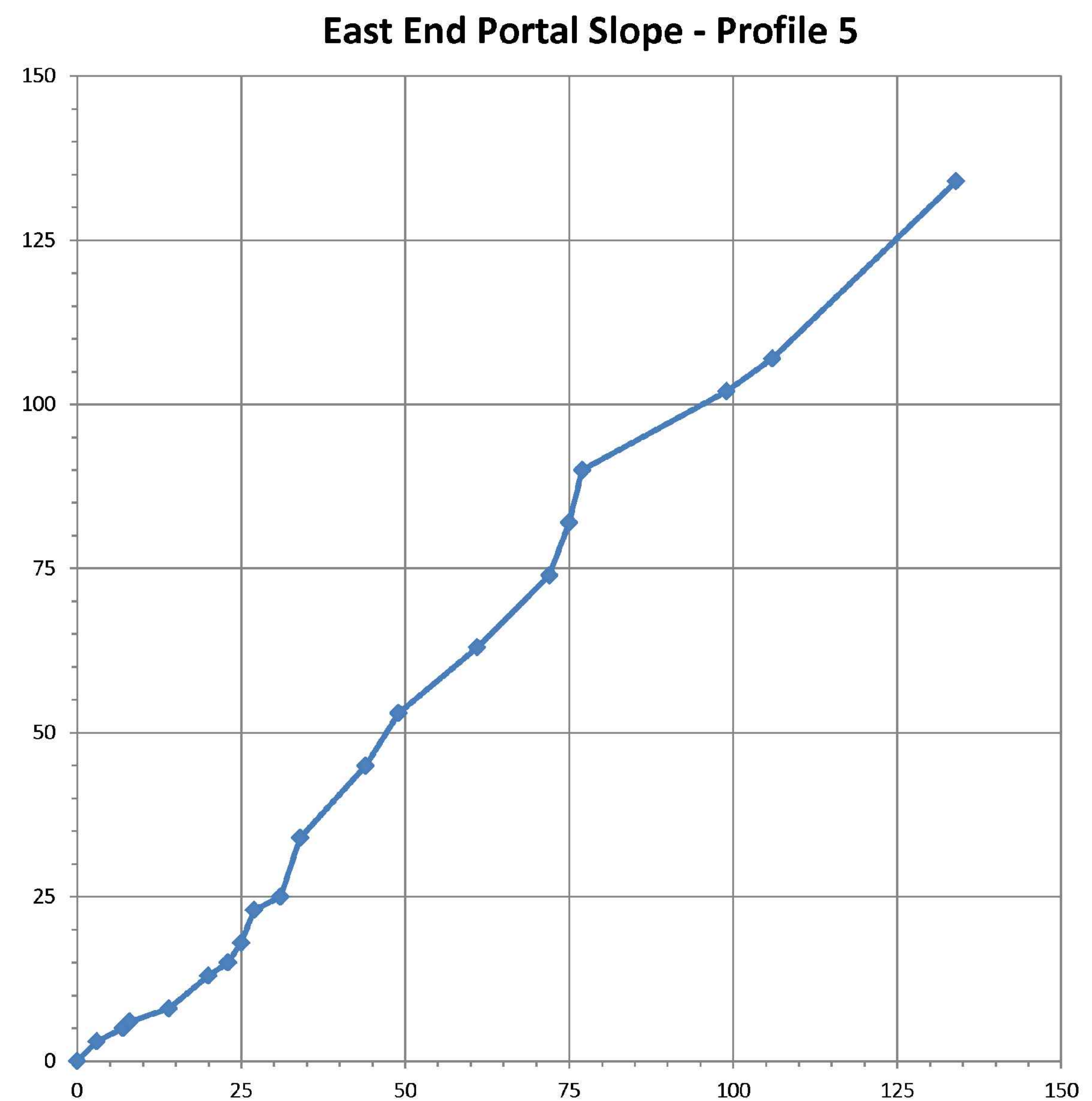

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Rock Stabilization Analysis
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FIGURE
11

