APPLICATION FOR A PERMIT TO CONSTRUCT AND OPERATE A CLASS I LANDFILL FACILITY
JUNGO DISPOSAL SITE
Humboldt County, Nevada

REPORT OF DESIGN

Revision 5

VOLUME I

Prepared For

Jungo Land and Investments, Inc.

Prepared by
Golder Associates Inc.
1009 Enterprise Way, Suite 350
Roseville, CA 95678
(916) 786-2424

Distribution:

(3) Copies – Jungo Land and Investments, Inc.
(2) Copies - Golder Associates Inc

April 2011 063-7079-100-200

APPLICATION FOR A PERMIT TO CONSTRUCT AND OPERATE A CLASS I LANDFILL FACILITY JUNGO DISPOSAL SITE

Humboldt County, Nevada

REPORT OF DESIGN

Revision 5

Prepared For

Jungo Land and Investments, Inc.

Prepared by
Golder Associates Inc.
1009 Enterprise Way, Suite 350
Roseville, CA 95678
(916) 786-2424

REGISTERED PROFESSIONAL ENGINEER - STATE OF NEVADA
4-1-11
RICHARD E.
KIEL
CIVIL
No. 08011

Golder Associates Inc.

Kenneth Haskell
Senior Consultant

Richard Kiel
Senior Consultant, P.E.

April 2011

063-7079-200

TABLE OF CONTENTS

VOLUME I

Section **Page**

Tables

Figures

Appendices

VOLUME II

Drawings

VOLUME III

PLAN OF OPERATIONS

1.0 INTRODUCTION

This report is submitted as an integral component of the application for a permit to construct and operate a Class I municipal solid waste landfill facility at the Jungo Disposal Site located in Humboldt County, Nevada. The Jungo Disposal Site is located approximately 30 miles west of Winnemucca, Nevada along Jungo Road as shown in ***Figure 1***. Jungo Land and Investments, Inc. (JLII) will be the landfill developer and operator.

This report (***Volume I***) includes the Report of Design and was prepared in accordance with Nevada Administrative Code (NAC), Sections 444.677 through 444.683. ***Volume II*** includes the site design and development drawings. The Plan of Operations is included in ***Volume III, which also includes the Monitoring Plan, Closure Plan, and Postclosure Monitoring Plan***.

The landfill is located on a 634-acre parcel that consists of Section 7 of Township 35N (T35N), Range 33E (R33E). The landfill disposal footprint encompasses 562-acres within Section 7, T35N, R33E. The site development will include a rail yard for unloading waste containers transported by the rail, an office trailer, and a maintenance shop. Additional detail on the supporting facilities is provided in the Plan of Operations.

This Report of Design is organized as follows:

- Section 1 – Introduction
- Section 2 – Report of Design
- Section 3 – References

The Engineering Design Report is divided into the following three volumes:

- Volume I includes the text, tables, and figures for the Report of Design various reports, and supporting appendices consisting of data, test results and engineering calculations;
- Volume II includes the site development drawings; and
- Volume III includes the Plan of Operations.

2.0 REPORT OF DESIGN

The Report of Design was prepared in compliance with NAC, Sections 444.680 and 444.681. The Report of Design provides a description of the physical setting, and therefore, Section 2.1 also discusses the site location with respect to the location restrictions specified in NAC, Section 444.678.

2.1 Physical Setting

2.1.1 Site Location, Zoning and NAC Location Restrictions

The Jungo Disposal Site is located at the southern end of Desert Valley approximately 30 miles west of Winnemucca along the south side of Jungo Road as shown in ***Figure 1***. Regionally, the site is located in an arid, relatively flat, desert basin bound by the Jackson Mountains to the west and northwest, the Antelope Range to the southwest, Alpha Mountain to the south, and the Eugene Mountains to the southeast. The site vicinity is sparsely vegetated with greasewood.

The site is located on a 634-acre parcel consisting of Section 7 of Township 35N, Range 33E of the Mount Diablo Baseline and Meridian. The Jungo Disposal Site is bounded by Union Pacific Railroad property to the northwest and elsewhere by publicly-owned land administered by the Bureau of Land Management (BLM). ***Figure 2*** shows the property parcel in relation to the railroad and surrounding property sections.

The Jungo Disposal Site and surrounding property are zoned M-3 – Open Land Use. An M-3 designation allows for conditional commercial uses, such as landfill disposal operations, provided such uses are approved by the Humboldt Planning Commission (HPC). The HPC issued JLII a Special Use Permit in April 2007 that allows the site to be developed as a municipal waste disposal site.

NAC 444.678 specifies location restrictions for Class I landfills. The Jungo Disposal Site satisfies these requirements as follows:

- **NAC 444.678 – General:**
    - NAC 444.678 (1) The site design includes all-weather access to the landfill including an access road surfaced with aggregate. The rail unloading area will include paved areas and areas surfaced with aggregate to provide dust control and all-weather access.
    - NAC 444.678 (2) and (3) The landfill design includes containment systems, controls, and monitoring systems that will prevent uncontrolled migration of landfill gas, control leachate, and prevent degradation of groundwater.
    - NAC 444.678 (4) The site has soil available that is workable and compactable for use in covering the refuse.
    - NAC 444.678 (5) The disposal site also conforms to Humboldt County's land use planning.
    - NAC 444.678 (6) The nearest public highway (Interstate 80) is more than 30 miles from the site.
    - NAC 444.678 (9) The nearest surface water body is more than 14 miles from the site. The landfill is located within 100 feet of the uppermost groundwater aquifer. However, to prevent degradation of the groundwater aquifer, the landfill design incorporates extensive protective measures consisting of low-permeability

containment systems, conservatively designed leachate control system, and landfill gas control systems. These protective measures are described in Section 2.3.

- **NAC 444.6783 – Airport Safety**: There are no airports within 10,000 feet of the site.
- **NAC 444.6785 – Floodplain**: The site is not located within a floodplain. The site is located within a desert basin where precipitation temporarily collects in shallow depressions until it evaporates or infiltrates into the underlying soils.
- **NAC 444.679 – Wetlands**: The site is not located within a wetland area. The nearest wetlands in Desert Valley are more than 25 miles to the north along Bottle Creek Slough.
- **NAC 444.6791 – Fault Areas**: The site is located in a region that is underlain by a thick sequence of sediments that are at least 6,000 feet thick and do not show any surficial evidence of faulting (i.e. scarps). There are no mapped faults within 200 feet of the landfill. The nearest quaternary fault to the site is located in the Eugene Mountains at a distance of approximately 3 miles from the site.
- **NAC 444.6793 – Seismic Impact Zones:** The Jungo Disposal Site is located within a seismic impact zone, which is defined as a location that has a 10 percent probability of exceedance in a 250 year period of experiencing a seismically induced peak ground acceleration of 0.1g or greater. As required by NAC 444.6793, the Jungo Disposal Site is designed to withstand the peak ground acceleration without damaging environmental containment systems and controls, including the liner and cover systems. Seismic impacts are evaluated in Section 2.3.
- **NAC 444.6795 – Unstable Areas:** The Jungo Disposal Site is not located in an area that is considered geologically unstable, such a landslide prone areas, karst terrain, or excessively soft soils that could result in foundation failure. The site soils are expected to experience consolidation under loading by refuse. However, the landfill is designed to accommodate the settlement without adversely affecting the liner system as discussed in Section 2.3.

Additional detail on location restrictions is provided in the Plan of Operations.

2.1.2 Climate and Hydrology

The site is located in an arid region, where precipitation is controlled primarily by the rain-shadow effects imposed by the Sierra Nevada range located 150 miles to the west. The Jackson Mountains located on the west side of Desert Valley, cause a similar orographic effect, but of a lesser magnitude (Berger, 1995).

Precipitation results primarily from thunderstorms in the summer, and snow and rain in the winter. The mean annual precipitation is estimated to be approximately 8 inches. The mean annual precipitation in Winnemucca located 30 miles to the east (1897-2006) is 8.3 inches. Three different sources (Western Regional Climate Center; World Climate.com; and Berger, 1995) provide mean annual precipitation values ranging between 7.97 inches to 9.1 inches for a precipitation gauge at the Jungo-Meyers Ranch, located approximately 4 miles west of the Jungo Disposal Site. This precipitation gauge was measured from 1968 to 1986.

Temperatures in the summer months occasionally exceed 100° F. Winters are cool with temperatures often below 0°F. Based on data from Rye Patch Reservoir located 14 miles to the south, evaporation from free water sources is approximately 48-inches per year (Cohen, 1966). The prevailing wind direction in Desert Valley is toward the west-southwest.

The 25-year, 24-hour storm event is estimated to be 1.62 inches (NOAA, 2006).

