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GEOTECHNICAL ENGINEERING SERVICES
PROPOSED SCA #2 ASH LANDFILL
SUNNYSIDE COGENERATION FACILITY
CARBON COUNTY, UTAH

PSI PROJECT No: 0710281

April 6, 2012

Prepared For:

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1.0 INTRODUCTION

This report presents the results of PSI's geotechnical study completed for the proposed SCA #2 Ash Landfill in Carbon County, Utah. The purpose of our subsurface explorations and geophysical surveys was to characterize the subsurface profile of the site in order to evaluate the global and local slope stability of the proposed ash landfill. This report provides an evaluation of existing groundwater conditions as well as geotechnical recommendations regarding erosion control and construction considerations for the proposed ash landfill. The study was performed in general accordance with PSI's proposal (PSI Proposal No. 57449r1) dated November 14, 2011 and authorized by Mr. Rusty Netz with Sunnyside Cogeneration on November 14, 2011.

2.0 PROJECT DESCRIPTION

Based on the information provided to PSI by Twin Peaks, Sunnyside Cogeneration Associates (SCA) is considering the construction of a new ash landfill to be located in a side canyon just southeast of the existing SCA facility. The site is located to the southeast of the SCA Refuse Pile in portions of the NE ¼ of Section 7 & NW ¼ of Section 8, Township 15 South, and Range 14 East in West Carbon, Utah. Additional access to the site will be required across portions of the SW ¼ of Section 5, Township 15 South, Range 14 East.

The proposed landfill concept is currently designed to hold approximately 2.7 to 3.2± million cubic yards of combined ash materials. The landfill will be constructed using similar ash placement methods as what is currently being used on the SCA #1 ash landfill located about two (2) miles southwest of the project site. PSI understands that the landfill will be constructed by placing ash from the bottom (toe) upward to the top of the landfill. SCA has indicated a plan to place the ash in cells in a terraced configuration with terraces being 30 to 40 feet high with 2.5 horizontal to 1 vertical faces. Each terrace will be set back approximately 15 feet from the previous terrace to form a bench. This process will then continue until completion of the landfill over a period of approximately eight to twelve years. Calculations performed as part of this geotechnical study indicate that a landfill configuration as described above would be stable and appropriate for the site. Conceptual designs indicate diversion of periodic surface water at the top of the landfill, away from the placed ash materials.

2.1 Previous Geotechnical Data

Twin Peaks provided PSI with a copy of the following documents for reference prior to the start of our study, including a copy of the geotechnical scope of work and engineering report for the existing SCA #1 ash landfill:

- Executive Summary, Proposed SCA #2 Ash Landfill, Sunnyside Cogeneration Facility, prepared by Twin Peaks;
- Portions of the Sunnyside Cogeneration Facility Geotechnical Report, prepared by others;
- January 1997 Engineering Report, Phase 2 of the SCA #1 Ash Disposal Facility, Permit No. UGW070002, Sunnyside Cogeneration Associates, prepared by others;
- Geotechnical Scope of Work for the existing SCA #1 ash landfill site, prepared by others;
- January 1997 Hydrologic Characterization, Sunnyside Ash Landfill Expansion, prepared by Maxim Technologies;

3.0 SITE DESCRIPTION

Based on a review of the provided information and available geologic maps for the site, PSI understands the proposed landfill site is underlain by colluvial and alluvial deposits. PSI also understands that some combined coal mine refuse material was previously placed in the northeast portion of the proposed landfill site from the late 1980s to the early 1990s.

The site of the proposed ash landfill is currently undeveloped with surface vegetation consisting of pinyon pine, juniper, Salina wildrye, Indian ricegrass, black sagebrush and birchleaf mountain mahogany with gravel, rock and boulders at the surface. Steeper areas of the canyon contain mostly rock outcroppings. Annotated photographs of the project site are included in Appendix A.

3.1 USGS Topographic Map

The topographic survey map published by the USGS entitled, “Sunnyside, Utah” was reviewed for ground surface features in the area of the proposed ash landfill. Based on this review, the natural ground surface elevations in the project vicinity range from 6400 to 6720 feet National Geodetic Vertical Datum (NGVD) of 1929. The site slopes down from a high in the northeast to a low in the southwest with a central canyon running down the center. This is reasonably consistent with the site-specific topographic information provided to PSI for use in our study. A depiction of the USGS map for the project vicinity is included on Figure A-1 in Appendix A.

3.2 USDA SCS Soil Survey

The “Soil Survey of Carbon Area, Utah,” (June 1988 edition) published by the United States Department of Agriculture Soil Conservation Service, was reviewed for general near-surface soil information within the general project vicinity. This information indicates that there are four(4) soil groups within the vicinity of the proposed project. The mapped soil units are summarized in the following, as provided by the SCS and also depicted on Figure A-2 in Appendix A.

Gerst-Strych-Badland complex, 3 to 50 percent slopes (Soil Group No. 36) is found on mountain slopes and the toe of mountain slopes between elevations of 6,100 feet and 7,200 feet. The Gerst portion of the complex consists of shallow well drained soil formed in residuum and colluvium mainly from sandstone and shale. Strych soil is very deep and well drained and is formed in alluvium derived predominantly from sandstone and shale. The Badland portion of the complex consists of steep and very steep nearly barren areas of shale that are dissected by many intermittent drainageways.

Strych very stony loam, 3 to 15 percent slopes (Soil Group No. 113) consists of very deep, well drained soil found on benches and outwash plains. Specifically, it is found in the mouths of canyons near Helper and Sunnyside and south of Pace Canyon and along the north fork of Gordon Creek, near Cedar Bench. This soil is formed in glacial outwash and alluvium derived from sandstone and shale.

Strych very stony loam, dry, 3 to 30 percent slopes (Soil Group No. 114) consists of very deep, well drained soil found on alluvial fans and terraces. Specifically, it is found at the foot of Book Cliffs, extending from Horse Canyon to the town of Wattis. This soil is formed in glacial outwash and alluvium derived from sandstone and shale.

Travessilla-Rock outcrop-Gerst complex (Soil Group No. 121) is found on canyonsides in the area of Jack Creek and along the Book Cliffs, extending from Price Canyon to Sunnyside. Travessilla soil is a shallow well drained soil formed in residuum and colluvium derived predominantly from sandstone and shale. The Gerst portion of the complex consists of shallow well drained soil formed in residuum and colluvium mainly from sandstone and shale. The Rock outcrop portion consists of areas of exposed sandstone and siltstone.