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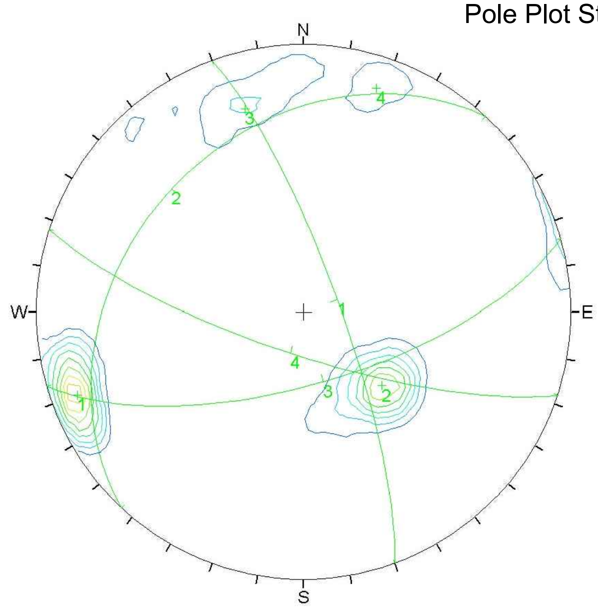
Large Blocks
East End Cut Slope

I-80 Bridge and Tunnel Rehabilitation Scope B
Rock Stabilization Analysis
Elko County, Nevada

FIGURE

12

Pole Plot Stereonet

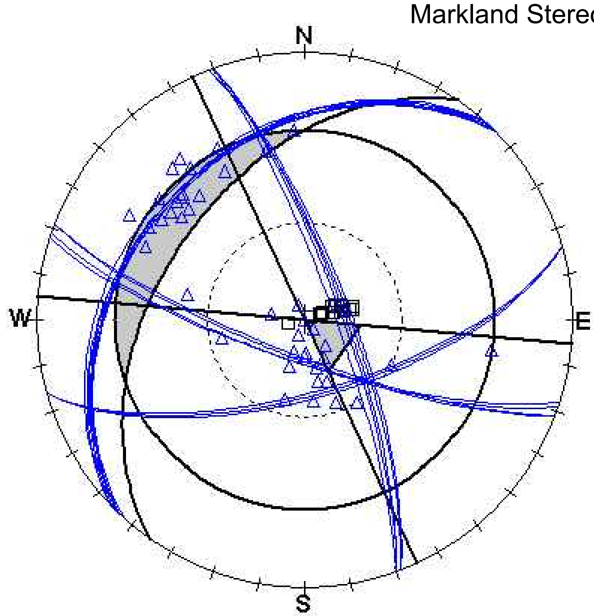


Orientations

ID	Dip / Direction
1	79 / 070
2	33 / 313
3	68 / 164
4	77 / 198

Equal Area
Lower Hemisphere
60 Poles
60 Entries

Markland Stereonet Analysis



Dip Angle = 45deg
Dip Direction = 315deg
Friction Angle = 30deg

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West End Stereonets

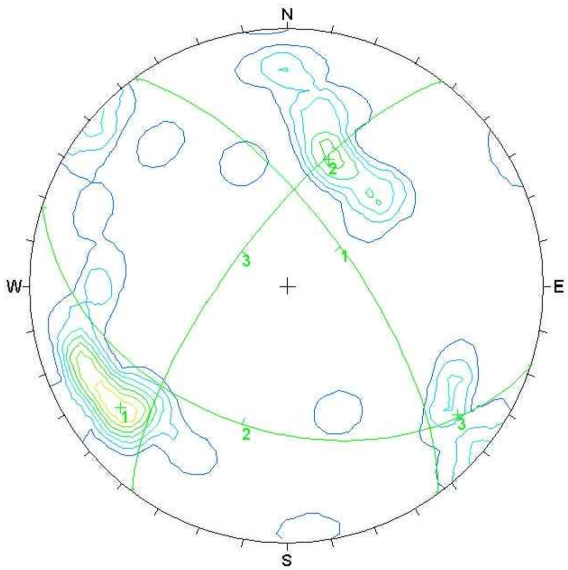
I-80 Bridge and Tunnel Rehabilitation Scope B
Rock Stabilization Analysis
Elko County, Nevada