2.1.3 Topography and Drainage

The Antelope Range, Alpha Mountain, and Eugene Mountains form the topographic divides at the southern end of the Desert Valley near the general location of the Jungo Disposal Site. The low-points of these topographic divides range from approximately 4,250 feet msl to 4,400 msl.

The valley floor is relatively flat. At the southern end of the valley, the elevations generally range from 4,180 feet msl to 4,155 feet msl and slope from the southeast to the northwest. At the Jungo Disposal Site, the elevations range from a high of 4,177 feet msl at the southeast corner of the property to a low of 4,172 feet msl at the southwest corner of the property.

Desert Valley is a 1,052 square mile hydrographic sub-area within the Black Rock Desert Region hydrographic basin. Streams from the surrounding mountains are ephemeral and rarely discharge to the valley floor and instead infiltrate through the upper coarse alluvial fans or evaporate (Berger, 1995). Precipitation or snow melt on the valley floor accumulates in localized depressions until it infiltrates or evaporates.

At the Jungo Disposal Site, these shallow depressions are on the order of several inches deep. During normal precipitation events, water accumulates in the depressions until it evaporates or infiltrates into the subsurface soils. In the event of intense storms, it is possible that localized depressions may fill and then sheet flow to the next depressions located to the north or west. This is consistent with the United States Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS) (2007), which estimates that ponding may occur locally to depths of 6 to 12 inches.

Large alkali flats are located in larger depressions that are approximately 2 miles west and north of the Jungo Disposal Site with surface elevations ranging from 4,162 to 4,164 feet msl. These alkali flats are located in an area identified by Berger (1995) as containing hard-pan, or low-permeability clays and silts, which impede infiltration.

2.1.4 Geology

The following sections describe the regional geologic conditions and the site-specific geologic conditions based on subsurface explorations completed as part of the initial site characterization program.

2.1.4.1 Regional Geology

Desert Valley lies within the Basin and Range Geomorphic Province, which is characterized by north-south trending uplifted mountain ranges adjacent to down-dropped valleys or basins. Desert Valley is a north-south trending structural basin with a relatively flat valley floor that is approximately 55-miles long and 12-miles wide. Mountain ranges provide topographic boundaries at the edges of the valley floor. The Jungo Disposal Site is located in the southernmost portion of Desert Valley. This end of Desert Valley is bound to the west by the Jackson Mountains, to the southwest by the Antelope Range, and to the east by the Eugene Mountains. ***Figure 3*** shows the regional geologic map.

The lithology of the area is comprised of two major types - consolidated rock and basin fill. The rock is found in the surrounding mountains and underlying the valley basin fill sediments. Predominant rock types include Tertiary-age volcanic flows, clastic sedimentary rocks from the Jurassic or Triassic ages, and Permian-age or older volcanic rocks. These formations are generally considered to have low permeability.

At the base of the mountains, alluvial fans consisting of eroded sediments from the mountains occur. These cone-shaped deposits contain coarse sediments typically deposited by stream and debris flows.

The alluvial deposits range from partly consolidated to unconsolidated fill material. The Older Alluvium unit, consisting of poorly sorted, subangular to subrounded sand to cobbly gravel, has been identified to occur along several range fronts, high on the alluvial fans. Toward the valley floor, this unit underlies the Younger alluvium of the valley floor. The Older Alluvium grades finer toward the valley center and becomes partially consolidated.

In addition to the coarser-grained alluvial fan sediments, the basin contains aeolian, lacustrine, and volcanic deposits. During late Pleistocene time, Desert Valley was inundated by ancient Lake Lahontan, which reached depths of nearly 200 feet in Desert Valley. The contact between Older Alluvium and Younger Alluvium is typically drawn at the elevation of the highest Lake Lahontan terrace. The Younger Alluvium, located on the valley floor and beneath stream channels, includes Pleistocene and Recent lake sediments, shoreline deposits, stream deposits, and aeolian deposits. The total depth of the sediments in the basin is estimated to be 6,000 to 7,000 feet in the vicinity of the Jungo Disposal Site (Berger, 1995).

Older Alluvium, consisting of poorly-sorted, subangular to subrounded sand to cobbly gravel, generally occurs along the range fronts and grades finer toward the toes of the alluvial fans. Older Alluvium is primarily dissected alluvial fan deposits that are coarser than the Younger Alluvium found on the valley floors and in stream channels.

2.1.4.2 Site Geology

An initial site characterization program was completed to evaluate the site-specific geologic conditions, hydrogeologic conditions, and engineering properties of the underlying soils. This initial characterization program consisted of the examination and logging of the surficial soils, the completion of five borings to depths of 100 to 145 feet below ground surface (bgs), and the completion of a geotechnical laboratory testing program.

The site soils are mapped as Younger Alluvium and classified as Boton Playas described as unconsolidated alluvial sediments (USDA, NRCS, 2006). Boton soils consist of volcanic ash and loess over lacustrine deposits. The surficial soils are relatively uniform and consist of silty fine sands, classified as a silty sand (denoted as "SM") in accordance with the Unified Soils Classification System (USCS).

Five borings were completed using hollow-stem auger drilling methods under the observation of a Golder engineer who logged the soil samples and recorded groundwater conditions. The locations of the borings are shown in ***Figure 4***. Soil samples were collected at 5-foot intervals using a combination of standard split spoon samplers, modified California samplers, and Shelby (thin-walled) tubes.

The standard split spoon samples and modified California samples were obtained in accordance with the Standard Method for Penetration Test and Split Barrel Sampling of Soils as described in ASTM D1586. This sampling method consists of driving the split spoon sampler a distance of 18 to 24 inches into undisturbed soil. The number of blows required to drive a standard split spoon sampler the final 12-inches is known as the Standard Penetration Resistance "N", which provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. Because drilling mud was not used during the drilling program, caution should be used in interpreting N values in cohesionless soils below the groundwater surface. Soil samples obtained for consolidation characteristic testing were obtained by pushing a three-inch outer diameter thin-walled tube into the undisturbed soil in accordance with the Standard Practice for Thin Walled Tube Sampling of Soils as described in ASTM D 1587.

All samples were placed in air-tight sample bags or sealed directly in the thin-walled tube to minimize moisture loss during transport to the laboratory. Soil samples were classified in accordance with Golder technical procedures and the Unified Soil Classification System.

The four borings located near the corners of the property were converted into groundwater monitoring wells (MW-1 through MW-4). The boring completed in the middle of the site was abandoned by backfilling the boring annulus with a cement-bentonite grout.

Summary boring logs and monitoring well completion logs are included in ***Appendix A***. The blow counts (N-values) on the logs have not been corrected for overburden stresses.

Section 2.1.4.2.1 summarizes the subsurface geologic conditions observed. Section 2.1.4.2.2 summarizes the geotechnical engineering properties measured from the laboratory testing program.

2.1.4.2.1 Site Soil Conditions

The subsurface soils consist of interbedded sands, silts, and clays. ***Figures 5 and 6*** illustrate the subsurface lithology to a depth of 100 to 145 feet bgs, which is summarized below:

- **Upper Silty Sands**. The uppermost soils are predominately silty fine sands with occasional thin lenses of silt. These soils occur at the ground surface and extend to depths of approximately 35 to 40 feet bgs.
- **Upper Silty Clays and Clayey Silts**. A 10- to 18-foot thick layer of primarily silty clay and clayey silt underlies the uppermost silty sands.
- **Middle Sands**. At a depth of 55 to 60 feet bgs, the borings encountered predominately sands that are interbedded with silty sands and thin lenses of silts and silty clays. This soil zone was observed to be 18 to 30 feet thick.
- **Lower Clay and Clayey Silt**. A 12- to 20-foot thick clay layer was first encountered at a depth of 70 to 80 feet bgs. The upper portion of this layer is generally comprised of highly plastic and compressible clay, while the lower portion consists of low to moderately plastic clay.
- **Lower Sand and Silty Sand**. The deepest boring penetrated the lower clay and clayey silt zone at a depth of 115 feet and encountered interbedded sands, silty sands, and thin lenses of silt to the full depth of the boring at 145 feet.

As an approximate percentage of the lithologic section, clean sands comprise 10 percent, silty sands comprise 40 percent, silts comprise 10 percent, silty and sandy clays comprise 30 percent, and highly plastic clays comprise 10 percent.

2.1.4.2.2 Geotechnical Properties

Representative soil samples, collected from the borings during the investigation, were submitted to the laboratory for the following analyses:

- Moisture-density;
- Atterberg Limits;

- Grain-size distribution;
- Consolidation; and
- Consolidated-Undrained (CU) triaxial shear strength (with pore pressure measurements).

The key geotechnical properties are summarized below.