The SCS indicates the majority of the site to be mapped as Soil Group Nos. 36 (*Gerst-Strych-Badland complex, 3 to 50 percent slopes)* and 121 (*Travessilla-Rock outcrop-Gerst complex*).

4.0 FIELD EXPLORATIONS

4.1 Borings

Subsurface conditions at the project site were investigated by drilling two (2) borings designated as B-1 and B-2 on Figure A-2 in Appendix A. Boring B-1 was completed using ODEX drilling to a depth of approximately 50 feet. ODEX drilling consists of drilling with a cased downhole hammer. Upon completion of boring B-1 a permanent well was installed in the borehole to observe the groundwater level. The well was constructed of two inch PVC pipe with a 0.010 inch slotted screen pipe over the bottom 20 feet. The well was backfilled with 20/30 silica sand to a few feet below the ground surface followed by a bentonite seal and a concrete pad and manhole cover at the surface (see well log on Boring B-1). Boring B-2 was drilled to a depth of approximately 33 ½ feet below the existing ground surface using continuous flight hollow-stem auger drilling techniques to advance the boring. Practical auger refusal was encountered on cobbles and boulders at a depth of approximately 33 ½ feet in boring B-2.

The borings were located on the site by a member of our geotechnical staff using aerial photographs and project plans provided to PSI by the client. Drilling and sampling were performed under the direction of a PSI geotechnical engineer who maintained detailed logs of the subsurface materials and conditions encountered in the borings, and collected representative samples.

Samples of the soil were obtained at approximate 2½ to 5-foot intervals in the borings by driving a standard 2-inch (O.D.) split-spoon sampler into the soil a distance of 18-inches using a 140-lb hammer dropped from a height of 30-inches. The number of blows required to drive the sampler the last 12 inches is referred to as the standard penetration resistance, or N-value. The N-values provide a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as clays or silts. The samples were transported to our laboratory for further examination and testing. Boring B-2 was backfilled with the auger cuttings upon completion of drilling.

4.2 Exploratory Excavations

Exploratory test pits TP-1, TP-2, TP-3 and TP-4 were excavated using a client-provided trackhoe. The purpose of the exploratory excavations was to observe the near-surface soil conditions, coal refuse thickness (if any) and depth to the bedrock. The exploratory excavations were backfilled with the excavated on-site soils. The test pit backfill materials were not compacted. The approximate locations of the exploratory excavations are shown on Figure A-2 in Appendix A.

4.3 Geophysical Study

In order to further define the depth of overburden and the underlying bedrock and to assist with the generation of slope cross sections, PSI conducted Refraction Microtremor (ReMi®) testing along three profile line arrays within the area of the proposed ash landfill. A description of the ReMi® testing method is discussed in the following section. The approximate locations of the profile line arrays are shown on Figure A-2 in Appendix A.

4.3.1 Refraction Microtremor (ReMi®) Testing

The ReMi® method uses standard seismic refraction equipment and records microtremors (or background noise) in the area as the source. PSI performed ReMi testing along the profile line arrays as shown on the attached site plan, Figure A-2. Ambient background noise activities provided sufficient “noise energy” for the measurements. The background noise generates surface waves (including Rayleigh wave energy) that are detected and recorded by the twenty-four (24) channel geophone array.

The maximum depth of sampling using the ReMi® method is a function of the array length and the subsurface velocities. Once collected, the data was checked for accuracy and fidelity. Multiple data samples were recorded for each array at the site. To assure a robust profile is being made, both individual recordings and multiple summed recordings were evaluated. The first step in data reduction was to produce a velocity spectrum of the recorded data. This process involves computing a surface wave phase velocity dispersion spectral ratio image using p-tau (slant spectra) and Fourier transforms across the array. This process is described in Louie, 2001. The resulting spectrum is in the slowness-frequency (p-f) domain. The p-f transformation helps segregate the Rayleigh wave arrivals from the other P and S seismic wave arrivals.

The normal mode dispersion can be distinguished from the aliasing and wave-field transformation truncation artifact trends in the spectra. Picking of the surface wave dispersion curve is done along the envelope of the lowest phase velocities. The data processing includes interactive forward modeling of the normal mode dispersion data using the picks from the p-f plots. The modeling process iterates on phase velocity at each period (or frequency), to provide a shear velocity profile as a function of depth beneath the site. The process and resulting velocity profiles are able to identify velocity inversions within the subsurface profile, which allow multiple subsurface soils or bedrock layers.

The resulting shear wave velocity profiles were used to assist in differentiating between the overburden soil deposits and the underlying bedrock. The ReMi® average shear wave velocity profile plot and the supporting documentation are shown on Figures C-1, C-2, and C-3 in Appendix C for the data lines collected at the site as depicted on Figure A-2.

5.0 LABORATORY TESTING

Representative samples of the native soils and waste ash (combined fly ash/bed ash material) were tested to evaluate physical and engineering properties. The laboratory testing program of the native soils included natural water content, percent retained on the No. 4 sieve (gravel content), and percent passing the No. 200 sieve (fines content). Testing on samples of the combined ash materials included direct shear testing, unconfined compressive strength, moisture-density relationship testing, and sieve analysis.

A summary of the laboratory test results is included in Table 1 below. For detailed descriptions of the soils and conditions observed in borings and test pits, please refer to the boring logs, Figures B-2 through B-7 in Appendix B.

Table 1: Summary of Laboratory Test Results

Material Description	Water Content (%)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Internal Friction Angle (Φ)	Gradation		
					Gravel (%)	Sand (%)	Silt/Clay (%)
Carbon Refuse	5-9	-	-	-	10-22	37-52	26-53
Sandy Silt (ML)	9	-	-	-	13	32	55
Silty sand with gravel (SM)	5-7	-	-	-	26-35	32-38	33-38
Silty gravel with sand (GM) / (GP-GM)	2-5	-	-	-	40-76	15-30	9-31
Bulk combined ash sample from stockpile	-	88	24	32	2	50	48

5.1 Unconfined Compressive Strength Tests on Ash Material

A bulk sample of the combined ash material was moisture conditioned and placed in a total of three (3) 4x8 inch cylinder molds. The cylinder molds were then placed in a low temperature drying oven (105^{o} F to 100^{o} F) for a period of three days and then taken out of the oven to continue drying for another two days. The samples were then removed from the molds and prepared for the unconfined compression tests on day five at which time they were broken.