FIGURE

13

Pole Plot Stereonet

North of Fault Zone

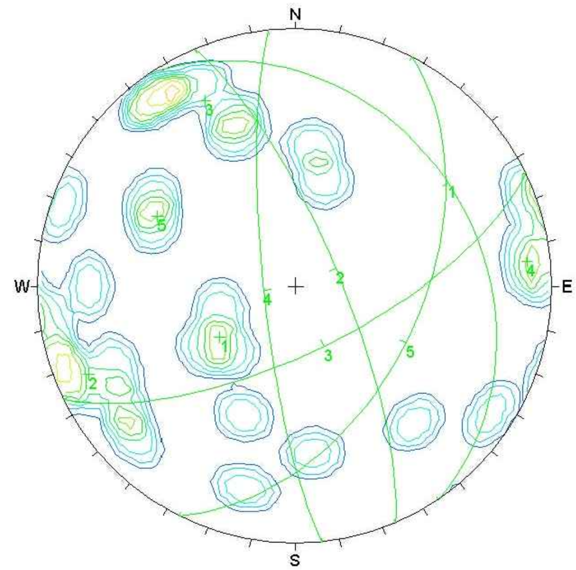


Orientations

ID	Dip / Direction
1	69 / 054
2	43 / 198
3	72 / 307

Equal Area
Lower Hemisphere
40 Poles
40 Entries

South of Fault Zone

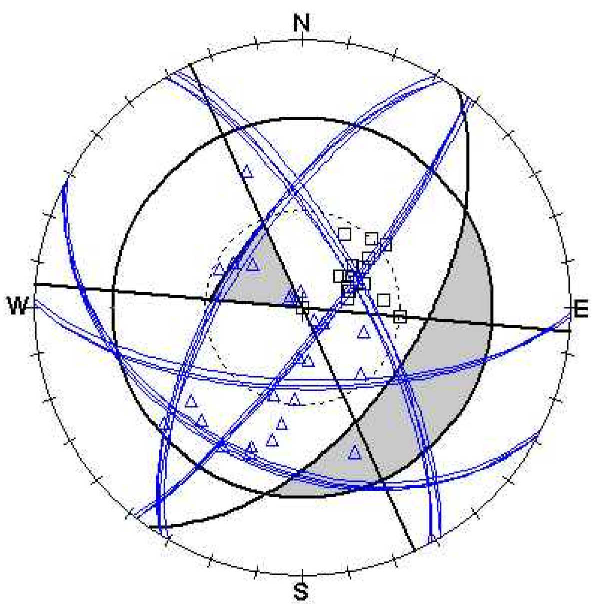


Orientations

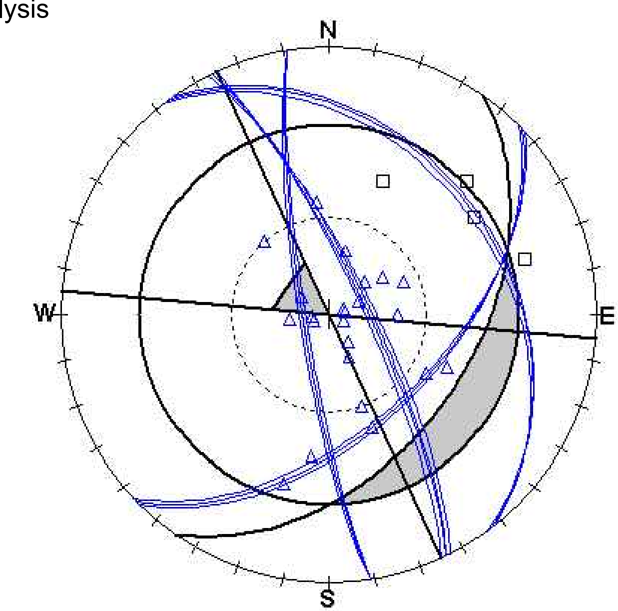
ID	Dip / Direction
1	29 / 056
2	76 / 067
3	69 / 154
4	80 / 264
5	50 / 117

Equal Area
Lower Hemisphere
28 Poles
28 Entries

Markland Stereonet Analysis



Dip Angle = 55deg
Dip Direction = 125deg
Friction Angle = 30deg



Dip Angle = 45deg
Dip Direction = 125deg
Friction Angle = 30deg

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East End Stereonets

I-80 Bridge and Tunnel Rehabilitation Scope B
Rock Stabilization Analysis
Elko County, Nevada

FIGURE

14

APPENDIX A – STEREONET AND MARKLAND ANALYSES

STEREONET KINEMATIC ANALYSIS

GENERAL

Discontinuity: Any plane of weakness in the rock mass including joints, faults, shears, beds, foliation, etc.