- SPT blow counts (N-Values, uncorrected) in the sands above the water table range from approximately 16 to 20 per foot in the upper 5 to 10 feet and generally increase to 40 at a depth of 45 to 55 feet. Based on the blow-counts, the sands are considered to be dense.
- The moisture content of the upper silty sand layer ranged from 12 to 18 percent. The dry density of the upper silty sand ranged from 92 to 106 pounds per cubic foot (pcf).
- The dry density of the silty clays ranged from 86 to 89 pcf. The dry density of the highly plastic clay was measured to be 59 pcf in two samples.
- The Plasticity Index of the silty clay (CL) layers ranged from 20 to 30. The Plasticity Index for the highly plastic clay (CH) was measured between 56 and 72.
- Consolidation tests indicated that the soils are generally normally consolidated. In some cases, the soils might be slightly over-consolidated, which may be related to the recent regional decline in groundwater levels and the older groundwater declines that occurred after ancient Lake Lahontan dried up. The primary compression index (C_c) for the silty clay and silt (CL-ML) was measured to be 0.16 to 0.26 with an initial void ratio of 0.9 to 1.0. The primary compression index (C_c) for the highly plastic clay (CH) was measured to be 0.66 to 0.71 with an initial void ratio of approximately 1.87. The lowest coefficient of consolidation (C_v) values were generally measured at between 0.01 to 0.08 ft/day, which corresponded to the high plasticity clays and some of the low plasticity clays.
- CU-triaxial shear strength tests measured effective stress strength parameters for the silty clays that can be defined by a friction angle of 26 to 27 degrees and cohesion of 1,500 to 2,100 psf. The measured effective stress strength parameters for the highly plastic clays can be defined by a friction angle of 19 to 21 degrees and cohesion of 800 to 975 psf.

Appendix B includes a summary of laboratory tests that were completed as part of the initial site characterization.

2.1.4.2.3 Summary

The initial site characterization indicates that the site is underlain by interbedded sands, silts, and clays. Four soil sequences were identified throughout the site in all five borings and included an upper silty sand, an upper silty clay and clayey silt, a middle sand, and a lower clay and clayey silt. One boring extended to a depth of approximately 145 feet encountered a fifth soil sequence consisting of a lower sand and silty sand. The base of the landfill, as described in Section 2.3 will be founded in the upper silty sand. Groundwater was first encountered in the middle sand layer at a depth of approximately 59 to 60 feet. The upper silty clay layer occurs between the base of the landfill and first occurrence of groundwater.

The initial site characterization also indicated that the soils are normally consolidated, the silty clays and silts are moderately compressible, and that the highly plastic clay at a depth of approximately 80 to 90 feet is highly compressible.

As described in Section 2.3, the compressive characteristics of the underlying soils pose a significant constraint to the height and weight of refuse that can be placed on the liner. Excessive settlement of the foundation could result in adverse drainage grades on the landfill. Due to the critical aspect of these geotechnical properties, additional geotechnical borings will be completed prior to the construction of the base containment system as follows:

- A minimum of six borings will be installed for each module (each module is approximately 55-acres in area). After the explorations are completed for the first one or two modules, the number of borings may be increased or decreased based on the variability of the observed subsurface conditions. For the five borings that were completed for the initial characterization, the soil conditions were relatively consistent.
- The borings will be installed to depths of at least 200 to 300 feet.
- The underlying silts and clays will be sampled for further consolidation testing
- Additional standard penetration test (SPT) measurements will be collected to allow for confirmation of the initial liquefaction assessment (Section 2.3.4.4 and Appendix K).

The above geotechnical investigations and testing will be used to confirm the current lithologic and geotechnical model. If these investigations indicate differing conditions, the lithologic and geotechnical model will be updated and the landfill design modified if necessary. Potential landfill design modifications may include changes to the drainage grades on top of the base liner or potential changes in the landfill heights. All design changes will be submitted the Nevada Department of Environmental Protection for review and approval prior to implementation.

2.1.5 Hydrogeology

2.1.5.1 Regional Hydrogeology

The partially-consolidated to unconsolidated basin-fill deposits in Desert Valley comprise the primary water-bearing unit. The deposits generally function as a single, heterogeneous aquifer rather than one with defined, contiguous fine-grained aquitards layered between coarser-grained water-bearing units. Most shallow groundwater occurs in unconfined conditions, while groundwater found at depth below finer-grained deposits occurs in semi-confined conditions (Berger, 1995).

Most groundwater recharge in the Desert Valley basin occurs as precipitation that falls on the mountains surrounding the basin. The primary mechanisms for recharge are for the rainfall or snowmelt to infiltrate exposed weathered and/or fractured bedrock or for the runoff to percolate through the coarser-grained alluvial fan deposits. With most recharge occurring at the higher elevations, groundwater at the eastern and western valley margins primarily flows from the higher elevations downgradient toward the center of the basin. Additional groundwater recharge does occur in the subsurface from the Quinn and Kings River Valleys located in the northern portion of the basin (Berger, 1995).

A groundwater divide bisects the Desert Valley basin from east to west. The location of the divide has been shown to migrate over time in response to changes in groundwater elevations, but has generally remained in the central to southern-central area of the basin located north of the Jungo Disposal Site. Groundwater on the northern side of the divide flows to the north toward the Quinn River and discharges out the northwestern side of the basin at Pine Valley. Groundwater on the southern side of the divide flows to the southwest and likely exits the basin near the Antelope Range (Berger, 1995). The Jungo Disposal Site is located on the southern side of the groundwater divide.

Ranges of horizontal groundwater hydraulic conductivities have been estimated by calculating average values for the different lithologic units encountered in the upper 180 feet of saturated basin-fill deposits in Desert Valley. Using the lithologic data, areas of the basin were roughly categorized as having horizontal conductivities either greater than 50 feet/day (ft/day) or less than 50 ft/day (Berger, 1995). Most of the basin was estimated to have a horizontal conductivity less than 50 ft/day ($<1.7\text{x}10^{-2}$ cm/s), while some smaller areas of the basin were estimated to have conductivities greater than 50 ft/day. The Jungo Disposal Site is located in an area identified by Berger (1995) to have horizontal hydraulic conductivities less than 50 ft/day.

Prior to 1985, groundwater withdrawal in the Desert Valley basin occurred primarily for agriculture and irrigation, with lesser amounts for domestic and livestock use. In 1985, significant mining and associated dewatering operations began in the northern portion of the valley on the northern side of the groundwater divide at the Sleeper Mine. Basin-wide groundwater elevations measured in 1991 demonstrated that groundwater elevations in the basin had been affected by the mine dewatering. In general, elevations measured in 1991 (while mine dewatering was occurring) were lower than those from "pre-development" (late 1950's to early 60's) conditions. ***Figure 7*** compares groundwater flow directions in 1991 with estimated pre-development conditions in 1975. These 1991 measurements also showed that the groundwater divide had migrated toward the south. The Desert Valley basin groundwater elevations range from approximately 4100 to 4130 feet mean sea level (msl) (Berger, 1995).

Appendix C includes an evaluation of historical groundwater levels for the Desert Valley Basin. The data review determined that, in the vicinity of the Jungo Road site, groundwater levels have declined approximately 10 feet over the past 30 years. The decline is attributed to past and current groundwater withdrawal for agricultural use and mine dewatering. As such, with continued agricultural groundwater use and other development-related uses, groundwater elevations in the vicinity of the site will continue to decline, and therefore, they are not likely to exceed pre-development levels (i.e., approximately 1975 levels) and indeed may not rise beyond present day levels.

The data review presented in ***Appendix C*** also indicates that groundwater levels in the basin are not impacted by seasonal changes including periods with above average precipitation. ***Figure 8*** shows the historical trends in groundwater elevations for wells within the southern portion of the basin in comparison to annual precipitation values. ***Figure 9*** shows the locations of these reference wells. As indicated in ***Figure 8***, groundwater levels did not respond to several above-average rainfall years occurring in the late 1990's (1996, 1997, 1999, and 2001). To the contrary, groundwater levels continued to decline an additional two to three feet. Basin development and long-term groundwater use patterns (e.g., groundwater extraction for irrigation) appear to be a more significant factor in groundwater elevation change than annual precipitation.

2.1.5.2 Site Hydrogeology

In soil borings completed during the site investigation, first groundwater was encountered at approximately 59 to 60 feet bgs in the middle sand/silty sand layer, above a layer of low to high plasticity clay. Four of the five borings were converted to groundwater monitoring wells. The well locations are shown on ***Figure 4*** and the well construction details are provided in ***Appendix A***. Quarterly depth-to-water measurements have been taken in the wells since their installation in January 2007. These measurements indicate that groundwater occurs at elevations similar to those recorded in the initial soil borings. Therefore, first-encountered groundwater occurs under unconfined, water-table conditions, consistent with the regional hydrogeologic model.