The primary purpose of the unconfined compressive strength test is to obtain the undrained compressive strength of soils that possess sufficient cohesion to permit testing in the unconfined state. Unconfined compressive strength is the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test. In this test method, unconfined compressive strength is taken as the maximum load obtained per unit area or the load per unit area at 15 percent axial strain, whichever is obtained first during the performance of a test. For the unconfined compressive strength test, the shear strength is calculated to be half of the compressive stress at failure. Based on the results of our tests, the unconfined compressive strength of the ash material ranged from 40 to 48 psi (5760 to 6910 psf). The test results are presented on Figure B-11 in Appendix B.

6.0 SUBSURFACE CONDITIONS

6.1 Soil and Bedrock

The subsurface soil and bedrock observed in the borings and exploratory excavations generally consist of alluvial and colluvial materials (silty sands with gravel and silty gravel with sands) underlain by lean clays and sandy silt with cobbles and boulders. The soils are underlain by a relatively impervious layer of shale bedrock. The depth to the shale bedrock varied from approximately 14 to 50 feet below the existing grades in the areas explored. The depth to bedrock generally was shallower in the southeast portion of the site in the lower elevations and in the steeper portions of the canyon. Combined coal mine refuse material was encountered to a depth of approximately 29 feet in boring B-2 and to a depth of approximately 6 feet in exploratory excavation TP-1. PSI was informed that this material had been placed in the northeast portion of the site from the late 1980's to the early 1990's.

Standard Penetration resistance, N-Values, ranged from approximately 32 to greater than 50 blows per foot in the overburden soils and greater than 50 blows per foot in the shale bedrock.

The subsurface profile described is a generalized interpretation provided to highlight the major subsurface stratification features and material characteristics. The boring and exploratory excavation logs included in Appendix B should be reviewed for more specific information. These records include soil description, stratifications, standard penetration resistances, location of samples, and laboratory test data. The stratifications shown on the logs represent the soil conditions only at the boring or exploratory excavation locations. The stratifications indicated on the logs represent the approximate boundary between subsurface materials. The actual transitions may be gradual. Subsurface materials and conditions may vary across relatively short distances at the site and may become apparent with additional explorations or excavation. If soil conditions are found to be different than described herein, we should be allowed to reevaluate our recommendations if necessary.

6.2 Groundwater

Groundwater was encountered in boring B-1 at a depth of approximately 20 feet below existing grades. Groundwater was not observed in Boring B-2 or the exploratory excavations during drilling/excavation operations. Groundwater is expected to remain 10 feet or more below the ground surface in the vicinity of the landfill and not anticipated to come into contact with any ash materials. Similarly, the groundwater is expected to remain perched atop the shale bedrock as it moves in a general northeast to southwest direction.

7.0 GEOTECHNICAL RECOMMENDATIONS

7.1 Earthquake and Seismic Design Considerations

A search of the U.S. Geological Survey National Earthquake Hazard Reduction Program (NEHRP) database resulted in the following probabilistic ground motion values at the bedrock elevation for the project site located at latitude 39.5399° and longitude -110.3806°.

Table 2: Seismic Parameters

Period (seconds)	2% Probability of Exceedence in 50 years (%g)	10% Probability of Exceedence in 50 years (%g)	Max. Spectral Acceleration parameters	Design Spectral Acceleration parameters	
PGA	0.159	0.065	---	---	
0.2 (S_s)	0.312	0.144	S_{ms} = 0.312	S_{Ds}= 0.208	T_0= 0.066
1.0 (S_1)	0.104	0.051	S_{m1} = 0.104	S_{D1}= 0.069	T_s= 0.332

$S_{ms} = F_a S_s$ $S_{Ds} = ⅔ * S_{ms}$ $T_0 = 0.2 * S_{D1}/S_{Ds}$

$S_{m1} = F_v S_1$ $S_{D1} = ⅔ * S_{m1}$ $T_s = S_{D1}/S_{Ds}$

7.2 Slope Stability Analysis

Based on the information obtained from the site topography, subsurface evaluation, geophysical study (ReMi), site reconnaissance and other information from available geologic maps, cross-sections were developed for use in the slope stability analyses shown in Appendix D. For this report a total of six (6) cross-sections were developed to model the long term global stability of the overall landfill design, the intermediate stability during construction and to evaluate the local shorter term stability of the ash benches that will be used throughout the construction phases of the landfill.

The slope stability analyses were performed with GSTABL7. The Pre and Post processor STEDwin v.2 was used to develop the cross sections for analyses. The computer output associated with the stability analyses are presented in Appendix D.

The calculation of the factor of safety against instability of a slope was performed using either the simplified Bishop method, applicable to circular shaped failure surface or the simplified Janbu method, applicable to failure surfaces of general shape. GSTABL7 features unique random techniques for generation of potential failure surfaces for subsequent determination of the more critical surfaces and their corresponding factors of safety. Circular, sliding block or more general irregular surfaces may be generated and analyzed using random search techniques or specific input of the coordinates of a given potential failure surface. The program is coded to handle heterogeneous soils systems, anisotropic soil strength properties, static groundwater and surface water, pseudo-static earthquake loading, surcharge boundary loads, tieback loading and geogrids

The Modified (or Simplified) Bishop's Method is a method for calculating the stability of slopes. It is an extension of the Method of Slices. By making some simplifying assumptions, the problem becomes statically determinate and suitable for hand calculations:

- forces on the sides of each slice are horizontal

The method has been shown to produce factor of safety values within a few percent of the "correct" values.