Dip: A 2-digit whole number from 0° to 90° indicating the angle of inclination of a planar surface, such as a discontinuity, from horizontal, measured perpendicular to the strike and in the vertical plane.

Dip Direction: 3-digit whole numbers from 0° to 360° indicating direction in which the discontinuity is dipping.

Strike: 3-digit whole number 0°to 360° indicating the direction taken by a structural surface, e.g. a bedding or fault plane, as it intersects the horizontal i.e. the direction of a horizontal line in a planar surface .

Joint Set: A group of approximately parallel joints of similar dip and dip direction.

Bed Set: A group of approximately parallel beds of similar dip and dip direction.

Planar Failure: Planar failures are those in which movement occurs by sliding on a single discrete surface that approximates a plane.

Wedge Failure: Wedge failures occur when rock masses slide along two intersecting discontinuities both of which dip out of the cut slope at an oblique angle to the cut face, forming a wedge-shaped block.

Toppling Failure: Toppling failures most commonly occur in rock masses that are subdivided into a series of slabs or columns formed by a set of fractures that strike approximately parallel to the slope face and dip steeply into the face.

INTRODUCTION

In general, most rock slope failures occur along discontinuities, or planes of weaknesses, within a mass of rock (or rock mass). The most common types of failures include planar, wedge and toppling failures. Stereonets are used to identify the discontinuities that have the kinematic potential to develop into these failures. Furthermore, stereonets can be used to identify optimum slope geometries during a design phase of a project.

The following is a basic explanation on the use of stereonets in identifying potential failure mechanisms in a rock slope. For a more comprehensive discussion of the principles of stereonet analysis for rock slope analysis the work by Hoek and Bray (1981), Hoek and Brown (1980), and Goodman (1976) should be referenced.

STEREONET PROJECTION

Stereonets permit a three-dimensional analysis of discontinuities to be represented and analyzed in two dimensions. This allows the orientations of planes in space to be accurately represented and easily visualized as illustrated by Figure 1A & B. In stereonet analyses, discontinuities are assumed to be planar. There are three possible representations of a plane in space on stereonets. They are poles, dip vectors, and great circles, as illustrated in Figure 1C.

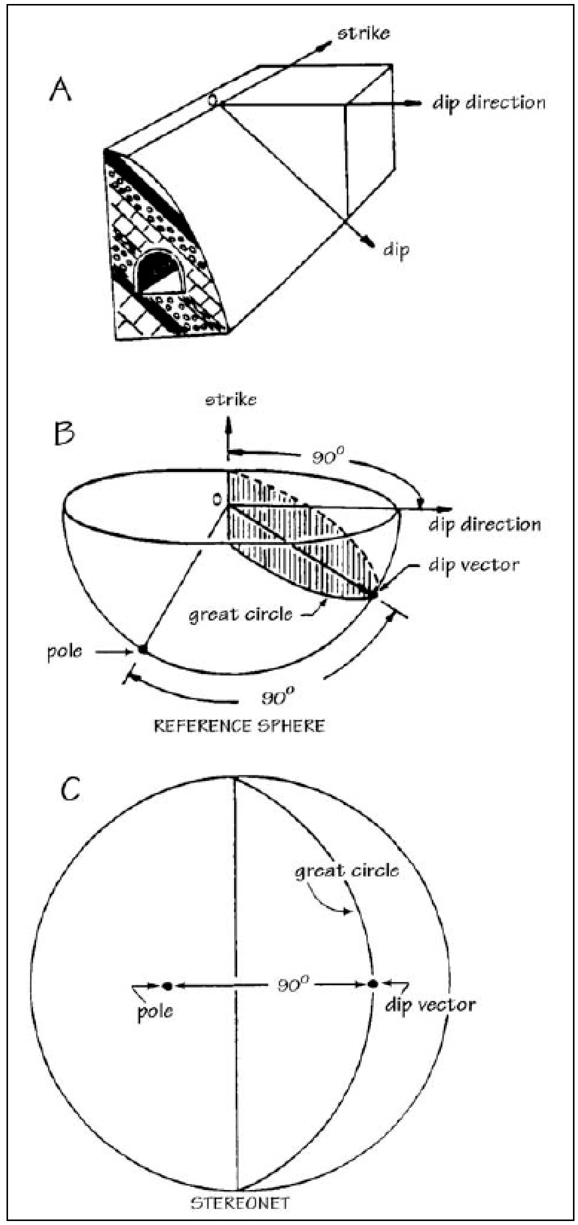


Figure 1. A) Illustration of strike and dip of bedding surface. B) Lower hemisphere illustration of 3 methods for representing a single plane in space. C) Stereonet projection of part B (from Watts, 2003)