As described above, water levels in the basin have decreased, and in the area of the site have declined approximately 10 feet over the past 30 years. Current depth to groundwater at the site is approximately 59 to 60 feet bgs (***Table 3***). Therefore, assuming a return to 1975 groundwater levels, the highest anticipated groundwater levels at the site are estimated to be approximately 50 feet bgs.

Water-level measurements collected between January 2007 and November 2007 have exhibited no seasonal variation. The maximum change in elevation during 2007 is less than one foot. In April 2007, a pressure transducer and datalogger were installed in well MW-1 to allow for continuous recording of groundwater levels to further evaluate any potential seasonal or other short-term variation.

A groundwater contour map of the site has been prepared using groundwater elevations measured in the site wells (***Figure 10***). Based on these measurements, groundwater flows toward the southwest, consistent with the basin flow net prepared by Berger (1995). The gradient is estimated to be 0.0003.

Groundwater Velocity

Rising head slug tests were conducted in each well on February 2, 2007 to determine the hydraulic conductivity of the middle sand and silty sand. With these data, hydraulic conductivities were calculated for each well. To determine a hydraulic conductivity for the site, the geometric mean of the four individual well conductivities was calculated. As such, the hydraulic conductivity at the site is estimated to be 1.2×10^{-4} cm/s. The slug test data is presented in ***Appendix D***.

Using the calculated gradient (i), the hydraulic conductivity (K), and the estimated effective porosity of the water-bearing zone (n_e), the approximate groundwater seepage velocity can be calculated using Darcy's Law ($v = Ki/n_e$). An effective porosity value of 0.15 for the sandy zones is assumed, based on information from Cohen (1963). Groundwater seepage velocity beneath the site is estimated to be 2.4×10^{-7} cm/s (0.25 feet per year [ft/yr]).

Vertical Gradient

The landfill is likely located in an area of groundwater discharge. According to Berger (1995), most precipitation and groundwater recharge occurs in the mountains surrounding the basin, with recharge through fractured rock or intermittent streamflow along coarse channel deposits on alluvial fans (located along the edges of the valley). In addition, Berger (1995) states that some recharge may occur in the south-central portion of the valley (north of Jungo) in the area covered by active sand dunes. Note that during extremely wet years, when the playa is saturated, there may be some minor recharge from the playa.

All four site groundwater monitoring wells are screened in the shallowest water-bearing zone beneath the site. There are no deeper wells at the site capable of providing data on the vertical hydraulic gradient beneath the site. Berger (1995) suggested that only small vertical gradients existed in Desert Valley prior to development. Both upward and downward vertical gradients were determined for wells located near the Sleeper Mine in the northern part of the valley (Berger, 1995). These vertical gradients were apparently induced by mine dewatering and artificial recharge in the immediate area of the nested wells. Based on the lack of development near the Jungo area, and based on the work by Berger (1995), it is likely that only small vertical gradients exist in this area.

There is a 12- to 20-foot thick clay layer (lower clay and clayey silt) beneath the first water-bearing zone (middle sands). Vertical, downward flow through the lower clay layer could occur if there was a downward gradient beneath the site. Given that there may be small vertical gradients in the area, if there was a 1/10-foot difference in head between the upper water-bearing zone and the next deeper sand layer, the vertical gradient would be approximately 0.008 (0.1 foot/12 feet [intervening clay layer minimum thickness]). Berger (1995) estimated the vertical hydraulic conductivity for fine-grained deposits in Desert Valley to be 3 x 10^{-6} cm/sec. Using an effective porosity of 0.05 for the clay layer (Berger, 1995), if the vertical gradient was upward (the site is likely located in an area of groundwater discharge with an upward gradient), the potential upward groundwater flow velocity would be 0.5 ft/yr. The theoretical minimum transport time for water to flow through the underlying clay layer would be approximately 24 to 40 years. However, due to path tortuosity in clay layers, the actual transport time through the clay layer would likely be longer, due to the greater length of the flow path traveled.

2.1.6 Seismicity

As defined by NAC 444.6793, the site is located within a seismic impact zone, which means the site is located in an area with a 10 percent or greater probability that the maximum horizontal acceleration in lithified earth material will exceed 0.10g in a 250-year period.

Quaternary faults within a 10-mile radius of the site tend to be limited in length (10 km or less), and therefore, these faults have a limited capacity to generate large earthquakes. The nearest Quaternary fault is the Eugene Mountains Fault located approximately 3 miles southeast of the site. The nearest significant fault is the Western Humboldt Range fault zone located more than 20 miles southeast of the site.

Using the 2002 United States Geological Survey (USGS) database, Golder initially estimated the design peak ground acceleration (PGA) to be 0.28g (in bedrock) at the Jungo Disposal Site for an earthquake event with a 10 percent probability of exceedance in a 250-year period. Subsequent to this initial evaluation, the USGS seismic ground motion database was updated in 2008 to include the latest, state-of-the-art relationships between earthquake magnitude, distance from the epicenter, and peak bedrock accelerations. Using the 2008 USGS seismic hazard mapping database, the revised estimated design bedrock PGA for this site is 0.25g.

The Jungo Disposal Site landfill containment systems and environmental controls are designed to withstand an earthquake event resulting in a PGA of 0.25g without compromising the integrity of the containment systems and environmental controls. Section 2.3.4.4 describes these seismic impact evaluations.

2.2 Landfill Capacity and Site Development

2.2.1 Refuse Quantities and Landfill Capacity

The Jungo Disposal Site will serve as a regional disposal site for portions of Northern California and possibly Humboldt County. Although waste from Humboldt County is currently disposed of in the Humboldt County Regional Landfill, should the Humboldt County Commission so desire, the Jungo Disposal Site will accept local waste. Any refuse from Humboldt County will be delivered to the site in refuse hauling trucks. Refuse from Northern California will be delivered to the site by rail. Northern California includes the metropolitan Bay Area, including the nine county San Francisco Bay Area, and including tributary communities along the rail route. The metropolitan Bay Area and the tributary communities along the rail route have a total population of approximately 8 to 9 million. Refuse from Northern California will comprise more than 95% of the waste stream, which is estimated to be up to an average of 4,000 tons/day.

The site will accept only municipal solid waste (MSW). Typically, MSW from Northern California is processed to remove recyclable or compostable materials including selected metals, plastics, and greenwaste. In addition, a screening program exists to remove hazardous waste before it is loaded into waste containers. The screening program is described in the Operating Plan (***Volume III***).

The waste will be comprised of residential, commercial and selected special wastes, which will include construction and demolition (C&D) wastes, and waste tires. Wastes will be containerized for rail delivery to the disposal site. At the point of loading, most wastes will be commingled. Exceptions to commingling can include tires and inerts. No hazardous wastes will be accepted. Specific waste handling procedures have been developed for several of the segregated waste types accepted for disposal. The Jungo Disposal Site will not accept bulky metal waste, medical waste, liquid waste, and sludges.

JLII anticipates that the majority of the waste stream will be comprised of municipal solid waste from residential and commercial sources. The estimated quantities of the MSW and the various special wastes are summarized as follows:

- Residential and Commercial MSW: 70% to 100%
- Contaminated Soils (non-hazardous): 0 to 30%
- C&D Wastes: 0 to 15%
- Tires: 0 to 15%

The quantities of special wastes may vary seasonally depending on the length and size of the disposal contracts. However, residential and commercial MSW is expected to comprise at least 70 percent of the waste stream, and at times, may comprise the entire waste stream.

Figures 11 and 12 illustrate the landfill base grading system and the final refuse fill geometry, respectively. The maximum refuse thickness is 200 feet at the center of the landfill. The maximum refuse height extends approximately 200 feet above the surrounding grades at the center of the landfill.

The disposal volume is approximately 104 million cubic yards. Based on an estimated in place effective density of 1,100 pounds/cubic yard (pcy), the landfill has a refuse capacity of approximately 57.1 million tons. Effective density is defined as the weight of disposed refuse divided by the total volume occupied by refuse and soil cover. For initial planning, it assumed that approximately 600,000 tons of refuse will be disposed annually. Accordingly, this disposal rate would result in a projected life of 95 years. The projected life will decrease as the disposal tonnages increase.

The base grades have been designed to maximize the separation between the bottom of the liner system and groundwater. The minimum separation distance is approximately 24 to 26 feet at the sumps after settlement of the base grades due to the weight of the overlying refuse. The average separation distance will be approximately 37 to 38 feet following base settlement induced by refuse loading (Section 2.3.4.1). Section 2.3 describes the containment systems and controls used to protect the underlying groundwater from potential impacts of leachate and landfill gas.