$$F = \frac{\sum[\frac{c' + ((W/b) - u)\tan\phi'}{\psi}]}{\sum[(W/b)\sin\alpha]}$$

where

$$\psi = \cos\alpha + \frac{\sin\alpha\tan\phi}{F}$$

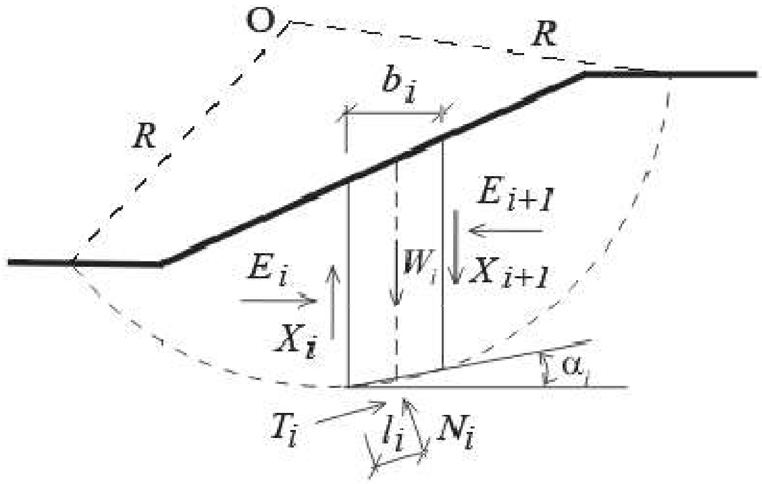
c' is the effective cohesion

φ' is the effective internal angle of internal friction

b is the width of each slice, assuming that all slices have the same width

W is the weight of each slice

u is the water pressure at the base of each slice



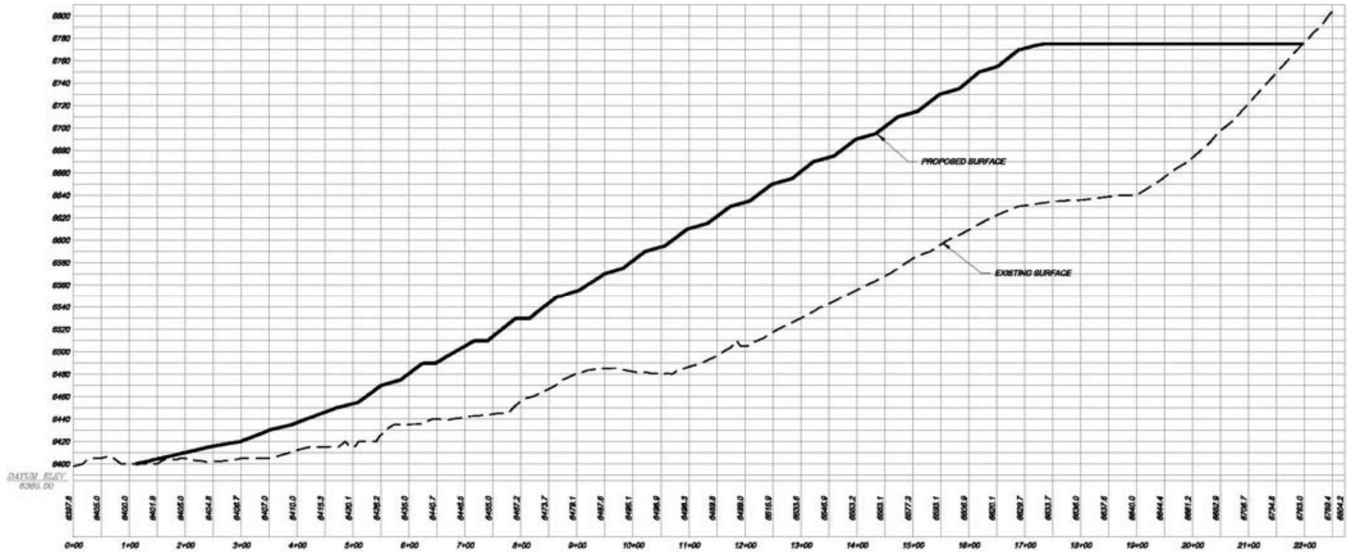
The modified Bishop slope stability method is a procedure generally grouped with a methodology know as the method of slices. In this methodology, a two-dimensional portrayal of the slope being analyzed is divided into a series of slices and the forces of each slice are summed with the resulting resisting forces divided by the driving forces generating the factor of safety. The methodology also factors the shear strength parameters by a trial factor of safety and an iterative solution applied until the resulting factor of safety equals the trial factor of safety.

7.2.1 Global Long Term Stability Analyses

The engineering cross-section for the long term global stability model consists of combined fly ash / bed ash material underlain by predominately colluvial and alluvial deposits and shale bedrock. Groundwater elevations for the analyses were considered to be in a perched condition over the shale bedrock.

A sliding block analysis (simplified Janbu method) and circular analysis (Modified Bishop) were used to model the long term global stability of the model. Based on the information obtained from the site topography provided, subsurface soil and groundwater information, and geophysical testing, Cross Section E-E shown below was used in the analyses. A copy of the Conditional Use Permit Submittal which also includes Section E-E is included in Appendix A. It should be noted that Section E-E was based on a conceptual plan prepared prior to our geotechnical study and will likely be updated with new terrace height/width parameters for the final design plans.

Section E-E



Soil and Bedrock Strengths

SPT blow count correlations, laboratory test results and shear wave velocity profiles were used to approximate effective shear strength parameters used in the long term global stability analysis. The following table summarizes the parameters used in the model.

Table 3: Effective Shear Strength Soil Parameters

Description of Soil	Unit Weight of Soil, pcf		Effective Shear Strength	
	Moist	Saturated	C' (psf)	ϕ'
Ash	80	85	800	32
Silty gravel with sand (SM) (GM)	120	125	0	34
Gravel with silt, sand and cobbles (GP-GM)	140	145	0	38
Shale bedrock	150	155	25,000	0

Static Slope Stability Analysis

A static slope stability analysis was performed on Cross-Section E-E using the computer program GSTABL7 for both a sliding block and circular failure mode. A factor of safety against sliding of 2.9 to 3.0 was calculated for the static condition for both failure modes. A minimum factor of safety against sliding for the static condition of 1.5 is recommended per ASTM E 2277-03 "Standard Guide for Design and Construction of Coal Ash Structural Fills" and also in accordance with the guidelines presented in USACE Manual EM 1110-2-1902 "Slope Stability".