Traditionally, geologists have used *poles* to represent planes on an equatorial stereonet. A pole is formed by passing a line perpendicular to the plane through the center of the reference sphere (Figure 1B). The point where the line intersects the lower hemisphere is the pole and is projected upward to the stereonet. The direction of dip of a plane identified by a pole can be established by drawing a line from the pole through the center and on to the opposite side of the stereonet. The location where that line intersects the outside of the stereonet is the orientation of the direction of dip. In other words, on the stereonet the direction of dip of a plane represented by a pole is 180^{o} from the pole. The amount of dip identified by a pole increases the closer it appears to the edge of the stereonet.

A *great circle* is formed by the intersection of the plane in space with the lower half of the reference sphere (Figure 1B). The stereonet projection of this intersection is an arc called a cyclographic trace of the plane, but commonly referred to as a great circle (Marshak and Mitra, 1988).

The *dip vector*, like the pole is a single point, except that it is plotted in the direction of the dip located at the midpoint of the great circle representation of the plane (Figure 1C). The relationship between a pole and dip vector is that they are 90^{o} apart from each other on a stereonet in the orientation of dip of the plane.

A dip vector, conversely from the pole, directly depicts the dip direction and dip value of the plane in space. Dip direction is identified by drawing a line from the center of the stereonet through the dip vector to the edge of the stereonet. For the amount of dip the closer the dip vector is to the center, the steeper the dip.

Each of the representations has its own advantages and uses. Poles and dip vectors are used to represent individual discontinuities as single points, keeping the stereonet less cluttered than if large numbers of great circles are used. On the other hand, great circles are used to represent slope faces so that they stand out clearly and the relationships between them and the individual discontinuities may easily be examined. Also, great circles are useful when representing clusters in wedge analyses as described later.

DISCONTINUITY CLUSTER ANALYSIS

Unless a rock mass is severely fractured, several distinct clusters or discontinuity sets or populations will be obvious when discontinuities are plotted on a stereonet. The orientations of discontinuities in a rock mass are related to its geologic history. Discontinuity sets have the potential to have considerable range in both dip direction and amount of dip. Outlier or random discontinuities will be displayed as individual points. To characterize a discontinuity set contouring of a stereonet is commonly performed. Contouring of a stereonet is a statistical evaluation of the orientations of discontinuities that aids in the identification of the concentrated center and extent of a discontinuity set. An example of stereonet contouring is shown in Figure 2.