The excavation will generate a total excavation volume of approximately 13 million cubic yards of soil, of which approximately 6 million cubic yards will be used to construct the liner system and the final cover system. Approximately 7 million cubic yards of soil are available for daily and intermediate soil cover use, which requires a refuse to soil cover ratio of approximately 14:1. As described in the Plan of Operations, measures will be incorporated to limit soil usage for refuse disposal through the extensive use of Alternative Daily Cover (ADC) materials and a reduced soil cover thickness.

2.2.2 Site Development

The site development is illustrated in the landfill design drawings provided in ***Volume II***. The landfill disposal boundary is located 100 feet from the west, south, and east property boundaries. The disposal boundary is located 200 to 300 feet from the north property boundary to allow the development of a rail yard for unloading waste containers.

As shown in ***Drawing 3 (Volume II)***, the landfill will be developed with 10 modules measuring approximately 56 acres each in area, where each module will contain a sump to collect and remove leachate. The site will be developed in phases with phases consisting of the construction of approximately 20 to 30-acres of the base liner. The landfill will also be closed in phases as the site is developed. Drawings 10 through 16 illustrate the landfill development, including base liner, refuse fill, and final cover construction at 2, 10, 25, 50 and 75 years following initial site development. During site development, areas that reach final grade will be allowed to consolidate and settle for a minimum 5-year period prior to final cover construction to reduce post-closure settlement impacts on the final cover. The final closure phase is an exception where the final cover will be constructed within 6-months of the placement of the last refuse shipment.

The Jungo Disposal Site will include the following facilities:

- A rail yard for unloading and loading waste containers;
- An administrative trailer;
- An equipment maintenance shop; and
- A break-room trailer for equipment operators and laborers.

Figure 13 shows the anticipated location of the initial facilities development. The locations of the equipment maintenance shop, and administrative and break-room trailers are anticipated to be temporary and may be relocated on the site as the landfill is developed. Specifically, the equipment maintenance shop and break-room trailer are likely to be periodically relocated near the active disposal area to reduce the travel time for equipment and site personnel.

The administration trailer and break-room will provide potable water and restrooms for site personnel. Wastewater will be discharged to a septic system located at the northwestern boundary of the site. Percolation tests will be completed to properly size and design the septic system.

The landfill perimeter will be bounded by a fence that is located 20 to 70 feet from the outer limits of the refuse disposal area.

2.3 Containment System and Environmental Controls

The Jungo Disposal Site is designed with an extensive system of low-permeability containment layers, high-permeability leachate controls to minimize leachate head on the liner, and a landfill gas collection and disposal system to control landfill gas. The containment system design and environmental controls include the following enhanced landfill gas and leachate control features to provide additional groundwater protection:

A double liner system with primary and secondary leachate collection

- A high capacity leachate collection and removal system LCRS on top of a composite liner system. The high capacity will limit maximum leachate build-up to a fraction of an inch and thereby reduce the leakage potential of leachate.
- Additional pipes will be incorporated in the LCRS system that can be tied to a gas control system. This allows the potential to develop a vacuum on top of the liner to minimize the potential for the migration of landfill gas through the liner.

The following sections describe details of the containment systems and controls, and the engineering analyses used to support the landfill design.

2.3.1 Liner Design and Base Grading

The base liner system consists of a prescriptive composite liner system, in accordance with NAC 444.681, comprised of the following components from top to bottom on the floor of the landfill (see Drawing 4, Volume II):

- 1-foot-thick operations soil layer;
- 1-foot thick gravel blanket for the primary LCRS with a permeability of 1 cm/s or greater;
- central leachate collection piping within each module to provide redundant leachate capacity;
- 16-oz geotextile cushion;
- 60-mil high-density polyethylene (HDPE) primary geomembrane;
- 2-foot thick compacted low-permeability soil liner with a permeability (k) less than or equal to $1x10^{-7}$ cm/s;
- A secondary geocomposite drainage layer for the secondary LCRS; and
- A 60-mil high-density polyethylene (HDPE) secondary geomembrane

On the side-slopes, the base liner system is comprised of the following components from top to bottom:

- 2-foot-thick operations soil layer;
- Geocomposite drainage layer (geonet with geotextile heat-bonded to both sides) for the LCRS;
- 60-mil HDPE primary geomembrane;
- 2-foot thick compacted low-permeability soil liner ($k \leq 1x10^{-7}$ cm/s).
- A secondary geocomposite drainage layer for the secondary LCRS; and
- A 60-mil high-density polyethylene (HDPE) secondary geomembrane

The base grading plan is shown in ***Figure 11***. The landfill will be divided into 10 modules with each module measuring approximately 56 acres each in area. The modules will be oriented in a north-south direction and the base grades are designed to minimize excavation depths and thereby maximize separation between the top of the composite liner system and groundwater. The floor of the landfill is graded at two percent toward the center of each module. The flowlines within each module extend from a center ridgeline oriented in an east-west direction and slope at a 1 percent grade to the north or south perimeter of the landfill. At the center ridge-line, the high end of the LCRS flow-line is near the existing ground surface. The maximum excavation depth is approximately 32 feet at each sump. Accordingly, the excavation plan results in the following groundwater separation distances immediately following construction (measured between groundwater and the top of the primary geomembrane liner):

- A minimum of 29 feet at the sump locations;
- A maximum of 60 feet at the center of the landfill; and
- An average separation distance of approximately 45 feet.

Based on the subsurface explorations described in Section 2.1.4, a generalized lithologic sequence indicates that the site is underlain by interbedded sands silts and clays. Two relatively low-permeability silt and clay layers were observed in each of the five borings, which appear to be relatively consistent across the site. The upper most silt and clay layer occurs at a depth of approximately 35 feet and 50 feet bgs, and therefore, provides another low-permeability barrier layer between the landfill and groundwater, which occurs at a depth of 59 to 60 feet bgs.

The existing site soils will not meet the permeability requirements for the low-permeability soil liner. Therefore, either suitable clay soils will be imported, or the on-site soils will be admixed with bentonite to produce a soil liner material with a permeability of 1×10^{-7} cm/s or less. Typically, an admixture of 5 to 9 percent of sodium bentonite by dry weight is required to produce a soil material that will meet this permeability requirement. The admixture percentage depends on the initial composition of the soils and the source of the bentonite. For the admix alternative, a bentonite admix design will be completed as part of the final drawings and technical specifications that will be prepared prior to the construction of each liner phase. In addition, construction quality assurance testing requirements will be established to verify that permeability requirements are achieved.

Construction Quality Assurance (CQA) will be completed during the liner construction activities to ensure that the construction complies with the liner design plans and specifications. Following each liner construction project, a certification report will be prepared and submitted to provide documentation that the construction activities were completed in accordance with the design plans and applicable federal and state regulations. A Nevada registered civil engineer will supervise CQA activities and certify the report.

Typical CQA activities will include, but are not limited to the following:

- Verification of the low-permeability soil materials including material quality, thickness, and compaction;
- Verification of the LCRS gravel including material quality and thickness;
- Observation and inspection of the geosynthetic materials for conformance with the engineering plans and specifications;
- Conformance testing of soil and geosynthetic materials;
- Documentation of construction procedures, and identification and resolution of construction problems; and

- Preparation of a CQA report providing documentation that the closure activities and construction complied with the project plans and specifications.

Table 2 summarizes the minimum CQA Plan requirements.

2.3.2 Leachate Collection and Removal System (LCRS)

The landfill liner system design includes a blanket LCRS (***Drawing 4, Volume II***) that has a high hydraulic capacity that is designed to collect leachate while minimizing leachate head build-up on the liner. The maximum leachate head on the liner is estimated to be only a fraction of one-inch, which is considerably less than the 12-inch (30 centimeter) maximum depth allowed by NAC 444.681. The leakage potential of a liner system is reduced by decreasing the potential head build-up on the liner system.

On the floor of the landfill, the LCRS will consist of a one-foot thick gravel blanket with a 6-inch diameter HDPE drainage pipe located on the center of the flow-line of each module (***Drawing 4***). Leachate collected within each module will be conveyed to a 2-foot deep, gravel filled sump measuring approximately 40 feet by 40 feet in plan area. Liquids will be extracted from an HDPE riser pipe using either submersible pumps or a pneumatic pump system.

Based on very conservative estimates of leachate generation, the LCRS on the floor has a factor of safety of more than 200 for hydraulic capacity. Similarly, the side-slope drainage geocomposite drainage layer has a minimum factor of safety of approximately 50 for hydraulic capacity. Section 2.3.4.3 further describes the design hydraulic capacity of the LCRS. The LCRS design is very conservative in that the design leachate generation rates are expected to significantly exceed actual leachate generation rates. Golder's experience with leachate generation in landfills located in arid regions indicates that very little leachate will be generated at the Jungo Disposal Site during operation.