Pseudo-Static Stability Analysis

A pseudo-static analysis was performed to estimate the global stability under seismic conditions. For the pseudo-static analysis, a uniform cyclic shear stress (Seed and Idriss, 1971) was used in the model as outlined below.

$$\tau_{ave} = 0.65 * \sigma_0 * \frac{a_{max}}{g} * r_d$$

where:

τ_{ave} = uniform cyclic shear stress

σ_0 = total vertical stress

a_{max} = peak ground surface acceleration

r_d = stress reduction factor

Based on the results of our stability analyses, a factor of safety against sliding of 2.4 to 2.5 was calculated for the pseudo-static condition for both the block and circular failure modes. A minimum factor of safety against sliding for the pseudo-static condition of 1.2 is recommended per ASTM E 2277-03 “Standard Guide for Design and Construction of Coal Ash Structural Fills” and also in accordance with the guidelines presented in USACE Manual EM 1110-2-1902 “Slope Stability”.

Table 4: Summary of Global Long Term Stability Analyses

Description	Location	Method	Factor of Safety
Global Stability block failure mode (static)	Cross Section E-E	Simplified Janbu	2.9
Global Stability block failure mode (pseudo-static)	Cross Section E-E	Simplified Janbu	2.4
Global Stability circular failure mode (static)	Cross Section E-E	Modified Bishop	3.0
Global Stability circular failure mode (pseudo-static)	Cross Section E-E	Modified Bishop	2.5

7.2.2 Intermediate Stability Analyses

Similar to the global long term stability model, the engineering cross-sections for the intermediate stability models consisted of combined ash material underlain by predominately colluvial and alluvial deposits and shale bedrock. Groundwater elevations for the analyses were considered to be in a perched condition over the shale bedrock with groundwater able to migrate below the ash landfill through the granular soils above the shale bedrock. Since construction staging of the landfill is not known at this time, the intermediate stages used in our analyses may not be indicative of the actual construction staging of the landfill. Two cross sections were generated and used in the intermediate stability analyses and are presented in Appendix D along with the results of the analyses.

A sliding block analysis (simplified Janbu method) was used to model the stability of the intermediate cross sections.

<u>Soil and Bedrock Strengths</u>

SPT blow count correlations, laboratory test results and shear wave velocity profiles were used to approximate total shear strength parameters used in the intermediate stability analyses.

Table 5: Total Shear Strength Soil Parameters

Description of Soil	Unit Weight of Soil, pcf		Total Shear Strength, psf	
	Moist	Saturated	C	ϕ
Ash	80	85	600	30
Silty gravel with sand (SM) (GM)	120	125	0	32
Gravel with silt, sand and cobbles (GP-GM)	140	145	0	36
Shale bedrock	150	155	25,000	0

<u>Static Slope Stability Analysis</u>

A static slope stability analysis was performed on two intermediate cross sections using the computer program GSTABL7 and a sliding block failure mode. This method models potential failure surfaces for granular soil. A factor of safety against sliding ranging from 3.1 to 3.5 was calculated for the static condition. A minimum factor of safety against sliding for the static condition of 1.5 is recommended per ASTM E 2277-03 “Standard Guide for Design and Construction of Coal Ash Structural Fills” and also in accordance with the guidelines presented in USACE Manual EM 1110-2-1902 “Slope Stability”.

Pseudo-Static Stability Analysis

A pseudo-static analysis was performed to estimate the global stability under seismic conditions. Based on the results of our sliding block stability analysis, a factor of safety against sliding ranging from 2.5 to 2.7 was calculated for the pseudo-static condition. A minimum factor of safety against sliding for the pseudo-static condition of 1.2 is recommended per ASTM E 2277-03 “Standard Guide for Design and Construction of Coal Ash Structural Fills” and also in accordance with the guidelines presented in USACE Manual EM 1110-2-1902 “Slope Stability”.

Table 6: Summary of Intermediate Stability Analyses

Description	Location	Method	Factor of Safety
Intermediate Stability block failure mode (static)	Intermediate Cross Section 1	Simplified Janbu	3.5
Intermediate Stability block failure mode (pseudo-static)	Intermediate Cross Section 1	Simplified Janbu	2.7
Intermediate Stability block failure mode (static)	Intermediate Cross Section 2	Simplified Janbu	3.1
Intermediate Stability block failure mode (pseudo-static)	Intermediate Cross Section 2	Simplified Janbu	2.5

7.2.3 Local Short Term Stability Analyses (Ash Benches)

The engineering cross-section for the local short term stability models consisted of ash benches constructed with terraces 30 to 60 feet in height and modeled with 2.5 horizontal to 1 vertical faces, 2 horizontal to 1 vertical faces and 1.5 horizontal to 1 vertical faces. A 15 foot wide bench is provided between terraces. The geometry for the models was based on our discussions with the project Civil Engineer and differ from the original concept.

Waste Ash Properties

As noted earlier, laboratory testing on samples of the ash materials included direct shear testing, moisture-density relationship testing, unconfined compressive strength testing and sieve analysis. The results of the laboratory testing program were used to approximate total shear strength parameters used in the short term stability analyses. The following table summarizes the parameters of the ash used in the model.

Table 7: Total Shear Strength Soil Parameters

Description of Soil	Unit Weight of Soil, pcf		Total Shear Strength, psf	
	Moist	Saturated	C	ϕ
Ash	80	85	600	30

Static Slope Stability Analysis

A static slope stability analysis was performed on each cross section noted above using the computer program GSTABL7 with the modified Bishop Method of slices to evaluate the potential for slope movement. This method calculates the factor of safety for hundreds of potential slope movement surfaces generated through the slope cross-section. This method models potential failure surfaces for granular soil. The following factors of safety against sliding were calculated for the static condition for the various cross sections.

Table 8: Summary of Short Term Static Stability Analyses (Ash Benches)

Description	Cross Section (Ash Bench)	Bench Height (ft.)	Method	Factor of Safety
Short term stability circular failure mode (static)	1.5 H : 1 V	30	Modified Bishop	2.6
Short term stability circular failure mode (static)	1.5 H : 1 V	40	Modified Bishop	2.2
Short term stability circular failure mode (static)	2 H : 1 V	30	Modified Bishop	3.0
Short term stability circular failure mode (static)	2 H : 1 V	40	Modified Bishop	2.5
Short term stability circular failure mode (static)	2.5 H : 1 V	30	Modified Bishop	3.3
Short term stability circular failure mode (static)	2.5 H : 1 V	40	Modified Bishop	2.9
Short term stability circular failure mode (static)	2 H : 1 V	60	Modified Bishop	2.1

A minimum factor of safety against sliding for the static condition of 1.5 is recommended per ASTM E 2277-03 “Standard Guide for Design and Construction of Coal Ash Structural Fills” and also in accordance with the guidelines presented in USACE Manual EM 1110-2-1902 “Slope Stability”.