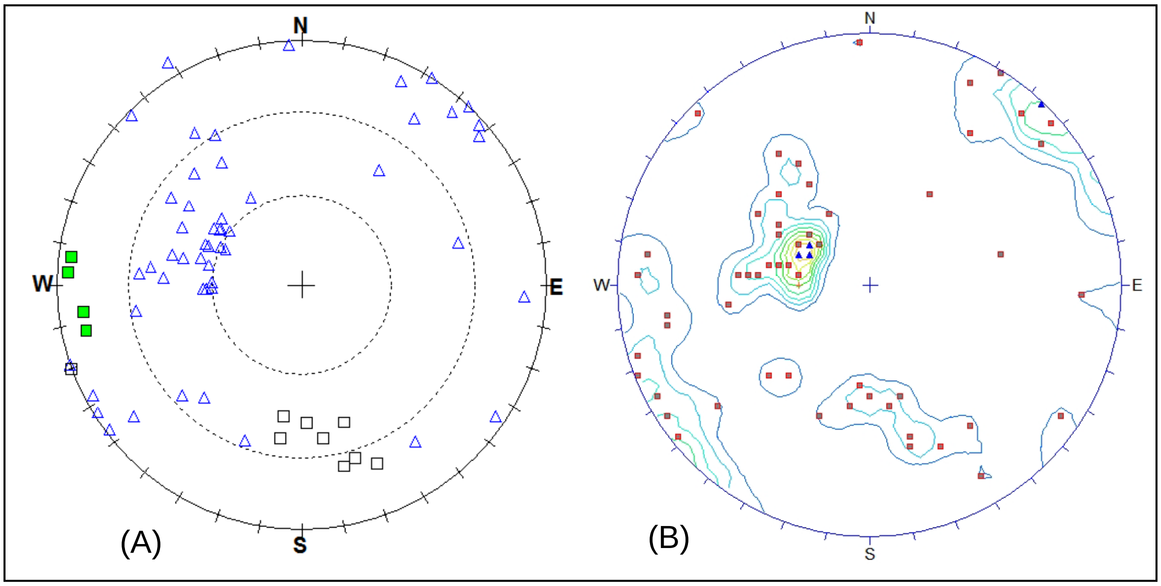


Figure 2. (A) Example of a pole plot stereonet. (B) Data set from example (A) contoured into discontinuity sets.

It should be noted that sliding (or failure) can occur along any of the discontinuities in a discontinuity set. Therefore, while performing a kinematic analysis, discussed later, variation of orientation of a discontinuity set needs to be considered. This can be accomplished by evaluating the entire range of values of a discontinuity set, or utilizing statistical methods, such as assuming that the set is more accurately represented by a range encompassing the mean one standard deviation from the mean to characterize a discontinuity set.

SLOPE GEOMETRY

The geometry of a slope can be described by a dip direction and amount of dip. The simplest manner in which to visualize the geometry of a slope on a stereonet is with a great circle. Figure 3 illustrates a slope orientation with the use of a great circle.

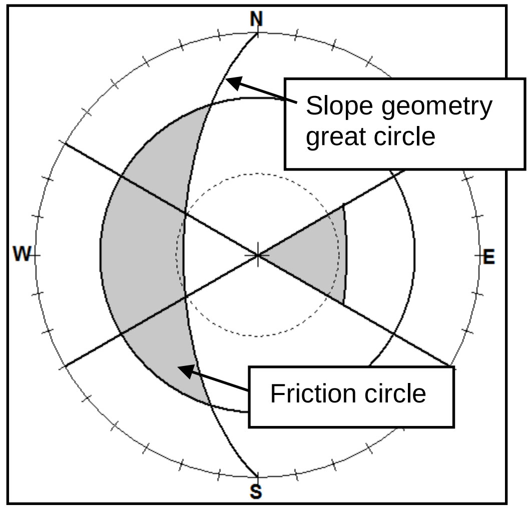


Figure 3. Stereonet illustrating the orientation of the slope face, friction circle, and critical zone.

ANALYSIS ASSUMPTIONS/PROCEDURES

The stereonet procedure is a "cohesion-equals-zero" analysis, in which the effects of cohesion are ignored. When this assumption is made, the fundamental limiting equilibrium equation for calculating a safety factor reduces to:

FS = Tan ϕ'/ Tan θ.

Where ϕ' and θ are the friction angle (or angle of internal friction) and dip of the discontinuity, respectively. The discontinuity frictional angle (ϕ') is represented on a stereonet by placing a circle drawn at a distance measured from the outside of the stereonet equal to ϕ'. Typically the friction angle would represent the mean base friction angle for that set of discontinuities. Figure 4 illustrates this friction circle on a stereonet.

If a discontinuity dip vector dips greater than the friction angle and direction of dip of the discontinuity is nearly parallel with the dip direction of the slope, the discontinuity dip vector will plot within the friction circle and the safety factor against sliding on the discontinuity is considered less than 1.0. Whenever the dip value is less than the friction angle, the safety factor is greater than 1.0 and the dip vector will plot outside of the friction circle on the stereonet.

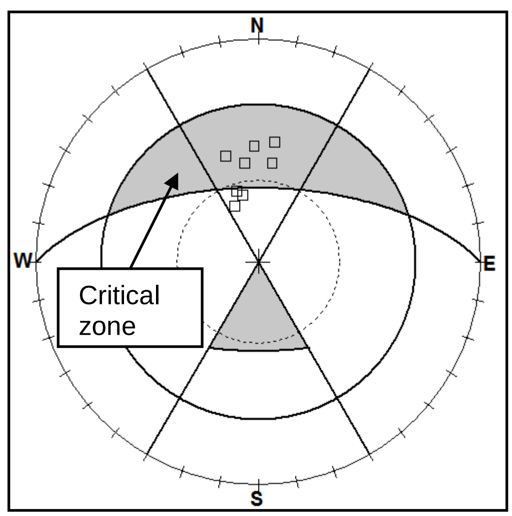


Figure 4. Illustration of kinematically possible planar failure.