Extracted leachate will be used for dust control over constructed, lined modules. In the event that the collected leachate exceeds the dust control needs, the excess leachate will be re-circulated within the landfill. However, such recirculation volumes are expected to be very small with a negligent impact on the moisture content of the waste or depth of leachate head on the liner.

Leachate pipes will be designed to withstand the weight of the refuse without crushing or buckling. HDPE pipes with a size-dimension ratio (SDR) of 11 or less can readily withstand the loads imposed by 200 feet of refuse.

2.3.3 Landfill Gas Control

A gas control system will be used to collect and dispose of landfill gas. At a minimum, the gas control will comply with Federal New Source Performance Standards (NSPS) and Emission Guidelines and require a Title V Permit (40 CFR Part 70 and NAC 445B) prior to operating the gas controls. Conceptually, the landfill gas system will consist of a system of horizontal and vertical gas wells, HDPE collection and header pipes, and condensate sumps. Initially, landfill gas will be disposed of using flares. A Waste-To-Energy (WTE) system may be used to dispose of gas and generate electricity if such a system is determined to be feasible for the Jungo Disposal Site.

Refuse gas wells will be installed as the refuse is placed, or alternatively drilled into the refuse after refuse placement. Operation of the gas control system will not occur until there is sufficient amount of methane to operate a flare disposal system. For landfills that receive 12 to 20 inches of annual precipitation in the western U.S., this typically requires 1 to 2 million tons of refuse in place and a minimum of 2 to 4 years of decomposition. Due to the arid climate of the Jungo Disposal Site, a longer time period may be required before sufficient gas is generated for flare operations.

As a further groundwater protective measure, perforated gas extraction pipes will be incorporated in the LCRS layer to allow gas withdrawal from above the liner system. This will further reduce the potential of gas migration from the liner system.

2.3.4 Liner Engineering Evaluations

To ensure the liner performs as intended, engineering evaluations described in the following sections were completed to evaluate foundation settlement, slope stability, leachate drainage capacity, drainage, and closure design.

2.3.4.1 Base Settlement

The placement of refuse changes the stresses acting on the foundation soils, which will result in settlement of the soils supporting the liner system for portions of the landfill. This settlement will tend to result in flatter drainage grades along the liner system in the future. The analyses presented in this section evaluate the magnitude of the calculated settlements and the resulting impact on the future drainage capacity of the LCRS.

A conservative settlement model was developed for the initial landfill design. The model used the five general lithologic layers described in Section 2.1.4, and incorporated the thickest observed clay layers. In addition, the model considered soils to a depth of 300 feet. The soils between a depth of 145 feet and 300 feet were modeled to consist of soils similar to that observed in the upper 145 feet.

Foundation settlement calculations included in ***Appendix E*** indicate a maximum settlement of 16 feet in the center of the landfill and 4.5 feet of settlement at the sumps. The resulting post-settlement drainage grades will range from 0.2 to 0.9 percent along the flowlines. Base settlement on the LCRS hydraulic capacities is discussed in Section 2.3.4.3.

Due to the need to maintain positive drainage on top of the liner for leachate control, base settlement is a critical design feature for this landfill. Therefore, additional borings will be completed for each module as described in 2.1.4.2.3. Prior to the final design of each module, additional soil testing will be completed to determine whether the current settlement model is appropriate or requires modification. Appropriate design changes will be implemented if necessary. Potential landfill design modifications may include changes to the drainage grades on top of the base liner or potential changes in the landfill heights. Substantive design changes will be submitted the Nevada Department of Environmental Protection for review and approval prior to implementation.

In addition to evaluating changes to the drainage grades on the liner, the potential impact of the elongation of the liner system on the slope was considered. Assuming zero settlement at the slope crest and 4.5 feet of settlement at the sump, the resulting liner elongation is computed to be 2.3 percent.

Accepted industry standards for textured HDPE geomembranes (GRI-GM13) specifies that these materials include a minimum elongation at yield of 12 percent and elongation at break of 120 percent. Elongation at yield represents the maximum strain under which the liner performs elastically. Prudent design standards limits strains below the yield strains.

The maximum strain of 2.3 percent is more than 5 times lower than the yield strain specified by GRI-GM13. The addition of negligible strains due to seismically-induced permanent displacements (Section 2.3.4.4) to the above relatively low settlement induced strains will result in overall strains that are well below the maximum yield strain for textured HDPE geomembranes.

Settlement monitoring will be completed during filling to verify settlement predictions. The settlement monitoring program will consist of vibrating wire sensor plates installed below the base of the liner system using the RST vibrating wire liquid level settlement monitoring system or equivalent. This type of instrumentation extends liquid filled tubes between a stationary reservoir at the edge of the landfill to sensor plates placed underneath the landfill. A vibrating wire transducer is used to measure the hydraulic pressure due to the elevation difference between the reservoir and sensor plate. As the landfill base grades settle, the measured hydraulic pressure will increase.

Figure 14 shows the proposed monitoring system array for the first constructed landfill cell located in the northeast corner of the site. Monitoring sensors will be included at intervals of approximately 100 feet along the LCRS flowline where the postconstruction settlements are the most critical as discussed in Section 2.3.4.3. Additional sensors will be included on the floor of the landfill as shown in Figure 14. The monitoring system will be included as part of the final construction plans and specifications provided to NDEP for review prior to construction. The specifications will also include provisions for establishing survey control for reservoir stations.

Settlement monitoring data will be collected at least quarterly during operations. The results of the settlement monitoring will be presented as part of two comprehensive landfill performance reviews as described in the July 2011 Groundwater Protection Evaluation Plan prepared by Golder. These landfill performance reviews will be completed at the end of the construction sequences shown in Drawings 11 and 12 (Volume II). The settlement monitoring will review the actual results to predicted results, confirm whether the results are within predicted parameters, and if necessary, review changes to landfill design in the event the initial settlement measurements are greater than those predicted. This will allow design changes, such as reducing the final grading height, to be made prior to the development of adverse drainage grades.

2.3.4.2 Slope Stability

Slope stability evaluations were completed to verify adequate stability under static and design seismic conditions. The primary failure mode of concern is the potential failure along the liner system, which generally has relatively low interface shear strengths. Potential failure of the underlying foundation soils was also considered.

Slope stability was evaluated using the computer program SLIDE (V. 3.047), which uses a two-dimensional method of slices and limit equilibrium methods to calculate factors of safety. The program was used to search for the failure plane with the lowest factor of safety.

Key assumptions common to the foundation and refuse slope stability analyses are summarized below.

- The shear strength of the refuse was modeled by a linear failure envelope represented by an internal angle of friction of 30 degrees and a cohesion of 200 pounds per square foot (psf), which is within the range of refuse strength parameters reported by Singh and Murphy (1990). These parameters are close to the values recommended by Kavazanjian (2001), which presents a refuse shear strength model with an internal friction angle of 33 degrees with a minimum shear strength of 500 psf.

- The unit weight of the total waste fill mass was assumed to be 70 pcf, which is a typical value for the unit weight of the refuse. Total weight is higher than effective density because it includes the weight of cover materials. This unit weight is conservative for a site that will extensively use ADC to minimize soil cover use.

- The critical liner interface is expected to occur between the compacted clay and textured HDPE geomembrane or compacted clay/geocomposite drainage layer. The design interface shear strength was assumed to be defined by an effective friction angle of 12 degrees with no cohesion. Based on Golder's experience in performing interface shear strength tests on liner materials, this design interface shear strength is expected to be conservative. Interface direct shear strength testing will be completed once the low-permeability soils are identified as part of the final liner design plans. Interface direct shear testing will be completed as part of the Construction Quality Assurance Program to ensure that the minimum design liner interface shear strength is achieved.

- Seismic stability was initially evaluated using the simplified seismic design procedure developed by Bray et. al. (1998) and assuming a design PGA value of 0.28g for bedrock. Additional analyses were subsequently completed using the updated PGA estimate of 0.25g and a more rigorous approach to model ground motions within the landfill as discussed in Section 2.3.4.4 and Appendix K.

- The underlying sands were assumed to have a shear strength corresponding to a friction angle of 30 degrees. The low plasticity clay was assumed to have a friction angle of 27 degrees based on the result of laboratory testing. The highly plastic clay was assumed to have a friction angle of 20 degrees based on the results of laboratory testing.

- The underlying clays were assumed to be drained during loading. Preliminary analyses indicate that the clays will reach 80 percent consolidation within 4 years of loading. Therefore, the development of excess pore pressures should be negligible.