Pseudo-Static Stability Analysis

A pseudo-static analysis was performed to estimate the local stability under seismic conditions. The following factors of safety against sliding were calculated for the pseudo-static condition for the various cross sections.

Table 9: Summary of Short Term Pseudo-Static Stability Analyses (Ash Benches)

Description	Cross Section (Ash Bench)	Bench Height (ft.)	Method	Factor of Safety
Short term stability circular failure mode (pseudo- static)	1.5 H : 1 V	30	Modified Bishop	2.3
Short term stability circular failure mode (pseudo-static)	1.5 H : 1 V	40	Modified Bishop	1.9
Short term stability circular failure mode (pseudo-static)	2 H : 1 V	30	Modified Bishop	2.6
Short term stability circular failure mode (pseudo-static)	2 H : 1 V	40	Modified Bishop	2.2
Short term stability circular failure mode (pseudo-static)	2.5 H : 1 V	30	Modified Bishop	2.8
Short term stability circular failure mode (pseudo-static)	2.5 H : 1 V	40	Modified Bishop	2.5
Short term stability circular failure mode (pseudo-static)	2 H : 1 V	60	Modified Bishop	1.8

A minimum factor of safety against sliding for the pseudo-static condition of 1.2 is recommended per ASTM E 2277-03 “Standard Guide for Design and Construction of Coal Ash Structural Fills” and also in accordance with the guidelines presented in USACE Manual EM 1110-2-1902 “Slope Stability”.

7.3 Settlement

Total settlement consists of the sum of immediate or elastic settlement, primary consolidation settlement and secondary compression. Given the granular nature of the overburden and ash materials, consolidation settlement and secondary compression have been determined to be negligible. Immediate settlement is then calculated with the soil behaving as a linear elastic material.

Provided the recommended subgrade preparation recommendations presented herein are properly performed in establishing the base of the landfill and in placing the ash materials, total settlement has been estimated to be on the order of 6 to 8 inches. These estimates are based on the proposed construction and geometry discussed herein. Given the relative granular nature of the overburden and ash material, settlement of the material should occur relatively quickly after initial placement. Thus, the majority of expected settlement should occur during construction as the ash materials are placed.

7.4 Erosion Control

During the various construction phases of the ash landfill, periodic slope maintenance will likely be required of the ash slopes, especially following heavy rain events. Ash surfaces should be graded or sloped at the end of each day to provide positive drainage and prevent the ponding of water or the formation of runoff channels that could erode slopes. Any washouts or gullies that form should be immediately repaired to prevent the potential for the slopes to become unstable. Vegetation mats, rip-rap, diversion ditches or other erosion control methods could also be used to help prevent erosion.

7.5 Surface Water Control / Seepage

Design of the ash landfill should also account for surface runoff into the landfill. All efforts should be made to divert surface runoff from entering the landfill.

Based on our review of the current concept plans, we understand it is planned to divert storm water runoff from areas east and south of the proposed ash landfill to minimize contact with the ash materials. Each terrace of the ash landfill would also collect storm runoff and channel it to a main collection ditch which would then flow into a proposed detention pond.

7.6 Site Preparation and Earthwork

PSI recommends that the ground surface within the proposed construction limits of the ash landfill and other areas to receive fill or ash refuse be cleared of vegetation, soil containing significant amounts of roots and organics, disturbed soil and other unsuitable materials.

If the subgrade is disturbed during construction, loose, disturbed soils should be over-excavated to firm, undisturbed soil and backfilled with properly compacted fill.

7.7 Excavations

Excavations and fill placement should occur in a safe manner and in accordance with local, state and federal safety regulations. During wet weather, earthen berms or other methods should be used to prevent runoff water from entering the excavations. The bottom of the excavations should be sloped to a collection point. Collected water within the trench excavations should be discharged to a suitable location outside the construction limits.

7.8 Fill Materials

In the event that soil fill materials are needed for site preparation prior to placement of ash in the landfill, the on-site granular soil (sand and gravel) is generally suitable for use as site grading fill. Imported fill should consist of well-graded granular material that is free of organic and other deleterious materials. Imported fill material should be approved by the Geotechnical Engineer prior to its delivery to the project site. Fill should meet the specifications presented in Table 10.

Table 10: Site Grading Fill Gradation Recommendations

Sieve Size	Site Grading Fill
	Percent Passing by Weight
3 inch	100
¾ inch	-
No. 4	-
No. 40	-
No. 200	35
Liquid Limit (LL)	≤ 35
Plasticity Index (PI)	≤ 15

7.9 Placement and Compaction of Material

7.9.1 Site Grading Fill

Fill materials used in site preparation work should be moisture conditioned to within two (2) percentage points of the optimum moisture content prior to placement. Fill should be placed in loose lifts not exceeding the capability of the compaction equipment. Loose lift thicknesses of six (6) to eight (8) inches are typically appropriate. Fill should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D 1557 Test Method.

7.9.2 Combined Fly Ash / Bed Ash

Ash is usually spread and leveled with a bulldozer, grader, or other equipment in lifts not exceeding 12 inches when loose. Individual lifts should be compacted as soon as the material has been placed and is moisture conditioned. The ash should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D 1557 Test Method.

8.0 CONCLUSIONS

PSI has reviewed various published geologic documents related to the site proposed for the SCA #2 Ash Landfill, various documents associated with the existing SCA #1 Ash Landfill and conducted a specific geotechnical investigation on the proposed SCA #2 site. The investigation included 2 boreholes, 4 test pits and 3 Refraction Microtremor tests. Laboratory testing was performed on several samples gathered during the field investigation and also performed on samples of the ash material to be placed in this landfill.