EXPLANATION OF MARKLAND TEST THEORY

Markland's test is an extremely valuable tool for identifying those discontinuity populations that could lead to failures in the rock mass and for eliminating other individual discontinuities and population clusters from consideration. However, it should be remembered that not every discontinuity that plots within the critical zone will result in a failure. There are many additional factors that can affect stability along discontinuities. The stereonet analysis is conservative owing to several assumptions that make the analysis possible. To begin, all of the discontinuities are assumed to be continuous and through going, when in reality many of them are not. Even a small percentage of intact rock along a discontinuity can be enough to make it safe from sliding.

The following is a description on how the Markland Analysis is used to identify potential plane, wedge, and toppling failures.

PLANE FAILURE- MARKLAND ANALYSIS

For sliding to occur on a single plane, the following geometrical conditions must be satisfied:

1. The plane on which sliding occurs must strike parallel or nearly parallel (within approximately ± 20^{o}) to the slope face,

2. The failure plane must "daylight" in the slope face. This means that its dip must be less than the dip of the slope face,

3. The dip of the failure plane must be greater than the angle of friction of this plane. and

4. Release surface which provide negligible resistance to sliding must be present in the rock mass to define the lateral boundaries of the slide.

The first three conditions described above can be represented on a stereonet in the form of a crescent-shaped zone (Figure 4), commonly referred to as the "critical zone". For discontinuity dip vectors, which lie within the critical zone, dip more steeply than the friction angle of the rock because they are inside the friction circle. They dip less steeply than the slope face because they lie outside the great circle representing the slope face.

To easily identify those dip vectors that occur within approximately ±20° of the slope face, reference lines are placed on the stereonet to aid in visualization. In the examples shown, the lines are drawn at ±30°of the slope face dip direction.

WEDGE FAILURE – MARKLAND ANALYSIS

Stereonet analyses for potential wedge failures are similar to stereonet analyses for plane failure. For a wedge failure to occur, the line made by the intersection of the planes creating the wedge must plunge more steeply than the friction angle and less steeply than the dip of the slope face and in a direction such that it daylights from the slope face.

To test for these conditions, a single great circle may be chosen and plotted on the stereonet to represent the discontinuities of each cluster. If any of the great circles representing clusters on the stereonet intersect within the crescent-shaped critical zone then the conditions are met and a wedge failure is kinematically possible (Figure 5). The intersection point provides the plunge and trend of the line of intersection and is read from a stereonet in the same manner as dip and dip direction.

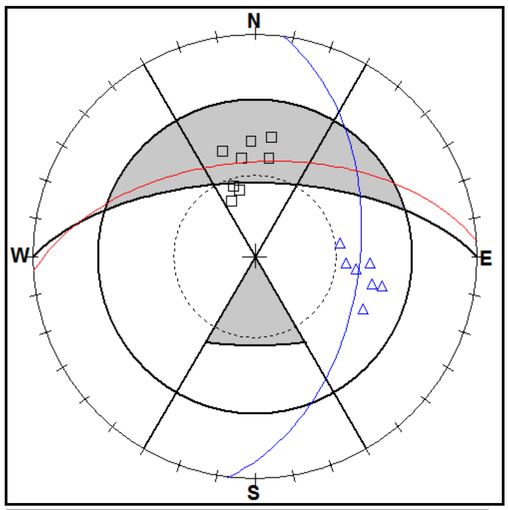


Figure 5. Intersection of great circles indicating potential wedge failure.