Appendix F includes the results of the slope stability analyses. For potential failure along the liner, a static factor of safety of 1.9 was computed, and permanent seismically induced displacements were calculated to be less than 1 inch. Displacements of up to 6 to 12-inches along the liner system are generally accepted as being within the tolerance limits of liner systems without resulting in adverse damage.

Potential failure of the foundation soils is not a critical failure mode since the shear strengths of the native soils are considerably higher than the assumed liner interface shear strength.

2.3.4.3 Leachate Generation and LCRS Capacity

A very conservative leachate generation model was developed to conservatively size the hydraulic capacity of the LCRS. A conservative approach was used to provide an additional level of environmental protection relative to leachate management.

The model was developed using the computer program Hydrologic Evaluation of Landfill Performance (HELP). ***Appendix G*** includes details on the HELP modeling for the Jungo Disposal Site. The conservatively developed HELP model estimates a peak leachate generation rate of 75 gallons/acre/day (gpad) for the Jungo Disposal Site. This estimated leachate generation rate is very high for an arid site with only 8-inches of average annual precipitation. This level of leachate generation is comparable to modern, composite-lined landfills in Northern California with an average annual average precipitation of 25 to 30 inches. Golder's experience with landfills in arid regions is that they produce very limited to no leachate. The proposed Rawhide Landfill in Nevada, which is located in an area with approximately 6-inches of annual precipitation, estimated no leachate production.

The design of the LCRS consists of a high permeability gravel blanket draining a 2 percent grades toward perforated HDPE collection pipes. The HDPE pipes drain at a one percent grade toward the perimeter of the landfill.

The impact of base settlement is most severe in a direction perpendicular to the refuse slopes, which is in a direction parallel to the LCRS collection pipes. The settlement calculations indicate a post-settlement grade of approximately 0.2 percent along these pipes. Settlement impacts along the floor grades toward the LCRS pipes will be considerably less since the differential stresses and resulting differential settlements are less in the flow direction along the floor toward the pipes and the initial base grades along the flow will be 2 percent.

The table below summarizes the results of capacity and head predictions on the floor of the landfill draining toward the LCRS pipes and along the LCRS flowline. The critical portion of the drainage grades occurs along the outer 450 feet of the landfill where the greatest amount of differential loading occurs over the shortest distances.

Hydraulic Capacity Calculation Summary

LCRS Location	Before Settlement			After Settlement		
	Slope	Factor of Safety	Max. Predicted Head (Inch)	Slope	Factor of Safety	Max. Predicted Head (inch)
Landfill Floor	2.0%	450	0.02	1.5%	340	0.04
LCRS Flowline	1.0%	115	<6	0.2%	50	<6

Supporting calculations are summarized in Appendix G.

Given potential base settlement and the maximum leachate generation rates, the resulting factor of safety values and predicted maximum leachate head on the liner far exceeds minimum regulatory requirements. In addition, the peak leachate head depth on the landfill floor liner is estimated to be less than 0.1 inch.

2.3.4.4 Seismic Impact Evaluation

Appendix K includes a discussion of the seismic impact evaluations. These evaluations included the following:

- Permanent seismically induced displacements estimated to be zero to 0.6 inches, which is more than 10 times lower than the maximum allowable permanent seismically-induced displacement of 6 to 12 inches. The currently accepted practice for landfill liner design generally limit permanent displacements to 6 to 12 inches along landfill base liners. Permanent displacements within 6 to 12 inches are not expected to adversely compromise the liner integrity, leachate collection and removal system, or other environmental controls.
- A preliminary liquefaction assessment was completed using available data. The computed factor of safety against liquefaction is computed to be greater than 1.0. In most cases, the factor of safety is significantly larger than 1.0. These calculations in conjunction with the relatively high standard penetration test blow counts for silty sands and sandy silts, depth of

groundwater, and high vertical stresses imposed by the landfill indicate that liquefaction is unlikely at the Jungo Disposal Site.

As part of the more detailed subsurface drilling program discussed in Section 2.1.6, additional liquefaction analyses will be completed to confirm that liquefaction potential is negligible.

2.3.4.5 Drainage Controls During Operations

Drainage controls will be implemented during site development to control surface water run-on and run-off. Surface water run-on will be prevented by the following measures:

- A 4-foot high perimeter berm will be constructed to prevent run-on from shallow (6-inch to 12-inch) ponding that may occur locally following intense thunderstorms.
- Temporary retention basins will be located adjacent to module excavations to collect precipitation that occurs within the landfill excavation footprint. Water will be pumped from the temporary basin to the perimeter of the landfill. Drawing 10 illustrates an example of such a basin.

Surface water run-off will be controlled by ditches and down-drains that will be sized to accommodate a 25-year, 24-hour storm event in accordance with NAC 444.6885. A shallow ditch will be included around the perimeter of the site. The perimeter ditch will promote the accumulation of water until it exceeds the ditch depth and sheetflows westward to the surrounding grades where it will accumulate in shallow depressions until it evaporates or infiltrates into the underlying soil.

2.3.5 Closure Design

The prescriptive cover system (NAC 444.6891) requires a minimum 6-inch thick erosion layer underlain by minimum 18-inch thick infiltration layer. In addition, the permeability of the cover shall be equal to or less than $1x10^{-5}$ cm/s or less than the permeability of any component of the bottom liner, whichever is less.

A final cover system will be constructed over the waste at the Jungo Disposal Site as part of the closure activities. The final cover system is a prescriptive cover, in accordance with NAC 444.6891) consisting of the following components (***Drawing 8, Volume II***):

- A minimum 2-foot thick vegetative soil layer;
- A geocomposite drainage layer;
- A 60-mil HDPE geomembrane layer (textured on both sides); and
- A one-foot thick foundation layer.

The Jungo Cover System uses a vegetated, 2-foot thick erosion layer in place of the minimum 6-inch thick erosion layer. In addition, a geocomposite drainage layer, 60-mil HDPE geomembrane and a one-foot thick foundation layer are substituted for the minimum 18-inch thick infiltration layer. These modifications are necessary to result in a cover system that is no more permeable than the bottom liner in accordance with NAC 444.6891(a) (i.e. install low-permeability geomembrane component) and to establish an erosion resistant layer over the geomembrane layer that is capable of supporting the growth of native plants per NAC 444.6891(b).

The above cover system provides a low-permeability barrier that has permeability less than or equal to the base liner system. HELP modeling of the cover system indicates that a negligible amount of water will infiltrate through the cover. HELP analyses for the closed conditions are summarized in ***Appendix G***.

The Jungo Disposal Site will pursue an alternative Evapotranspirative (ET) final cover design once the landfill is in operation. An ET cover typically consists of 3 to 5 feet of soil that stores infiltration and then releases it through evapotranspiration. Based on Golder's experience with ET covers, the site climate and soil types appear suitable for an effective ET cover system. The alternative ET cover design will include supporting soil laboratory testing and unsaturated flow modeling. If the modeling results indicate that ET cover is equivalent or superior to the prescriptive cover system, then a field trial will be constructed on portions of the landfill that have achieved final grades. A work plan detailing the laboratory testing, modeling, and field trial program will be prepared and submitted to NDEP for review and approval.

2.3.5.1 Final Cover Grading

Figure 12 shows the final cover grades for Jungo Disposal Site landfill. The final cover grades reach a maximum elevation of 4,172 feet mean sea level (msl) and maintain a maximum side-slope inclination of 4H:1V (horizontal to vertical). To facilitate drainage and minimize erosion, 25-foot wide benches are incorporated into the side-slopes a maximum of every 50 feet vertically. The top surface will be graded at 5 percent to accommodate postclosure refuse settlements and maintain positive drainage.

2.3.5.2 Erosion

Final landfill slopes will be inclined no steeper than 4H:1V. Minimum final surface slopes will be 5 percent. To mitigate potential wind and water erosion, the vegetative layer thickness was increased from one foot to two feet.

As part of the closure activities, the integrity of the final site face will be maintained by the placement of a vegetative layer to provide erosion control. A Revegetation Plan is attached to this Report of Design in Appendix L.

A surface water erosion analysis was completed for the slopes using the Revised Universal Soil Loss Equation program, RUSLE Version 1.06 (United States Office of Surface Mining and Reclamation, 1998). The analysis conservatively assumes that the cover is poorly vegetated although the cover will be properly vegetated with suitable desert grasses. The results of the conservative analyses indicate an estimated maximum soil loss for the proposed final grades of 0.03 inches/year which is less than an average of approximately 1–inch over a 30 year postclosure period. The surface water erosion soil loss analysis is presented in ***Appendix H***.