PSI also investigated the possible presence of groundwater on the site. While ground water was not observed in Boring B-2 (upper east slope) or in any of the test pits, groundwater was observed in Boring B-1 at the lower west end of the site. No surface waters were present at the site or within the near proximity of the site. The granular surface soils (ranging from approximately 14 to 50 feet thick) on top of the relatively impervious shale bedrock will provide an adequately porous layer to convey any groundwater that does migrate under the proposed ash landfill. We expect that any migrating groundwater will move in a general northeast to southwest direction atop the shale bedrock and at least 10 feet below the ground surface in the vicinity of the landfill and not come into contact with the ash materials. We recommend placement of a low permeability soil cap (at least 6 inches at a hydraulic conductivity of 1x10^-3 cm/s or less) on top of the completed landfill with a native soil cover above that for revegetation. Surface water should be controlled to reduce the potential for erosion or ponding and observed erosion conditions should be repaired. Provided these recommendations are followed, we anticipate that the risk of water percolating through the ash material and into the groundwater is minimal.

PSI conducted several structural stability analyses for the proposed 400 ft high landfill in various possible configurations ranging from bench heights of 30 ft and cross slope section of 1.5H:1V up to a bench height of 60 feet and cross slope section of 2H:1V. All of the configurations modeled indicated short term and long term safety factors greater than the minimums recommended per ASTM E 2277-03 "Standard Guide for Design and Construction of Coal Ash Structural Fills" and also in accordance with the guidelines presented in USACE Manual EM 1110-2-1902 “Slope Stability”. We recommend that ash materials be placed in maximum 12-

inch lifts and should be compacted to a minimum 95%. With proper compaction, the expected settlement occurring in this landfill will have minimal impact.

Based on the results of our study, we are of the opinion that the site of the proposed SCA #2 ash landfill is suitable from a geotechnical engineering perspective, provided our recommendations for site preparation and placement of the ash materials are followed.

9.0 DESIGN REVIEW AND CONSTRUCTION SERVICES

PSI is available to assist as needed to review geotechnical related portions of the design documents or construction conditions. If site conditions are different than described in this report, PSI should be notified so that we can re-evaluate our recommendations if necessary.

10.0 GEOTECHNICAL RISK

The concept of risk is an important aspect of the geotechnical evaluation. The primary reason for this is that the analytical methods used to develop geotechnical engineering conclusions and recommendations do not comprise an exact science. The analytical tools which geotechnical engineers use are generally empirical and must be used in conjunction with engineering judgment and experience. Therefore, the conclusions, solutions and recommendations presented in the geotechnical evaluation should not be considered risk-free and, more importantly, are not a guarantee that the interaction between the soils and the proposed structure will perform as planned. The engineering conclusions and recommendations presented in the preceding sections constitute PSI's professional estimate of those measures that are necessary for the proposed ash landfill to perform according to the proposed design based on the information generated and referenced during this evaluation, and PSI's experience in working with these conditions.

11.0 LIMITATIONS

The recommendations submitted are based on the available subsurface information obtained by PSI, and information provided by Sunnyside Cogeneration Associates (SCA) and their design consultants. If there are revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes and/or other recommendations are required. If PSI is not retained to perform these functions, PSI cannot be responsible for the impact of those conditions on the performance of the project. The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

The Geotechnical Engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of Sunnyside Cogeneration Associates (SCA) and their design consultants for the specific application to the SCA #2 Ash Landfill project to be located in Carbon County, Utah.

PSI is committed to providing quality services to its clients, commensurate with their wants, needs, and desires. We appreciate the opportunity to provide our services on this project. If you have questions pertaining to this project or if we may be of further assistance, please call the undersigned.

Respectfully Submitted,
PROFESSIONAL SERVICE INDUSTRIES, INC.

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4/6/2012

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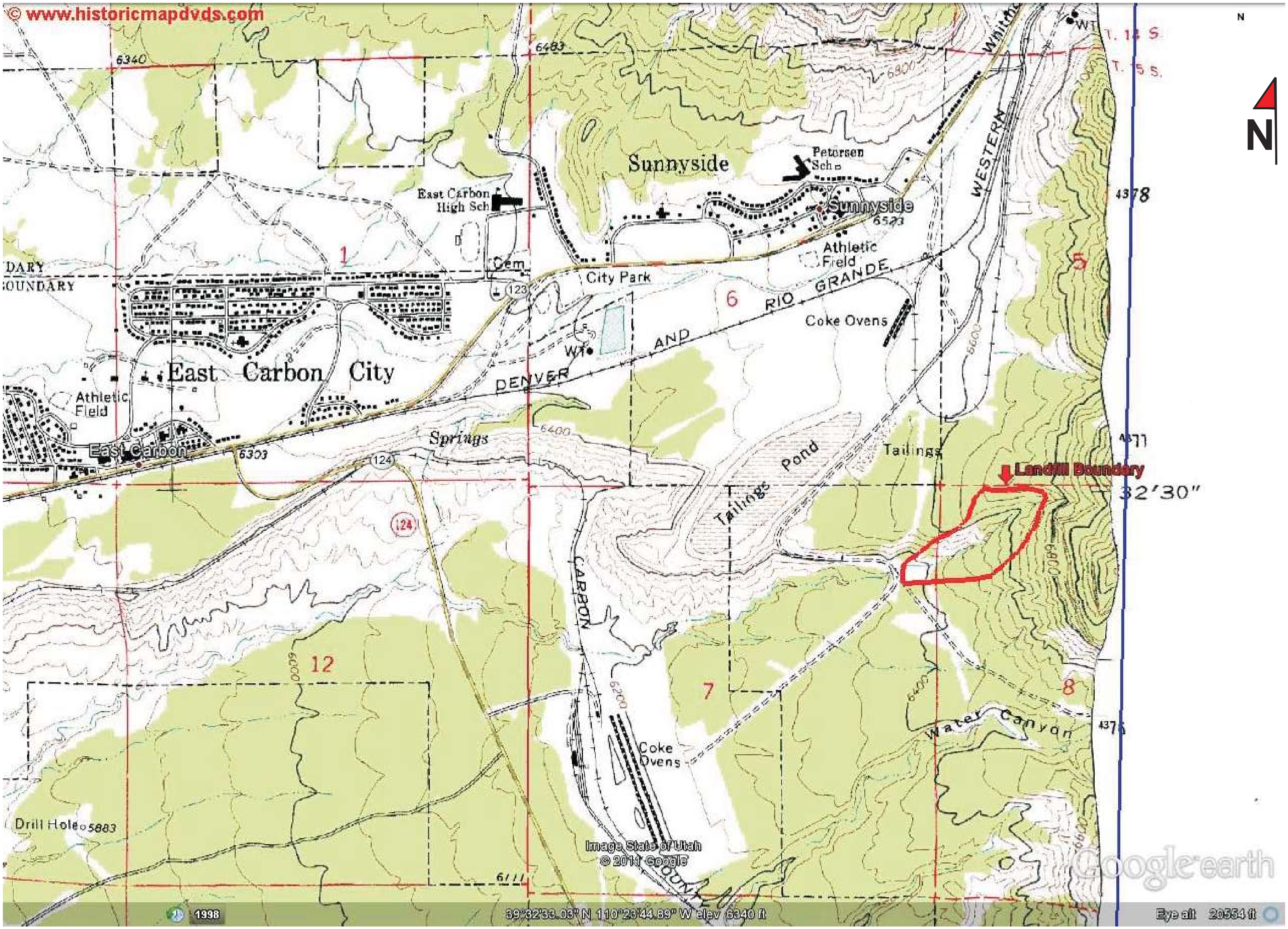
APPENDIX A