General conditions for wedge failures to be kinematically possible include:

1. The dip direction of the intersection of the discontinuities must approximate the dip direction of the rock cut slope (fall within the critical zone),

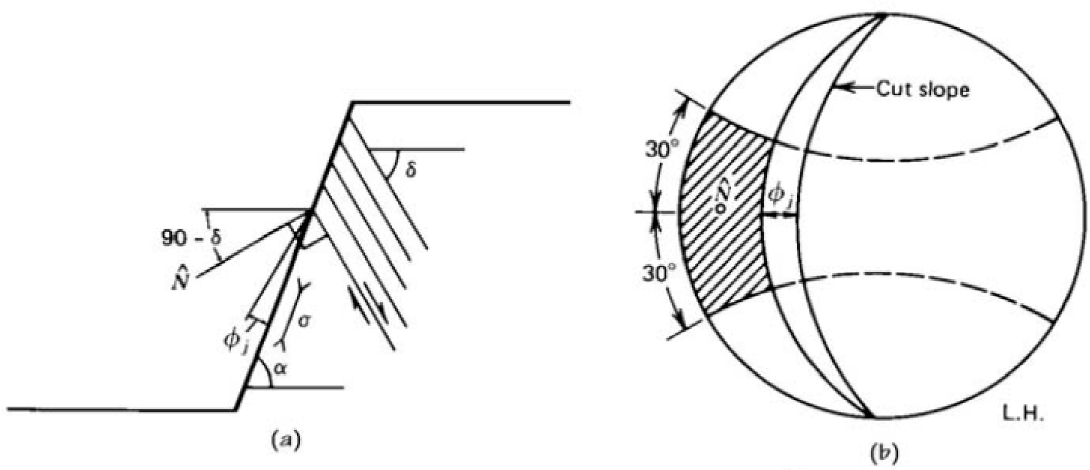
2. The plunge of the line of intersection of the discontinuities must be less than the dip of the rock cut slope,

3. The plunge of the line of intersection of the discontinuities must be greater than the friction angle.

Note: If the intersections of the great circles are close the boundary of the critical zone and the cutslope face, the wedges will be small and typically represent random rockfall and raveling. In contrast, if the intersections of the great circles are close to the outer boundary or frontier of the critical zone, then it is kinematically possible for a large wedge failure.

TOPPLING FAILURES – MARKLAND ANALYSIS

Goodman (1980, p. 265) discusses a stereonet procedure for kinematically identifying potential toppling failures. He states that interlayer slip must occur before large flexural deformations can develop. If the interlayer slip is controlled by the friction angle ϕ' toppling will occur if the normals to the toppling layers are inclined less steeply than a line inclined ϕ' degrees above the plane of the slope. The zone in which normals meet that condition is illustrated in Figures 6 (for a pole plot stereonet). In addition, toppling



Kinematic test for toppling. (*a*) $(90 - \delta) + \phi_j < \alpha$. (*b*) $\hat{N}$ must plot in the shaded zone.

Figure 6. Kinematic analysis of toppling. Taken from Goodman (1980).

will occur only if the layers strike nearly parallel to the strike of the slope, typically within 30^o. When using a dip vector stereonet, the shaded area for checking for potential toppling failures is rotated 90 degrees from that shown in Figure 6. Figure 7 is a stereonet in which shows the triangular and shaded toppling critical zone for discontinuities plotted as dip vectors. Discontinuity populations within the critical zone are subject to toppling. Note, these discontinuity populations can also act as release planes for the planar and wedge conditions above, assuming there is a population within the critical zone for planar or wedge failure conditions (Figures 4 and 5).

General conditions for flexural toppling failure include:

1. The strike of the discontinuities fall within 30° of strike of the rock cut slope.

2. Normal to the toppling plane (θ) must display plunge less than the dip of rock cut slope (ψ) and less than the friction angle (90°- θ) = (ψ - ϕ + 90°).

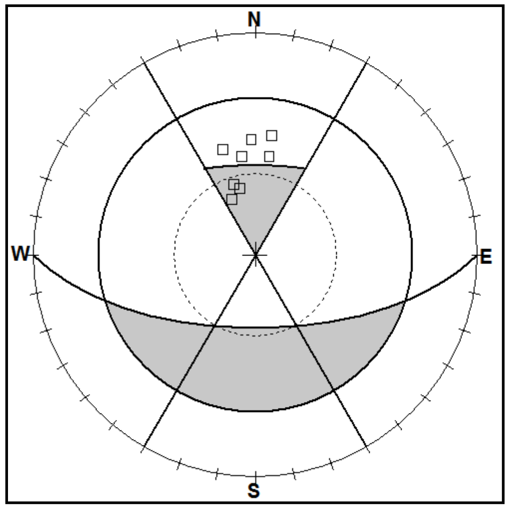


Figure 7. Check for toppling potential