Wind erosion was also evaluated for the Jungo Disposal Site. Golder consulted with wind erosion specialists working for the United States Department of Agriculture, Natural Resource Conservation Service (NRCS), Wind Research Unit. Because of the complexity of wind erosion calculations, which were primarily developed for agricultural applications, NRCS staff recommended the use of Single-Event Wind Erosion Evaluation Program (SWEEP, Ver. 1), which is a part of the Wind Erosion Prediction System (WEPS). WEPS is a process-based, continuous, daily time-step model that simulates weather, field conditions, and erosion.

Using SWEEP, wind erosion is estimated to be negligible if the Leaf Area Index (LAI) is less greater than approximately 0.3. The desert vegetation for closure is estimated to have an LAI that will vary seasonally from 0.2 to 0.4. Conservatively assuming an LAI value of 0.2 is representative of half of the year, Golder conservatively estimated that the annual soil erosion for the final cover would be less than 0.15 inches per year, or less than 4.5 inches over a 30-year postclosure period. If the LAI is 0.3 or greater, the estimated annual soil loss due to wind erosion is estimated to be negligible. The wind erosion soil loss analysis is presented in ***Appendix H***.

2.3.5.3 Postclosure Cover Settlement

Settlement analyses were performed to evaluate the impact of postclosure settlement on the final cover grades. Refuse settlement typically exhibits a large, rapid, initial settlement rate referred to as primary settlement, which is followed by a long-term, progressively decreasing, settlement rate that is referred to as secondary settlement. Primary settlement generally occurs within weeks to months of the initial refuse placement. However, secondary settlement occurs for many years as waste materials decompose and compress.

The calculated postclosure settlements assume that primary settlements are complete prior to closure, but secondary settlements will continue throughout the entire 30-year postclosure monitoring period. As indicated in ***Appendix I***, the postclosure grades following settlement will be greater than three percent, which is sufficient to promote positive drainage from the cover.

2.3.5.4 Cover Veneer Slope Stability

The stability of the cover system considers the potential occurrence of a failure within the final cover components. This failure mode is primarily a function of the interface strengths of the cover materials and the maximum final slope inclinations. Static stability analyses were completed using an infinite slope analysis and verified by the computer program XSTABL (v. 5.2.02). Yield accelerations were determined using the computer program XSTABL (v. 5.202). XSTABL uses two-dimensional, limit-equilibrium methods to evaluate stability.

Evaluation of the stability of the cover components was based on the following assumptions:

- The maximum cover grade was assumed to be 4H:1V (maximum slope in between benches). Following closure, settlement will reduce the slope height and inclination, which will tend to increase slope stability with time;
- The critical interface occurs between either the vegetative soil layer/geocomposite drainage layer or the geocomposite/textured geomembrane layer. Based on a shear strength data base prepared by Golder's Geosynthetics Laboratory (***Appendix F***), the critical shear strength parameters were assumed to be represented by an internal friction angle of 23 with no adhesion. At low normal loads, the interface shear strength between the textured geomembrane and underlying low-permeability soil layer is expected to be greater than 23 degrees based on Golder's data base.
- The simplified seismic design procedure by Bray et. al. (1998) was used to estimate seismic displacements for the cover system using the initial design PGA of 0.28g developed based on the USGS database. As discussed in Section 2.1.6, the design PGA has been revised to a lower value of 0.25g based on updated USGS seismic hazard mapping database.

The factor of safety for static conditions is calculated to be 1.7 (***Appendix F***). The results for the design seismic loading conditions indicate seismically induced permanent displacements of less than 4-inches. Based on current engineering practice, a maximum allowable seismically induced permanent displacement of 6 to 12 inches is acceptable for modern, geosynthetic landfill covers located in a seismic impact zone.

2.3.5.5 Surface Water Controls

Surface water controls will be installed on the final cover system to control surface water run-off and minimize erosion of the cover system. Drawing 7 illustrates a conceptual surface water drainage plan for the Jungo Disposal Site at closure. Surface water will be controlled by ditches on the slope benches, berms on the top-deck of the landfill, and down-drains along the side-slopes. All surface water controls are sized to accommodate the 25-year, 24-hour storm event (NAC 444.6885).

During site development, surface water will be managed as follows:

- Surface water run-off from active disposal areas will be directed to the interior sides of the landfill to temporary stormwater basins where it will be pumped to a temporary, lined, storage impoundment. The water will be sampled from the lined impoundment, and if free of waste constituents, it will be discharged to a run-on/run-off control basin where the water will be stored until it evaporates or infiltrates into the subsurface soils. A copy of the sampling results will be provided to the Nevada Department of Wildlife for review. If the water is impacted, it will be retained in the pond until it can be used as dust control over lined areas of the landfill footprint and/or evaporated within the basin.

- Surface water run-off from the permanent exterior slopes will be directed to a broad (28- to 30-foot wide), shallow perimeter ditch. The ditch ranges from 2 to 6 feet in depth and drains gradually to a stormwater run-on/run-off control basin where the water will evaporate or infiltrate into the subsurface soils.

Drawings 10 through 14 illustrate the above basin controls. For illustration purposes, the temporary lined basin and run-on/run-off control basin are shown in the southwest corner near the end of site development. During initial development these basins may be temporarily located closer the disposal operations and then relocated periodically in a southwesterly direction as the site development progresses. At closure, the liner for the temporary lined basin will be removed and disposed of in the landfill and then basin regarded to incorporate it as part of the final run-on/run-off control basin.

If ongoing sampling of the surface water from the temporary, lined basin indicates that the site operations are managed such that surface water consistently is not impacted by waste constituents, then JLII will propose, for NDEP's review and approval, the reduction of the frequency of sampling and testing or the elimination of the sampling and testing.

In the event that surface water in the temporary basin is impacted by waste constituents, JLII will investigate and evaluate the source(s) of the impacts. This will include, but not limited to, identifying and promptly repairing any erosion rills that expose refuse and allow contact between refuse and surface water and/or leachate seeps that may be commingling with surface water. If the Nevada Department of Wildlife determines that water impoundments may lead to wildlife mortality then an Industrial Artificial Pond Permit (IAPP) will be obtained. The temporary basin will be designed to accommodate a minimum 25-year, 24 hour storm event (1.62-inches).

At final build-out, the surface water run-off from the 25-year, 24 hour storm event is estimated to be 22 acre feet. The storage capacity of the final run-on/run-off control basin and perimeter ditches is approximately 37 acre-feet, which will accommodate more than a 25-year, 24-hour storm events while maintaining a minimum freeboard of one foot. More than two back-to-back 25-year storm events are required to result in an off-site stormwater release. ***Appendix J*** includes the stormwater calculations.

3.0 REFERENCES

Berger, 1995, Ground-Water Conditions and Effects of Mine Dewatering in Desert Valley, Humboldt and Pershing Counties, Northwestern Nevada, 1962-91. U.S. Geological Survey, Water-Resources Investigations Report 95-4119. Carson City, Nevada. 1995.

Bray, J.D., E.M. Rathje, A.J. Augello and S.M. Merry, *Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills*. Geosynthetics International, 1998, Vol.5, NOS. 1-2.

Cohen, Philip, 1963, Specific-yield and particle-size relations of Quaternary alluvium, Humboldt River valley, Nevada, U.S. Geological Survey Water-Supply Paper 1669-M, 24 p.

Cohen, Philip, 1966. Water in the Humboldt River Valley Near Winnemucca, Nevada, U.S. Geological Survey Water-Supply Paper 1816, 69 p.

Kavazanjian, E., 2001. *Mechanical properties of municipal solid waste*, Proceedings Sardinia 2001, 8th International Waste Management and Landfill Symposium, Cagliari, Italy, 415-424.

National Oceanic and Atmospheric Administration, 2006, Precipitation-Frequency Atlas of the United States, NOAA Atlas 14, Volume 1, Version 4.

Nevada Division of Water Resources (driller well logs, water-level database).

Singh, S. and Murphy, B. (1990). "Evaluation of the Stability of Sanitary Landfills" Geotechniques of Waste Fills – Theory and Practice, ASTM STP 1070, Arvid Landva, G. David Knowles, editors, American Society for Testing and Materials, Philadelphia.

United States Department of Agriculture, Natural Resources Conservation Service, 2006, *GIS Map Unit Description (Brief Generated), Humboldt County, Nevada, East Part.* Survey Area Version: 4. Survey Area Version Date: 12/12/2006.

United States Environmental Protection Agency, 2002, "Assessment and Recommendations for Improving the Performance of Waste Containment Systems," December 2002, EPA Publication EP/600/R-02/099.

USGS Scientific Investigations Report 2005-5291, 2005, Evaluation of Precipitation Estimates from PRISM for the 1961-90 and 1971-2000 Data Sets, Nevada.