USGS Topographic Site Vicinity Map - Figure A-1
USDA SCS Soils Map - Figure A-2
Ash Landfill Conditional Use Submittal (Dated 12-29-2009)
Site Photographs



Information To Build On
Engineering • Consulting • Testing

Project:
Proposed SCA #2 Ash Landfill
Sunnyside Cogeneration Facility
Carbon County, Utah

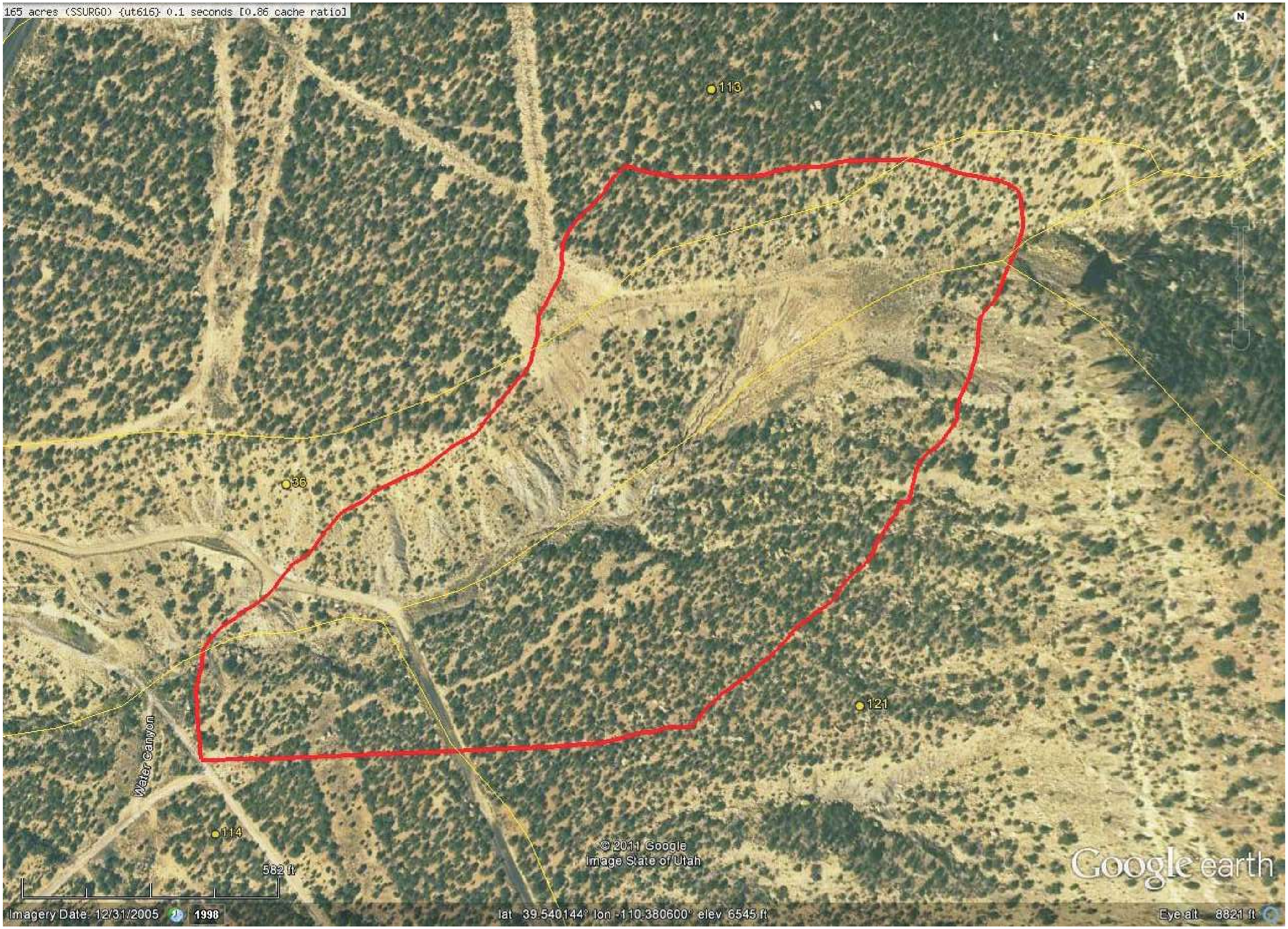
Project No: 0710281

VICINITY MAP

Drawn By: TKB

Reviewed By: KS

Figure: A-1



Gerst-Strych-Badland complex, 3 to 50 percent slopes (Soil Group No. 36)

Strych very stony loam, 3 to 15 percent slopes (Soil Group No. 113)

Strych very stony loam, dry, 3 to 30 percent slopes (Soil Group No. 114)

Travessilla-Rock outcrop-Gerst complex (Soil Group No. 121)

psi Information To Build On Engineering • Consulting • Testing	Project: Proposed SCA #2 Ash Landfill Sunnyside Cogeneration Facility Carbon County, Utah Project No: 0710281	USDA SCS SOILS MAP Drawn By: TKB — Scale: As shown Reviewed By: KS — Figure: A-2